Table of Contents

Chapter 5 - Roadway Design

5A General Information

5A-1----------General Information
A. Concept..... 1
B. References..... 1

5B Street Classifications

5B-1----------Street Classifications
A. General..... 1
B. Arterial Streets..... 1
C. Collector Streets..... 2
D. Local Streets..... 2
E. Private Streets..... 2

5C Geometric Design Criteria

5C-1----------Geometric Design Tables
A. General..... 1
B. Design Controls and Criteria..... 1
C. Roadway Design Tables..... 3
D. References..... 9

5C-2----------Geometric Design Elements
A. Level of Service..... 1
B. Sight Distance..... 2
C. Horizontal Alignment..... 6
D. Vertical Alignment..... 9
E. Pavement Crowns..... 12
F. Lane Width..... 12
G. Two-way Left-turn Lanes..... 12
H. Raised Median Width..... 13
I. Bridges..... 13
J. Clear Zone..... 13
K. Object Setback..... 15
L. Border Area..... 15
M. Curbs..... 15
N. Parking Lane..... 16
O. Cul-de-sacs..... 16
P. Shoulder Width..... 16
Q. Intersection Radii..... 17
R. Pavement Thickness..... 17
S. References..... 17

5D HMA Pavement Mixture Selection

5D-1----------HMA Pavement Mixture Selection
A. Scope……...……1
B. Definitions.....1
C. Design Checklist.....2
D. Material Properties……..5
E. Use of Mixture Selection Guide and Design Criteria Tables..…… 6
F. Example Plans……..……..6
G. Examples for Determination of Traffic ESALs.....7
H. Tables and Figures..…….....8

5E PCC Pavement Mixture Selection

5E-1----------PCC Pavement Mixture Selection
A. General Information……..…… 1
B. Cementitious Materials.....1
C. Supplementary Cementitious Materials.....3
D. Aggregates…….. 6
E. Chemical Admixtures..... 9
F. Water……..……10
G. Air-entrainment..... 10
H. Slump..…….....11
I. Concrete Mixtures……..…… 12
J. References..... 15

5F Pavement Thickness Design

5F-1----------Pavement Thickness Design
A. General……..…… 1
B. Pavement Thickness Design Parameters..... 2
C. Calculating ESAL Values.....9
D. Determining Pavement Thickness……..... 14
E. Example Pavement Thickness Design Calculations.....19
F. References…….22

5G PCC Pavement Joints

5G-1----------General Information for Joints
A. General Information……..……1
B. Crack Development……...……..... 2
C. Crack Control.....3
D. Considerations for Good Pavement Jointing..... 4
E. Load Transfer.....5

5G-2----------Types of Joints
A. Jointing……..... 1
B. Joint Spacing…….....…1
C. Joint Types…….……....1
D. Transverse Dowel Bar Size and Length…….... 14
E. Joint Reinforced Concrete Pavements..... 14
F. Miscellaneous PCC Pavement Jointing Figures……... 17
G. References…….22

5G-3----------Jointing Urban Intersections
A. Jointing Urban Transition Areas.....6
B. Jointing Cul-de-sacs.....15

5G-4----------Jointing Rural Intersections
A. Example 1: T-Intersection……..... 1
B. Example 2: Intersection at a Divided Highway.....4

5G-5----------Jointing Concrete Overlays
A. General Information……..... 1
B. Bonded Concrete Overlays..……..... 1
C. Unbonded Concrete Overlays…….....2
D. References.....4

5H Railroad Crossings

5H-1----------Railroad Crossings
A. Railroad Crossing Improvements……..... 1
B. Railroad Crossing Construction...……..... 1
C. Working with a Railroad.....……..... 1
D. Railroad Related Agencies in Iowa……..... 2
E. Railroad Companies in Iowa..... 2

5I Access Management

5I-1----------General Access Management
A. General Information……..……1
B. Access Permit Procedure..... 1
C. Definitions……..… 1
D. Entrance Type.....2
E. Access Management Principles..... 2
F. References……...3

5I-2----------Transportation System Considerations
A. Provide a Specialized Roadway System (Principle 1)……..……1
B. Limit Direct Access to Major Roadways (Principle 2)...……..……1
C. Promote Intersection Hierarchy (Principle 3)……..…..1
D. Locate Signals to Favor through Movements (Principle 4)...……..…...1
E. Provide a Supporting Street and Circulation System (Principle 10)…..…… 2

5I-3----------Access Location, Spacing, Turn Lanes, and Medians
A. Preserve the Functional Area of Intersections and Interchanges (Principle 5)…...1
B. Limit the Number of Conflict Points (Principle 6)……..……2
C. Separate Conflict Areas (Principle 7)……..…...2
D. Remove Turning Traffic from Through-traffic Lanes (Principle 8).....7
E. Use Nontraversable Medians to Manage Left Turn Movements (Principle 9).....8
F. References……10

5I-4----------Driveway Design Criteria
A. General……..……1
B. Width Measurement……..……1
C. Dimensions……..…….. 2
D. Sight Distance…….....4
E. Driveway Grades..... 5
F. Other Criteria..... 7
G. References.....8

5J Traffic Impact Studies

5J-1----------Traffic Impact Studies
A. General……..……1
B. Study Process.....1
C. Iowa DOT Access Permits……..3
D. References……...3

5K Pavement Preservation Program

5K-1----------General Information for Pavement Preservation Program
A. Concept……..…… 1
B. Definitions…..... 1
C. Benefits……..…….. 2

5K-2----------Pavement Preservation Process
A. Pavement Deterioration..……..…… 1
B. Evaluating Pavement Conditions.....2

5K-3----------Preventative Maintenance Treatment Type Selection
A. Introduction..……..……1
B. Flexible Pavement Treatment Types..... 1
C. Rigid Pavement Treatment Types…….……..3
D. References…….……..6

5L Permeable Interlocking Pavers

5L-1----------Permeable Interlocking Pavers
A. General……..……1
B. Structural Design..... 2
C. Hydraulic Design…….……... 3
D. Construction Elements……6
E. Maintenance……6

5M Complete Streets

5M-1----------Complete Streets
A. Background..……..……1
B. Design Guidance.....2
C. Design Elements.……..... 3
D. Traffic Calming……...……8
E. References……...9

General Information

A. Concept

The primary consideration of this chapter is that all new roadways and major reconstruction of existing corridors provide for safe, efficient, and economic transportation throughout the design life of the roadway. The values contained herein, specifically under design criteria, are to be considered basic design guidelines that will serve as framework for satisfactory design of new street and highway facilities. The Project Engineer is encouraged to develop the design based on this framework and tailored to particular situations that are consistent with the general purpose and intent of the design criteria through the exercise of sound engineering judgment.

The design criteria provided herein are divided into two classifications: preferred and acceptable. Designers should strive to provide a design that meets or exceeds the preferred criteria. Situations do arise that require special considerations; therefore, to eliminate hardships or problems, the Engineer may allow an exception to the preferred design criteria upon submittal of justification for such variances by the Project Engineer.

Cost effective design is encouraged along with the joint use of the transportation corridor and the consideration of the environment. The values contained herein are not intended as criteria for resurfacing, restoration, or rehabilitation projects.

B. References

The design for roadway facilities should comply with the current edition of the following references, unless a specific edition is cited:

Jurisdiction Supplemental Design Standards.

The American Association of State Highway and Transportation Officials (AASHTO). *A Policy on Geometric Design of Highways and Streets* ("Green Book").

The American Association of State Highway and Transportation Officials (AASHTO). *Roadside Design Guide*.

The U.S. Department of Transportation - Federal Highway Administration. *Manual on Uniform Traffic Control Devices (MUTCD)* and *Traffic Control Device Handbook.*

Transportation Research Board. *Highway Capacity Manual.*

The Institute of Transportation Engineers. *Transportation and Traffic Engineering Handbook.*

Street Classifications

A. General

The classifying of streets and highways is necessary for communication among engineers, administrators, and the general public. Streets can be classified based upon major geometric features (e.g. freeways, streets, and highways), route numbering (e.g. U.S., State, and County), or Administrative classification (e.g. National Highway System or Non-National Highway System). However, functional classification, the grouping of streets and highways by the character of service they provide, was developed specifically for transportation planning purposes and is the predominant method of classifying streets for design purposes. For urban areas, the functional classification hierarchy consists of major arterials, minor arterials, collectors, and local streets.

The information contained in this section is based on AASHTO criteria. The Project Engineer should use the various AASHTO publications and particularly the current edition of AASHTO's "Green Book" to verify the application of values provided herein when complex design conditions or unusual situations occur.

B. Arterial Streets

1. **Major (Principal) Arterial:** The major arterial (referred to as a principal arterial by AASHTO) serves the major center of activities of urbanized areas, the highest traffic volume corridors, the longest trip, and carries a high proportion of a total urban travel on a minimum of mileage. The system should be integrated both internally and between major rural connections.

   The major arterial system carries most of the trips entering and leaving the area as well as most of the through movements bypassing the central city. In addition, significant intra-area travel such as between central business districts and outlining residential areas, between major inner-city communities, and between major suburban centers, is served by major arterials. Frequently, the major arterial carries important intra-urban as well as inter-city bus routes. Finally, in urbanized areas, this system provides continuity for all rural arterials that intercept the urban boundary.

   Access to private property from the major arterial is specifically limited in order to provide maximum capacity and through movement mobility. Although, no firm spacing rule applies in all or even in most circumstances, the spacing between major arterials may vary from less than 1 mile in highly developed central areas to 5 miles or more in developed urban fringes.

2. **Minor Arterial:** The minor arterial inter-connects with and augments the major arterial system. It accommodates trips of moderate length at a somewhat lower level of travel mobility than major arterials. This system places more emphasis on land access but still has specific limits on access points. A minor arterial may carry local bus routes and provide intra-community continuity but ideally does not penetrate identifiable neighborhoods. This system includes urban connections to rural collector roads where such connections have not been classified as urban major arterials.

   The spacing of minor arterials may vary from 1/8 to 1/2 mile in highly developed areas to 2 to 3 miles in suburban fringes but is not normally more than 1 mile in fully developed areas.

C. Collector Streets

The collector street system provides both land access and traffic circulation within residential neighborhoods and commercial and industrial areas. It differs from the arterial system in that facilities on the collector system may penetrate residential neighborhoods, distributing trips from the arterials through the area to their ultimate destinations. Conversely, the collector street also collects traffic from local streets in residential neighborhoods and channels it into the arterial system. In the central business district, and in other areas of similar development and traffic density, the collector system may include the entire street grid.

1. **Major Collector:** This type of street provides for movement of traffic between arterial routes and minor collectors and may collect traffic, at moderately lower speeds, from local streets and residential and commercial areas. A major collector has control of access to abutting properties with a majority of access at local street connections. Normally, a slightly higher emphasis is placed on through movements than direct land access.

2. **Minor Collector:** This type of street provides movement of traffic between major collector routes and residential and commercial local streets as well as providing access to abutting property at moderate low speeds. Consideration for through movements and direct land access is normally equal.

D. Local Streets

Local streets allow direct access to abutting land and connections to the higher order street systems. They offer the lowest level of mobility and deliberately discourage major through traffic movements.

E. Private Streets

Certain Jurisdictions allow private streets in specific situations. Private streets are similar to the local streets but generally are located on dead-end roads less than 250 feet in length, short loop streets less than 600 feet in length, or frontage roads parallel to public streets. Design criteria for local private streets are not included in this manual. The Jurisdiction should be contacted to determine if they are allowed.

Geometric Design Tables

A. General

The following sections present two sets of design criteria tables - Preferred Roadway Elements (Table 5C-1.01) and Acceptable Roadway Elements (Table 5C-1.02). In general, the "Preferred" table summarizes design values taken from the AASHTO's "Green Book" that may be considered "preferred" while the "Acceptable" table represents AASHTO minimums or practical minimums not covered in AASHTO.

Designers should strive to provide a design that meets or exceeds the criteria established in the "Preferred" table. For designs where this is not practical, values between the "Preferred" and "Acceptable" tables may be utilized, with approval of the Engineer.

B. Design Controls and Criteria

The selection of various values for roadway design elements is dependent upon three general design criteria: functional classification, design speed, and adjacent land use.

1. **Functional Classification:** The first step in establishing design criteria for a roadway is to define the function that the roadway will serve (refer to Section 5B-1 for street classifications). The functional classification of the roadway is the basis for the cross-sectional design criteria shown in Tables 5C-1.01 and 5C-1.02. It also serves as the basis for the ultimate selection of design speed and geometric criteria.

   Under a functional classification system, design criteria and level of service vary according to the intended function of the roadway system. Arterials are expected to provide a high level of mobility for longer trip length; therefore, they should provide a higher design speed and level of service. Since access to abutting property is not their main function, some degree of access control is desirable to enhance mobility. Collectors serve the dual function of accommodating shorter trips and providing access to abutting property. Thus, an intermediate design speed and level of service is important. Local streets serve relatively short trip lengths and function primarily for property access; therefore, there is little need for mobility or high operating speeds. This function is reflected by use of lower design speeds and an intermediate level of service.

2. **Design Speed:** Design speed is the selected speed used to determine various geometric features of the roadway, including horizontal and vertical alignment. The design speed selected should be as high as practical to attain the desired degree of safety, mobility, and efficiency. It is preferred to select a design speed that is at least 5 mph greater than the anticipated posted speed limit of the roadway. Selecting a design speed equal to the posted speed limit may also be acceptable and should be evaluated on a project by project basis, subject to approval of the Engineer. Once the design speed is selected, all pertinent roadway features should be related to it to obtain a balanced design.

   In some situations, it may be impractical to conform with the desired design speed for all elements of the roadway (e.g. horizontal radius or clear zone). In these situations, warning signs or additional safety treatments may be required (e.g. warning signs or guard rail).

3. **Adjacent Land Use:** In addition to functional classification and design speed, the surrounding land use can impact the design elements of the roadway corridor as well. Land use can be categorized into three groups: residential, commercial, and industrial.

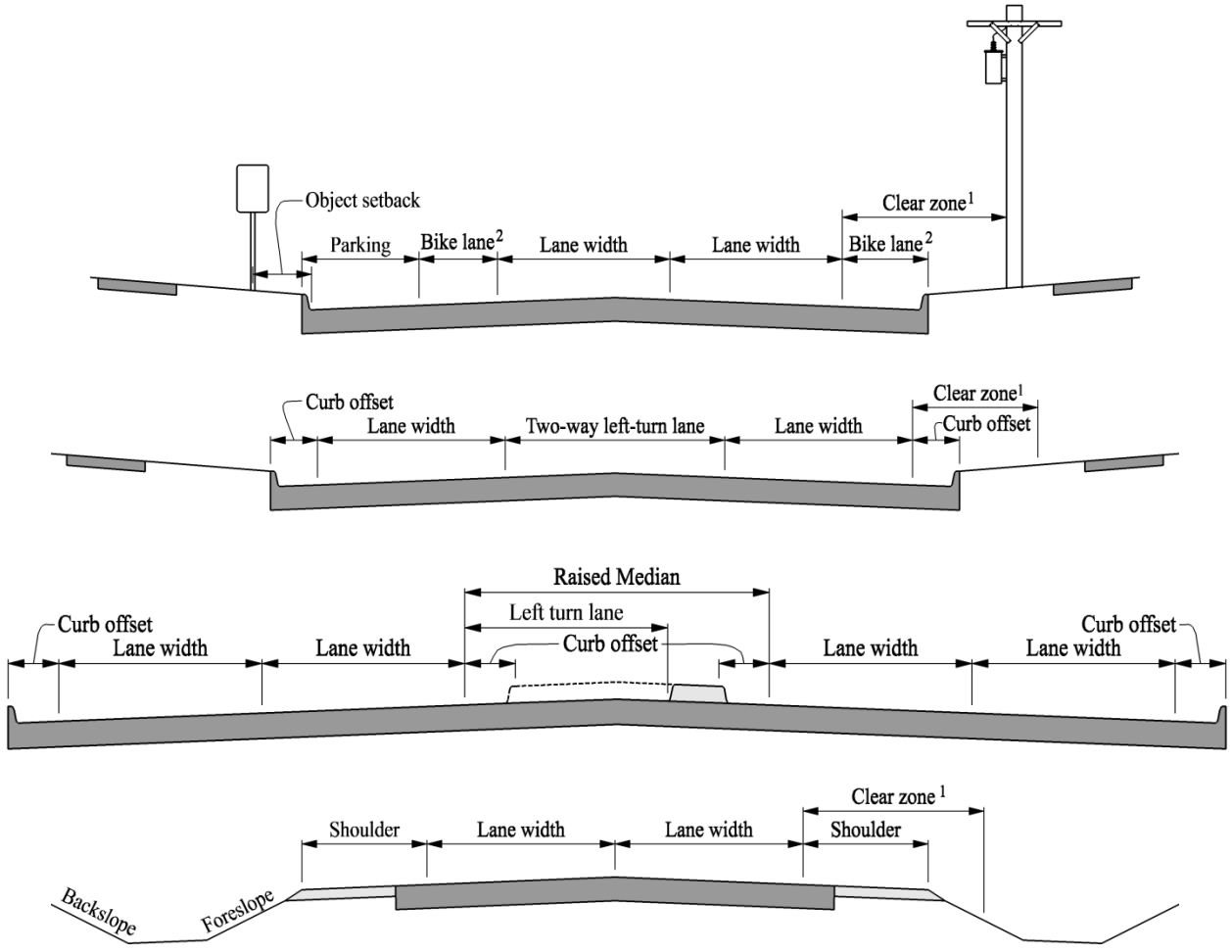
    a. Residential areas are regions defined by residential or multi-family zoning districts where single-family houses, apartment buildings, condominium complexes and townhome developments are located. Because these facilities typically have lower overall traffic volumes, low truck volumes, and are utilized primarily by drivers who are familiar with the roadway, some design values can be set at a lower level than for commercial or industrial areas.

    b. Commercial and industrial areas are highly developed regions generally defined by commercial and industrial zoning districts where factories, office buildings, strip malls, and shopping centers are or will be located. The areas typically require higher level design values due to increased traffic volumes, increased truck volumes, and decreased driver familiarity.

C. Roadway Design Tables

The following figures illustrate the location of various design elements of the roadway cross-section as specified in Tables 5C-1.01 and 5C-1.02.

Figure 5C-1.01: Roadway Design Elements



[1] Clear zone is measured from the edge of the traveled way.
[2] See Chapter 12 for bike lane requirements.

Table 5C-1.01: Preferred Roadway Elements

Elements Related to Functional Classification

Design Element	Local		Collector		Arterial	
	Res.	***C/I***	***Res.***	***C/I***	***Res.***	***C/I***
General						
Design level of service[1]	D	D	C/D	C/D	C/D	C/D
Lane width (single lane) (ft)[2]	10.5	12	12	12	12	12
Two-way left-turn lanes (TWLTL) (ft)	N/A	N/A	14	14	14	14
Width of new bridges (ft)[3]	See Footnote 3					
Width of bridges to remain in place (ft)[4]	----------	----------	----------	----------	----------	----------
Vertical clearance (ft)[5]	14.5	14.5	14.5	14.5	16.5	16.5
Object setback (ft)[6]	3	3	3	3	3	3
Clear zone (ft)	Refer to Tables 5C-1.03, 5C-1.04, and 5C-1.05					
Urban						
Curb offset (ft)[7]	2	2	2	3	3	3
Parking lane width (ft)	8	8	8	10	N/A	N/A
Roadway width with parking on one side[8]	26/31[9]	34	34	37	N/A	N/A
Roadway width without parking[10]	26	31	31	31	31	31
Raised median with left-turn lane (ft)[11]	N/A	N/A	19.5	20.5	20.5	20.5
Cul-de-sac radius (ft)	45	45	N/A	N/A	N/A	N/A
Rural Sections in Urban Areas						
Shoulder Width (ft)						
ADT: under 400	4	4	6	6	10	10
ADT: 400 to 1,500	6	6	6	6	10	10
ADT: 1,500 to 2000	8	8	8	8	10	10
ADT: above 2,000	8	8	8	8	10	10
Foreslope (H:V)	4:1	4:1	4:1	4:1	6:1	6:1
Backslope (H:V)	4:1	4:1	4:1	4:1	4:1	4:1

Res. = Residential, C/I = Commercial/Industrial

Elements Related to Design Speed

Design Element	Design Speed, mph[12]							
	25	***30***	***35***	***40***	***45***	***50***	***55***	***60***
Stopping sight distance (ft)	155	200	250	305	360	425	495	570
Passing sight distance (ft)	900	1090	1,280	1,470	1,625	1,835	1,985	2,135
Min. horizontal curve radius (ft)[13]	198	333	510	762	1,039	926	1,190	1,500
Min. vertical curve length (ft)	50	75	105	120	135	150	165	180
Min. rate of vertical curvature, Crest (K)[14]	18	30	47	71	98	136	185	245
Min. rate of vertical curvature, Sag (K)	26	37	49	64	79	96	115	136
Minimum gradient (percent)	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Maximum gradient (percent)	5	5	5	5	5	5	5	5

Note: For federal-aid projects, documentation must be provided to explain why the preferred values are not being met. For non-federal aid projects, the designer must contact the Jurisdiction to determine what level of documentation, if any, is required prior to utilizing design values between the "Preferred" and "Acceptable" tables.

Table 5C-1.01 Footnotes:

[1] Number of traffic lanes, turn lanes, intersection configuration, etc. should be designed to provide the overall specified LOS at the design year ADT. Two LOS values are shown for collectors and arterials. The first indicates the minimum overall LOS for the roadway as a whole; the second is the minimum LOS for individual movements at intersections.

[2] Width shown is for through lanes and turn lanes.

[3] Bridge width is measured as the clear width between curbs or railings. Minimum bridge width is based upon the width of the traveled way (lane widths) plus 4 feet clearance on each side; but no less than the curb-face to curb-face width of the approaching roadway. Minimum bridge widths do not include medians, turn lanes, parking, or sidewalks. At least one sidewalk should be extended across the bridge.

[4] See Table 5C-1.02, for acceptable values for width of bridges to remain in place.

[5] Vertical clearance includes a 0.5 foot allowance for future resurfacing.

[6] Object setback does not apply to mailboxes constructed and installed according to US Postal Service regulations, including breakaway supports.

[7] Values shown are measured from the edge of the traveled way to the back of curb. Curb offset is not required for turn lanes. On roadways with an anticipated posted speed of 45 mph or greater, mountable curbs are required. For pavements with gutterline jointing, the curb offset should be equal to or greater than the distance between the back of curb and longitudinal gutterline joint.

[8] Parking is allowed along one side of local or collector streets unless restricted by the Jurisdiction. Some jurisdictions allow parking on both sides of the street. When this occurs, each jurisdiction will set their own standards to allow for proper clearances, including passage of large emergency vehicles. Parking is normally not allowed along arterial roadways.

[9] For local, low volume residential streets, two free flowing lanes are not required and a 26 foot or 31 foot (back to back) roadway may be used where parking is allowed on one side or both sides respectively. For higher volume residential streets, which require two continuously free flowing traffic lanes, a 31 foot or 37 foot roadway should be used for one sided or two sided parking respectively.

[10] Some minimum roadway widths have been increased to match standard roadway widths. Unless approved by the Jurisdiction, all two lane roadways must comply with standard widths of 26, 31, 34, or 37 feet.

[11] Median width is measured between the edges of the traveled way of the inside lanes and includes the curb offset on each side of the median. Values include a left turn lane with a 6 foot raised median as required to accommodate a pedestrian access route (refer to Chapter 12) through the median (crosswalk cut through). At locations where a crosswalk does not cut through the median, the widths shown can be reduced by 2 feet to provide a 4 foot raised median.

[12] It is preferred to select a design speed that is at least 5 mph greater than the anticipated posted speed limit of the roadway. Selecting a design speed equal to the posted speed limit may also be acceptable and should be evaluated on a project by project basis, subject to approval of the Engineer.

[13] Values for low design speed (<50 mph) assume no removal of crown (i.e. negative 2% superelevation on outside of curve). Radii for design speeds of 50 mph or greater are based upon a superelevation rate of 4%. For radii corresponding to other superelevation rates, refer to the AASHTO's "Green Book."

[14] Assumes stopping sight distance with 6 inch object.

Table 5C-1.02: Acceptable Roadway Elements

Elements Related to Functional Classification

Design Element	Local		Collector		Arterial	
	Res.	***C/I***	***Res.***	***C/I***	***Res.***	***C/I***
General						
Design Level-of-Service[1]	D	D	D/E	D/E	D/E	D/E
Lane width (single lane) (ft)[2]	10	11	11	11	11	11
Two-Way Left-Turn Lanes (TWLTL) (ft)	N/A	N/A	12	12	12	12
Width of new bridges, (ft)[3]	See Footnote 3					
Width of bridges to remain in place (ft)[4]	20	22	24	24	26	26
Vertical clearance (ft)[5]	14.5	14.5	14.5	14.5	14.5	14.5
Object setback (ft)[6]	1.5	1.5	1.5	1.5	1.5	1.5
Clear zone (ft)	Refer to Tables 5C-1.03, 5C-1.04, and 5C-1.05					
Urban						
Curb offset (ft)[7]	1.5[8]	1.5[8]	1.5[8]	1.5[8]	2	2
Parking lane width (ft)	7.5	7.5	7.5	9	10	10
Roadway width with parking[9, 11]	26/31[10]	31	31	34[11]	34	34
Roadway width without parking[11]	26[10]	26	26	26	26	26
Raised median with left-turn lane (ft)[12]	N/A	N/A	18	18	18.5	18.5
Cul-de-sac radius (ft)	45	45	N/A	N/A	N/A	N/A
Rural Sections in Urban Areas						
Shoulder Width (ft)						
ADT: under 400	2	2	2	2	8	8
ADT: 400 to 1,500	5	5	5	5	8	8
ADT: 1,500 to 2,000	6	6	6	6	8	8
ADT: over 2,000	8	8	8	8	8	8
Foreslope (H:V)[13]	3:1	3:1	3:1	3:1	4:1	4:1
Backslope (H:V)	3:1	3:1	3:1	3:1	3:1	3:1

Res. = Residential, C/I = Commercial/Industrial

Elements Related to Design Speed

Design Element	Design Speed, mph[14]															
	25		***30***		***35***		***40***		***45***		***50***		***55***		***60***	
Stopping sight distance (ft)	155		200		250		305		360		425		495		570	
Passing sight distance (ft)	900		1,090		1,280		1,470		1,625		1,835		1,985		2,135	
Min. horizontal curve radius (ft)[15]	198		333		510		762		1,039		833		1,060		1,330	
Min. vertical curve length (ft)	50		75		105		120		135		150		165		180	
Min. rate of vert. curve, Crest (K)[16]	12		19		29		44		61		84		114		151	
Min. rate of vert. curve, Sag (K)	26		37		49		64		79		96		115		136	
Min. rate of vert. curve, Sag (K) based on driver comfort/overhead lighting[17]	14		20		27		35		44		54		66		78	
Minimum gradient (percent)[18]	0.5		0.5		0.5		0.5		0.5		0.5		0.5		0.5	
Maximum gradient (percent)[19]	R	C/I	R	C/I	R	C/I	R	C/I	R	C/I	R	C/I	R	C/I	R	C/I
Local	12	10	12	9	11	9	11	9	10	8	9	8	N/A	N/A	N/A	N/A
Collector	12	9	11	9	10	9	10	9	9	8	8	7	N/A	N/A	N/A	N/A
Arterial	N/A	N/A	9	9	8	8	8	8	N/A	7	N/A	7	N/A	6	N/A	6

R = Residential, C/I = Commercial/Industrial

Note: For federal-aid projects, proposed design values that do not meet the "Acceptable" table may require design exceptions. Design exceptions will be considered on a project-by-project basis and must have concurrence of the Iowa DOT when applicable. For non-federal aid projects, the designer should contact the Jurisdiction to determine what level of documentation, if any, is required prior to utilizing design values that do not meet the "Acceptable" table.

Table 5C-1.02 Footnotes:

[1] Number of traffic lanes, turn lanes, intersection configuration, etc. should be designed to provide the specified LOS at the design year ADT.

[2] Width shown is for through lanes and turn lanes.

[3] Bridge width is measured as the clear width between curbs or railings. Minimum bridge width is based upon the width of the traveled way (lane widths) plus 3 feet clearance on each side; but no less than the curb-face to curb-face width of the approaching roadway. Minimum bridge widths do not include medians, turn lanes, parking, or sidewalks. At least one sidewalk should be extended across the bridge.

[4] The values shown are the clear width across the bridge between curbs or railings. Values are based upon the width of the traveled way (lane width) and include a 1 foot and 2 foot offset on each side for collectors and arterials respectively. Values do not include medians, turn lanes, parking, or sidewalks. In no case should the minimum clear width across the bridge be less than the width of the traveled way of the approach road.

[5] Vertical clearance includes a 0.5 foot allowance for future resurfacing. Vertical clearance of 14.5 feet on arterials is allowed only if an alternate route with 16 feet of clearance is available.

[6] Object setback does not apply to mailboxes constructed and installed according to US Postal Service regulations, including breakaway supports.

[7] Values shown are measured from the edge of the traveled way to the back of curb. Curb offset is not required for turn lanes. On roadways with an anticipated posted speed of 45 mph or greater, mountable curbs are required. For pavements with gutterline jointing, the curb offset should be equal to or greater than the distance between the back of curb and longitudinal gutterline joint.

[8] At locations where a 1.5 foot curb offset is used, an alternative intake boxout, with the intake set back a minimum of 6 inches from the curb line, must be used to prevent intake grates from encroaching into the traveled way.

[9] Some jurisdictions allow parking on both sides of the street. When this occurs, each jurisdiction will set their own standards to allow for proper clearances, including passage of large emergency vehicles.

[10] For low volume residential streets, two free flowing lanes are not required and a 26 foot roadway may be used where parking is allowed on one side only. For higher volume residential streets, which require two continuously free flowing traffic lanes, a 31foot roadway should be used.

[11] Some minimum roadway widths have been increased to match standard roadway widths. Unless approved by Jurisdiction, all two lane roadways must comply with standard widths of 26, 31, 34, or 37 feet.

[12] Median width is measured between the edges of the traveled way of the inside lanes and includes the curb offset on each side of the median. Values include a left turn lane with a 6 foot raised median as required to accommodate a pedestrian access route (refer to Chapter 12) through the median (crosswalk cut through). At locations where a crosswalk does not cut through the median, the widths shown can be reduced by 2 feet to provide a 4 foot raised median.

[13] The use of 3:1 foreslopes is allowed, as shown, but may require a wider clear zone as slopes steeper than 4:1 are not considered recoverable by errant vehicles.

[14] It is preferred to select a design speed that is at least 5 mph greater than the anticipated posted speed limit of the roadway. Selecting a design speed equal to the posted speed limit may also be acceptable and should be evaluated on a project by project basis, subject to approval of the Engineer

[15] Values for low design speed (<50 mph) assume no removal of crown (i.e. negative 2% superelevation on outside of curve). Radii for design speeds of 50 mph or greater are based upon a superelevation rate of 6%. For radii corresponding to other superelevation rates, refer to the AASHTO's "Green Book."

[16] Assumes stopping sight distance with 2 foot high object.

[17] Use only if roadway has continuous overhead lighting.

[18] A typical minimum grade is 0.5%, but a grade of 0.4% may be used in isolated areas where the pavement is accurately crowned and supported on firm subgrade.

[19] Maximum gradient may be steepened by 2% for short distances and for one way downgrades.

Table 5C-1.03: Preferred Clear Zone Distances for Rural and Urban Roadways

Design Speed mph	Design Traffic ADT	Foreslope			Backslope or Parking		
		6:1 or flatter	*5:1 to 4:1*	*3:1*	*6:1 or flatter*	*5:1 to 4:1*	*3:1*
		In feet from edge of traveled way					
Urban 40 or less	All	For low-speed urban roadways, refer to Table 5C-1.05.					
Rural 40 or less	Under 750	10	10	*	10	10	10
	750 to 1,500	12	14	*	12	12	12
	1,500 to 6,000	14	16	*	14	14	14
	Over 6,000	16	18	*	16	16	16
Rural and Urban 45 to 50	Under 750	12	14	*	12	10	10
	750 to 1,500	16	20	*	16	14	12
	1,500 to 6,000	18	26	*	18	16	14
	Over 6,000	22	28	*	22	20	16
Rural and Urban 55	Under 750	14	18	*	12	12	10
	750 to 1,500	18	24	*	18	16	12
	1,500 to 6,000	22	30	*	22	18	16
	Over 6,000	24	32	*	24	22	18
Rural and Urban 60	Under 750	18	24	*	16	14	12
	750 to 1,500	24	32	*	22	18	14
	1,500 to 6,000	30	40	*	26	22	18
	Over 6,000	32	44	*	28	26	22

Source: Adapted from the *Roadside Design Guide*, 2006

Table 5C-1.04: Acceptable Clear Zone Distances for Rural and Urban Roadways

Design Speed mph	Design Traffic ADT	Foreslope			Backslope or Parking		
		6:1 or flatter	*5:1 to 4:1*	*3:1*	*6:1 or flatter*	*5:1 to 4:1*	*3:1*
		In feet from edge of traveled way					
Urban 40 or less	All	For low-speed urban roadways, refer to Table 5C-1.05.					
Rural 40 or less	Under 750	7	7	*	7	7	7
	750 to 1,500	10	12	*	10	10	10
	1,500 to 6,000	12	14	*	12	12	12
	Over 6,000	14	16	*	14	14	14
Rural and Urban 45 to 50	Under 750	10	12	*	10	8	8
	750 to 1,500	14	16	*	14	12	10
	1,500 to 6,000	16	20	*	16	14	12
	Over 6,000	20	24	*	20	18	14
Rural and Urban 55	Under 750	12	14	*	10	10	8
	750 to 1,500	16	20	*	16	14	10
	1,500 to 6,000	20	24	*	20	16	14
	Over 6,000	22	26	*	22	20	16
Rural and Urban 60	Under 750	16	20	*	14	12	10
	750 to 1,500	20	26	*	20	16	12
	1,500 to 6,000	26	32	*	24	18	14
	Over 6,000	30	36	*	26	24	20

Source: Adapted from the *Roadside Design Guide*, 2006

* Foreslopes steeper than 4:1 are considered traversable, but not recoverable. An errant vehicle can safely travel across a 3:1 slope, but it is unlikely the driver would recover control of the vehicle before reaching the bottom of the slope; therefore, fixed objects should not be present on these slopes or at the toe of these slopes.

Table 5C-1.05: Clear Zone for Low-speed (40 mph or less Design Speed) Urban Roadways

Roadway Classification	Distance from the Edge of the Traveled Way, feet[1]	
	Preferred	***Acceptable***
Arterial	10	7
Collector	8	5.5
Local	8	5.5

[1] Values in the table are measured from the edge of the traveled way. Parking lane, bike lane, and curb offset widths may be included as part of the clear zone; however, a minimum clear zone behind the back of curb of 6 feet (preferred) or 4 feet (acceptable) should be provided regardless of roadway classification. Clear zone requirements also apply along medians of divided roadways.

Source: Maze et al, 2008

D. References

American Association of State Highway and Transportation Officials (AASHTO) *Roadside Design Guide*. 3rd ed. Washington, DC. 2006.

Maze T. Hawkins N. et al. Clear Zone - A Synthesis of Practice and an Evaluation of the Benefits of Meeting the 10ft Clear Zone Goal on Urban Streets. Center For Transportation Research and Education. Iowa State University. 2008.

Geometric Design Elements

A. Level of Service

Level of service (LOS) is a measure of the operating conditions of a roadway facility. LOS is based upon traffic performance related to speed, travel time, freedom to maneuver, traffic interruptions, and comfort and convenience. The LOS ranges from A (least congested) to F (most congested). Refer to the Highway Capacity Manual for a more thorough discussion of the LOS concept.

Based upon the traffic capacity analysis, the number of lanes, turn lanes, and intersection controls should be selected to provide a design with the desired LOS for the design year traffic. Design year traffic is based upon a 20 year traffic projection. The current Highway Capacity Manual and the current AASHTO "Green Book" should be used for traffic projections and to determine the number of lanes and intersection configuration at the desired LOS.

The LOS for the roadway overall is based upon Average Daily Traffic (ADT), while the LOS at signalized intersections is based upon the peak hourly volume (PHV).

As a planning tool, the following tables are provided to indicate approximate capacities for two lane and four lane streets and highways and intersection capacity for four way stop and signalized intersections. These tables do not consider site specific details and should not be utilized for final design purposes.

Table 5C-2.01: Maximum ADT vs. LOS and Type of Terrain for Two Lane Highways

Terrain	**LOS**		
	B	***C***	***D***
Level	3,200 - 4,800	5,300 - 7,900	9,000 - 13,500
Rolling	1,800 - 2,800	3,500 - 5,200	5,300 - 8,000
Hilly	900 - 1,300	1,600 - 2,400	2,500 - 3,700

Table 5C-2.02: Reduced Capacity of Narrow Lanes with Restricted Lateral Clearance

Usable Shoulder Width or Clearance to Obstruction (feet)	**Two Lane Roadway** (percent of capacity of 12 feet lane)		
	12 feet lanes	***11 feet lanes***	***10 feet lanes***
6	100	93	84
4	92	85	77
2	81	75	68
0	70	65	58

Table 5C-2.03: Planning Capacity at LOS C[1], D, and E[2]
Two Way Arterial Streets (Non-intersection)

Number of Lanes	Turn Lanes	Capacity, VPD at LOS D			
		Minimal Side Friction	***Light (Residential) Side Friction***	***Moderate (Mixed Zoning) Side Friction***	***Heavy Side Friction***
Two Lanes Undivided	Without turn lanes	12,100	11,600	11,200	10,400
	With turn lanes	16,000	15,300	14,000	13,900
Four Lanes Undivided	Without turn lanes	24,300	23,400	23,400	21,900
	With left turn lanes or 5 lane with center TWLTL	32,100	30,900	30,900	29,100
Four Lanes Divided	Without turn lanes	27,100	26,200	26,100	23,300
	With left turn lanes	35,400	34,200	34,100	32,500
	With left and right turn lanes	37,500	36,200	34,400	34,400

LOS - Level of Service
TWLTL - Two-Way Left-Turn Lane
VPD - Vehicles per Day
[1] Capacity at LOS C may be determined by multiplying LOS D values above by 0.8.
[2] Capacity at LOS E may be determined by multiplying LOS D values above by 1.2.

Source: Adapted from "2000 Des Moines Area Daily Directional Capacities At Level of Service D" - Des Moines Area MPO

Table 5C-2.04: Approximate LOS C Service Volumes (VPH) for
Four Way Stop-controlled Intersections (Sum of all Four Legs)

Demand Split	Two Lanes on Each Street	Street 1: Two Lanes Street 2: Four Lanes	Four Lanes on Each Street
50/50	1,200	1,800	2,200
55/45	1,140	1,720	2,070
60/40	1,080	1,660	1,970
65/35	1,010	1,630	1,880
70/30	960	1,610	1,820

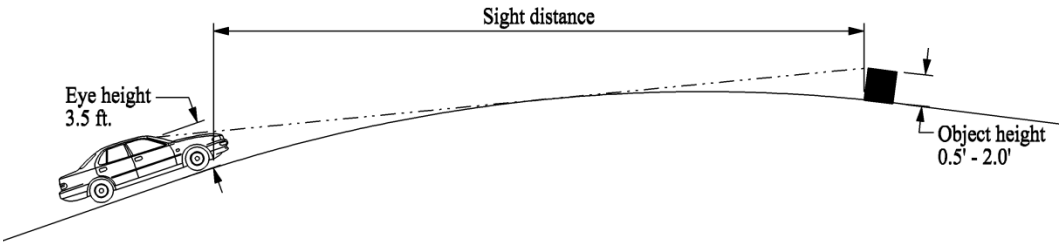
B. Sight Distance

The following information is taken from the 2004 AASHTO "Green Book." The Project Engineer should check the current edition of the AASHTO "Green Book" when specific information is needed to verify values provided.

1. **Stopping Sight Distances:** The minimum stopping sight distance is the distance required by the driver of a vehicle traveling at the design speed to bring the vehicle to a stop after an object on the road becomes visible. This distance directly affects the length and rate of curvature for vertical curves.

   The method for measuring stopping sight distance on vertical curves assumes a height for the driver's eye and a height for an object in the road. For a crest vertical curve, the sight distance is the distance at which an object in the road appears to the driver over the crest of the curve.

Figure 5C-2.01: Vertical Sight Distance Determination



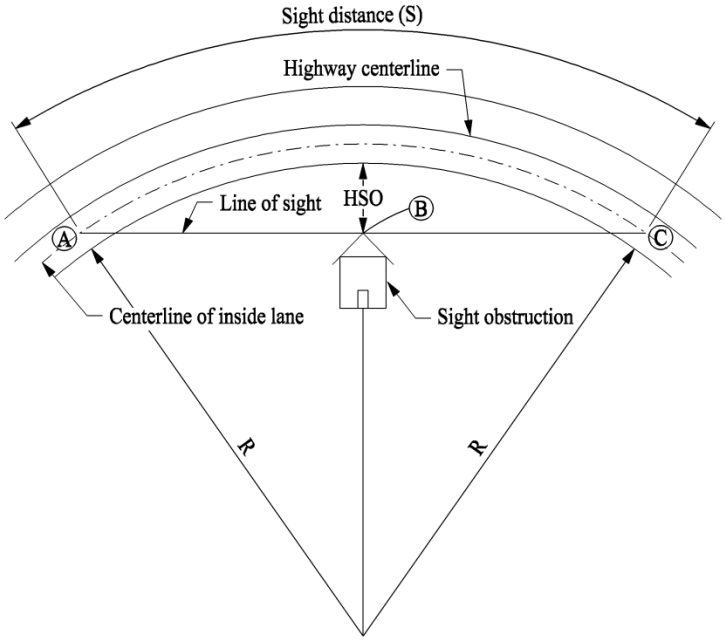
Stopping sight distance is calculated based upon an assumed height of the driver's eye and an assumed height of an object in the roadway. For all sight distance criteria, the height of the driver's eye is assumed to be 3.5 feet above the surface of the road, as recommended by AASHTO. Tables 5C-1.01 and 5C-1.02 in Section 5C-1 assume two different values for the height of the object in the roadway. The "Acceptable" values in Table 5C-1.02 use a 2 foot object height according to the current edition of the AASHTO "Green Book." The "Preferred" values in Table 5C-1.01 assume an object height of only 6 inches. This lower object height was the design value used in previous versions of the AASHTO "Green Book." The results of assuming a smaller object height for the preferred values in Table 5C-1.01 are higher required K values and longer vertical curves.

2. **Sight Distance on Horizontal Curves:** The horizontal alignment must provide at least the minimum stopping distance for the design speed at all points. This includes visibility around curves and roadside encroachments.

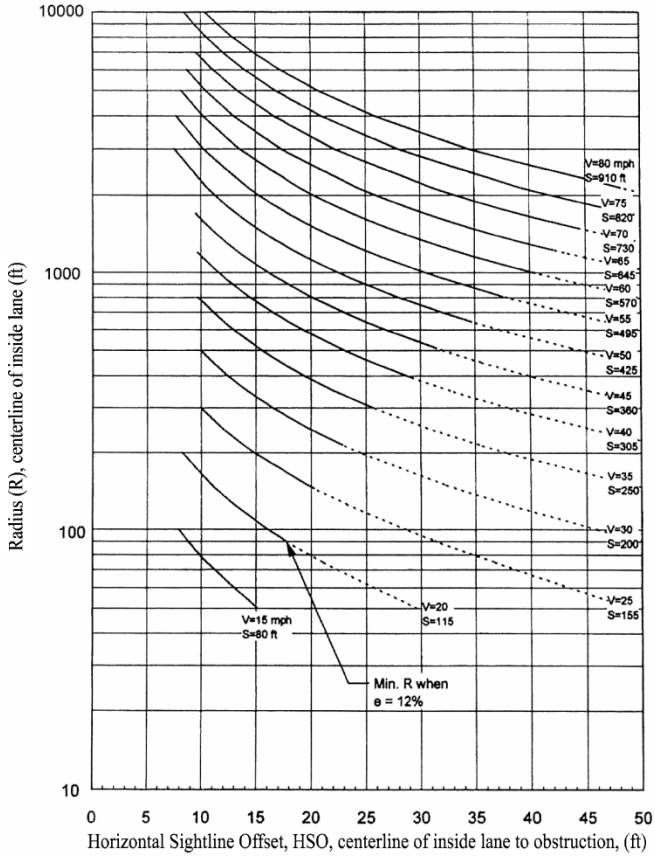
   Where there are sight obstructions such as walls, cut slopes, buildings, fences, bridge structures, or other longitudinal barriers on the inside of curves, an adjustment in the minimum radius of the curve may be necessary. In no case should sight distance be less than the stopping sight distance specified in Tables 5C-1.01 and 5C-1.02 in Section 5C-1. The sight distance design procedure should assume a 6 foot fence (as measured from finished grade) exists along all property lines except in the sight distance triangles required at all intersections.

   Available sight distance around a horizontal curve can be determined graphically using the method shown in Figures 5C-2.02 and 5C-2.03 below. From the center of the inside lane (Point A), a line is projected through the point on the obstruction that is nearest to the curve (Point B). The line is then extended until it intersects the centerline of the inside lane (Point C).

Figure 5C-2.02 and Figure 5C-2.03: Sight Distances for Horizontal Curves



HSO = Horizontal Sightline Offset



Source: Adapted from AASHTO "Green Book," 2004 Edition, Exhibits 3-53 and 3-54

3. **Passing Sight Distance:** Passing sight distance is the minimum sight distance that must be available to enable the driver of one vehicle to pass another safely and comfortably without interfering with oncoming traffic traveling at the design speed. Two lane roads should provide adequate passing zones at regular intervals. Minimum passing sight distances are shown in Tables 5C-1.01 and 5C-1.02 in Section 5C-1.

   Passing sight distance is measured between an eye height of 3.5 feet and an object height of 3.5 feet. On straight sections of roadway, passing sight distance is determined primarily by the vertical curvature of the roadway. On horizontal curves, obstructions adjacent to the roadway on the inside of the curve can limit sight distance. This is most common in a cut section where the adjacent terrain projects above the surface of the roadway. Passing sight distance should be verified using the methods described in the current edition of the AASHTO "Green Book."

4. **Intersection Sight Distance:** In addition to the stopping sight distance provided continuously in the direction of travel on all roadways, adequate sight distance at intersections must be provided to allow drivers to perceive the presence of potentially conflicting vehicles. Sight distance is also required at intersections to allow drivers of stopped vehicles to decide when to enter or cross the intersecting roadway. If the available sight distance for an entering or crossing vehicle is at least equal to the appropriate stopping sight distance for the major road, then drivers have sufficient sight distance to anticipate and avoid collisions. However, in some cases, this may require a major road vehicle to slow or stop to accommodate the maneuver by a minor road vehicle. To enhance traffic operations, intersection sight distances that exceed stopping sight distances are desirable along the major road.

   Each intersection has the potential for several different types of vehicular conflicts. The possibility of these conflicts actually occurring can be greatly reduced by providing proper sight distance and appropriate traffic controls. Each quadrant of an intersection should contain a triangular area free of obstructions that might block an approaching driver's view of potentially conflicting vehicles. This clear area is known as the sight triangle.

   a. **Sight Triangles:** Proper sight distance at intersections is determined through the establishment and enforcement of sight triangles. The required dimensions of the legs of the triangle depend on the design speed of the roadways and the type of traffic control provided at the intersection. Two types of clear sight triangles are considered in intersection design: approach sight triangles and departure sight triangles.
      1) **Approach Sight Triangles:** Approach sight triangles allow the drivers at uncontrolled or yield controlled intersections to see a potentially conflicting vehicle in sufficient time to slow or stop before colliding within the intersection. Although desirable at all intersections, approach sight triangles are not needed for intersections approaches controlled by stop signs or traffic signals.
      2) **Departure Sight Triangles:** A second type of clear sight triangle provides sight distance sufficient for a stopped driver on a minor-road approach to depart from the intersection and enter or cross the major road. Departure sight triangles should be provided in each quadrant of each intersection approach controlled by a stop sign.

      At signalized intersections, the first vehicle stopped on one approach should be visible to the driver of the first vehicle stopped on each of the other approaches. Left turning vehicles should have sufficient sight distance to select gaps in oncoming traffic.

      The recommended dimensions of the sight triangles vary with the type of traffic control used at an intersection because different types of controls impose different legal constraints on drivers and, therefore, result in different driver behavior. The AASHTO "Green Book"

contains the required procedures, equations, and tables for determining the required sight distance under various intersection and traffic control configurations.

b. Identification of Sight Obstructions within Sight Triangles: Within a sight triangle, any object at a height above the elevation of the adjacent roadways that would obstruct the driver's view should be removed or lowered if practical. Such objects may include buildings, parked vehicles, highway structures, roadside hardware, hedges, trees, bushes, unmowed grass, tall crops, walls, fences, and the terrain itself. Particular attention should be given to the evaluation of clear sight triangles at intersection ramp/crossroad intersections where features such as bridge railings, piers, and abutments are potential sight obstructions.

The determination of whether an object constitutes a sight obstruction should consider both the horizontal and vertical alignment of both intersecting roadways, as well as the height and position of the object. In making this determination, it should be assumed that the driver's eye is 3.5 feet above the roadway surface and that the approaching vehicle to be seen is 3.5 feet above the surface of the intersecting road.

C. Horizontal Alignment

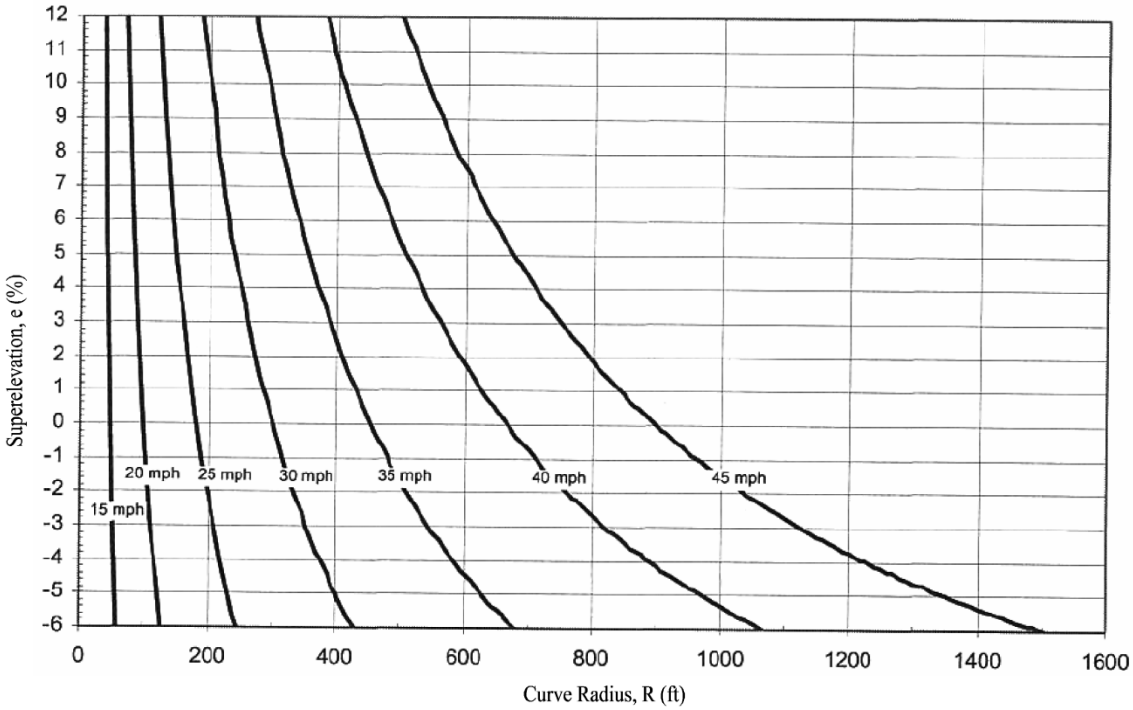
1. Roadway Curvature and Superelevation: On urban streets where operating speed is relatively low and variable, the use of superelevation for horizontal curves can be minimized. Although superelevation is advantageous for traffic operation, in urban areas the combination of wide pavements, the need to meet the grade of adjacent properties, the desire to maintain low speed operation, the need to maintain pavement profiles for drainage, and the frequency of cross streets and driveways and other urban features often combine to make the use of superelevation impractical or undesirable. Generally, the absence of superelevation on low speed urban streets is not detrimental to the motorist and superelevation is not typically provided on urban streets with a design speed of 45 mph or less.

The preferred radii shown in Section 5C-1, Table 5C-1.01 assume that a normal crown is maintained around a horizontal curve. With a standard 2% pavement cross-slope, this effectively results in a negative 2% superelevation for the outside lane. For roadways with a cross-slope other than 2%, including four lane and wider sections that utilize a steeper cross-slope for the outside lanes, the required curve radius should be determined from the guidance provided in the current AASHTO "Green Book" or from Figure 5C-2.04 below.

While superelevation on low speed urban roadways is not desirable, it may be necessary in situations where site conditions require a horizontal curve that cannot sustain traffic with the negative superelevation that results from maintaining the normal crown. For these situations, superelevation equal to the normal cross-slope may be provided for the outside lane. Section 5C-1, Table 5C-1.02 assumes the adverse crown in the outside lane of a curve is removed. For a roadway with a normal 2% cross-slope, this results in a superelevation of 2% across the width of the pavement. For roadways with cross-slopes other than 2%, the required radius and the resulting superelevation should be determined from the guidance provided in current AASHTO "Green Book" or from Figure 5C-2.04 below. The maximum superelevation for low speed urban roadways should not exceed the normal cross-slope or a maximum of 3%.

For roadways with design speeds of 50 mph or greater, superelevation of the roadway is acceptable and expected by motorists. The radii provided in Section 5C-1, Tables 5C-1.01 and 5C-1.02 are based upon superelevation rates of 4% and 6% respectively. The maximum superelevation rate in urban areas should not exceed 6%.

Figure 5C-2.04: Superelevation, Radius, and Design Speed for Low Speed (<50mph) Urban Street Design



Source: AASHTO "Green Book," 2004 Edition, Exhibit 3-17

2. **Intersection Alignment:** The centerline of a street approaching another street from the opposite side should not be offset. If the offset cannot be avoided, the offset should be 150 feet or greater for local streets. The centerline of a local street approaching an arterial or collector street from opposite side should not be offset unless such offset is 300 feet or greater.

3. **Adding, Dropping, or Redirecting Lanes:**

   a. **Dropping or Redirecting Through Lanes:** When dropping a lane, the minimum taper ratio to be used should be determined by the following formula, or from Table 5C-2.05:

      L = WS for velocities of 45 mph or more

      $L = \frac{WS^2}{60}$ for velocities of 40 mph or less.

      L = Minimum length of taper.
      S = Numerical value of posted speed limit or 85th percentile speed, whichever is higher.
      W = Width of pavement to be dropped or redirection offset.

      Preferably, taper ratios should be evenly divisible by five. Calculations that result in odd ratios should be rounded to an even increment of five. The table below utilizes the formulas to determine the appropriate taper ratio for dropping a 12 foot wide lane. The ratio remains constant for a given design speed while the length varies with the pavement width.

The procedure for determining minimum taper ratios for redirecting through lanes is the same as for lane drops, except for design speeds over 45 mph the use of reverse curves rather than tapers is recommended.

Table 5C-2.05: Length and Taper Ratio for Dropping 12 Foot Lane

Design Speed (mph)	25	30	35	40	45	50	55	60
Taper Ratio	10:1	15:1	20:1	25:1	45:1	50:1	55:1	60:1
Length (feet)	120	180	240	300	540	600	660	720

b. Adding Through or Turn Lanes: For design speeds of 45 mph or greater, a 15:1 lane taper should be used when adding a left or right turn lane. For design speeds less than 45 mph, a 10:1 taper may be used.

For design speeds less than 45 mph, shorter tapers that are squared off or taper at 1:1 may provide better "targets" for approaching drivers and give more positive identification to an added through lane or turn lane. For turn lanes, the total length of taper and deceleration length should be the same as if a standard taper was used. This results in a longer length of full width pavement for the turn lane. This design provides increased storage that may reduce the likelihood turning vehicles will back up into the through lane during peak traffic periods. The use of short taper sections must be approved by the Engineer.

Figure 5C-2.05: Adding or Dropping Lanes

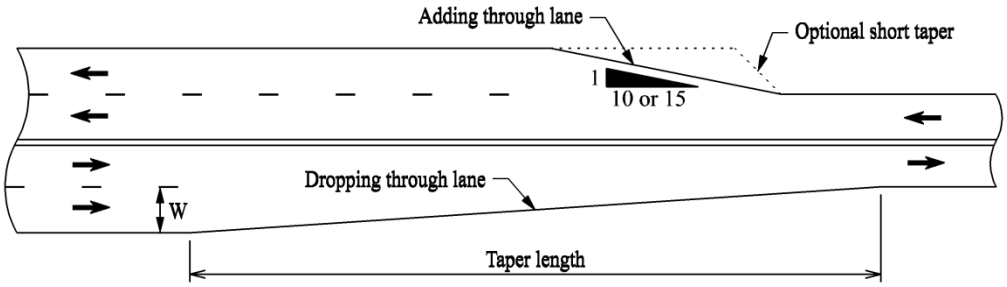
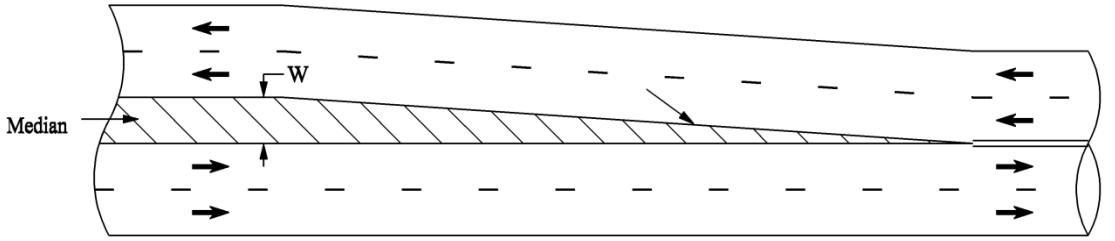


Figure 5C-2.06: Redirecting Through Lanes



D. Vertical Alignment

1. **Minimum Grades:** Flat and level grades on uncurbed pavements are preferred when the pavement is adequately crowned to drain the surface laterally. However, with curbed pavements, longitudinal grades must be provided to facilitate surface drainage. A typical minimum grade is 0.5%, but a grade of 0.4% may be used in isolated areas where the pavement is accurately crowned and supported on firm subgrade. The minimum allowance grade for bubbles and cul-de-sacs is 1%. Particular attention should be given to the design of stormwater inlets and their spacing to keep the spread of water on the traveled way within tolerable limits. Roadside channels and median swales frequently require grades steeper than the roadway profile for adequate drainage.

2. **Maximum Grades:** Grades for urban streets should be as level as practical, consistent with the surrounding terrain. The maximum design grades specified in Section 5C-1, Table 5C-1.02 should be used infrequently; in most cases grades should be less than the maximum design grade.

   Where sidewalks are located adjacent to a roadway, a maximum roadway grade of 5% is desirable. ADA requirements allow sidewalks adjacent to a roadway to match the running grade of the roadway, regardless of the resulting grade. However, sidewalk accessibility is greatly enhanced, especially over long distances, when grades are limited to 5% or less. It is recognized that meeting limitations will not be possible or practical in many situations; however, an attempt should be made to limit roadway grades to this level, especially in areas with high levels of anticipated pedestrian usage.

3. **Maximum Grade Changes:** Except at intersections, the use of grade breaks, in lieu of vertical curves, is not encouraged. However, if a grade break is necessary and the algebraic difference in grade does not exceed 1%, the grade break will be considered by the Engineer.

4. **Vertical Curves:** Vertical curves should be simple in application and should result in a design that is safe, comfortable in operation, pleasing in appearance, and adequate for drainage.

   The major control for safe operation on crest vertical curves is the provision of ample sight distances for the design speed. Minimum stopping sight distance should be provided in all cases. Wherever economically and physically feasible, more liberal stopping sight distances should be used. Furthermore additional sight distance should be provided at decision points.

   a. **Crest Vertical Curves:** Minimum lengths of crest vertical curves as determined by sight distance requirements are generally satisfactory from the standpoint of safety, comfort, and appearance. Figure 5C-2.06 shows the required length of crest vertical curve to provide stopping sight distance based upon design speed and change in grade.

   b. **Sag Vertical Curves:** Headlight sight distance is generally used as the criteria for determining the length of sag vertical curves. When a vehicle approaches a sag vertical curve at night, the portion of highway lighted ahead is dependent on the position of the headlights and the direction of the light beam. A headlight height of 2 feet and a 1 degree upward divergence of the light beam from the longitudinal axis of the vehicle is commonly assumed. For safety purposes, the sag vertical curve should be long enough that the light beam distance is the same as the stopping sight distance. Figure 5C-2.07 specifies the required sag curve length to meet the sight distance assumptions made above.

For both sag and crest vertical curves with a low algebraic difference in grade, sight distance restrictions may not control the design of the curve. In these cases, rider comfort and curve appearance are the primary considerations for vertical curve design. Generally, vertical curves with a minimum length (in feet) equal to three times the design speed (in mph) are acceptable.

If a roadway has continuous lighting, the length of sag vertical curve (L) may be based on passenger comfort instead of headlight sight distance. Use the following equation for the curve length:

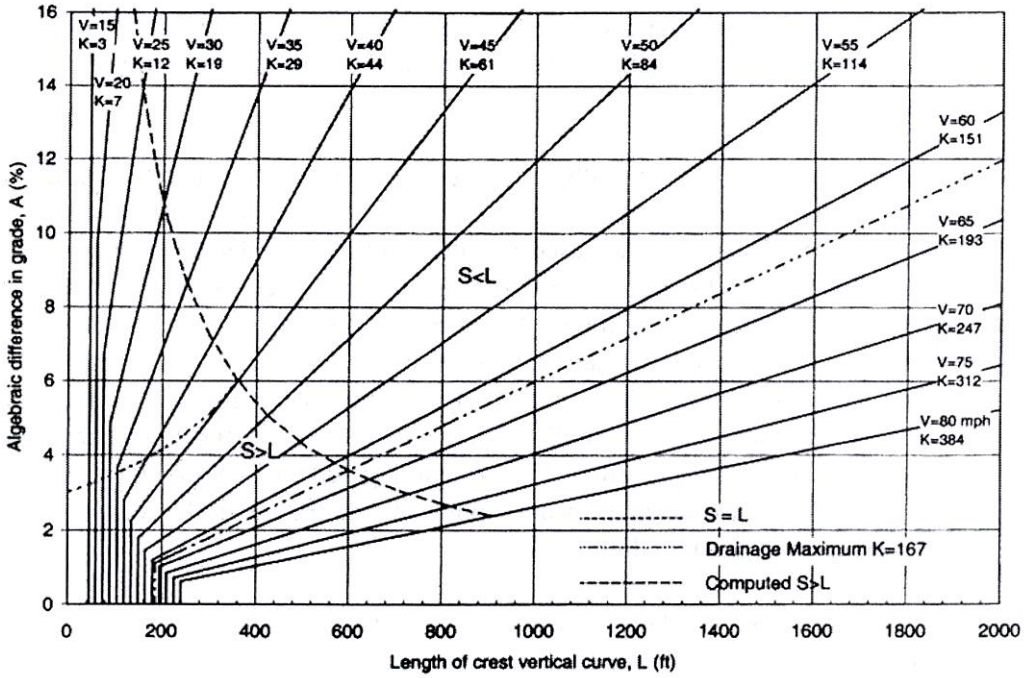
$$L = \frac{AV^2}{46.5}$$ where A = algebraic difference in grades, %
V = design speed, mph

(Equation 3-51 AASHTO Greenbook, 2011)

Drainage considerations also affect the design of vertical curves where curbs are utilized. Both crest and sag vertical curves that have a grade change from positive to negative (or vice versa) contain a level area at some point along the curve. Generally, as long as a grade of 0.30% is provided within 50 feet of the level area, no drainage problems develop. This criterion corresponds to a K value of 167 and is indicated by a dashed line in Figures 5C-2.06 and 5C-2.07 below. K values greater than 167 may be utilized, but additional consideration should be given to drainage in these situations.

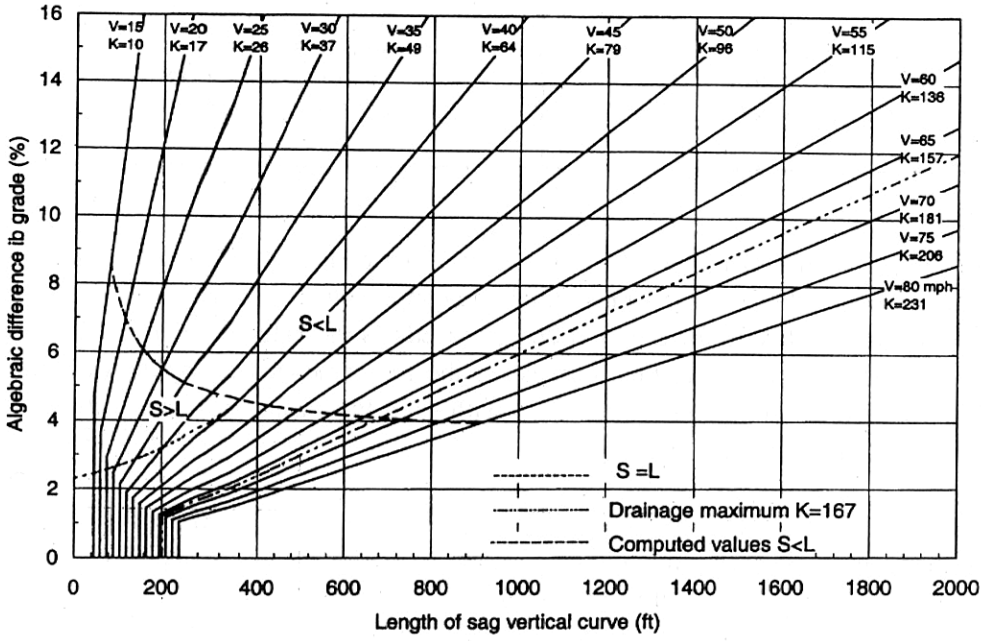
$$K = \frac{\mathrm{L(ft)}}{(g_2 - g_1)}$$ where g_1 and g_2 are in percent

Figure 5C-2.06: Design Controls for Crest Vertical Curves
for Stopping Sight Distance and Open Road Conditions



Source: "Green Book," Exhibit 3-71, 2004

Figure 5C-2.07: Design Controls for Sag Vertical Curves, Open Road Conditions



Source: "Green Book," Exhibit 3-78, 2004

5. **Intersection Grades:** The grade of the "through" street should take precedence at intersections. At intersections of roadways with the same classifications, the more important roadway should have this precedence. Side streets are to be warped to match through streets with as short a transition as possible, which provides a smooth ride. Consideration must be given to minimize sheet flow of stormwater across the intersection due to loss of crown on the side street.

   Carrying the crown of the side street into the through street is not allowed. In most cases the pavement cross-slope at the warped intersection should not exceed the grade of the through street.

   The maximum desirable grades of the through street at the intersection and the side street cross-slope should be 2% and should not exceed 3%. The maximum desirable approach grade of the side street should not exceed 4% for a distance of 100 feet from the curb of the through street.

   Establishing intersection spot grades by matching "curb corners" of intersecting streets is not recommended since it may result in an undesirable travel path from the through street to the side street because of the resulting bump on the side street centerline. At sidewalk curb ramps in intersections, the street grades may need to be warped at the curb line to ensure the resulting cross-slope at the bottom of the ramp does not exceed 2%. A detail of the jointing layout with staking elevations should be shown on the plans.

   ADA regulations set specific limits for crosswalk cross-slopes that directly impact street and intersection grades. ADA regulations limit the cross-slope to 2% (measured perpendicular to the direction of pedestrian travel) for crosswalks that cross a roadway with stop control (stop sign) at the intersection. For roadways without stop control (through movement or traffic signal) the cross-slope of the crosswalk is limited to 5%. Effectively, this requirement limits street grades to a maximum of 2% or 5% depending on intersection controls.

   For steep roadways without stop control, construction of a flattened "table" may be necessary to reduce the street grade to 5% or less at the location of the crosswalk. Crosswalk tables at these

locations must utilize vertical curves, appropriate for the design speed, to avoid a sudden change in grade at the intersection that could cause vehicles to bottom out or lose control.

For steep roadways with stop control, construction of a flattened "table" may utilize grade breaks or shortened vertical curves to reduce the street grade to 2% or less at the location of the crosswalk. A check should be made to verify that vehicles will not bottom out when traveling over the crosswalk table.

E. Pavement Crowns

The following typical pavement crowns are straight line cross-slope and are desirable sections.

1. **Urban Roadways (Curb and Gutter):** For streets with three or fewer travel lanes, the pavement crown should be 2%.

   For streets with four or more travel lanes, the pavement crown for all inside lanes, including left turn lanes, should be 2%. In order to reduce stormwater spread, the pavement crown for the outside lanes should be 3%.

   For all streets, auxiliary right turn lanes will have varying pavement crowns depending on the desired drainage pathway.

2. **Rural Roadways:** For pavement crowns, a 2% cross-slope is normal with 4% shoulder slope. Iowa DOT Standard Road Plans should be checked for Federal Aid, Farm to Market, and Secondary Roads.

F. Lane Width

The lane width of a roadway greatly influences the safety and comfort of driving. Narrow lanes force drivers to operate their vehicles closer to each other laterally than they would normally desire, resulting in driver discomfort, lower operating speeds, and reduced roadway capacity.

Tables 5C-1.01 and 5C-1.02 in Section 5C-1 indicate minimum lane widths based upon the roadway classification and adjacent land use. In addition to the lane width, a separate offset distance to the curb is required. This curb offset is not included in the lane widths listed.

Auxiliary lanes and turn lanes at intersections should be as wide as the adjacent through lanes. The width for turn lanes is measured to the face of curb. Because motorists are slowing in anticipation of making a turning movement, drivers are comfortable operating their vehicle closer to an adjacent obstacle (curb); therefore, turn lanes do not require a curb offset.

G. Two-way Left-turn Lanes (TWLTL)

Two-way left-turn lanes work well where design speeds are relatively low (25 to 50 mph) and there are no heavy concentrations of left turning traffic. The width of TWLTLs should be limited to a maximum of 14 feet to discourage left-turning motorists from pulling out into the TWLTL and stopping perpendicular to the direction of traffic, while they wait for oncoming traffic to clear.

H. Raised Median Width

A median is defined as the portion of a roadway separating opposing directions of the traveled way. The median width is expressed as the dimension between the edges of the traveled way and includes the left turn lanes, if any are present (refer to Section 5C-1, Figure 5C-1.01). The principal functions of a median are to separate opposing traffic, allow space for speed changes and storage of left turning and U-turning vehicles, minimize headlight glare, and provide width for future lanes. For maximum efficiency, a median should be highly visible both night and day and contrast with the traveled way lanes.

At unsignalized intersections on rural divided highways, the median should generally be as wide as practical. However, in urban areas, narrower medians appear to operate better at unsignalized intersections. If right-of-way is restricted, a wide median may not be justified if provided at the expense of a narrowed border area. A reasonable border width is needed to adequately serve as a buffer between private development along the road and the traveled way. Narrowing the border area may create operational issues similar to those that the median is designed to avoid. In addition, wide medians at signalized intersections result in increased time for vehicles to cross the median. This can lead to inefficient signal operation. Therefore, in urban areas, it is recommended that median width be only as wide as necessary to accommodate left turn lanes. Wider medians should only be used where needed to accommodate turning and crossing maneuvers by larger vehicles.

Medians and boulevards are not normally used on collector streets. However, when allowed, the median or boulevard should conform to the same design standards as set forth for arterial streets.

Median widths are also affected by sidewalk and crosswalk locations. Where a crosswalk cut through is present or proposed, medians (exclusive of any turn lanes) must be a minimum of 6 feet wide to comply with ADA regulations. These regulations require the placement of a 2 foot wide strip of detectable warnings at the curb line on both sides of the median. The detectable warnings must be separated by a minimum 2 foot strip without detectable warnings. Where the median has no curb, the detectable warnings must be placed along the edge of the roadway. At locations where a raised median is stopped short of the crosswalk, the 6 foot raised median and associated detectable warnings are not required, and a standard 4 foot raised median section may be used.

I. Bridges

The bridge widths listed in Section 5C-1, Tables 5C-1.01 and 5C-1.02 represent the clear roadway width (width between barrier rail faces). The widths shown do not account for barrier rail widths, sidewalk, recreational trails, etc.

For existing bridges, a structural analysis should be conducted. The existing bridge should be able to accommodate legal loads. Bridge guardrail should be upgraded if necessary.

J. Clear Zone

The AASHTO Roadside Design Guide (RDG) defines the clear zone as "the total roadside border area, starting at the edge of the traveled way, available for safe use by errant vehicles. This area may consist of a shoulder, a recoverable slope, a non-recoverable slope, and/or a clear runout area. The desired width is dependent upon the traffic volumes and speeds and on the roadside geometry."

The intent of the clear zone is to provide an errant vehicle that leaves the roadway with an unobstructed recovery area. This area, including medians on divided roadways, should be kept free of all unyielding objects, including utility and light poles, culverts, bridge piers, sign supports, and

any other fixed objects that might severely damage an out of control vehicle. Any obstruction that cannot be placed outside of the clear zone should be shielded by traffic barriers or guardrails.

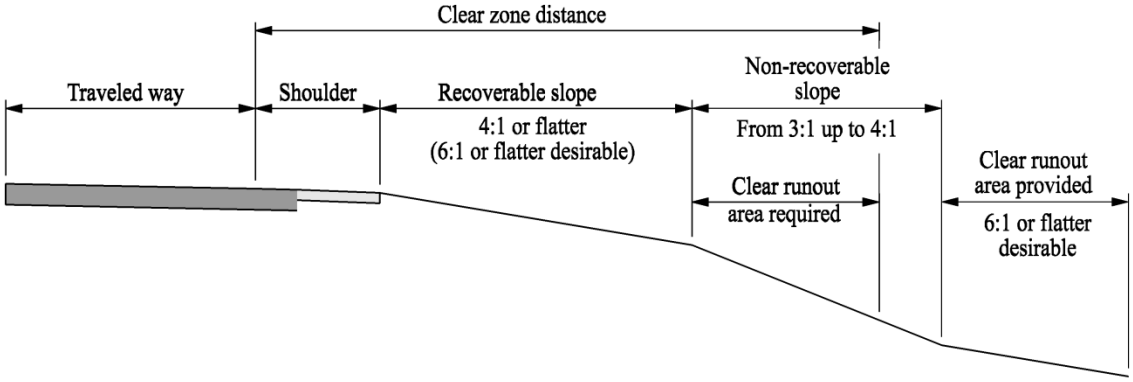
According to the AASHTO RDG, the width of this area varies based upon traffic volumes, design speed, and embankment slope.

Embankment slopes can be classified as recoverable, non-recoverable, or critical. Embankment slopes of 4:1 and flatter are considered recoverable. Drivers who encroach on recoverable slopes can generally stop their vehicles or slow them enough to return to the roadway safely.

A non-recoverable slope is defined as one that is passable, but from which most motorists will be unable to stop or to return to the roadway easily. Vehicles on such slopes are likely to reach the bottom before stopping. Embankments between 3:1 and 4:1 generally fall into this category. Since many vehicles will reach the toe of these slopes, the clear zone distance cannot logically end on a non-recoverable slope, and a clear runout area at the base of the slope is required. Fixed objects should not be present on a non-recoverable slope.

A critical slope is one on which a vehicle is likely to overturn. Slopes steeper than 3:1 generally fall into this category. If a slope steeper than 3:1 begins closer to the traveled way than the suggested clear zone, a barrier might be warranted if the slope cannot be flattened.

Figure 5C-2.08: Clear Zone Components



Source: Adapted from *Roadside Design Guide*, 2006

For horizontal curves, an adjustment factor may be applied to the clear zone width taken from Section 5C-1, Tables 5C-1.03 or 5C-1.04. This adjustment is only required at selected locations. Widening the clear zone should be considered along the outside of curves when crash history suggests the need for additional clear zone width, or whenever the radius of the curve is less than 2,860 feet, the design speed is 55 mph or greater, and the curve occurs on a normally tangent alignment (one where the curve is preceded by a tangent more than a mile in length).

The clear zone along an urban section may contain minor obstructions (traffic signs, mailboxes, etc.). In addition, along lower (<40 mph design speed) urban roadways, larger objects designed to "break-away" when struck by a vehicle may also be located within the clear zone (light poles, cast-iron fire hydrants, etc.). All objects, however, should be kept free from the object setback zone as described in the next section.

K. Object Setback

Like clear zone, object setback is intended to provide an area adjacent to the roadway that is clear of obstructions. However, the purpose of the object setback is to provide an operational clearance to increase driver comfort and avoid a negative impact on traffic flow. It also improves aesthetics, provides an area for snow storage and, in areas with curbside parking, provides a clear area to open car doors.

As discussed in the previous section on clear zone, minor obstructions and larger "breakaway" objects may be located in the clear zone on lower speed roadways (<40 mph design speed), but must be kept free from the object setback. Mailboxes constructed and installed according to US Postal Service regulations, including breakaway supports, may be located within the object setback area.

Additional object setback, as measured from the back of curb, may be required around radii at intersections and driveways in order to provide sufficient clearance to keep the overhang of a truck from striking an object.

L. Border Area

Border area is the area between the roadway and the right-of-way line and is sometimes referred to as the "parking" in urban areas. The grade for the border area is normally 1/2 inch per foot. The border area between the roadway and the right of way line should be wide enough to serve several purposes including provision of a buffer space between pedestrians and vehicular traffic, sidewalk space, and an area for both underground and above ground utilities such as storm sewer, traffic signals, parking meters, and fire hydrants. The border area also provides snow storage and aesthetic features such as grass or other landscaping features. The border width ranges from 14 to 16 feet, including the sidewalk width. Traffic signals, utility poles, fire hydrants, and other utilities should be placed as far back of the curb as practical for safety reasons. Breakaway features should be built when feasible and as an aid to safety considerations.

Table 5C-2.08: Preferred Border Area

Street Classification	Border Area Width (feet)
Major/minor arterial	16
Collector	14.5
Local streets	14

M. Curbs

1. **Curb Offset:** The curb offset is measured from the back of curb to the edge of the lane. The curb offset increases driver comfort and roadway safety. The presence of the curb, and potential vehicle damage and loss of control resulting from striking the curb, causes drivers to move away from the curb, reducing the effective width of the through lane. Due to this driver reaction, and to accommodate the flow of drainage and intake structures, an offset between the curb and the edge of the traveled way is provided.

   The curb offset widths specified in Section 5C-1, Tables 5C-1.01 and 5C-1.02 do not necessarily indicate the width of the curb and gutter or the location of a longitudinal joint; however, the width of the curb and gutter can affect the required width of the curb offset. The presence of a longitudinal joint near the curb (gutterline jointing) can be a limiting factor for usable lane width as some drivers are uncomfortable driving on or near the joint line. This is especially true for HMA roadways with PCC curb and gutter. For pavements with a longitudinal joint line near the

gutter, the curb offset should be equal to or greater than the width of the curb and gutter section. In addition, grates and special shaping for curb intakes and depressions for open-throat intakes should be located within the curb offset width and should not encroach into the lane.

2. **Curb and Gutter:** Typically, a curb and gutter cross-section should consists of a 6 inch high, 6 inch wide curb with a concrete gutter section. If the design speed is 40 mph or below, an 8 inch curb may be used for certain arterial and collector streets. For design speeds greater than 40 mph, a 1 foot wide, 6 inch high sloped curb with a minimum 2 foot gutter offset should be used.

N. Parking Lane

Where curbed sections are used, the curb offset width may be included as part of the parking lane.

1. Parking lanes are not allowed on arterial streets.

2. Although on-street parking may impede traffic flow, parallel parking may be allowed by the Jurisdiction on urban collectors where sufficient street width is available to provide parking lanes.

3. Parking lane width determinations should include consideration for the potential use of the lane as a through or turn lane for moving traffic either during peak hours or continuously. If this potential exists, additional parking width should be provided.

O. Cul-de-sacs

A local street open at one end only should have a cul-de-sac constructed at the closed-end. The minimum radius for cul-de-sacs is 45 feet, which may be increased in commercial areas or if significant truck traffic is anticipated. The border area around the cul-de-sac should be the same as the approach street. The transition radius with the approach street will be 50 feet for residential streets and 75 feet for commercial and industrial streets.

P. Shoulder Width

Shoulders accommodate stopped vehicles, emergency use, and provide lateral support of the subbase and pavement. In some cases, the shoulder can accommodate bicyclists. Where no curb and gutter is constructed a soil, granular, or paved shoulder will be provided.

Desirably, a vehicle stopped on the shoulder should clear the pavement edge by 2 feet. This preference has led to the adoption of 10 feet as the desirable shoulder width that should be provided along high volume facilities. In difficult terrain and on low volume highways, usable shoulders of this width may not be practical.

Where roadside barriers, walls, or other vertical elements are used, the graded shoulder should be wide enough that these vertical elements can be offset a minimum of 2 feet from the outer edge of the usable shoulder. It may be necessary to provide a graded shoulder wider than used elsewhere on the curved section of a roadway or to provide lateral support for guardrail posts and/or clear space for lateral dynamic deflection required by the particular barrier in use. On low volume roads, roadside barriers may be placed at the outer edge of the shoulder; however, a minimum of 4 feet should be provided from the traveled way to the barrier.

Q. Intersection Radii

Minimum curb return radii are shown in Table 5C-2.09 below. Where truck traffic is significant, curb return radii should be provided according to the current AASHTO "Green Book;" turning templates are used in this design. The Iowa DOT has an Iowa truck vehicle that can be used to check the proposed radii for truck routes.

Table 5C-2.09: Curb Return Radii Based Upon Roadway Classification

Roadway Classification	Arterial	Collector	Local - Commercial/ Industrial	Local - Residential
Arterial	Special*	Special*	30'	30'
Collector	Special*	30'	30'	25'
Local - Commercial/Industrial	30'	30'	25'	25'
Local - Residential	30'	30'	25'	25'

*Special design required. Use turning templates.

R. Pavement Thickness

Refer to Section 5F-1 for pavement thickness determination and design.

S. References

American Association of State Highway and Transportation Officials (AASHTO). *A Policy on Geometric Design of Highways and Streets* ("Green Book"). Washington, DC. 2004.

American Association of State Highway and Transportation Officials (AASHTO). *Roadside Design Guide*. 3rd ed. Washington, DC. 2006.

Des Moines Area Metropolitan Planning Organization (MPO). *Des Moines Area Daily Directional Capacities At Level of Service D*. Des Moines. 2000.

HMA Pavement Mixture Selection

A. Scope

This section is intended for the engineers and technicians who specify asphalt paving material criteria for urban projects, generally ranging from low to medium volume, up to 10M ESALs (3,300 trucks per day). Vehicle volumes exceeding 10M ESAL20, or projects outside of these design standards, may require more detailed design and/or expert consultation. The section provides a step-by-step process for determining the appropriate mixture criteria and gives the designer additional background information on specific mixture criteria. The section is intended to assist in selecting the mixture criteria that best satisfy the project demands and limitations. Statewide use of this section will improve the standard application of current accepted gyratory mix design technology.

B. Definitions

Equivalent Single Axle Load (ESAL): A standard unit of pavement damage created by a single pass of a vehicle axle.

Car axle = 0.0002 ESAL 18kip truck axle = 1.0 ESAL 24kip truck axle = 3.0 ESAL

$ESAL_{20}$: Estimated cumulative ESALs over a 20 year period.

N: The number of gyratory compaction revolutions at which HMA mixture properties are measured.
N_{ini} represents initial field compaction.
N_{des} represents 20 years of traffic loading.
N_{max} represents a factor of safety against excessive vehicle loads.

Gyratory Mix Design: A laboratory process for achieving desired pavement performance by determining the optimum proportions of aggregates and asphalt binder for hot mix asphalt using a SHRP Superpave gyratory compactor.

Nominal Maximum Aggregate Size (NMAS): The mixture size designation used for the combined aggregate gradation. Defined as one sieve size larger than the first sieve to retain more than 10%.

Lift Designation (Surface, Intermediate, Base): The terms for the lifts of mixture in the hot mix asphalt pavement structure. The surface lift is the top lift, generally 1 to 2 inches thick. The intermediate lift(s) is one or more lifts placed under the surface lift, generally 2 to 4 inches thick. The base lift(s) is all mixture placed below the intermediate lift, generally limited to full depth construction.

Performance Graded (PG): National asphalt binder grading system, developed by AASHTO, based on high and low pavement operating temperatures (°C).

"Rule of 90": The "rule of 90" adds the binder grade numbers. For example, PG 58 -28 would have a value of 86 (58+28). If the "rule of 90" value for a binder grade is above 90, it may be a modified binder with an additional cost.

C. Design Checklist

Designers should follow the steps below to ensure that the material criteria selected will best meet the needs of the project and the constraints of the owner agency.

1. **Determine the Level of Traffic Forecasted for the Next 20 Years:** Both current and future traffic levels are needed to determine the appropriate Hot Mix Asphalt (HMA) mixture for the project. Even if the project is not expected to remain in place for 20 years, the material selection levels are based on 20 year values. Common values are average daily traffic (ADT) for the current year, ADT for the 20 year forecast, and percent trucks. In addition to these annualized daily values, the designer should consider potential seasonal high truck volumes, and give particular attention to point sources and future development areas that may generate heavy truck volumes, like quarries, industrial parks, and bus lanes. Seasonal truck volumes may reflect a rate of pavement loading well in excess of the annualized values.

2. **Understand the Pavement Section Design or Rehabilitation Strategy:** In order to make the proper mixture selection, the designer must have knowledge of the proposed pavement construction or rehabilitation and intended pavement performance. The thickness of the pavement will also affect the material and mixture selection. For example, mixture selection for new construction or reconstruction may be different from mixture selection to correct a rutting problem. Particular parameters include required structural thickness, existing pavement cross section and condition (dominant distress patterns), traffic patterns and speed, and past maintenance.

3. **Determine the Regional Climate Conditions:** Iowa's 1 day low pavement temperature ranges approximately 5 °C from north to south. Adjusted for 98% reliability, the values range from -29 °C to -24 °C. The 7 day high pavement temperature across the state only varies by 3 °C. These values are computed from daily high air temperatures. Adjusted for 98% reliability, the pavement temperature values range from 56 °C to 59 °C. Climate details for a specific location can be obtained from the LTPPBind software package available on the FHWA website (http://www.fhwa.dot.gov/research/tfhrc/programs/infrastructure/pavements/ltpp/dwnload.cfm). See Figures 5D-1.01 and 5D-1.02.

4. **Compute the Anticipated 20 Year Pavement Loading:** The design pavement loading is the starting point for selecting the material and mixture selection criteria. The design pavement loading is measured in Equivalent Single Axle Loads (ESAL), not ADT. To determine the design ESALs on the project, use the traffic conditions from Step 1 and compute the $ESAL_{20}$. Use the examples outlined in Examples 5D-1.01 and 5D-1.02, for two lane, two way traffic; use Example 5D-1.03 for urban multi-lane situations. Design ESAL levels for HMA criteria selection are divided into relatively large brackets. The high value of each bracket is three times the minimum value. While a firm understanding of the traffic and pavement loading is important, good approximations of truck traffic are normally sufficient to determine the design $ESAL_{20}$.

5. **Identify Any Special Conditions that Impact the Pavement:** The standard selection process is based on high speed traffic with a broad distribution of vehicle types. There are numerous special conditions that may, through engineering judgement, require changes in the standard pavement materials/mixture selection. These special conditions are outlined below.

   a. **Heavy Trucks:** If the pavement's history has regularly been impacted by heavy trucks, the designer may consider increasing either the binder high temperature grade, the mix designation (ESAL level), or both. Typical examples of this condition are routes adjacent to quarries, grain elevators, or regional commercial freight distribution centers.

   b. **Slow/Stop/Turning:** Urban roadways normally require slower running speeds and often include signed or signaled intersections. The pavement loading condition significantly increases at slower speeds (less than 45 MPH) and stopped vehicles at intersections. The designer may consider increasing the high temperature binder grade and/or the percent of crushed aggregate to account for this condition. Economics will determine if the higher grade of binder can be applied to the whole project, or just the impacted length of pavement (i.e. intersection and approaches).

   c. **Durability:** Many low-volume asphalt pavements are more susceptible to failure due to long term aging than to rutting or fatigue. For pavements with good maintenance histories and more than 15 years between overlays, the designer may want to ensure that the mixture selection will provide adequate durability and, if economically necessary, sacrifice some reliability against rutting or fatigue. This can be accomplished through the selection of a lower compaction level and/or the selection of a softer grade of binder.

   d. **Urban vs. Rural:** Separate from the issue of traffic speed, rural projects that pass through urban locations should consider mix sizes (NMAS) that will appeal to the pedestrian traffic. In general, smaller mix sizes will have a better surface appearance than larger mix sizes. The designer can specify smaller mix sizes than those provided in the material selection guide table, but should also consider the availability of the aggregates when making that decision. Similarly, the designer may choose to use a larger mix size on rural sections for the purpose of reducing the asphalt binder content in the mixture.

   e. **New Construction vs. Overlay:** The design guide takes into account the major pavement performance parameters (rutting, fatigue, and low temperature cracking). When an overlay is placed, there are existing pavement distresses that influence the performance of the overlay. Generally, low temperature cracking will be less of a factor because reflective cracking from the existing pavement will minimize the stresses that cause low temperature cracking. If the existing pavement is cold-in-place rehabilitated (or a stress relief layer is placed) below the overlay, then low temperature cracking should be considered in the design.

   f. **Seasonal Traffic:** Seasonal traffic occurs over a relatively short period of time and may create pavement damage in excess of the normal traffic. For example, grain harvest, Iowa State Fair, festivals, etc. may generate higher volumes (in terms of ESALs) of traffic for a short period of time. This does not only take into account traffic volumes, but also pavement loads.

   g. **Mixture Workability:** Smaller mixture sizes are easier to use for hand work.

6. **Select the HMA Mixture Criteria for Each Pavement Layer:** Using the information developed in steps 1 through 5, select the PG binder grade, mixture size, mix design level, and aggregate properties.

   a. **PG Asphalt Binder Grade:** The designer should select a binder that nominally satisfies 98% temperature reliability for both the 7 day high pavement temperature and the 1 day low pavement temperature (see 5D-1, C, 3). The designer should select a conventional binder that best satisfies the project conditions. The standard conventional binders in Iowa are PG 58 -28 and PG 64 -22. Typically in urban areas, residential and collectors will normally use PG 58 -28. For arterials, a PG 64 -22 should be considered. If the designer selects other binder grades to address special considerations, they should recognize the cost impact of specifying non-standard binders. The "rule of 90" (see definitions) can be applied to determine if a binder may require a modifier to achieve the specified properties. "Rule of 90" values above 90 may be modified binders and will increase the binder cost by 15 to 50%.

   b. **HMA Mixture Size:** Each mixture size (NMAS) is a function of the available aggregates, project conditions, and lift thickness. Minimum lift thickness is a function of density and mixture constructability. The following table shows the minimum lift thickness for the following mix sizes:

Mix Size	Minimum Lift Thickness
3/8"	1"
1/2"	1 1/2"
3/4"	2"
1"	3"

   c. **Mix Design Level:** Based on the projected $ESAL_{20}$ value, seasonal traffic loading and current pavement distress, the designer must select a mix design level. The boundaries of the design levels are not absolute, so the designer should take into consideration the assumptions used to compute the ESAL value. A conservative $ESAL_{20}$ value of 400,000 may justify the selection of a 300,000 ESAL mixture instead of a 1,000,000 ESAL mixture. Conversely, a project that computes 900,000 $ESAL_{20}$ in a developing area may consider a 3,000,000 ESAL mixture.

   d. **Aggregate Properties:** The mixture design criteria (Table 5D-1.02) is derived from Iowa DOT Materials I.M. 510. Table 5D-1.02 specifies a 15% increase in percent crushed aggregate for surface and intermediate mixes 1 M ESALs and less to account for slow, stop, and turning conditions. Table 5D-1.02 also changes the aggregate source type for the 100K ESAL and 300K ESAL surface and intermediate mixtures from Type B to Type A.

      The specifications allow the designer to specify the minimum percent crushed and aggregate source type (Type A or Type B). This will be a local decision based on past performance and available aggregates. The actual percent crushed needed to achieve the mix design gyratory compaction volumetrics will vary with the quality of the aggregates used. Both the specified percent crushed and the gyratory compaction volumetrics must be satisfied by the HMA mixture.

7. **Check for Availability of Materials to Meet the Mix Design Criteria:** Review the mix design criteria selected in step 6 and determine if the binder and aggregates required to meet the mix design criteria are readily available or accessible at a reasonable cost. Contact local producers and/or district materials engineers, if the designer plans to use non-standard criteria. Imported aggregates and modified binders generally cause higher costs. The designer should be ready to

justify the mix selection decision.

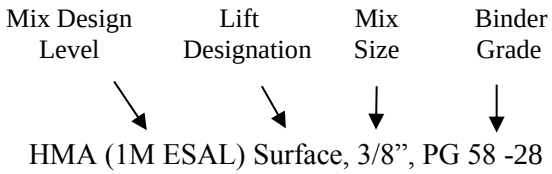
8. **Place Mix Criteria in the Project Plans and Proposal:** The following information should be placed in the plans and proposal:

   a. **Traffic and $ESAL_{20}$ Projections:** The traffic and $ESAL_{20}$ projections should be listed on the title sheet of the plans. The $ESAL_{20}$ value should coincide with the selected mix design level. If seasonal ESALs are used for design, the title sheet should note that the $ESAL_{20}$ value is based on seasonal loading. The following is an example title sheet.

Traffic	
Current ADT	_____
Future ADT	_____
Present Trucks	_____
$ESAL_{20}$	_____

   b. **HMA Mixture:** Each HMA mixture bid item is defined by the ESAL level, lift designation, and aggregate size. The mixture properties for each mixture level are specified in the specifications and Table 5D-1.02. If the designer specifies a different percent crushed aggregate, this should be identified in the bid item note on the plans. The designer should avoid placing the mix size in additional sections of the plans to minimize errors associated with duplication. The exception to this guide would be a bid item note or tabulation intended to identify locations of different mix sizes for the same lift.

   c. **Asphalt Binder Grade PG XX -XX:** The asphalt binder grade should be specified in the bid item. The designer should avoid placing the binder grade in additional sections of the plans to minimize errors associated with duplication. The exception to this guide would be a bid item note or tabulation intended to identify binder use when multiple binders are specified. The following is an example bid item.



D. Material Properties

1. **Typical PG Grades and Their Application:** Two grades (PG 58 -28 and PG 64 -22) are common conventional binders used in Iowa. A PG 58 -28 asphalt binder will cover the climate range for a majority of projects in Iowa with 98% reliability. Some locations along northern tier counties may desire a PG 52 -34 for better low temperature protection but will lose protection against rutting. Southern counties may consider a PG 64 -22 binder for better high temperature properties but will lose protection against low temperature cracking.

2. **Aggregate Source Properties:** Aggregate source properties are classified as Type A or Type B quality. The source properties are defined in Iowa DOT Specifications Section 4127. The mixture criteria listed in Table 5D-1.02 define the aggregate type for each mixture level specified for the project. Each individual source of aggregate is expected to meet these criteria. The designer may specify a different aggregate type in the bid item note.

3. **Aggregate Consensus Properties:** Aggregate consensus properties are listed in Table 5D-1.02 for each mixture level. These properties include percent crushed aggregate, fine aggregate angularity, clay content (sand equivalent), and flat & elongated particles. These aggregate properties are measured on the combined aggregate, not individual aggregates.

   If the designer specifies a value different from Table 5D-1.02, the value selected should be based on the local practice and desired pavement performance. The HMA mixture must satisfy both the percent crushed aggregate and laboratory compaction volumetric criteria. The percent crushed aggregate specified is interdependent on the compaction level and the quality of the aggregate.

E. Use of Mixture Selection Guide and Design Criteria Tables

Two tables in Subsection H are provided to assist designers with the selection of HMA materials for projects. The HMA Mixture Selection Guide (Table 5D-1.01) provides the project designer with a set of standard material selections that will satisfy most projects. The HMA Mixture Design Criteria (Table 5D-1.02) is derived from Iowa DOT Materials I.M. 510 and provides the mix designer with the detailed mix criteria for each mixture level. The mixture selection guide and mixture design criteria represent the current understanding of accepted HMA properties for application on urban routes.

The HMA Mixture Selection Guide (Table 5D-1.01) represents commonly used mixture parameters, but does not preclude the project designer from deviating from the "recommended" values. The designer should understand the impact of any modification. The first two columns define the standard mixture levels based on traffic loading. Daily ESAL and daily truck values are given as a general comparison to the design $ESAL_{20}$ ranges. These may be used to examine seasonal traffic conditions in relation to the 20 year value. The middle columns establish lift thickness and mix size relationships. It should be noted that Table 5D-1.01 does not address required pavement thickness to meet structural needs (Section 5F-1). The Bid Item Designation column ties the mixture levels to the bid items. The final column gives a general statewide guide for the estimated binder content. Local binder content experience may be more appropriate for project estimated quantities. This table does not address the need for special friction aggregate. In general terms, urban routes do not require special friction aggregate.

As mentioned earlier, the HMA Mixture Design Criteria (Table 5D-1.02) is derived from Iowa DOT Materials I.M. 510. However, the table differs from I.M. 510. For the surface and intermediate levels in the 100K to 1M ESAL mixes, the percent of crushed aggregate was increased 15% and the aggregate type was improved from B to A. A different aggregate type and the percent crushed aggregate may be specified by the designer for the project. These values established in the table are prescribed for each mixture and care should be exercised if altered by the project designer. The designer should only change these values when familiar with the material properties and mixture performance for the local area. The bid item plan note must include these values, if it differs from the value in Table 5D-1.02.

F. Example Plans

1. **Title Page:** The traffic and $ESAL_{20}$ projections should be listed on the title sheet of the plans. The $ESAL_{20}$ value should coincide with the selected mix design level. If seasonal ESALs are used for design, the title sheet should note that the $ESAL_{20}$ value is based on seasonal loading.

2. **Typical Section:** Lift thickness should be shown on the typical section. The lift thickness should match or exceed the recommended lift thickness for the mixture size selected, provided compactive requirements are also achieved. The lift should be designated as surface, intermediate, or base. Mixture size or design $ESAL_{20}$ level should not be added to the typical

section (it is specified in the bid item).

3. **Bid Items:** Unless otherwise specified, each bid item covers the mixture and binder grade selected. The corresponding bid item note must specify the minimum percent crushed aggregate, if it differs from the value in Table 5D-1.02.

G. Examples for Determination of Traffic ESALs

Similar to pavement thickness design, the HMA mixture is designed for the frequency and size of the load applied to the pavement. While it is important to have a good understanding of the traffic, it is possible to select the asphalt paving materials based on reasonable approximations. If the designer has actual traffic data, including a distribution of truck types and loads, the current annual ESAL value can be computed from the AASHTO pavement design tables. For most projects however, the designer will determine estimated values based on a general familiarity with the route. The following examples can be used to approximate the design $ESAL_{20}$ for a project.

Example 5D-1.01: Two Lane, Two Way Traffic, Low Volume Street

Step	Task	Values
1	Given: Current AADT Percent Trucks Percent Annual Growth Rate Design Period	1,000 5% 2% 20 years
2	Base Year Design ESALs *[from Section 5F-1, Table 5F-1.08]*	8,000 ESALs
3	Growth Factor *[from Section 5F-1, Table 5F-1.11]*	24.3
4	Compute $ESAL_{20}$ *[8,000 ESALs x 24.3]*	194,400 ESALs
5	Select HMA mixture design level *[from Table 5D-1.01, HMA Mixture Selection Guide]*	**HMA 300K**

Example 5D-1.02: Two Lane, Two Way Traffic, High Volume Street

Step	Task	Values
1	Given: Current AADT Percent Trucks Percent Annual Growth Rate Design Period	10,000 3% 3% 20 years
2	Base Year Design ESALs *[from Section 5F-1, Table 5F-1.08]*	50,000 ESALs
3	Growth Factor *[from Section 5F-1, Table 5F-1.11]*	26.9
4	Compute $ESAL_{20}$ *[50,000 ESALs x 26.9]*	1,345,000 ESALs
5	Select HMA mixture design level *[from Table 5D-1.01, HMA Mixture Selection Guide]*	**HMA 3M**

Example 5D-1.03: Four Lane Street

Step	Task	Values
1	Given: Current AADT Percent Trucks Percent Annual Growth Rate Design Period	15,000 5% 2% 20 years
2	Base Year Design ESALs *[from Section 5F-1, Table 5F-1.10]*	75,000 ESALs
3	Growth Factor *[from Section 5F-1, Table 5F-1.11]*	24.3
4	Compute $ESAL_{20}$ *[75,000 ESALs x 24.3]*	1,822,500 ESALs
5	Select HMA mixture design level *[from Table 5D-1.01, HMA Mixture Selection Guide]*	**HMA 3M**

H. Tables and Figures

Table 5D-1.01: HMA Mixture Selection Guide

Daily Trucks	Daily ESALs	Design $ESAL_{20}$ (Millions)	Layer Designation	Lift Thickness[3]			Mix Size[1]	Bid Item Designation	Binder Content[2]
				min	*rec*	*max*			
< 40	< 15	< 0.1	Surface	1.5	1.5	2.5	1/2”	HMA 100K	6.00
			Intermediate	1.5	1.5	3			
			Base	1.5	1.5	3			
40 to 125	15 to 40	0.1 to 0.3	Surface	1.5	1.5	2.5	1/2”	HMA 300K	6.00
			Intermediate	1.5	1.5	3			
			Base	1.5	1.5	3			
125 to 350	40 to 140	0.3 to 1.0	Surface	1.5	1.5	2.5	1/2”	HMA 1M	6.00
			Intermediate	1.5	1.5	3			
			Base	1.5	1.5	3			
350 to 990	140 to 400	1.0 to 3.0	Surface	1.5	2	2.5	1/2”	HMA 3M	6.00
			Intermediate	2	3	4	3/4”		5.50
			Base	2	3	4	3/4”		6.00
990 to 3,300	400 to 1,350	3.0 to 10.0	Surface	1.5	2	2.5	1/2”	HMA 10M	6.00
			Intermediate	2	3	4	3/4”		5.50
			Base	3	4	5	1”		5.25

[1] The common mix size is shown. When other mix sizes are used, the minimum lift thickness also changes (See Section 5D-1, C, 6, b).
[2] These values are for estimating quantities only. The actual asphalt binder content is established in the approved JMF.
[3] Some lift thickness values in this guide may conflict with traffic control or allowable compaction criteria.

Table 5D-1.02: SUDAS HMA Mixture Design Criteria
(derived from Iowa DOT Materials I.M. 510)

Mix Designation	Gyratory Density					Friction[2]			Aggregate[3]			
	N_{ini} - N_{des} - N_{max}	Initial % G_{mm} (max)	Design % G_{mm} (target)	Maximum % G_{mm} (max)	VFA	Film Thickness	Filler: Binder	Type 4 (min)	Type 3 (min)	Type 2 (min)	Quality Type	Crush (min)	FAA (min)	Sand Equiv. (min)
HMA 100K S-I	7 - 68 - 104	92.5	97.0	98.5	75-85	8.0-13.0	0.6-1.4				A[1]	60[1]	---	40
HMA 100K B											B	45	---	
HMA 300K S-I	7 - 68 - 104	92.0	96.5	98.0	70-80	8.0-13.0	0.6-1.4				A[1]	60[1]	---	40
HMA 300K B		92.5	97.0	98.5	75-85						B	45	---	
HMA 1M S L-4	7 - 76 - 117	90.5	96.0	98.0	65-78	8.0-15.0	0.6-1.4	50			A	75[1]	40	40
HMA 1M S														
HMA 1M I											A[1]	60[1]		
HMA 1M B	7 - 68 - 104	92.0	96.5	98.0	70-80						B	45	---	
HMA 3M S L-4	7 - 86 -134	89.5	96.0	98.0	65-78	8.0-15.0	0.6-1.4	50			A	75	40	40
HMA 3M S L-3								80	45	(25)				
HMA 3M S												60		
HMA 3M I														
HMA 3M B	7 - 76 - 117	90.5	96.5	98.0	65-78						B	45		
HMA 10M S L-3	8 - 96 - 152	89.0	96.0	98.0	65-78	8.0-15.0	0.6-1.4	80	45	(25)	A	75	43	45
HMA 10M I														
HMA 10M B	7 - 86 - 134	89.5	96.0	98.0	65-78						B	75	40	
For mix design levels exceeding 10M ESALs, see Iowa DOT Materials I.M. 510.														
For 100K, 300K, and 1M ESAL mixes, the Engineer may designate Type B quality aggregate due to local availability														

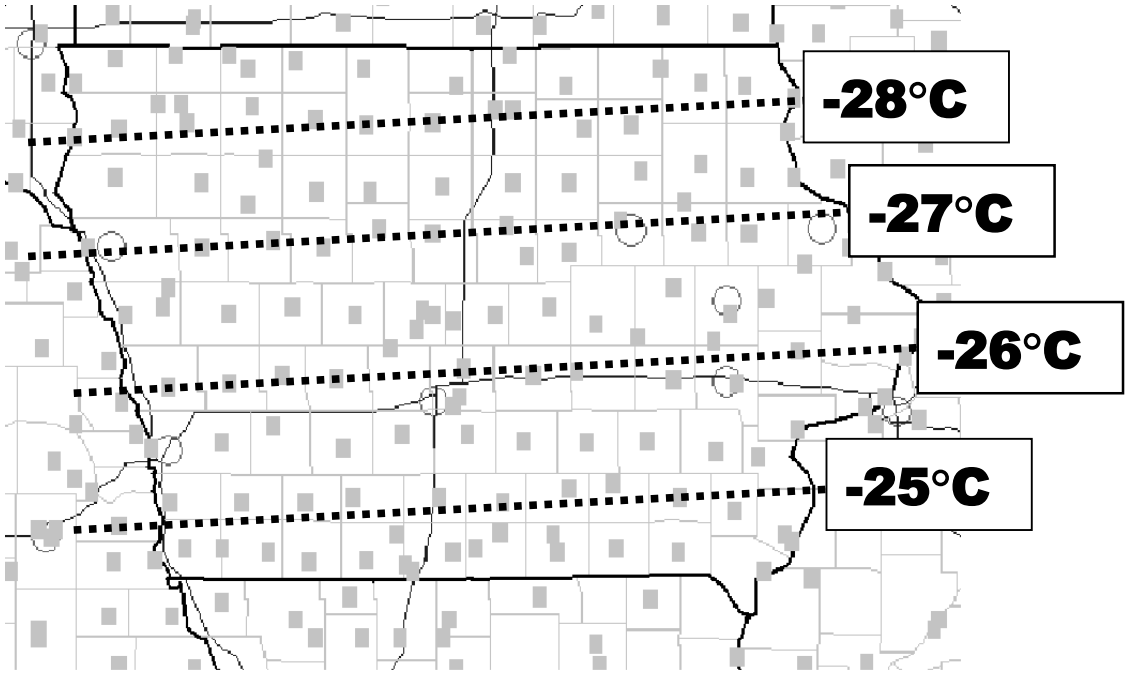
[1] Requirements differing from Iowa DOT Materials I.M. 510; for surface and intermediate mixes, aggregate quality improved from B to A and percent crushed aggregate increased by 15%.

[2] See Iowa DOT Standard Specifications Article 2303.02

[3] Flat & Elongated 10% maximum at a 5:1 ratio

Figure 5D-1.01: High (7 Day) Pavement Temperature at 98% Reliability

Figure 5D-1.02: Low (1 Day) Pavement Temperature at 98% Reliability

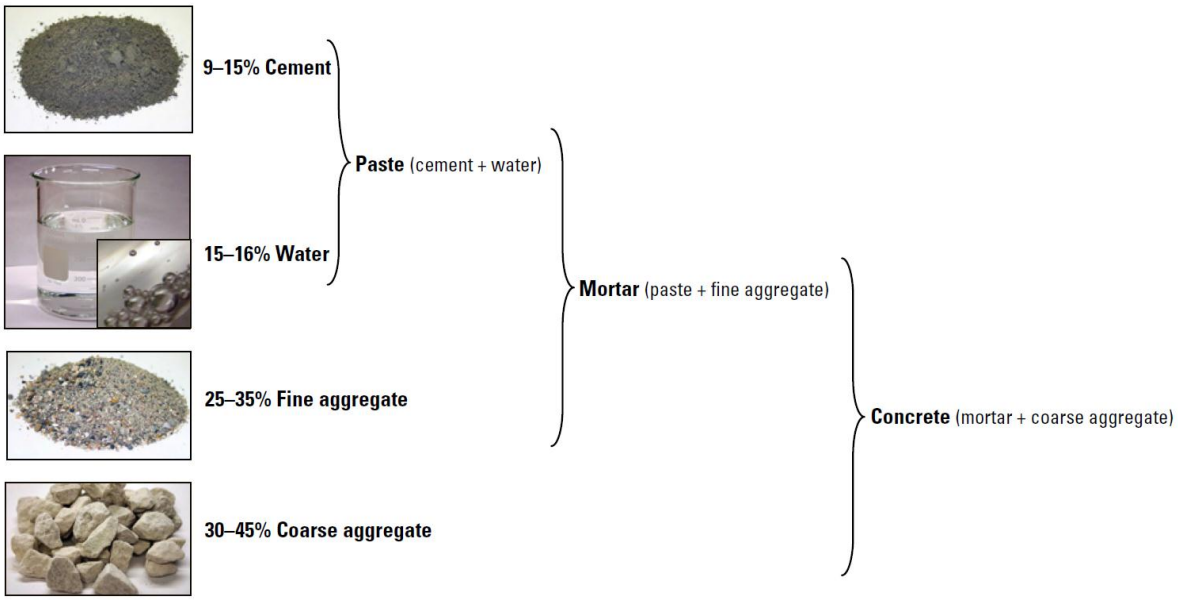


PCC Pavement Mixture Selection

A. General Information

Concrete is basically a mixture of two components, paste and aggregates. Cement and water form the paste, which binds the aggregates, usually sand and gravel or crushed stone, into a solid rocklike mass. The paste hardens due to a chemical reaction of the cement and water, known as hydration. In addition to the basic ingredients, supplementary cementitious materials (SCMs) and chemical admixtures may be included in the paste. This section will introduce the pavement designer to the PCC mixture components and their characteristics and behaviors so the optimum mixture selection for concrete pavements can be determined. It should be noted, the SUDAS Specifications reference the Iowa DOT Specifications for concrete mix materials, design, and proportions.

Figure 5E-1.01: Concrete is Basically a Mixture of Cement, Water/Air, and Aggregates (percentages are by volume)



Sources: Taylor et al, 2006

B. Cementitious Materials

Cementitious materials are classified as either hydraulic cements or pozzolans. The difference between the two is their reaction when mixed with water.

1. **Hydraulic Cements:** Hydraulic cements chemically react with water through a process called hydration. The compounds produced during hydration affect the setting, hardening, and strength gains of hydraulic cement mixtures. The hydration process occurs until the hardening of the concrete is complete and the strength gains have ceased. Portland cement, the most common type of hydraulic cement, contains hydraulic calcium silicates, calcium aluminates, calcium aluminoferrites, and calcium sulfate (gypsum). The reaction of the water being in contact with

the hydraulic cement produces a byproduct of calcium silicate hydrate (C-S-H) and calcium hydroxide (CH). Blended cements, a manufactured blend of Portland cement and one or more supplementary cementitious materials (SCMs), are also hydraulic cements.

a. Types of Hydraulic Cements: As outlined by ASTM C 1157, a performance specification, there are six types of hydraulic cements.

1) **Type GU:** General use
2) **Type HE:** High early strength
3) **Type MS:** Moderate sulfate resistance
4) **Type HS:** High sulfate resistance
5) **Type MH:** Moderate heat of hydration
6) **Type LH:** Low heat of hydration

If an "R" is added after the type name, HE-R, it denotes low reactivity with alkali-reactive aggregates.

b. Types of Portland Cement: There are five different types of Portland cements that are required to meet the specifications of ASTM C 150/AASHTO M 85.

1) **Type I:** Normal
2) **Type II:** Moderate sulfate resistance
3) **Type III:** High early strength
4) **Type IV:** Low heat of hydration
5) **Type V:** High sulfate resistance

c. Types of Blended Cements: Blended hydraulic cements are a combination of two or more types of fine materials. They can be used the same way as Portland cements. Typical materials that are combined are Portland cement and SCMs, including ground granulated blast-furnace slag (GGBF slag), fly ash, silica fume, calcined clay, pozzolans, or hydrated lime. These combinations that make blended hydraulic cements must conform to ASTM C 595 requirements. The two main classes of ASTM C 595 cements are:

1) **Type IS (X):** Portland blast-furnace slag cement
2) **Type IP (X):** Portland-pozzolan cement

The "X" stands for a percentage of the SCM included in the blend. Type IS (40) contains 40% by mass of slag.

d. Selection of Cement: The selection of cement is important when considering which type to use on the job. The main aspect to consider when selecting which cement is right for the job is the availability of cements. Types I and II are available almost everywhere, where Types III, IV, and V are less common in certain areas. Blended cements are available almost everywhere. The Iowa DOT and SUDAS Specifications allow Types I and II cements to be used in pavements, in addition to Types IP and IS blended cements.

2. Pozzolans: Pozzolans do not react when solely mixed with water; they require a source of calcium hydroxide (CH) to hydrate. The most common source of CH comes from the hydration of hydraulic cements, which produces both calcium silicate hydrates (C-S-H) and CH (a less desirable product). When combined with water and CH, pozzolans form additional C-S-H. This additional C-S-H contributes to concrete strength and impermeability of the cement mixtures. Common pozzolans include fly ash, silica fume, and natural pozzolans such as calcined clay, calcined shale, and metakaolin.

Table 5E-1.01: Cementitious Materials

Cementitious Materials

	Hydraulic cements	Pozzolans (or materials with pozzolanic characteristics)
	Portland cement Blended cement	
Supplementary cementitious materials	GGBF slag Class C fly ash	GGBF slag Class C fly ash Class F fly ash Natural pozzolans (calcined clay, calcined shale, metakaolin) Silica fume

Simple Definitions

Cement (hydraulic cement)—material that sets and hardens by a series of nonreversible chemical reactions with water, a process called hydration.

Portland cement—a specific type of hydraulic cement.

Pozzolan—material that reacts with cement and water in ways that improve microstructure.

Cementitious materials—all cements and pozzolans.

Supplementary cementitious materials—cements and pozzolans other than portland cement.

Blended cement—factory mixture of portland cement and one or more SCM.

Source: Taylor et al, 2006

C. Supplementary Cementitious Materials

Supplementary Cementitious Materials (SCMs) are a common addition to the mix in modern concrete mixtures. SCMs can contribute to the concrete through either hydraulic or pozzolanic activity or both. For example, GGBF slags are hydraulic materials, and Class F fly ashes are typically pozzolanic. Class C fly ash has both hydraulic and pozzolanic characteristics. There are four main types of SCMs are fly ash, ground granulated blast furnace slag (GGBF slag), natural pozzolans, and silica fume.

Fly ash is the most commonly used SCM and includes two types, Class C and Class F. Substituting fly ash in concrete mixes can reduce the amount of water required for workability but delays the setting time of the concrete. The addition of fly ash causes a slower but longer reaction rate in the concrete. As a result, the heat of hydration is reduced, and the setting time of the mix is delayed.

1. **Effects of SCMs:** The addition of SCMs to a concrete mixture affects a wide variety of properties for the concrete. Tables 5E-1.02 and 5E-1.03 indicate the effects of SCMs on fresh and hardened concrete properties. The modified properties include the following:

    a. **Fresh Properties:** Fly ash and GGBF slag increase the workability of the concrete, while silica fume can reduce workability at concentrations greater than 5%.

    b. **Durability/Permeability:** SCMs generally increase the durability of the concrete by reducing the permeability of the concrete mix. As a result, the concrete is less susceptible to chloride penetration. The quality of the SCMs and the work practices of the contractor are important to realize these benefits.

c. Resistance: Alkali-silica reactivity can be controlled with SCMs. The optimum dosage of fly ash has proven to reduce reactivity. Silica resistance is also improved with the addition of SCMs by reducing the reactive elements that contribute to expansive sulfate reactions. Class F fly ash is more effective than Class C, and GGBF slag is beneficial in sulfate environments. SCM content in concretes subject to freezing should not exceed 50%; however, they are still durable if over 50% is used.

Table 5E-1.02: Effects of SCMs on Fresh Concrete Properties

	Fly ash				Natural pozzolans		
	Class F	Class C	GGBF slag	Silica fume	Calcined shale	Calcined clay	Metakaolin
Water requirements	↓↓	↓↓	↓	↑↑	↔	↔	↑
Workability	↑	↑	↑	↓↓	↑	↑	↓
Bleeding and segregation	↓	↓	↕	↓↓	↔	↔	↓
Air content	↓↓*	↓*	↓	↓↓	↔	↔	↓
Heat of hydration	↓	↕	↓	↔	↓	↓	↓
Setting time	↑	↕	↑	↔	↑	↑	↔
Finishability	↑	↑	↑	↕	↑	↑	↑
Pumpability	↑	↑	↑	↑	↑	↑	↑
Plastic shrinkage cracking	↔	↔	↔	↑	↔	↔	↔

Sources: Thomas and Wilson (2002b); Kosmatka, Kerkhoff, and Panarese (2003)

* Effect depends on properties of fly ash, including carbon content, alkali content, fineness, and other chemical properties.

Key: ↓ reduced
↓↓ significantly reduced
↑ increased
↑↑ significantly increased
↔ no significant change
↕ effect varies

Source: Taylor et al, 2006

Table 5E-1.03: Effects of SCMs on Hardened Concrete Properties

	Fly ash				Natural pozzolans		
	Class F	Class C	GGBF slag	Silica fume	Calcined shale	Calcined clay	Metakaolin
Early strength	↓	↔	↓	↑↑	↓	↓	↑↑
Long-term strength	↑	↑	↑	↑↑	↑	↑	↑↑
Permeability	↓	↓	↓	↓↓	↓	↓	↓↓
Chloride ingress	↓	↓	↓	↓↓	↓	↓	↓↓
ASR	↓↓	↕	↓↓	↓	↓	↓	↓
Sulfate resistance	↑↑	↕	↑↑	↑	↑	↑	↑
Freezing and thawing	↔	↔	↔	↔	↔	↔	↔
Abrasion resistance	↔	↔	↔	↔	↔	↔	↔
Drying shrinkage	↔	↔	↔	↔	↔	↔	↔

Sources: Thomas and Wilson (2002b); Kosmatka, Kerkoff, and Panarese (2003)

Key: ↓ reduced
↓↓ significantly reduced
↑ increased
↑↑ significantly increased
↔ no significant change
↕ effect varies

Source: Taylor et al, 2006

2. **Limitations on the Use of SCMs:** Table 5E-1.04, which is adapted from ACI 218, provides recommended maximum amounts of SCMs for concrete exposed to deicing chemicals, such as Iowa concrete pavements. The Iowa DOT and SUDAS Specifications limit the usage of SCMs below the ACI maximum amounts. By those specifications, the maximum allowable fly ash substitution rate is 20%, and the GGBF slag substitution rate is limited to no more than 35% by weight (mass). The total mineral admixture substitution rate cannot exceed 40%. When Type IP or IS cement is used in the concrete mixture, only fly ash substitution is allowed. Substitution of Type I/II cement with both GGBF slag and fly ash is only allowed in ready mix concrete mixtures.

Table 5E-1.04: Cementitious Materials Requirements for Concrete Exposed to Deicing Chemicals

Cementitious Materials*	**Maximum Percent by Total Cementious Materials by Mass****	
	ACI Values	***Iowa Values***
Fly ash and natural pozzolans	25	20
GGBF slag	50	35
Silica fume	10	0
Total of fly ash, GGBF slag, silica fume, and natural pozzolans	50***	40
Total of natural pozzolans and silica fume	35***	20

* Includes portion of supplementary cementitious materials in blended cements.
** Total cementitious materials include the summation of portland cements, blended cements, fly ash, slag, silica fume, and other pozzolans.
*** Silica fume should not constitute more than 10% of total cementitious materials and fly ash or other pozzolans must not constitute more than 25% of cementitious materials.

Source: Taylor et al, 2006

D. Aggregates

Aggregates account for 60% to 75% of concrete by volume and are seldom susceptible to moisture and chemical changes, making them an important ingredient in concrete mixtures. Aggregates influence the concrete's freshly mixed and hardened properties, mixture proportions, and economy. They must be durable and free of any absorbed materials, clay, and materials that effect the interaction with the cement.

1. **Types of Aggregates:**
   - **a. Carbonate Rock:** Mainly limestone and dolomite with low porosity and low absorption rate.
   - **b. Granite:** Igneous rocks composed mainly of silica and silicates with the highest modulus of elasticity of any rock type available.
   - **c. Gravel and Sand:** Typically mixtures of many minerals and rocks. Gravel and sand from shale, siltstone, or unsound material rich rocks tend to be unsound and not recommended. Sand and gravel from higher elevations and that have been smoothed by water are best.
   - **d. Manufactured Aggregates:** Produced by crushing rocks into smaller pieces. Least likely to be contaminated, but mixtures with manufactured aggregates tend to be harder to work with and require more water.
   - **e. Recycled Aggregates:** Made from crushing concrete pavement and mixed with new aggregates. Typically has a higher absorption rate.

The following aggregate properties are important to consider when mixing concrete: gradation, durability, particle shape, surface texture, absorption, coefficient of thermal expansion, and resistance to freezing and thawing

2. **Gradation:** Gradation is a measure of the size distribution of aggregate particles, determined by passing aggregate through sieves of different sizes (ASTM C 136 / AASHTO T 27). Grading is most commonly shown as the percentage of material passing sieves with designated hole sizes. Aggregates are classified as coarse or fine by ASTM C 33/AASHTO M 6/M 80 as follows:

   - **a. Coarse Aggregate:** Coarse aggregate consists of gravel, crushed gravel, crushed stone, or crushed concrete that is retained on the No. 4 sieve. The maximum size of a coarse aggregate is generally 3/8 inch to 1 1/2 inch.
     1) Coarse aggregate requirements allow a wide range in selection.
     2) If the proportion of fine aggregate to total aggregate produces concrete of good workability, the grading for a given maximum size coarse aggregate can be varied moderately without appreciably affecting a mixture's cement and water requirements.
     3) Coarse aggregate size is limited by local availability, the maximum fraction of the minimum concrete thickness or reinforcing spacing, and the ability of the equipment to handle the concrete.

   - **b. Fine Aggregate:** Fine aggregate consists of natural sand, manufactured sand, or combinations of the two that pass the No. 4 sieve. Very fine particles (passing the No. 100 sieve) are limited by specifications because they have extremely high surface-to-volume ratios that require more paste.
     1) A relatively wide range in fine aggregate gradation is allowed.
     2) If the water to cementitious materials (w/cm) ratio is kept constant and the ratio of fine to coarse aggregate is chosen correctly, a wide range in grading can be used without a measurable effect on strength.
     3) Generally, increasing amounts of fine material will increase the water demand of concrete.

c. Well-graded Aggregate: It is important to maximize the amount of aggregate in concrete mixtures because they are more chemically and dimensionally stable than cement paste. This is done by selecting the best aggregate grading for the job.

1) Aggregates with a variety of sizes are optimum because smaller particles fill the voids between larger particles, maximizing the aggregate volume.
2) Aggregate size and grading is important when trying to achieve the preferred water content. Smaller aggregates require more paste because of the high surface to volume ratios, and vice versa.
3) Mixtures with properly graded aggregates tend to have less permeability and shrinkage, will be easier to handle, and will be the most economical.

d. Combined Aggregate Grading: The most important grading in a concrete mixture is the combined aggregate, utilizing the coarse and fine aggregates. Aggregates with a smooth grading curve will generally provide better performance than a gap-graded system.

1) Research on air-entrained concrete has indicated the w/cm ratio could be reduced by more than 8% using combined aggregate gradation.
2) If problems develop due to a poor gradation, consider using alternative aggregates, blending aggregates, or conducting a special screening of existing aggregates.

3. Durability: Aggregates containing minerals (see Table 3-12 in *Design and Control of Concrete Mixtures*) can react with alkali hydroxides and expand when exposed to moisture, cracking the concrete. Some rock types are potentially susceptible to alkali-silica reactivity (ASR) (shown in Table 3-13 from *Design and Control of Concrete Mixtures*). ASR and alkali carbonate reactivity can be avoided by expansion tests and petrography. Aggregates with coarse surfaces may be susceptible to freeze-thaw damage and cause D-cracking, damaging the concrete. The abrasion resistance of an aggregate indicates the quality; the higher the resistance, the higher the quality. The maximum percent of abrasion allowed for crushed stone is 50% and for gravel is 35% by the Iowa DOT and SUDAS Specifications, as determined according to AASHTO T 96.

a. Durability Determination: Durability of aggregates allowed for use in pavements by the Iowa DOT and SUDAS Specifications is based on service history; geologic correlation; and testing, including abrasion, freeze-thaw, and objectionable materials.

b. Durability Classes: Based on the durability determination, aggregates are designated as follows.

1) Class 2 Durability: No deterioration of pavements of non-interstate segments of the road system after 15 years and only minimal deterioration in pavements after 20 years of age. Class 2 is the default durability requirement by the SUDAS Specifications for aggregates used in mixes for urban pavements.
2) Class 3 Durability: No deterioration of pavements of non-interstate segments of the road system after 20 years of age and less than 5% deterioration of the joints after 25 years.
3) Class 3i Durability: No deterioration of pavements of the interstate road system after 30 years of service and less than 5% deterioration of the joints after 35 years.

4. Particle Shape: Aggregate shapes are described as either cubic, flat, or elongated. The use of flat and elongated particles should be limited to 15% the total mass of aggregate in an effort to reduce water demands. Rough textured, angular, and elongated particles require more water to make concrete mixtures smooth and workable. Angular aggregates have higher flexural and compressive strengths along with a higher skid resistance.

5. Surface Texture: Different textures may be used in a mixture as long as the mixture is properly proportioned with the varying textures. Rough textures are advantageous because of better

bonding and interlocking. If coarse surfaces are used, the particle sizes should be reduced and drainage should be improved around the base.

6. **Absorption:** It is important to know what state of moisture the aggregates are in during batching so that the w/cm ratio can be adjusted.

   a. Wet aggregates contribute undesired moisture.

   b. Saturated surface dry - neither absorbing water from nor contributing water to the concrete mix - is the ideal moisture level.

   c. An increased w/c ratio increases shrinkage and reduces strength.

   d. Aggregates with high absorption values result in variations of concrete quality.

7. **Coefficient of Thermal Expansion (CTE):** Similar to freeze-thaw resistance, an aggregate's CTE is how much it changes in size during temperature changes. Low CTE values are desirable because their size changes the least and tend to crack less. In Iowa, CTE is generally not a problem because of the prevalent use of limestone aggregate, which has a low CTE as illustrated in Table 5E-1.05.

Table 5E-1.05: Typical CTE Values for Common PCC Ingredients

	Coefficient of Thermal Expansion 10^{-6}/°F
Aggregate	
Granite	4 to 5
Basalt	3.3 to 4.4
Limestone	3.3
Dolomite	4 to 5.5
Sandstone	6.1 to 6.7
Quartzite	6.1 to 7.2
Marble	2.2 to 4
Cement paste (saturated)	10 to 11
Steel	6.1 to 6.7

Note: These values are for aggregates from specific sources, and different aggregate sources may provide values that vary widely from these values.

Source: Taylor et al, 2006

8. **Freeze-thaw Resistance:** The freeze-thaw resistance of aggregates is related to its porosity, absorption, permeability, and pore structure. If too much water is absorbed and the concrete aggregates freeze, the expanding aggregates can potentially destroy the concrete. This degradation of the concrete aggregate is known as D-cracking (see photo below). D-cracking can be reduced by selecting aggregates that have a better freeze-thaw resistance, reducing the maximum particle size, and by installing an effective drainage system to pull water out from underneath the pavement. The use of higher quality aggregates is recommended to increase freeze-thaw resistance. As noted, Class III limestone aggregates will provide greater freeze-thaw resistance than Class II. Some gravel sources will also outperform the Class II aggregates, but the actual freeze-thaw resistance should be verified prior to use.

E. Chemical Admixtures

Any ingredient other than portland cement, supplementary cementitious materials, water, and aggregates is considered an admixture. There are eleven different types of chemical admixtures, the four most common are air-entraining admixtures, water-reducing admixtures, retarding admixtures, and accelerating admixtures. Reasons for using admixtures are to reduce the cost of concrete construction, assist in construction operations, obtain certain properties in concrete, and maintain the quality of concrete over longer periods of time. Admixtures are used to complement acceptable cementing practices and should not be used to substitute them.

1. **Air-entraining Admixtures:** Air-entraining admixtures are the most common type of admixture used. Air-entraining admixtures have the ability to control and entrap air bubbles in concrete, providing the user with a more durable and workable concrete. These admixtures affect concrete by improving freeze-thaw resistance, increasing workability, improving deicer resistance, and reducing sulfate and alkali reactivity.

   Keep in mind for every 1% entrained air, concrete loses about 5% of its compressive strength. The Iowa DOT and SUDAS Specifications require air content of the unconsolidated concrete ahead of the paver to be 8% ± 2%, with the goal to have a minimum of 5% air entrained in the hardened concrete.

2. **Water-reducing Admixtures:** Water-reducing admixtures are implemented to control and reduce the amount of water in a concrete mixture. They typically reduce water content 5% to 10% by sacrificing the reduction of slump. However, strength is increased anywhere from 10% to 25% because of the reduction in the w/cm ratio, and concrete with water-reducing admixtures tend to have good air retention.

3. **Retarding and Accelerating Admixtures:** Retarding admixtures delay the setting time of concrete; therefore decreasing the rate of slump loss and extending the workability of the concrete. The bleeding capacity and rate of concrete is also increased. Retarding admixtures allow more time to place concrete on difficult jobs, allow for special finishing techniques, and offset the adverse effects hot weather has on setting time.

   Accelerating admixtures are used to accelerate the setting of concrete. The accelerating admixtures speed up the hydration process, setting, and strength gains at early ages. Calcium Chloride ($CaCl_2$) is the most commonly used accelerating admixture. It should be added as part of the mixing water.

F. Water

Any drinkable, potable water may be used as mixing water for concrete. ASTM C 1602 provides specifications for using mixing water in concrete mixtures. Sources of water in a concrete mixture come from batch water, ice (if used during high-temp weather), free moisture on aggregate, and water in admixtures.

Any non-potable water may have adverse effects on the strength and set time of the concrete and should be tested for strength, setting time, alkali levels, sulfate levels, chloride levels, total solids, and corrosion of reinforcements

G. Air-entrainment

Air-entrained concrete is used to improve freeze-thaw resistance when exposed to water and deicing chemicals. Air-entrained concrete is produced by using air-entrained cement or by adding an air-entraining admixture. The Iowa DOT and SUDAS Specifications require air content of the unconsolidated concrete ahead of the paver to be 8% ± 2%. The goal is to have a minimum of 5% entrained air in the hardened concrete.

1. **Benefits:** The primary benefits of air-entrained concrete include the following.

   a. Significant improvement of freeze-thaw and deicer-scaling resistance
      1) Air bubbles are created during the entrainment process. The air bubbles allow the pressure of freezing water to be released in air voids instead of in the concrete, which would eventually destroy it. Figure 11-16 in *Design and Control of Concrete Mixtures* demonstrates the improved durability of air-entrained concrete.
      2) Air voids on the surface of the concrete relieve pressure buildups and reduce surface scaling that have detrimental effects on the life of the concrete.

   b. Higher resistance to sulfate and alkali-silica reactivity

   c. Improved workability

   d. Reduced segregation and bleeding in freshly mixed concrete

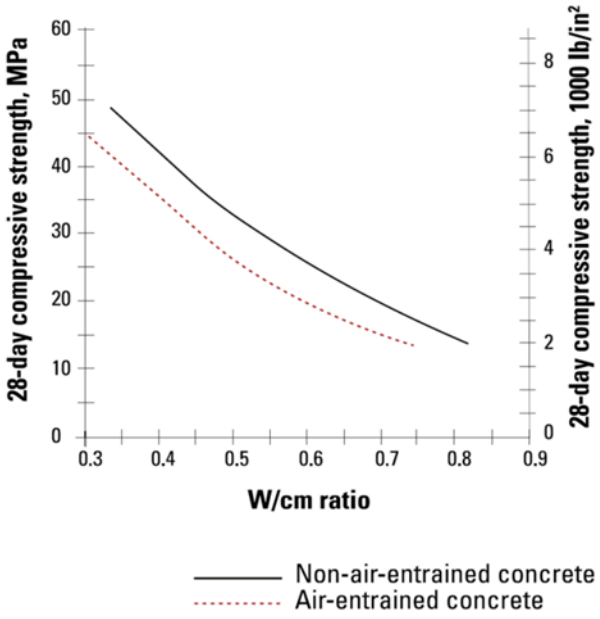
2. Factors that Affect the Air Content:

- Cement
- Aggregates
- Mixing water
- Slump
- Temperature
- Supplementary cementitious materials
- Admixtures
- Mixing acting
- Transportation and handling
- Finishing techniques

H. Slump

The slump of concrete is an indicator of the workability and a measurement of concrete consistency. Slump is not an indicator of the quality of the concrete. The slump of concrete is affected by changes in aggregates, cements, admixtures, water, and air. Workability, along with air content, is one of the primary concrete properties that can be manipulated during the process. Keep in mind that adding water to the mix will increase the w/cm ratio and lower the strength of the concrete. The SUDAS Specifications require that slump be no less than 1/2 inch or no more than 2 1/2 inches for machine finish and no less than 1/2 inch and no more than 4 inches for hand finish. Figure 5E-1.02 is an example of the strength values for mixes with differenct w/cm ratios; the higher the w/cm ratio, the lower the strength.

Figure 5E-1.02: Effect of w/cm Ratio on Strength of Concrete



I. Concrete Mixtures

1. **Concrete Mix Design and Mix Proportioning:** The terms mix design and mix proportioning are often incorrectly used interchangeably.

   a. **Mix Design:** Mix design is the process of determining required and specifiable properties of a concrete mixture, i.e., concrete properties required for the intended use, geometry, and exposure conditions. Workability, placement conditions, strength, durability, and cost should be considered in concrete mix designs.

   b. **Mix Proportioning:** Mix proportioning is the process of determining the quantities of concrete ingredients for a given set of requirements. The objective of proportioning concrete mixtures is to determine the most economical and practical combination of readily available materials to produce a concrete that will have the required properties.

2. **Concrete Mix Specifications:** There are two types of concrete mix specifications, prescriptive and performance mixes. With prescriptive mixes, the materials, proportions, and construction methods are specified, and satisfactory performance is anticipated. If performance requirements are utilized, functional requirements, such as strength, durability, and volume changes are specified, and the contractor and concrete producer are expected to develop concrete mixtures that meet those requirements.

   a. **SUDAS Concrete Mix Design Specifications:** The SUDAS Specifications for PCC pavement refer to the Iowa DOT's PCC pavement specifications and materials instructional memorandums (I.M.s) for concrete mix designs, proportions, and materials. Iowa DOT mixes are prescriptive and include specifications for concrete mix materials, proportions, and construction methods. There are no concrete performance requirements, and satisfactory performance is expected if all specifications are followed.

   b. **SUDAS Concrete Mix Material Specifications:** SUDAS references the Iowa DOT Specifications for concrete mix material, including the following.

Material	Iowa DOT Specifications Section	Iowa DOT Materials I.M.
Cement	4101	401
SCMs		
Fly Ash	4108	491.17
GGBF Slag	4108	491.14
Fine Aggregate	4110	409 (Source)
Coarse Aggregate (3 gradations)	4115	409 (Source)
Water	4102	
Admixtures		
Air entrainment	4103	403
Retarding and water reducing	4103	403
Accelerating (calcium chloride)	2529.02	403

c. SUDAS Concrete Mix Proportioning Specifications: The concrete mixes currently used in Iowa were developed in the 1950s. Classes A, B, and C were specified for concrete paving. As originally developed, Classes A and B, with minimum design compressive strengths of 3,500 psi and 3,000 psi respectively, were utilized for rural county paving. Class C concrete, with a higher compressive strength of a minimum of 4,000 psi and a w/cm ratio of less than 0.45, was the standard for primary roads. With its history of proven performance, Class C concrete is now the standard for all concrete road paving in Iowa. In areas where early opening strength is desired, such as intersections and driveways, an M mix can be substituted for C mix. M mix has a higher cement content, which accelerates the heat of hydration and set time of the concrete.

Unless the designer otherwise specifies, the contractor can choose any of the Iowa DOT Class C mixes and the materials that are allowed within the specifications. Generally, economy, workability, and availability of materials are key factors in the decision making process of the contractor and the concrete supplier.

Iowa DOT Materials I.M. 529 establishes the mix proportions for the various concrete mixes used by the Iowa DOT and SUDAS. Each mixture has specific requirements for the coarse and fine aggregates as well as the type of cement, including SCMs. The mix proportions include unit volumes for all materials.

1) Mix Designation:

Example: C-4WR-S35

- The first letter indicates the class of concrete
- The first number indicates the percentages of fine aggregate and coarse aggregate
  - 2 is composed of 40% fine and 60% coarse
  - 3 is composed of 45/55
  - 4 is composed of 50/50
  - 5 is composed of 55/45
  - 6 is composed of 60/40
  - 7 is composed of 65/35
  - 8 is composed of 70/30
  - 57 is composed of 50/50
- The WR indicates water reducer is used in the mixture
- SCMs are then indicated with their percentage of cementitious material substitution. C and F fly ashes are indicated with a C and F, respectively. GGBF slag is indicated with an S. The percentage of substitution is indicated after the SCM letter.
- The example designates a Class C concrete mix, a combined aggregate composed of 50% fine aggregate and 50% coarse aggregate, water reducer admixture, and 35% GGBF slag cementitious material substitution.

2) Mix Proportions: Iowa DOT Materials I.M. 529 provides material proportioning for the various Iowa DOT concrete mixes and includes basic absolute volumes of cement, water, air, and fine and coarse aggregate per unit volume of concrete (cy/cy). Target and maximum w/cm ratios are provided for each of the mix classes. Also included is guidance for calculation of fly ash and GGBF slag cementitious material substitution of cement.

3) Admixtures: Sources of Iowa DOT approved are provided in Iowa DOT Materials I.M. 403, along with their maximum dosages. Generally, the maximum dosages are as recommended by the manufacturers. Do not exceed the maximum dosages unless further consulted with the manufacturer and the Engineer.

3. **Modification of the Standard Concrete Mix Specifications:** While care should be exercised, achieving the required properties in the concrete may require making adjustments to the materials selected, to materials proportions, or even to other factors such as temperature, as follows.

   a. **Workability:** Water content, proportion of aggregate and cement, aggregate properties, cement characteristics, admixtures, and time and temperature can be adjusted to achieve the desired workability. The slump test (ASTM C 143 / AASHTO T 119) is most often used to measure the workability of fresh concrete.

   b. **Stiffening and Setting:** The rates of stiffening and setting of a concrete mixture are important because they affect its ability to be placed, finished, and sawed. Stiffening and setting can be affected by the following in the concrete mixture: cementitious materials, chemical admixtures, aggregate moisture, temperature, and water-cementitious materials (w/cm) ratios.

   c. **Bleeding:** Techniques can be used to prevent and minimize bleeding. These techniques (Kosmatka 1994) include reducing the water content, w/cm, and slump; increasing the amount of cement or supplementary cementitious materials in the mix; increasing the fineness of the cementitious materials; using properly graded aggregate; and using certain chemical admixtures such as air-entraining agents may reduce bleeding.

   d. **Air-void System:** The air-void system is important to concrete durability in environments subject to freezing and thawing. It includes total air content, spacing factors, and specific surface. The air-void system can be controlled with cement, supplementary cementitious materials, aggregates, and workability. The air-void system in the field will be affected by changes in the grading of the aggregate, water, admixture dosage, delays, and temperature.

   e. **Density:** Conventional concrete used in pavements has a density in the range of 137 to 150 lb/yd^3. Density varies depending on the amount and density of the aggregates, the amount of entrained air, the amount of water, and the cement content. Density is affected by the following factors: density of the material in the mixture, mostly from coarse aggregates; moisture content of the mixture; and relative proportions of the materials, mainly water.

   f. **Strength:** Strength and rate of strength gain are influenced by water-cementitious materials ratio, cement chemistry, SCMs, chemical admixtures, aggregates, and temperature. Changes in the environmental conditions and variation in materials, consolidation, and curing affect the strength at a specified age and affect strength development with age. Increased temperatures will increase early strength but may decrease long-term strength gain.

   g. **Volume Stability:** Concrete experiences volume changes as a result of temperature and moisture variations. To minimize the risk of cracking, it is important to minimize the tendency to change in volume by considering paste content, aggregates, and curing.

   h. **Permeability and Frost Resistance:** Permeability is a direct measure of the potential durability of a concrete mixture. Lower permeability is achieved by the following factors.
      - Increasing the cementitious materials content
      - Reducing the water-cementitious materials ratio
      - Using supplementary cementitious material at dosages appropriate to the expected likelihood of freezing water
      - Using good curing practices
      - Using materials resistant to the expected form of chemical attack

- Using aggregates that have proven to resist D-cracking. Reducing maximum coarse aggregate size will reduce the risk of damage if aggregates prone to damage are unavoidable.
- Ensuring that a satisfactory air-void system is provided

J. References

Kosmatka, S.H., B. Kerkhoff, W.C. Panarese. *Design and Control of Concrete Mixtures*. EB001.14. Skokie, IL: Portland Cement Association. 2002.

Taylor, P.C., S.H. Kosmatka, G.F. Voigt, et al. *Integrated Materials and Construction Practices for Concrete Pavement: A State-of-the-Practice Manual.* FHWA HIF-07-004. National Concrete Pavement Technology Center/Center for Transportation Research and Education, Iowa State University. 2006.

Pavement Thickness Design

A. General

The AASHO road test (completed in the 1950s) and subsequent AASHTO *Guide for the Design of Pavement Structures* (AASHTO Design Guide) provide the basis for current pavement design practices. To design a pavement by the AASHTO method, a number of design parameters must be determined or assumed. This section will explain the parameters required to design the pavement thickness of both concrete and hot mix asphalt roadways. The same parameters can be used for input data in computer programs on pavement determinations. The program used should be based on AASHTO design methods.

Even though the AASHTO Design Guide is several years old, it is still used throughout the industry for pavement thickness design. A newer design program called the Mechanistic-Empirical Pavement Design Guide (MEPDG) is available, however, it is costly and requires a great deal of data to be effective. The MEPDG does not generate a pavement thickness, it is set up to analyze the failure potential for a given thickness design. It is not generally used by local agencies. Each of the paving associations provides software programs for calculating pavement thickness. One HMA program is called I-Pave and was developed for the Asphalt Paving Association of Iowa. Another HMA program is called PaveXPress and was developed for the National Asphalt Pavement Association (NAPA). The American Concrete Pavement Association (ACPA) developed the PCC software, which is called StreetPave. The programs can be accessed through the respective websites of the paving associations. Users should be aware of the required inputs for the software programs, as well as the specific system defaults that cannot be changed or do not fit the project design criteria. If the program defaults do not match the project circumstances, the software program should not be used.

Historically municipalities have resorted to a one-size-fits-all approach by constructing standard pavement thicknesses for certain types of roadways without regard to traffic volumes or subgrade treatments. In an effort to show the effect of varying traffic loads and subgrade treatments on pavement thickness, this section provides comparison tables showing the various rigid and flexible pavement thicknesses calculated according to the AASHTO pavement design methodology. The ESAL and pavement thickness values shown in the tables are dependent upon the design parameters used in the calculations. The assumed parameters are described in the corresponding tables. The pavement designer should have a thorough understanding of the parameters and their reflection of actual site conditions prior to using them to select a pavement thickness. Projects that have traffic or site conditions that differ significantly from the values assumed herein should be evaluated with a site specific pavement design.

Engineers need to examine their agency's standard pavement foundation support system based on good engineering practices and the level of service they desire for the life of both HMA and PCC pavements. It is important to understand the characteristics of the soil and what cost-effective soil manipulation can be achieved, whether an aggregate subbase is used or not. If different soil types are encountered, and an aggregate subbase is not used, properly blending and compacting the soil will help reduce differential movement and help prevent cracking. Good designs, followed by good construction practices with a proper inspection/observation program, are critical to realize the full performance potential of either pavement type.

Designs that improve the foundation will extend the pavement life, improve the level of service throughout the life of the pavement, and provide more economical rehabilitation strategies at the end of the pavement's life for both HMA and PCC pavements. Although the initial cost to construct the pavement will undoubtedly be higher than placing the pavement on natural subgrade, the overall life cycle costs will be greatly improved.

Definitions of the pavement thickness design parameters are contained in Section 5F-1, B. Section 5F-1, C defines the process for calculating ESAL values. Section 5F-1, D provides the comparison tables discussed in the previous paragraph. Finally, example calculations are shown in Section 5F-1, E.

The pavement designer should be aware of the parameters that are required for the project under design. If those project design parameters differ from the parameters used to calculate the typical pavement thicknesses provide in this section, then a specific design set to meet the specific project parameters should be undertaken.

B. Pavement Thickness Design Parameters

Some of the pavement thickness design parameters required for the design of a rigid pavement differ from those for a flexible pavement. Table 5F-1.01 summarizes the parameters required for the design of each pavement structure.

Table 5F-1.01: Summary of Design Parameters for Pavement Thickness

Section	Description	Flexible HMA	Rigid JPCP/JRCP
5F-1, B, 1	**Performance Criteria**		
	a. Initial Serviceability Index	X	X
	b. Terminal Serviceability Index	X	X
5F-1, B, 2	**Design Variables**		
	a. Analysis Period	X	X
	b. Design Traffic	X	X
	c. Reliability	X	X
	d. Overall Standard Deviation	X	X
5F-1, B, 3	**Material Properties for Structural Design**		
	a. Soil Resilient Modulus	X	
	b. Modulus of Subgrade Reaction		X
	c. Concrete Properties		X
	d. Layer Coefficients	X	
5F-1, B, 4	**Pavement Structural Characteristics**		
	a. Coefficient of Drainage	X	X
	b. Load Transfer Coefficients for Jointed		X
	c. Loss of Support		X

The following considerations should be used when designing pavement thickness for flexible and rigid pavements.

1. **Performance Criteria (Serviceability Indexes):** Condition of pavements are rated with a present serviceability index (PSI) ranging from 5 (perfect condition) to 0 (impossible to travel).

    a. **Initial Serviceability Index (P_o):** The initial serviceability index (P_o) is the PSI immediately after the pavement is open. At the AASHO road test, values of 4.5 for rigid pavement and

4.2 for flexible pavement were assumed. These values are listed in the 1993 AASHTO Design Guide.

b. Terminal Serviceability Index (P_t): The terminal serviceability index (P_t) is considered to be the PSI that represents the lowest acceptable level before resurfacing or reconstruction becomes necessary.

The following values are recommended for terminal serviceability index.

Table 5F-1.02: Terminal Serviceability Indexes (P_t) for Street Classifications

P_t	Classifications
2.00	Secondary Roads and Local Residential Streets
2.25	Minor Collectors, Industrial, and Commercial Streets
2.50	Major Collectors and Arterials

c. Serviceability Loss: The predicted loss or drop in serviceability (ΔPSI) is the difference between initial and terminal serviceability ($P_o - P_t$). The ΔPSI is the basis for the pavement design.

2. Design Variables:

a. Analysis Period: This refers to the period of time for which the analysis is to be conducted. The recommended analysis period is 50 years for both concrete and asphalt pavements.

b. Design Traffic: An estimate of the number of Equivalent 18,000 pound Single Axle Loads (ESALs) during the analysis period is required. This value can be estimated based on:

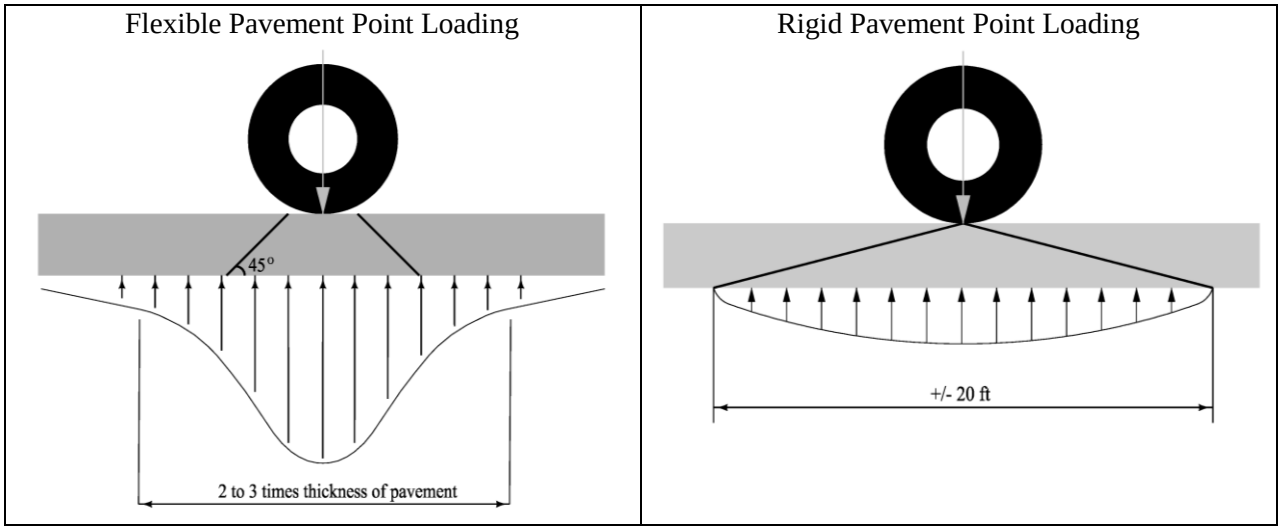
- the Average Annual Daily Traffic (AADT) in the base year,
- the average percentage of trucks expected to use the facility,
- the average annual traffic growth rate, and
- the analysis period.

It should be noted that it is not the wheel load but rather the damage to the pavement caused by the wheel load that is of particular concern. As described above, the ESAL is the standard unit of pavement damage and represents the damage caused by a single 18,000 pound axle load. Therefore, a two-axle vehicle with both axles loaded at 18,000 pounds would produce two ESALs. However, since vehicle configurations and axle loads vary, AASHTO has established a method to convert different axle loads and configurations to ESALs. For example, a 34,000 pound tandem axle produces approximately 1.9 ESALs for rigid pavement (1.1 for flexible pavement). Summing the different ESAL values for each axle combination on a vehicle provides a vehicle's Load Equivalency Factor (LEF). The LEF can then be applied to the assumed truck mix and the AADT to determine ESALs.

Section 5F-1, C details the steps involved in ESAL calculations and provides examples for both rigid and flexible pavements. ESAL tables for rigid and flexible pavements, and the corresponding assumptions used to create them, are provided for both two lane and four lane facilities.

The need for separate ESAL tables for flexible and rigid pavements is based on the inherent ability of each type of pavement to distribute a point loading. Rigid pavements have the ability to distribute the load across the slab. A point loading on a flexible pavement is more localized. This results in different ESAL factors for the two types of pavements. This is shown graphically in Figure 5F-1.01.

Figure 5F-1.01: Flexible vs. Rigid Point Loading Distribution



c. Reliability [R (%)]: Reliability is the probability that the design will succeed for the life of the pavement. Because higher roadway classification facilities are considered more critical to the transportation network, a higher reliability is used for these facilities. The following reliability values were assumed for the calculations.

Table 5F-1.03: Reliability for Flexible and Rigid Pavement Design

Street Classification	Reliability
Local Streets	80%
Collector Streets	88%
Arterial Streets	95%

d. Overall Standard Deviation (S_o): The Overall Standard Deviation is a coefficient that describes how well the AASHO Road Test data fits the AASHTO Design Equations. The lower the overall deviation, the better the equations models the data. The following ranges are recommended by the AASHTO Design Guide.

Table 5F-1.04: Overall Standard Deviation (S_o) for Rigid and Flexible Pavements

Pavement Type	Range of Values		Value Used
	Low	***High***	
Rigid Pavements	0.30	0.40	0.35
Flexible Pavements	0.40	0.50	0.45

3. Material Properties for Structural Design:

a. Soil Resilient Modulus (M_R): The important variable in describing the foundation for pavement design is the Soil Resilient Modulus (M_R). M_R is a property of the soil that indicates the stiffness or elasticity of the soil under dynamic loading.

The Soil Resilient Modulus measures the amount of recoverable deformation at any stress level for a dynamically loaded test specimen. The environment can affect pavement performance in several ways. Temperature and moisture changes can have an effect on the strength, durability, and load-carrying capacity of the pavement and roadbed materials. Another major environmental impact is the direct effect roadbed swelling, pavement

blowups, frost heave, disintegration, etc. can have on loss of riding quality and serviceability. If any of these environmental effects have the potential to be present during the life cycle of the pavement, the M_R should be evaluated on a season by season basis, and a seasonal modulus developed.

The purpose of using seasonal modulus is to qualify the relative damage a pavement is subject to during each season of the year and treat it as part of the overall design. An effective soil modulus is then established for the entire year, which is equivalent to the combined effects of all monthly seasonal modulus values.

For the purposes of this section, the M_R value was calculated based on the proposed CBR values of 3, 5, and 10. The normal soils in Iowa have in situ CBR values of 1 to 3. In order to successfully develop a foundation CBR of 10, it is most likely going to involve use of a subgrade that is stabilized with cement, fly ash, or other product. NCHRP Project 1-28 provides a relationship between CBR value and M_R value (see equation below). Using this equation, the corresponding M_R values for CBR values of 3, 5, and 10 are shown. For further information regarding the relationship between soil types and bearing values, see Section 6E-1.

Relationship between CBR and M_R values per NCHRP Project 1-28:

$$M_R = 1941.488\ X\ CBR^{0.6844709}$$

CBR Value	M_R Value
3	4120
5	5840
10	9400

For flexible pavement design, Figure 2.3 from Chapter 2 of the 1993 AASHTO Design Guide was used to estimate the effective M_R value taking into account seasonal variability. Frozen conditions were assumed for the months of December, January, and February. Due to spring wetness and thawing conditions, the M_R value for the months of March and April were assumed to be 30% of normal conditions.

For rigid pavement design, the M_R value is used to calculate the modulus of subgrade reaction, k.

b. Modulus of Subgrade Reaction (k, k_c): Several variables are important in describing the foundation upon which the pavement rests:

- k - The modulus of subgrade reaction for the soil;
- k_c - A composite k that includes consideration of subbase materials under the new pavement
- M_R - Soil resilient modulus

1) Modulus of Subgrade Reaction, k: For concrete pavements, the primary requirement of the subgrade is that it be uniform. This is the fundamental reason for specifications on subgrade compaction. In concrete pavement design, the strength of the soil is characterized by the modulus of subgrade reaction or, as it is more commonly referred to, "k". An approximate relationship between k and M_R published by AASHTO is:

$k = M_R/19.4$

The resilient modulus is used to calculate the modulus of subgrade reaction. Refer to Section 6E-1 for the relationship between k and M_R.

2) Composite Modulus of Subgrade Reaction, k_c: In many highway applications the pavement is not placed directly on the subgrade. Instead, some type of subbase material is used. When this is done, the k value actually used for design is a "composite k" (k_c), which represents the strength of the subgrade corrected for the additional support provided by the subbase.

The analysis of field data completed as a part of the Iowa Highway Research Board (IHRB) Project TR-640 showed that the modulus of subgrade reaction and the drainage coefficient for 16 PCC sites, which ranged in ages between 1 and 42 years, were variable and found to be lower in-situ than typical parameters used in thickness design. This indicates a loss of support over time. This change in support is already partially reflected in the AASHTO serviceability index to a degree.

Similar to the procedures used to estimate the effective M_R value for flexible pavement design, the AASHTO Design Guide provides procedures for estimating the k_c value taking into account potential seasonal variability. For the purposes of this manual, the k_c value was calculated using Table 3.2 and Figures 3.3 through 3.6 from Chapter 3 of the 1993 AASHTO Design Guide. Again, frozen conditions were assumed for the months of December, January, and February. Due to spring wetness conditions, the M_R value for the months of March and April were assumed to be 30% of normal conditions.

c. Concrete Properties: PCC - Modulus of Elasticity (E_c) and Modulus of Rupture (S'_c).

The Modulus of Rupture (S'_c) used in the AASHTO Design Guide equations is represented by the average flexural strength of the pavement determined at 28 days using third-point loading (ASTM C 78).

The Modulus of Elasticity for concrete (E_c) depends largely on the strength of the concrete. Typical values are from 2 to 6 million psi. The following equation provides an approximate value for E_c:

$$E_c = 6{,}750\ (S'_c)$$

where:

S'_c = modulus of rupture [28 day flexural strength of the concrete using third point loading (psi)]

The approximate relation between modulus of rupture (S'_c) and compressive strength (f_c) is

$$S'_c = 2.3\ f_c^{0.667} (\text{psi})$$

d. Layer Coefficients: Structural layer coefficients (a_i values) are required for flexible pavement structural design. A value for these coefficients is assigned to each layer material in the pavement structure in order to convert actual layer thickness into the structural number (SN). These historical values have been used in the structural calculations. If specific elements, such as a Superpave mix or polymer modified mix are used, the designer should adjust these values to reflect differing quality of materials.

The following table shows typical values for layer coefficients.

Table 5F-1.05: Layer Coefficients

Component	Coefficient	Minimum Thickness Allowed
Surface / Intermediate Course		
HMA with Type A Aggregate	0.44*	2
HMA with Type B Aggregate	0.44*	2
HMA with Type B Class 2 Aggregate	0.40	
Base Course		
Type A binder placed as base	0.40	2
HMA with Type B Aggregate	0.40	2
Asphalt Treated Base Class I	0.34*	4
Bituminous Treated Aggregate Base	0.23	6
Asphalt Treated Base Class II	0.26	4
Cold-Laid Bituminous Concrete Base	0.23	6
Cement Treated Granular (Aggregate) Base	0.20*	6
Soil-Cement Base	0.15	6
Crushed (Graded) Stone Base	0.14*	6
Macadam Stone Base	0.12	6
PCC Base (New)	0.50	
Old PCC	0.40**	
Crack and Seated PCC	0.25 to 0.30	
Rubblized PCC	0.24	
Cold-in-Place Recycled Asphalt Pavement	0.22 to 0.27	
Full Depth Reclamation	0.18	
Subbase Course		
Soil-Cement Subbase (10% cement)	0.10	6
Soil-Lime Subbase (10% lime)	0.10	6
Modified Subbase	0.14	4
Soil-Aggregate Subbase	0.05*	4

* Indicates coefficients taken from AASHTO Interim Guide for the Design of Flexible Pavement Structures.

** This value is for reasonably sound existing concrete. Actual value used may be lower, depending on the amount of deterioration that has occurred.

Source: AASHTO, Kansas State University, and Iowa DOT

4. Pavement Structural Characteristics:

a. Coefficient of Drainage: Water under the pavement is one of the primary causes of pavement failure. Water, either from precipitation or groundwater, can cause the subgrade to become saturated and weaken. This can contribute to pavement pumping under heavy loads.

C_d - The coefficient of drainage for rigid pavement design used to account for the quality of drainage.

M_i - The coefficient of drainage for flexible pavement design used to modify layer coefficients.

At the AASHO road test, the pavements were not well drained as evidenced by the heavy pumping that occurred on some of the test sections. The cross-sections were elevated and drainage ditches were provided. However, edge drains, which are used frequently in today's street and highway construction, were not evaluated at the AASHO road test. Edge drains are an effective deterrent to pumping and associated pavement distress.

In selecting the proper C_d or M_i value, consideration must be given to two factors: 1) how effective is the drainage, and 2) how much of the time is the subgrade and subbase in a saturated condition? For example, pavements in dry areas with poor drainage may perform as well as pavements built in wet areas with excellent drainage.

The following definitions are offered as a guide.

- Excellent Drainage: Material drained to 50% of saturation in 2 hours.
- Good Drainage: Material drained to 50% of saturation in 1 day.
- Fair Drainage: Material drained to 50% of saturation in 7 days.
- Poor Drainage: Material drained to 50% of saturation in 1 month.
- Very Poor Drainage: Material does not drain.

Based on these definitions, the C_d or M_i value for the road test conditions would be 1.00. A value of 1.00 would have no impact on pavement thickness or the number of ESALs a section would carry. Lower values increase the required pavement thickness; higher values decrease the required pavement thickness. Based on Tables 2.4 and 2.5 from the 1993 AASHTO Design Guide, the analysis assumed a fair quality of drainage and 1% to 5% exposure to saturation for the drainable base sections.

b. Load Transfer Coefficients for Jointed and Jointed Reinforced Pavements: One item that distinguishes PCC pavement is the type of joint used to control cracking and whether or not steel dowels are used in the joint for load transfer. Each of these designs provides a different level of transfer of load from one side of a pavement joint to the other. To adjust projected pavement performance for these various designs, the load transfer coefficient or "J" factor is used.

c. Loss of Support: The loss of support factor is included in the design of rigid pavements to account for the potential loss of support arising from subgrade erosion and/or differential vertical soil movement. It is treated in the design process by diminishing the composite k-value based on the size of the void that may develop beneath the slab. The primary failure mode of rigid pavements at the AASHTO road test was loss of support caused by pumping of fines from underneath the slab. The loss of support variable was only applied on natural subgrades since the presence of water in the joint area contributes to the pumping of the subgrade fines through the joints and cracks. With granular subbase and longitudinal drains, the water is drained away from the joints and cracks much quicker.

Pavement design parameters within the PCC thickness design software programs often do not adequately reflect actual pavement foundation conditions except immediately after initial construction. Field data from testing completed at 16 Iowa sites showed lower coefficient of drainage values than those assumed in design, indicating that a potential migration of natural soils into the aggregate subbase over time may cause some loss of support. This in turn lowers the overall modulus of subgrade reaction. The results of the field testing indicating this loss of support due to mixing of the subgrade and subbase will need to be further validated by additional research. In order to maintain a high drainage coefficient, it is important to maintain separation between the soil subgrade and the aggregate subbase. One method of providing the separation is with a geotextile layer.

In most cases for local, low volume PCC roads, aggregate subbases do not influence thickness design to any measurable degree. According to MEPDG analysis for low volume PCC roadways (less than 1,000 ADT and 10% trucks), aggregate subbase thicknesses greater than 5 inches do not appear to improve the International Roughness Index (IRI) or reduce slab cracking.

Based on the IHRB TR-640 research with a limited data set of 16 Iowa sites, it was noted that a PCC pavement with an optimized foundation of granular subbase, subdrains, and a geotextile separation layer between the subgrade and subbase is likely to maintain a higher pavement condition index (PCI) over time than a PCC pavement on natural subgrade. The lower the variability and the higher the coefficient of drainage with an optimized foundation, the higher the pavement condition will be for a given period of time. Since the PCI prediction model from the IHRB research was developed based on a limited data set, it must be further validated with a larger pool of data. However, designers should consider the benefits of optimizing the foundations under their pavements to improve long-term serviceability.

C. Calculating ESAL Values

To estimate the design ESALs, the following procedure may be used. A more thorough analysis may also be performed using the procedures found in Appendix D of the 1993 AASHTO Design Guide or computer programs based on that procedure.

1. Obtain an estimate of the design AADT for the beginning, or base year of the analysis period.

2. Obtain an estimate of the average percentage of the AADT that will be trucks.

3. Select the base year design lane ESALs from Tables 5F-1.07 through 5F-1.10, depending upon whether the facility is two lane, four lane, rigid, or flexible.

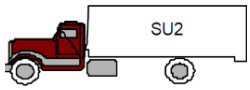
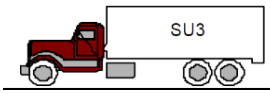
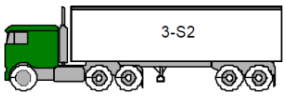
4. Select the growth factor from Table 5F-1.11 based on the average annual traffic growth rate and the analysis period.

5. Multiply the base year design lane ESALs, by the growth factor to obtain the total ESALs for the analysis period.

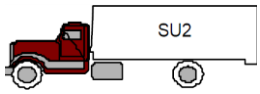
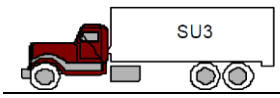
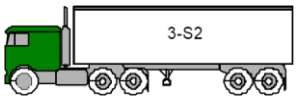
Table 5F-1.06 summarizes the inputs and calculations that went into creating Tables 5F-1.07 through 5F-1.10.

Table 5F-1.06: Truck Mixture for Urban Roadways and Determination of Truck ESAL Factor

A. Low Volume Streets

Vehicle Type		Percent of Total Trucks	Loading	Percent of Truck Type	Vehicle Weight (lbs)	Axle Type *S - Single* *TA - Tandem*	Axle Load (lbs)	ESAL Factor (per axle) Rigid	ESAL Factor (per axle) Flexible	LEF (by Vehicle) Rigid	LEF (by Vehicle) Flexible
	Single Unit (2 axles) (Class 5/6 Truck)	65%	Empty	30%	14,500	Front - S	7,000	0.019	0.024		
						Rear - S	7,500	0.025	0.032		
			Partial Load (50% Capacity)	50%	20,500	Front - S	8,000	0.033	0.041		
						Rear - S	12,500	0.212	0.242		
			Fully Loaded	20%	26,000	Front - S	9,000	0.053	0.066		
						Rear - S	17,000	0.785	0.799	*0.3033*	*0.3313*
	Dump Trucks - 3 axles (Class 7/8 truck) (doesn't address cheater axles)	15%	Empty	50%	22,000	Front - S	10,000	0.083	0.101		
						Rear - TA	12,000	0.026	0.018		
			Fully Loaded	50%	54,000	Front - S	20,000	1.558	1.52		
						Rear - TA	34,000	1.9	1.099	*1.7835*	*1.369*
	Semis (5 axles)	20%	Empty	20%	26,000	Front - S	12,000	0.178	0.206		
						Rear - TA	7,000	0.003	0.002		
						Trailer - TA	7,000	0.003	0.002		
			Partial Load (50% Capacity)	60%	53,000	Front - S	13,000	0.251	0.282		
						Rear - TA	20,000	0.208	0.138		
						Trailer - TA	20,000	0.208	0.138		
			Fully Loaded	20%	80,000	Front - S	20,000	1.558	1.52		
						Rear - TA	34,000	1.9	1.099		
						Trailer - TA	34,000	1.9	1.099	*1.5086*	*1.1204*
						Composite Load Equivalency Factor (LEF) for "Trucks"				**0.76639**	**0.644775**

B. High Volume Streets

Vehicle Type		Percent of Total Trucks	Loading	Percent of Truck Type	Vehicle Weight (lbs)	Axle Type *S - Single* *TA - Tandem*	Axle Load (lbs)	ESAL Factor (per axle) Rigid	ESAL Factor (per axle) Flexible	LEF (by Vehicle) Rigid	LEF (by Vehicle) Flexible
	Single Unit (2 axles) (Class 5/6 Truck)	30%	Empty	30%	14,500	Front - S	7,000	0.019	0.024		
						Rear - S	7,500	0.025	0.032		
			Partial Load (50% Capacity)	50%	20,500	Front - S	8,000	0.033	0.041		
						Rear - S	12,500	0.212	0.242		
			Fully Loaded	20%	26,000	Front - S	9,000	0.053	0.066		
						Rear - S	17,000	0.785	0.799	*0.3033*	*0.3313*
	Dump Trucks - 3 axles (Class 7/8 truck) (doesn't address cheater axles)	10%	Empty	50%	22,000	Front - S	10,000	0.083	0.101		
						Rear - TA	12,000	0.026	0.018		
			Fully Loaded	50%	54,000	Front - S	20,000	1.558	1.52		
						Rear - TA	34,000	1.9	1.099	*1.7835*	*1.369*
	Semis (5 axles)	60%	Empty	20%	26,000	Front - S	12,000	0.178	0.206		
						Rear - TA	7,000	0.003	0.002		
						Trailer - TA	7,000	0.003	0.002		
			Partial Load (50% Capacity)	60%	53,000	Front - S	13,000	0.251	0.282		
						Rear - TA	20,000	0.208	0.138		
						Trailer - TA	20,000	0.208	0.138		
			Fully Loaded	20%	80,000	Front - S	20,000	1.558	1.52		
						Rear - TA	34,000	1.9	1.099		
						Trailer - TA	34,000	1.9	1.099	*1.5086*	*1.1204*
						Composite Load Equivalency Factor (LEF) for "Trucks"				**1.1745**	**0.90853**

Assumptions:
ESAL factors for individual axles were determined from AASHTO Design Guide.
Assumed an initial pavement thickness of 8 inches and SN of 3.25 (does not significantly impact ESAL factor - +/- 3%)
Assumed a terminal serviceability of 2.25.
ESAL factors do not account for directional split. i.e. if 2 lane roadway with total AADT and 50/50 directional split, divide ESAL factor in half.
Vehicle weights and payload obtained from US Department of Energy Fact #621: Gross Vehicle Weight vs. Empty Vehicle Weight.
Assumed maximum single axle load of 20,000 lbs (including steering axle) and tandem axle load of 34,000 lbs.
Percent of fully loaded, partially loaded, and empty vehicles were estimated. No supporting documentation.

Source: 2010 Iowa DOT traffic count data (unless otherwise noted).

Table 5F-1.07: Base Year Design ESALs for Two Lane *Rigid Pavement*

	Two Way, Base Year AADT							
% Trucks	***1,000***	***2,000***	***3,000***	***4,000***	***5,000***	***10,000***	***15,000***	***20,000***
1	1,000	3,000	4,000	9,000	11,000	21,000	32,000	43,000
2	3,000	6,000	8,000	17,000	21,000	43,000	64,000	86,000
3	4,000	8,000	13,000	26,000	32,000	64,000	96,000	129,000
4	9,000	17,000	26,000	34,000	43,000	86,000	129,000	171,000
5	11,000	21,000	32,000	43,000	54,000	107,000	161,000	214,000
6	13,000	26,000	39,000	51,000	64,000	129,000	193,000	257,000
7	15,000	30,000	45,000	60,000	75,000	150,000	225,000	300,000
8	17,000	34,000	51,000	69,000	86,000	171,000	257,000	343,000
9	19,000	39,000	58,000	77,000	96,000	193,000	289,000	386,000
10	21,000	43,000	64,000	86,000	107,000	214,000	322,000	429,000
12	26,000	51,000	77,000	103,000	129,000	257,000	386,000	514,000
14	30,000	60,000	90,000	120,000	150,000	300,000	450,000	600,000
16	34,000	69,000	103,000	137,000	171,000	343,000	514,000	686,000
18	39,000	77,000	116,000	154,000	193,000	386,000	579,000	772,000
20	43,000	86,000	129,000	171,000	214,000	429,000	643,000	857,000
22	47,000	94,000	141,000	189,000	236,000	472,000	707,000	943,000
24	51,000	103,000	154,000	206,000	257,000	514,000	772,000	1,029,000
26	56,000	111,000	167,000	223,000	279,000	557,000	836,000	1,115,000
28	60,000	120,000	180,000	240,000	300,000	600,000	900,000	1,200,000
30	64,000	129,000	193,000	257,000	322,000	643,000	965,000	1,286,000

Assume two lane roadway with 50/50 directional split of base year AADT
Values within "box" assume a low volume mix of trucks

Table 5F-1.08: Base Year Design ESALs for Two Lane *Flexible Pavement*

	Two Way, Base Year AADT							
% Trucks	***1,000***	***2,000***	***3,000***	***4,000***	***5,000***	***10,000***	***15,000***	***20,000***
1	1,000	2,000	4,000	7,000	8,000	17,000	25,000	33,000
2	2,000	5,000	7,000	13,000	17,000	33,000	50,000	66,000
3	4,000	7,000	11,000	20,000	25,000	50,000	75,000	99,000
4	7,000	13,000	20,000	27,000	33,000	66,000	99,000	133,000
5	8,000	17,000	25,000	33,000	41,000	83,000	124,000	166,000
6	10,000	20,000	30,000	40,000	50,000	99,000	149,000	199,000
7	12,000	23,000	35,000	46,000	58,000	116,000	174,000	232,000
8	13,000	27,000	40,000	53,000	66,000	133,000	199,000	265,000
9	15,000	30,000	45,000	60,000	75,000	149,000	224,000	298,000
10	17,000	33,000	50,000	66,000	83,000	166,000	249,000	332,000
12	20,000	40,000	60,000	80,000	99,000	199,000	298,000	398,000
14	23,000	46,000	70,000	93,000	116,000	232,000	348,000	464,000
16	27,000	53,000	80,000	106,000	133,000	265,000	398,000	531,000
18	30,000	60,000	90,000	119,000	149,000	298,000	448,000	597,000
20	33,000	66,000	99,000	133,000	166,000	332,000	497,000	663,000
22	36,000	73,000	109,000	146,000	182,000	365,000	547,000	730,000
24	40,000	80,000	119,000	159,000	199,000	398,000	597,000	796,000
26	43,000	86,000	129,000	172,000	216,000	431,000	647,000	862,000
28	46,000	93,000	139,000	186,000	232,000	464,000	696,000	929,000
30	50,000	99,000	149,000	199,000	249,000	497,000	746,000	995,000

Assume two lane roadway with 50/50 directional split of base year AADT
Values within "box" assume a low volume mix of trucks

Table 5F-1.09: Base Year Design ESALs for Four Lane *Rigid Pavement*

	Two Way, Base Year AADT							
% Trucks	***2,000***	***5,000***	***10,000***	***15,000***	***20,000***	***25,000***	***30,000***	***35,000***
1	3,000	6,000	13,000	19,000	26,000	32,000	39,000	45,000
2	5,000	13,000	26,000	39,000	51,000	64,000	77,000	90,000
3	8,000	19,000	39,000	58,000	77,000	96,000	116,000	135,000
4	10,000	26,000	51,000	77,000	103,000	129,000	154,000	180,000
5	13,000	32,000	64,000	96,000	129,000	161,000	193,000	225,000
6	15,000	39,000	77,000	116,000	154,000	193,000	231,000	270,000
7	18,000	45,000	90,000	135,000	180,000	225,000	270,000	315,000
8	21,000	51,000	103,000	154,000	206,000	257,000	309,000	360,000
9	23,000	58,000	116,000	174,000	231,000	289,000	347,000	405,000
10	26,000	64,000	129,000	193,000	257,000	322,000	386,000	450,000
12	31,000	77,000	154,000	231,000	309,000	386,000	463,000	540,000
14	36,000	90,000	180,000	270,000	360,000	450,000	540,000	630,000
16	41,000	103,000	206,000	309,000	412,000	514,000	617,000	720,000
18	46,000	116,000	231,000	347,000	463,000	579,000	694,000	810,000
20	51,000	129,000	257,000	386,000	514,000	643,000	772,000	900,000
22	57,000	141,000	283,000	424,000	566,000	707,000	849,000	990,000
24	62,000	154,000	309,000	463,000	617,000	772,000	926,000	1,080,000
26	67,000	167,000	334,000	502,000	669,000	836,000	1,003,000	1,170,000
28	72,000	180,000	360,000	540,000	720,000	900,000	1,080,000	1,260,000
30	77,000	193,000	386,000	579,000	772,000	965,000	1,157,000	1,350,000

Table 5F-1.10: Base Year Design ESALs for Four Lane *Flexible Pavement*

	Two Way, Base Year AADT							
% Trucks	***2,000***	***5,000***	***10,000***	***15,000***	***20,000***	***25,000***	***30,000***	***35,000***
1	2,000	5,000	10,000	15,000	20,000	25,000	30,000	35,000
2	4,000	10,000	20,000	30,000	40,000	50,000	60,000	70,000
3	6,000	15,000	30,000	45,000	60,000	75,000	90,000	104,000
4	8,000	20,000	40,000	60,000	80,000	99,000	119,000	139,000
5	10,000	25,000	50,000	75,000	99,000	124,000	149,000	174,000
6	12,000	30,000	60,000	90,000	119,000	149,000	179,000	209,000
7	14,000	35,000	70,000	104,000	139,000	174,000	209,000	244,000
8	16,000	40,000	80,000	119,000	159,000	199,000	239,000	279,000
9	18,000	45,000	90,000	134,000	179,000	224,000	269,000	313,000
10	20,000	50,000	99,000	149,000	199,000	249,000	298,000	348,000
12	24,000	60,000	119,000	179,000	239,000	298,000	358,000	418,000
14	28,000	70,000	139,000	209,000	279,000	348,000	418,000	487,000
16	32,000	80,000	159,000	239,000	318,000	398,000	478,000	557,000
18	36,000	90,000	179,000	269,000	358,000	448,000	537,000	627,000
20	40,000	99,000	199,000	298,000	398,000	497,000	597,000	696,000
22	44,000	109,000	219,000	328,000	438,000	547,000	657,000	766,000
24	48,000	119,000	239,000	358,000	478,000	597,000	716,000	836,000
26	52,000	129,000	259,000	388,000	517,000	647,000	776,000	905,000
28	56,000	139,000	279,000	418,000	557,000	696,000	836,000	975,000
30	60,000	149,000	298,000	448,000	597,000	746,000	895,000	1,045,000

Assumes four lane roadway with 50/50 directional split of two way base year ADT and 60% of trucks in design lane

Table 5F-1.11: Growth Factor

Design Period Years (n)	Average Annual Traffic Growth Rate, Percent (r)					
	No Growth	*1%*	*2%*	*3%*	*4%*	*5%*
1	1.0	1.0	1.0	1.0	1.0	1.0
2	2.0	2.0	2.0	2.0	2.0	2.1
3	3.0	3.0	3.1	3.1	3.1	3.2
4	4.0	4.1	4.1	4.2	4.2	4.3
5	5.0	5.1	5.2	5.3	5.4	5.5
6	6.0	6.2	6.3	6.5	6.6	6.8
7	7.0	7.2	7.4	7.7	7.9	8.1
8	8.0	8.3	8.6	8.9	9.2	9.5
9	9.0	9.4	9.8	10.2	10.6	11.0
10	10.0	10.5	10.9	11.5	12.0	12.6
11	11.0	11.6	12.2	12.8	13.5	14.2
12	12.0	12.7	13.4	14.2	15.0	15.9
13	13.0	13.8	14.7	15.6	16.6	17.7
14	14.0	14.9	16.0	17.1	18.3	19.6
15	15.0	16.1	17.3	18.6	20.0	21.6
16	16.0	17.3	18.6	20.2	21.8	23.7
17	17.0	18.4	20.0	21.8	23.7	25.8
18	18.0	19.6	21.4	23.4	25.6	28.1
19	19.0	20.8	22.8	25.1	27.7	30.5
20	20.0	22.0	24.3	26.9	29.8	33.1
21	21.0	23.2	25.8	28.7	32.0	35.7
22	22.0	24.5	27.3	30.5	34.2	38.5
23	23.0	25.7	28.8	32.5	36.6	41.4
24	24.0	27.0	30.4	34.4	39.1	44.5
25	25.0	28.2	32.0	36.5	41.6	47.7
26	26.0	29.5	33.7	38.6	44.3	51.1
27	27.0	30.8	35.3	40.7	47.1	54.7
28	28.0	32.1	37.1	42.9	50.0	58.4
29	29.0	33.5	38.8	45.2	53.0	62.3
30	30.0	34.8	40.6	47.6	56.1	66.4
31	31.0	36.1	42.4	50.0	59.3	70.8
32	32.0	37.5	44.2	52.5	62.7	75.3
33	33.0	38.9	46.1	55.1	66.2	80.1
34	34.0	40.3	48.0	57.7	69.9	85.1
35	35.0	41.7	50.0	60.5	73.7	90.3
36	36.0	43.1	52.0	63.3	77.6	95.8
37	37.0	44.5	54.0	66.2	81.7	101.6
38	38.0	46.0	56.1	69.2	86.0	107.7
39	39.0	47.4	58.2	72.2	90.4	114.1
40	40.0	48.9	60.4	75.4	95.0	120.8
41	41.0	50.4	62.6	78.7	99.8	127.8
42	42.0	51.9	64.9	82.0	104.8	135.2
43	43.0	53.4	67.2	85.5	110.0	143.0
44	44.0	54.9	69.5	89.0	115.4	151.1
45	45.0	56.5	71.9	92.7	121.0	159.7
46	46.0	58.0	74.3	96.5	126.9	168.7
47	47.0	59.6	76.8	100.4	132.9	178.1
48	48.0	61.2	79.4	104.4	139.3	188.0
49	49.0	62.8	81.9	108.5	145.8	198.4
50	50.0	64.5	84.6	112.8	152.7	209.3

Growth Factor = $\frac{[(1+r)^n]-1}{r}$ for values of n > 0

D. Determining Pavement Thickness

Once the ESALs have been determined, the pavement thickness may be determined by comparing the calculated ESAL value to Tables 5F-1.13 through 5F-1.18. These tables provide recommended pavement thicknesses for various subgrade conditions, roadway types, and pavement types. Due to established policies in many jurisdictions across the state, the minimum pavement thickness for streets on natural subgrade was set at 7 inches for rigid pavement and 8 inches for flexible pavement. For pavements with a granular subbase, the minimum thickness was set at 6 inches for both pavement types. As noted in the thickness tables, whenever a thickness was calculated that was less than the minimum, the minimum was used.

Tables 5F-1.13 through 5F-1.18 were developed according to the guidelines of the AASHTO Design Guide. The AASHTO pavement design methodology is based upon the results of the AASHO Road Test, which was a series of full scale experiments conducted in Illinois in the 1950s. The design methodology that grew out of the Road Test considers numerous factors that affect the performance of a pavement. Table 5F-1.12 describes the assumptions used in the development of the pavement thickness tables. An explanation of each variable, as well as a recommended range, is provided in the AASHTO Guide.

For projects with unique conditions such as unusual soils, high truck volumes, significant drainage problems, or where specialized subgrade or subbase treatments are utilized, a special design may be warranted. The values in the tables above have been selected to represent typical conditions. An effort has been made not to be overly conservative in the establishment of the design parameters. For this reason, the designer is cautioned against deviating from the values presented in the tables above unless materials testing and/or project site conditions warrant such deviation.

Table 5F-1.12: Parameter Assumptions Used for Pavement Thickness Design Tables

Subbase:	**Natural**			**4" Granular**			**6" Granular**			**8" Granular**			**10" Granular**			**12" Granular**		
CBR Value:	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**	**3**	**5**	**10**
Rigid Pavement Parameters																		
Initial Serviceability Index, P_o	4.5																	
Terminal Serviceability Index, P_t	Local Roads = 2.00 Collector Roads = 2.25 Arterials = 2.50																	
Reliability, R	Local Roads = 80% Collector Roads = 88% Arterial Roads = 95%																	
Overall Standard Deviation, S_o	0.35																	
Loss of Support, LS	1			0														
Soil Resilient Modulus, M_R Per NCHRP Project 128 $M_R = 1941.488 \times CBR^{0.6844709}$	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400
Subbase Resilient Modulus, E_{SB} * Assumed	Not Applicable			30,000														
Modulus of Subgrade Reaction, k, and Composite Modulus of Subgrade Reaction, k_c Use AASHTO Chapter 3, Table 3.2 and Figures 3.3 - 3.6 to determine	252	327	469	263	332	455	284	354	477	308	379	504	332	406	535	356	433	566
Adjusted k or k_c for Loss of Support Use AASHTO Part 2, Figure 3.6	85	105	160	263	332	455	284	354	477	308	379	504	332	406	535	356	433	566
Coefficient of Drainage, C_d	1.00			1.10														
Modulus of Rupture, S'_c $S'_c = 2.3 \times f_c^{0.667}$ * Assumed 4,000 psi concrete	580																	
Modulus of Elasticity, E_c $E_c = 6,750 \times S'_c$ * Assumed 4,000 psi concrete	3,915,000																	
Load Transfer, J	J = 3.1 (Pavement Thickness < 8") J = 2.7 (Pavement Thickness ≥ 8")																	
Flexible Pavement Parameters																		
Initial Serviceability Index, P_o	4.2																	
Terminal Serviceability Index, P_t	Local Roads = 2.00 Collector Roads = 2.25 Arterials = 2.50																	
Reliability, R	Local Roads = 80% Collector Roads = 88% Arterial Roads = 95%																	
Overall Standard Deviation, S_o	0.45																	
Layer Coefficients	Surface/Intermediate Course = 0.44 Base Course = 0.40 Granular Subbase = 0.14																	
Soil Resilient Modulus, M_R Per NCHRP Project 128 $M_R = 1941.488 \times CBR^{0.6844709}$	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400	4120	5840	9400
Effective Soil Reslient Modulus, MR Use AASHTO Chapter 2, Figure 2.3 to determine	2460	3480	5580	2460	3480	5580	2460	3480	5580	2460	3480	5580	2460	3480	5580	2460	3480	5580
Coefficient of Drainage, M_i	1.00			1.15														

The following flowchart depicts a summary of the analysis process.

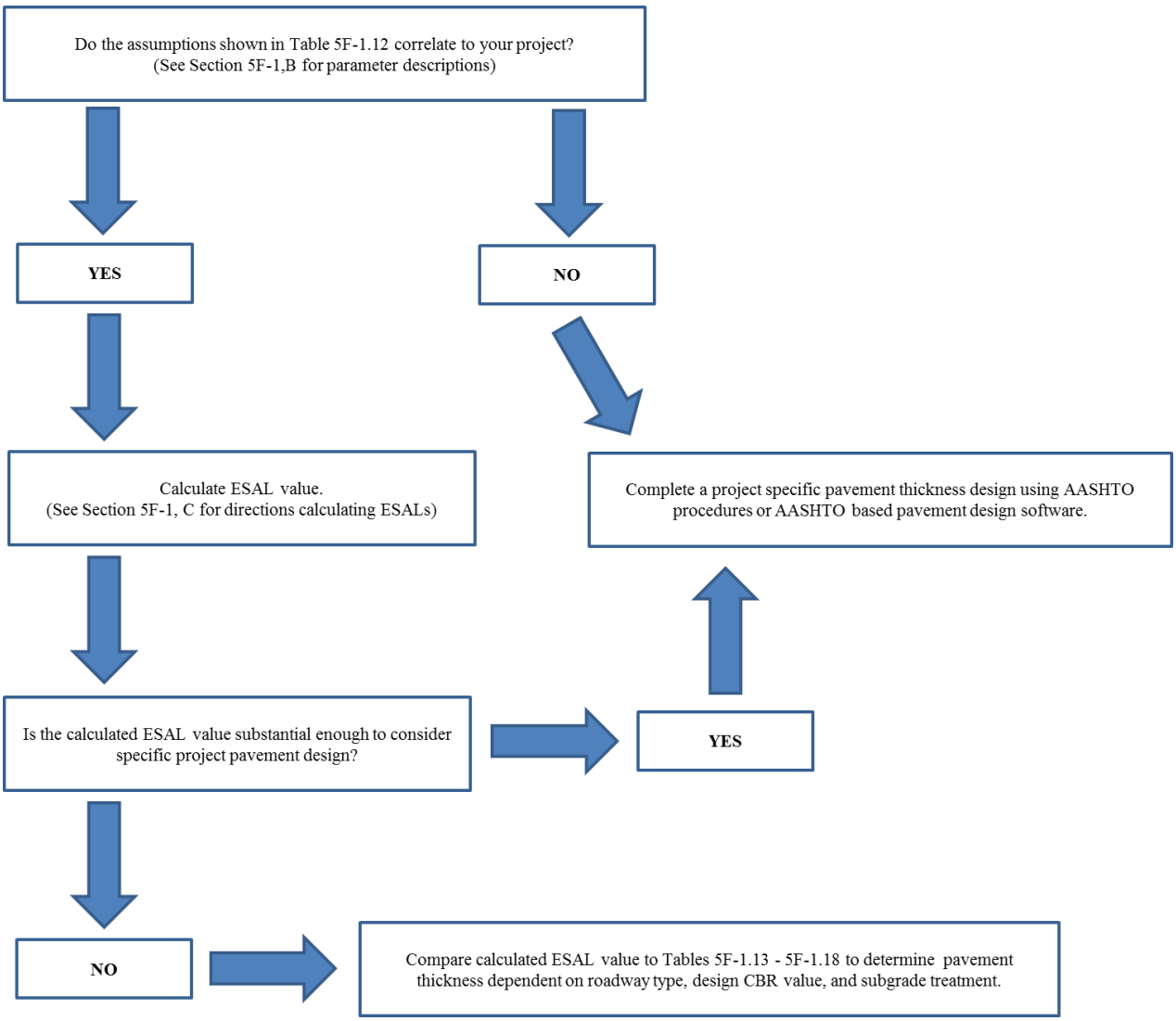


Table 5F-1.13: Required Thickness for Rigid Pavement - *Local Roads*

Subbase	Natural Subgrade			4" Granular			6" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10
300,000	7**	7**	7**	6*	6*	6*	6*	6*	6*
500,000	7**	7**	7**	6*	6*	6*	6*	6*	6*
750,000	7	7**	7**	6*	6*	6*	6*	6*	6*
1,000,000	7	7	7	6	6	6*	6	6*	6*
1,500,000	7.5	7.5	7.5	6.5	6.5	6	6.5	6	6
2,000,000	8	8	7.5	7	6.5	6.5	7	6.5	6.5
3,000,000	8	8	8	7.5	7	7	7.5	7	7
4,000,000	8	8	8	7.5	7.5	7.5	7.5	7.5	7
5,000,000	8.5	8.5	8	8	8	7.5	8	8	7.5

***** The value shown represents a 6 inch minimum; the actual value is less.
****** The value shown represents a 7 inch minimum; the actual value is less.

Table 5F-1.14: Required Thickness for Rigid Pavement - *Collector Roads*

Subbase	Natural Subgrade			4" Granular			6" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10
750,000	7	7	7	6	6	6*	6	6	6*
1,000,000	7.5	7.5	7.5	6.5	6	6	6.5	6	6
1,500,000	8	8	8	7	6.5	6.5	7	6.5	6.5
2,000,000	8	8	8	7.5	7	7	7	7	7
3,000,000	8.5	8	8	8	7.5	7.5	8	7.5	7.5
4,000,000	8.5	8.5	8.5	8	8	8	8	8	8
5,000,000	9	9	8.5	8	8	8	8	8	8
7,500,000	9.5	9.5	9.5	8.5	8.5	8	8.5	8	8
10,000,000	10	10	9.5	9	8.5	8.5	9	8.5	8.5
12,500,000	10.5	10	10	9	9	9	9	9	9
15,000,000	10.5	10.5	10.5	9.5	9.5	9	9.5	9.5	9
17,500,000	11	10.5	10.5	9.5	9.5	9.5	9.5	9.5	9.5
20,000,000	11	11	11	10	10	9.5	10	9.5	9.5

***** The value shown represents a 6 inch minimum; the actual value is less.

Table 5F-1.15: Required Thickness for Rigid Pavements - *Arterial Roads*

Subbase	Natural Subgrade			4" Granular			6" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10
1,000,000	8	8	8	7	7	6.5	7	6.5	6.5
1,500,000	8	8	8	7.5	7.5	7	7.5	7.5	7
2,000,000	8.5	8.5	8	8	8	7.5	8	7.5	7.5
3,000,000	9	9	8.5	8	8	8	8	8	8
4,000,000	9.5	9	9	8.5	8	8	8	8	8
5,000,000	9.5	9.5	9.5	8.5	8.5	8	8.5	8.5	8
7,500,000	10	10	10	9	9	9	9	9	9
10,000,000	10.5	10.5	10.5	9.5	9.5	9	9.5	9.5	9
12,500,000	11	11	11	10	10	9.5	10	10	9.5
15,000,000	11.5	11.5	11	10.5	10	10	10	10	10
17,500,000	11.5	11.5	11.5	10.5	10.5	10	10.5	10.5	10
20,000,000	12	12	11.5	10.5	10.5	10.5	10.5	10.5	10.5
22,500,000	12	12	12	11	11	10.5	11	11	10.5
25,000,000	12	12	12	11	11	11	11	11	10.5
27,500,000	---	---	12	11.5	11	11	11	11	11
30,000,000	---	---	---	11.5	11.5	11	11.5	11.5	11

Table 5F-1.16: Required Thickness for Flexible Pavements - *Local Roads*

Subbase	Natural Subgrade			4" Granular			6" Granular			8" Granular			10" Granular			12" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10
300,000	9.5	8.5	8**	8	7	6*	7	6	6*	6.5	6*	6*	6*	6*	6*	6*	6*	6*
500,000	10	9	8	8.5	7.5	6	8	7	6*	7	6	6*	6.5	6*	6*	6*	6*	6*
750,000	10.5	9.5	8	9	8	6.5	8.5	7.5	6	7.5	6.5	6*	7	6*	6*	6	6*	6*
1,000,000	11	10	8.5	9.5	8.5	7	9	7.5	6	8	7	6*	7.5	6	6*	6.5	6*	6*
1,500,000	11.5	10.5	9	10	9	7.5	9.5	8	7	8.5	7.5	6	8	6.5	6*	7	6	6*
2,000,000	12	11	9.5	10.5	9.5	8	10	8.5	7	9	8	6.5	8.5	7	6*	7.5	6.5	6*
3,000,000	---	11.5	10	11	10	8.5	10.5	9	7.5	9.5	8.5	7	9	8	6	8	7	6*
4,000,000	---	12	10.5	11.5	10.5	9	11	9.5	8	10	9	7.5	9.5	8	6.5	8.5	7.5	6*
5,000,000	---	---	10.5	12	11	9	11.5	10	8.5	10.5	9	7.5	10	8.5	7	9	8	6

***** The value shown represents a 6 inch minimum; the actual value is less.
****** The value shown represents an 8 inch minimum; the actual value is less.

Table 5F-1.17: Required Thickness for Flexible Pavements - *Collector Roads*

Subbase	Natural Subgrade			4" Granular			6" Granular			8" Granular			10" Granular			12" Granula		
ESAL/CBR	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10	3	5	1
750,000	11.5	10.5	9	10	9	7.5	9	8	6.5	8.5	7.5	6*	8	6.5	6*	7	6*	6
1,000,000	12	10.5	9	10.5	9	7.5	9.5	8.5	7	9	7.5	6	8	7	6*	7.5	6	6
1,500,000	12.5	11.5	9.5	11	10	8	10.5	9	7.5	9.5	8.5	6.5	9	7.5	6	8	7	6
2,000,000	13	12	10	11.5	10.5	8.5	11	9.5	8	10	8.5	7	9.5	8	6.5	8.5	7.5	6
3,000,000	14	12.5	10.5	12.5	11	9	11.5	10	8.5	11	9.5	8	10	8.5	7	9	8	
4,000,000	---	13	11	13	11.5	9.5	12	10.5	9	11.5	10	8	10.5	9	7.5	10	8.5	6
5,000,000	---	13.5	11.5	13.5	12	10	12.5	11	9	11.5	10.5	8.5	11	9.5	8	10	8.5	
7,500,000	---	14	12	14	12.5	10.5	13.5	12	10	12.5	11	9	11.5	10	8.5	11	9.5	7
10,000,000	---	---	12.5	---	13	11	14	12.5	10.5	13	11.5	9.5	12.5	10.5	9	11.5	10	
12,500,000	---	---	13	---	13.5	11.5	---	12.5	10.5	13.5	12	10	13	11	9	12	10.5	8
15,000,000	---	---	13.5	---	14	12	---	13	11	14	12.5	10.5	13	11.5	9.5	12.5	10.5	8
17,500,000	---	---	13.5	---	14	12	---	13.5	11.5	---	12.5	10.5	13.5	12	10	12.5	11	
20,000,000	---	---	14	---	---	12.5	---	13.5	11.5	---	13	11	14	12	10	13	11.5	

***** The value shown represents a 6 inch minimum; the actual value is less.

Table 5F-1.18: Required Thickness for Flexible Pavements - *Arterial Roads*

Subbase	Natural Subgrade			4" Granular			6" Granular			8" Granular			10" Granular			12" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10
1,000,000	13.5	12	10	11.5	10.5	8.5	11	9.5	8	10	9	7	9.5	8	6.5	8.5	7.5	5.5
1,500,000	14	12.5	11	12.5	11	9	11.5	10.5	8.5	11	9.5	8	10	8.5	7	9.5	8	6
2,000,000	---	13	11	13	11.5	9.5	12.5	11	9	11.5	10	8	10.5	9	7.5	10	8.5	6.5
3,000,000	---	14	12	14	12.5	10.5	13	11.5	9.5	12.5	10.5	9	11.5	10	8	10.5	9	7.5
4,000,000	---	---	12.5	---	13	11	13.5	12	10	13	11	9.5	12	10.5	8.5	11.5	9.5	8
5,000,000	---	---	13	---	13.5	11	14	12.5	10.5	13.5	11.5	9.5	12.5	11	9	11.5	10	8
7,500,000	---	---	13.5	---	14	12	---	13.5	11	14	12.5	10.5	13.5	11.5	9.5	12.5	11	9
10,000,000	---	---	14	---	---	12.5	---	14	11.5	---	13	11	14	12.5	10	13.5	11.5	9.5
12,500,000	---	---	---	---	---	13	---	---	12	---	13.5	11.5	---	13	10.5	14	12	10
15,000,000	---	---	---	---	---	13.5	---	---	12.5	---	14	11.5	---	13	11	14	12.5	10
17,500,000	---	---	---	---	---	13.5	---	---	13	---	---	12	---	13.5	11	---	12.5	10.5
20,000,000	---	---	---	---	---	14	---	---	13	---	---	12.5	---	14	11.5	---	13	10.5
22,500,000	---	---	---	---	---	14	---	---	13.5	---	---	12.5	---	14	11.5	---	13.5	11
25,000,000	---	---	---	---	---	---	---	---	13.5	---	---	12.5	---	---	12	---	13.5	11
27,500,000	---	---	---	---	---	---	---	---	13.5	---	---	13	---	---	12	---	14	11.5
30,000,000	---	---	---	---	---	---	---	---	14	---	---	13	---	---	12.5	---	14	11.5

E. Example Pavement Thickness Design Calculations

1. Two Lane Local Roadway, PCC

AADT = 1,000
Trucks = 2%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.07) = 3,000
Growth Factor (from Table 5F-1.11) = 84.6
3,000 ESALs X 84.6 = 253,800 ESALs

By referring to Table 5F-1.13 and rounding up the ESAL calculation to 300,000 (see below), the pavement thickness alternatives are either 6 inches or 7 inches depending on the presence of granular subbase.

Subbase	Natural Subgrade			4" Granular			6" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10
300,000	7**	7**	7**	6*	6*	6*	6*	6*	6*
500,000	7**	7**	7**	6*	6*	6*	6*	6*	6*
750,000	7	7**	7**	6*	6*	6*	6*	6*	6*

***** The value shown represents a 6 inch minimum; the actual value is less.
****** The value shown represents a 7 inch minimum; the actual value is less.

2. Two Lane Local Roadway, HMA

AADT = 1,000
Trucks = 2%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.08) = 2,000
Growth Factor (from Table 5F-1.1) = 84.6
2,000 ESALs X 84.6 = 169,200 ESALs

By referring to Table 5F-1.16 and rounding up the ESAL calculation to 300,000 (see below), the pavement thickness alternatives range from 6 inches to 9.5 inches depending on the CBR value and subbase treatment selected.

Subbase	Natural Subgrade			4" Granular			6" Granular			8" Granular			10" Granular			12" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10
300,000	9.5	8.5	8**	8	7	6*	7	6	6*	6.5	6*	6*	6*	6*	6*	6*	6*	6*
500,000	10	9	8	8.5	7.5	6	8	7	6*	7	6	6*	6.5	6*	6*	6*	6*	6*
750,000	10.5	9.5	8	9	8	6.5	8.5	7.5	6	7.5	6.5	6*	7	6*	6*	6	6*	6*

***** The value shown represents the 6 inch minimum; the actual value is less.
****** The value shown represents an 8 inch minimum; the actual value is less.

3. Two Lane Collector Roadway, PCC

AADT = 5,000
Trucks = 4%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.07) = 43,000
Growth Factor (from Table 5F-1.11) = 84.6
43,000 ESALs X 84.6 = 3,637,800 ESALs

By referring to Table 5F-1.14 and rounding up the ESAL calculation to 4,000,000 (see below), the pavement thickness alternatives range from 8 inches to 8.5 inches depending on the CBR value and subbase treatment selected.

Subbase	Natural Subgrade			4" Granular			6" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10
750,000	7	7	7	6	6	6*	6	6	6*
1,000,000	7.5	7.5	7.5	6.5	6	6	6.5	6	6
1,500,000	8	8	8	7	6.5	6.5	7	6.5	6.5
2,000,000	8	8	8	7.5	7	7	7	7	7
3,000,000	8.5	8	8	8	7.5	7.5	8	7.5	7.5
4,000,000	8.5	8.5	8.5	8	8	8	8	8	8
5,000,000	9	9	8.5	8	8	8	8	8	8

***** The value shown represents a 6 inch minimum; the actual value is less.

4. Two Lane Collector Roadway, HMA

AADT = 5,000
Trucks = 4%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.08) = 33,000
Growth Factor (from Table 5F-1.11) = 84.6
33,000 ESALs X 84.6 = 2,791,800 ESALs

By referring to Table 5F-1.17 and rounding up the ESAL calculation to 3,000,000 (see below), the pavement thickness alternatives range from 6 inches to 14 inches depending on the CBR value and subbase treatment selected.

Subbase	Natural Subgrade			4" Granular			6" Granular			8" Granular			10" Granular			12" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10	3	5	1
750,000	11.5	10.5	9	10	9	7.5	9	8	6.5	8.5	7.5	6*	8	6.5	6*	7	6*	6
1,000,000	12	10.5	9	10.5	9	7.5	9.5	8.5	7	9	7.5	6	8	7	6*	7.5	6	6
1,500,000	12.5	11.5	9.5	11	10	8	10.5	9	7.5	9.5	8.5	6.5	9	7.5	6	8	7	6
2,000,000	13	12	10	11.5	10.5	8.5	11	9.5	8	10	8.5	7	9.5	8	6.5	8.5	7.5	6
3,000,000	14	12.5	10.5	12.5	11	9	11.5	10	8.5	11	9.5	8	10	8.5	7	9	8	(
4,000,000	---	13	11	13	11.5	9.5	12	10.5	9	11.5	10	8	10.5	9	7.5	10	8.5	6

***** The value shown represents the 6 inch minimum; the actual value is less.

5. Four Lane Arterial Roadway, PCC

AADT = 15,000
Trucks = 5%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.09) = 96,000
Growth Factor (from Table 5F-1.11) = 84.6
96,000 ESALs X 84.6 = 8,121,600 ESALs

By referring to Table 5F-1.15 and rounding up the ESAL calculation to 10,000,000 (see below), the pavement thickness alternatives range from 9 inches to 10.5 inches depending on the CBR value and subbase treatment selected.

Subbase	Natural Subgrade			4" Granular			6" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10
1,000,000	8	8	8	7	7	6.5	7	6.5	6.5
1,500,000	8	8	8	7.5	7.5	7	7.5	7.5	7
2,000,000	8.5	8.5	8	8	8	7.5	8	7.5	7.5
3,000,000	9	9	8.5	8	8	8	8	8	8
4,000,000	9.5	9	9	8.5	8	8	8	8	8
5,000,000	9.5	9.5	9.5	8.5	8.5	8	8.5	8.5	8
7,500,000	10	10	10	9	9	9	9	9	9
10,000,000	10.5	10.5	10.5	9.5	9.5	9	9.5	9.5	9
12,500,000	11	11	11	10	10	9.5	10	10	9.5

6. Four Lane Arterial Roadway, HMA

AADT = 15,000
Trucks = 5%
Annual Growth Rate = 2%
Design Period = 50 years

Base Year Design ESALs (from Table 5F-1.10) = 75,000
Growth Factor (from Table 5F-1.11) = 84.6
75,000 ESALs X 84.6 = 6,345,000 ESALs

By referring to Table 5F-1.18 and rounding up the ESAL calculation to 7,500,000 (see below), the pavement thickness alternatives range from 9 inches to 14 inches depending on the CBR value and subbase treatment selected.

Subbase	Natural Subgrade			4" Granular			6" Granular			8" Granular			10" Granular			12" Granular		
ESAL/CBR	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10	3	5	10
1,000,000	13.5	12	10	11.5	10.5	8.5	11	9.5	8	10	9	7	9.5	8	6.5	8.5	7.5	5.5
1,500,000	14	12.5	11	12.5	11	9	11.5	10.5	8.5	11	9.5	8	10	8.5	7	9.5	8	6
2,000,000	---	13	11	13	11.5	9.5	12.5	11	9	11.5	10	8	10.5	9	7.5	10	8.5	6.5
3,000,000	---	14	12	14	12.5	10.5	13	11.5	9.5	12.5	10.5	9	11.5	10	8	10.5	9	7.5
4,000,000	---	---	12.5	---	13	11	13.5	12	10	13	11	9.5	12	10.5	8.5	11.5	9.5	8
5,000,000	---	---	13	---	13.5	11	14	12.5	10.5	13.5	11.5	9.5	12.5	11	9	11.5	10	8
7,500,000	---	---	13.5	---	14	12	---	13.5	11	14	12.5	10.5	13.5	11.5	9.5	12.5	11	9
10,000,000	---	---	14	---	---	12.5	---	14	11.5	---	13	11	14	12.5	10	13.5	11.5	9.5

F. References

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Van Til, C.J., et al. *NCHRP Report 1-28: Evaluation of AASHO Interim Guides for Design of Pavement Structures* (AASHTO Design Guide). The American Association of State Highway and Transportation Officials. 444 North Capitol St NW Suite 225, Washington, D. C.1972.

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General Information for Joints

A. General Information

The need for a jointing system in concrete pavements results from the desire to control the location and geometry of transverse and longitudinal cracking. Without jointing, uncontrolled cracking occurs due to stresses in the pavement from shrinkage, temperature and moisture differentials, and applied traffic loadings.

A good jointing plan will ease construction by providing clear guidance. The development of a jointing plan requires the designer to think about not only the specific project requirements but also the entire project jointing system. Jointing layouts in some parts of a project can have a substantial impact on other parts. In order to control concrete pavement cracking and subsequently maintain structural integrity, designers need to develop an understanding of how to complete jointing layouts of mainline pavements and intersections to obtain a comprehensive jointing system. This will allow a check on the pattern, type of joints, and matching joints to their purpose.

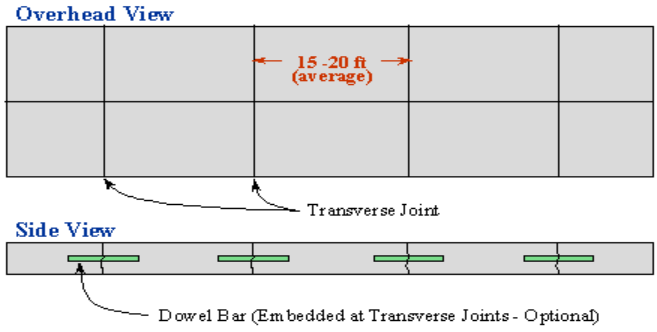
There are three types of jointing systems for concrete pavements, including:

- Jointed plain concrete pavement
- Jointed reinforced concrete pavement
- Continuously reinforced concrete pavement

This section deals primarily with jointed plain concrete pavements (JPCP) with tie bars or dowel bars only at joints as shown in Figure 5G-1.01. The function of the bars in JPCP is to provide load transfer across the joints, either through tie bars that hold the adjacent slabs together and maintain aggregate interlock or through dowel bars that provide mechanical load transfer even with slab movement.

Some cities specify jointed reinforced concrete pavements (JRCP), sometimes referred to as distributed steel reinforcing pavements. Section 5G-2 discusses jointed reinforced pavements. Jointed reinforced pavements allow for longer spacing between transverse joints by utilizing bar mats to hold midpanel cracks together and maintain structural integrity of the slab. Jointed reinforced pavements should not be confused with continuously reinforced concrete pavement, CRCP, which has very few or no joints.

Figure 5G-1.01: Jointed Plain Concrete Pavement (JPCP)



The primary benefits of jointing include:

- Crack control.
- Accommodating slab movements.
- Providing desirable load transfer.

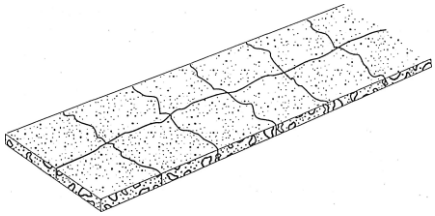
Secondary benefits of jointing include:

- Dividing the pavement into practical construction increments (i.e. traffic lanes, pavement widening).
- Providing traffic guidance.

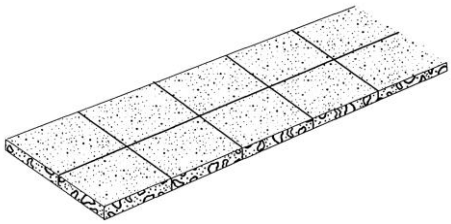
B. Crack Development

Crack development results from stress that exceeds the strength of the concrete due to concrete drying shrinkage, subgrade restraint, temperature/moisture differentials, applied traffic loads, and the combined effects of restrained curling and warping. It is highly desirable to control the location and geometry of transverse and longitudinal cracking in pavements by using properly designed and constructed joints. Without this control, cracking occurs in a random pattern similar to Figure 5G-1.02.

Figure 5G-1.02: Effect of Jointing on Crack Control



Crack Pattern Without Jointing



Properly Jointed Pavement

Cracking can be broken into two categories - initial and mature.

1. **Initial Cracking:** Initial cracking occurs within a few hours to a few months after the pavement has been placed. It may be caused by the following conditions.

    a. **Concrete Shrinkage (loss of volume):** Concrete shrinkage is caused by contraction of concrete from the following.

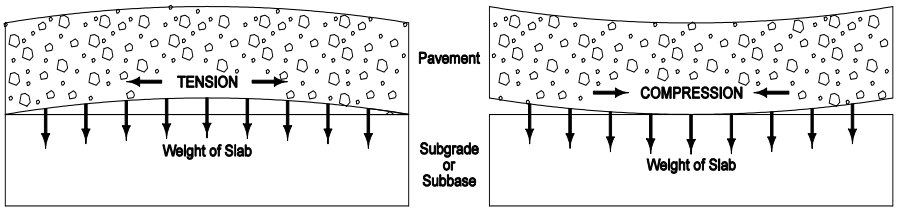
        1) **Temperature Change During Hydration:** The heat of hydration and temperature of pavement normally peak a short time after final set. After peaking, the temperature of concrete declines due to reduced hydration activity and lower air temperature during the first night of pavement life. As the temperature of concrete drops, the concrete contracts or shrinks. If severe air temperature changes occur within the first few hours after construction, high tensile stresses may cause transverse cracking to occur.

        2) **Loss of Water During Hydration (drying shrinkage):** Drying shrinkage results from the reduction of volume through loss of mix water. Concrete mixes for roadway applications require more mix water than is required for hydration (water consumed through chemical reactions with cement). The extra water helps provide adequate workability for placing and finishing operations. During consolidation and hardening, most of the excess water bleeds to the surface and evaporates. With the loss of the water, the concrete has less volume.

b. Subgrade and Subbase Restraint: Subgrade or subbase friction resists the contraction of the pavement from reduced volume and temperature. This resistance produces tensile stresses within the concrete.

c. Curling and Warping: Curling is the result of temperature changes through the depth of the pavement. Daytime curling occurs when the top portion of the slab is at a higher temperature than the bottom portion. Because of the higher temperature, the top expands more than the bottom, causing the tendency to curl. Subgrade and subbase friction and the weight of the slab are factors that help to counteract the daytime curling. During the night, the effects of curling are reversed. See Figure 5G-1.03.

Figure 5G-1.03: Daytime and Nighttime Curling



Daytime Curling Nighttime Curling

Warping results from a moisture differential from the top to the bottom of the slab. The top of the slab is normally drier than the bottom. The decrease in moisture content causes contraction at the top of the slab, which helps to counteract daytime curling. This contraction causes stresses in the concrete, which can lead to cracking.

2. Mature Cracking: Mature cracking occurs several months or years after pavement is placed. As traffic loads are applied to the pavement, along with temperature and moisture changes, tensile strain/stress will develop in concrete as the result of:

a. Curling and warping in combination with repetitive traffic loads.

b. Poorly designed and constructed pavement joints that do not provide proper load transfer across the joints and pavement slab.

c. Poor foundation support due to unsuitable or non-uniform soils and excessive subgrade moisture.

C. Crack Control

Cracking can be minimized by the following:

1. Properly designed and constructed joints and joint layout that account for load transfer.

2. Proper timing of sawing of joints.

3. Sawing of joints in the correct locations.

4. Proper curing of concrete to prevent high initial shrinkage and cracking of hardening concrete.

5. Constructing a quality foundation with uniform, stable subgrade, drainable subbase, and longitudinal subdrains. See Chapter 6 - Geotechnical for additional guidance.

D. Considerations for Good Pavement Jointing

In order to design a suitable pavement jointing system, the following considerations have been included in the jointing layout steps covered in this design section. The following elements need to be considered for adequate jointing:

1. **Joint Purpose:** Transverse and longitudinal joints are used to control cracking of the pavement by relieving internal curing and loading stresses. Transverse joints serve to control cracking resulting from contraction of the pavement.

2. **Climate and Environmental Conditions:** Depending upon temperature and moisture changes that occur at the time of construction, expansion and contraction of the slab will occur, resulting in stress concentrations, warping, and curling.

3. **Slab Thickness:** Pavement thickness counteracts curling stresses and deflections. Thicker pavements are less prone to curling.

4. **Load Transfer:** Load transfer is desirable across any concrete pavement joint. However, the amount of load transfer provided varies for each joint type, aggregate interlock, and the type of bar.

5. **Joint Spacing vs. Thickness:** Maximum joint spacing is dependent on pavement thickness; thinner pavement requires closer joint spacing than thick pavement.

6. **Traffic:** Traffic is an extremely important consideration in joint design. Traffic classification, channelization, and, particularly, the amount of truck traffic influence the load transfer requirements for long-term performance.

7. **Concrete Material and Construction Characteristics:** Specific materials and their combinations can affect concrete strength and joint requirements. When special mixes outside of standard mixes are proposed to meet project conditions and requirements, the materials selected for the concrete can influence slab shrinkage. Substandard materials and construction practices can have a detrimental effect on joint performance. For example, poor coarse aggregate can lead to D-cracking, which initially occurs along pavement joints, or over-vibration can lead to low air content, which can lead to early deterioration of pavement joints.

8. **Subbase Type:** The support values and interface friction characteristics of different subbase types affect movement and support of the slabs.

9. **Shoulder Design:** The shoulder type (curb, tied concrete, asphalt, granular, or earth) affects edge support and ability of mainline joints to transfer load. Widened outer lanes are also effective for helping maintain load transfer.

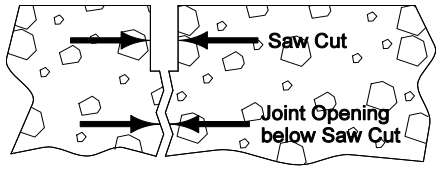
10. **Past Performance:** Performance observations and records can be used to establish standard joint design (what has worked and what has not).

E. Load Transfer

For jointed concrete pavement to perform adequately, traffic loadings must be transferred effectively from one side of the joint to the other. This is commonly referred to as load transfer, which is measured by joint effectiveness. If a joint is 100% effective, it will transfer approximately one half of the applied load. Field evaluation of load transfer is calculated by measuring the deflection on each side of a joint from the applied load. Load transfer is necessary for jointed concrete pavements to perform well. Adequate load transfer lowers deflections and reduces faulting, spalling, and corner breaks. Table 5G-1.01 shows that joint efficiency drops considerably when the joint opening below the sawcut line starts to exceed 1/8 inch.

Table 5G-1.01: Joint Opening Below the Saw Cut vs. Joint Efficiency

Joint Opening Below Saw Cut	Joint Efficiency
1/16”	> 50%
1/8”	< 50%
1/4”	0%



The following factors contribute to load transfer across joints:

- Aggregate interlock
- Mechanical load transfer devices
- Uniform, stable foundation, including quality subgrade and drainable subbase

1. Aggregate Interlock: Aggregate interlock is the interlocking action between aggregate particles at the face of the joint. It relies on the shared interaction between aggregate particles at the irregular crack face that forms below the sawcut. This form of load transfer has been found to be the most effective form of load transfer on streets with short joint spacings and low truck volumes. Increased aggregate interlock load transfer and minimized faulting will result from the following:

a. Longitudinal tiebars and/or keyways.
   1) Typically used in longitudinal contraction and construction joints.
   2) Tiebars provide little load transfer themselves, but they do hold the slabs relatively tight together to maintain aggregate interlock.

b. Shorter joint spacings (e.g. 15 feet or less).

c. Larger crushed stone in the concrete mix
   1) Larger (greater than 1 inch) aggregates are helpful in maintaining load transfer, especially for larger joint openings.
   2) Generally, crushed stone aggregates perform better for aggregate interlock than rounded aggregates because the angular aggregates create a rougher joint face.

2. Mechanical Load Transfer: Aggregate interlock alone may not always provide sufficient load transfer in transverse joints for highway pavements and streets subject to heavy truck traffic. Under these circumstances, dowel bars should be used.

a. Dowel Bar Benefits: Dowel bars are smooth round bars placed across joints to transfer loads without restricting horizontal joint movement. The benefits of dowel bars are as follows:
   1) They keep slabs in horizontal and vertical alignment.
   2) Since dowel bars span the joint, daily and seasonal joint openings do not affect load transfer across doweled joints as much as they do undoweled joints.

3) Dowel bars lower deflection and stress in concrete slabs, and reduce the potential for faulting, pumping, and corner breaks.
4) Dowel bars increase pavement life by effectively transferring the load across the joint.

b. Dowel Bar Use: Historically, dowel bars have been used to provide additional mechanical load transfer where traffic exceeds 200 trucks per day (or 100 trucks per lane), or accumulated design traffic exceeds 4 to 5 million ESALs. Typically, this truck traffic level will require at least an 8 inch thick slab, which is generally regarded as the minimum thickness to accommodate dowels.

3. **Quality Subgrades and Subbases:** A proper foundation for pavements reduces joint deflection, assists in aggregate interlock, and improves and maintains joint effectiveness under repetitive loads. Quality subgrades and subbases not only provide this support but also provide an all-weather working platform and stable smooth trackline for paving equipment. For design guidance for pavement subgrades and subbases, see Chapter 6 - Geotechnical.

It should be noted the subbase type and the subgrade support *k*-value have an effect on stresses in pavement slabs. The stiffer the foundation, the greater the slab's curl and warping stresses will be. Therefore, a shorter transverse joint spacing should be used.

4. Skewed Joints: Upon approval of the Jurisdictional Engineer, transverse contraction joints for undoweled pavements may be skewed counterclockwise (right ahead) 4 to 5 feet. Skewed joints are effective in decreasing the dynamic loading in the joint area by distributing the transfer of load. Each wheel on an axle crosses a skewed joint at a separate time. This reduces stresses and deflections in the concrete slab and helps reduce pumping and faulting. Also, the joints are 4 or 5 inches longer which increases the slab support area. The use of skewed joints is more appropriate for rural low volume roads and is not as practical for urban conditions due to the need to have right angle jointing patterns at intersections. A word of caution: If random cracks occur, they normally are at somewhat right angles and can create a pie shaped piece of pavement when they cross a skewed joint.

Types of Joints

A. Jointing

PCC pavement joints are necessary primarily to control the location of cracks that occur from natural and dynamic loading stresses. They accommodate stresses that develop from slab curling and warping due to moisture and temperature differentials and traffic loading. In addition, joints divide the pavement into suitable construction increments or elements. Standard design considerations include joint types, spacing, load transfer, and sealing. This section deals with the proper selection and layout of contraction, construction, and isolation joints.

B. Joint Spacing

Joint spacing for unreinforced concrete pavements depends on slab thickness, concrete aggregate, subgrade/subbase support, and environmental conditions. Transverse joint spacing should be limited to 24T (T is slab thickness) for pavements on subgrades and granular subbases or 21T if the pavement is placed on stabilized subbases, existing concrete, or asphalt. Transverse joint spacing is12 feet for pavements 6 inches thick, 15 feet for pavements 7 to 9 inches thick, and 20 feet for pavements over 9 inches thick. Longitudinal joint spacing for two lane streets, where lane delineation is not necessary, should be limited to a maximum of 10 feet. For multi-lane streets, where lane delineation is desired, longitudinal joint spacing is typically 10 to 13 feet. Generally, transverse joint spacing should not exceed 150% of the longitudinal joint spacing. Table 5G-2.01 provides transverse and longitudinal joint spacings for standard two lane streets.

Table 5G-2.01: Transverse Joint Requirements

Pavement Thickness	Transverse Joint Type	Transverse Joint Spacing
6”	C	12’
7”	C	15’
8”	CD[1]	15’
9”	CD[1]	15’
≥ 10”	CD[1]	20’

[1] No dowels within 24” of the back of curb

Source: SUDAS Specifications Figure 7010.901

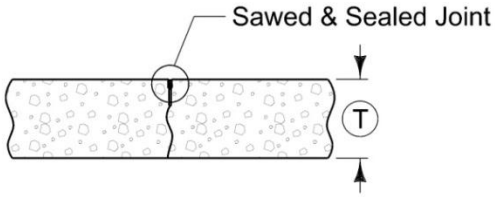
C. Joint Types

Contraction joints for concrete pavements are generally sawed. Transverse joints can be sawed with conventional sawing or early concrete sawing equipment. Longitudinal joints are formed with conventional sawing . Some joints, including construction joints, are formed. The figures in this subsection are derived from SUDAS Specifications Figure 7010.101.

1. **Transverse Contraction Joints:** Contraction joints constructed transversely across pavement lanes are spaced to control natural initial and mature cracking of the concrete pavement. Under certain conditions, such as rapidly dropping air temperature during the night, transverse cracks may occur early. Therefore, early formation of the transverse joints is required.

   a. **Plain Contraction Joints:** Plain contraction joints are normally used in local streets and minor collectors where load transfer is not a major factor. Load transfer for plain contraction joints occurs through the adjacent irregular fractured faces. Generally, they are used when the slab thickness is less than 8 inches. The joints are constructed by sawing to a depth of T/3. Plain contraction joints are sometimes used when the pavement thickness is 9 inches or greater such as at intersections in boxouts near curbs where load transfer is not a concern. Approved early concrete sawing equipment may be used to cut the joint to a depth of 1 1/4 inch. For sealing, the joint width must be a minimum of 1/4 inch wide.

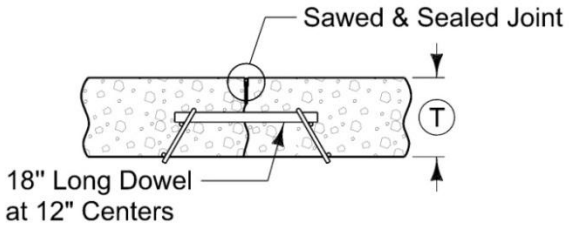
Figure 5G-2.01: 'C' Plain Contraction Joint



   b. **Doweled Contraction Joints:** Dowel bars are used to supplement the load transfer produced by aggregate interlock. The joints are sawed to a depth of T/3 and are spaced at 15 foot intervals for slab thickness of 9 inches or less and 20 feet for slabs greater than 9 inches thick. The dowels are placed at the mid-depth in the slab so they can resist shear forces as traffic loads cross the joint; thus helping reduce deflection and stress of the joint. The need for doweled contraction joints depends on subgrade/subbase support and the truck traffic loadings the roadway is to provide. They are usually used on streets or roadways where the pavement thickness is 8 inches or greater and where the pavement is subject to heavier truck traffic, generally more than 100 trucks per lane per day. Early entry concrete sawing can be used for 'CD' joints.

   Dowels should not be placed closer than 24 inches from the back of the curb on streets with quarter point or third point jointing. If gutterline jointing is used, place the first dowel in the traffic lane 6 inches from the joint.

Figure 5G-2.02: 'CD' Doweled Contraction Joint

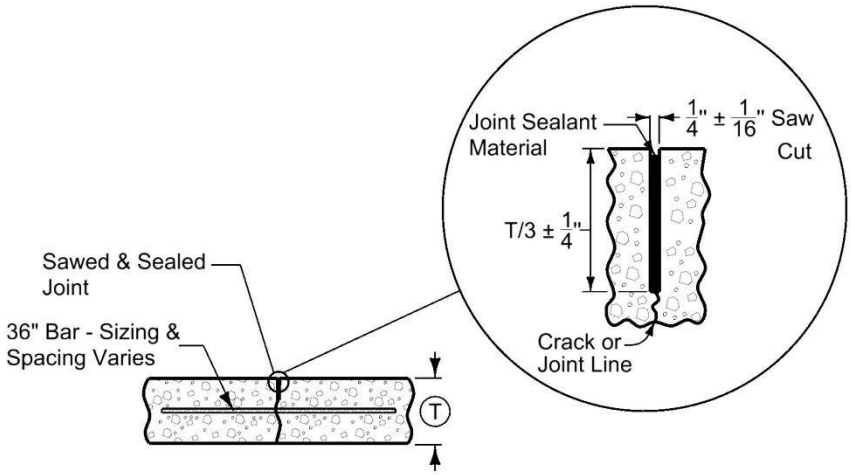


2. **Longitudinal Contraction Joints:** Longitudinal contraction joints release stresses from restrained warping and dynamic loading. Under certain conditions, such as rapidly dropping air temperature during the night, longitudinal cracks may occur early. Therefore, early formation of the joint is required.

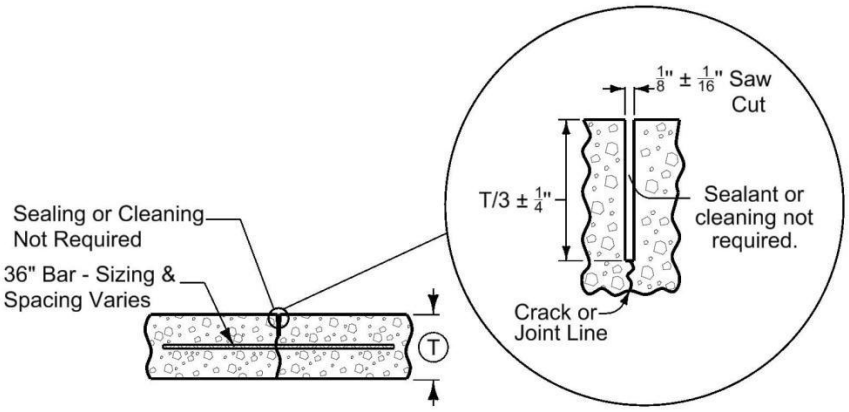
Typically, sawed longitudinal joints are sealed. However, since the slabs on either side of the longitudinal contraction joint are tied by a reinforcing bar, the Jurisdictional Engineer may approve not sealing the joint. The need to seal the joint is reduced due to the tied connection and the fact the joint will not open. The depth of cut for sawed longitudinal joints is T/3, regardless of the method of sawing used. The width of the sealed joints is 1/4 inch ± 1/16 inch. The maximum width of the unsealed joints is 1/8 inch ± 1/16 inch.

A longitudinal joint is usually placed at the center of the pavement to allow the pavement to hinge due to lane loading and help delineate separation of opposing traffic. Controlling cracking and proper constructability are the primary functions of longitudinal contraction joints. Lane delineation is a secondary function.

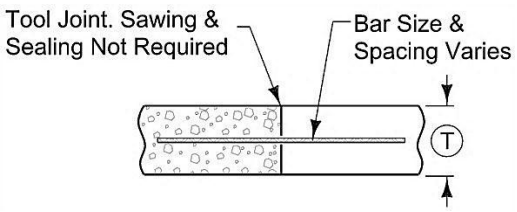
Figure 5G-2.03: Longitudinal Contraction Joints



'L' Joint with Sealant



'L' Joint without Sealant



'BT', Butt Joint

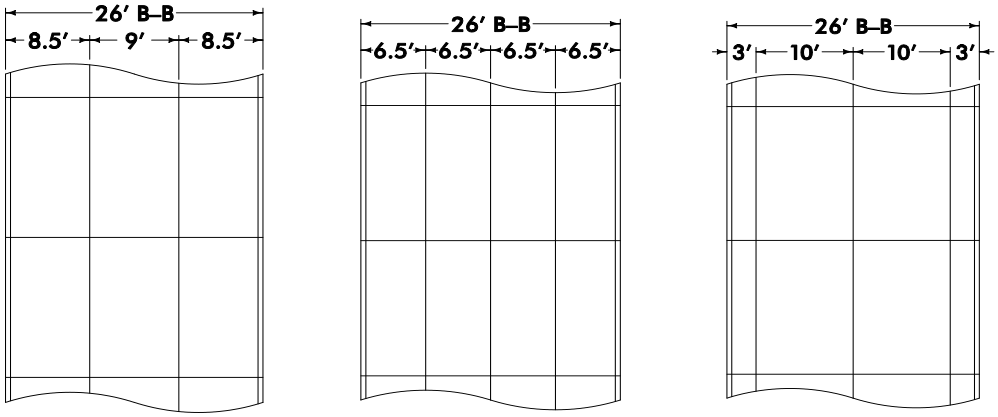
An important consideration when establishing the distance between longitudinal joints for jointed plain concrete pavements is the prevention of random longitudinal cracking at the quarter point, which is the midpoint between the centerline and the back of the curb. Pavements less than 9 inches thick may not crack through a longitudinal joint placed close to the gutter, which could cause longitudinal cracks at the quarter point. For this reason, it is preferred to use quarter point jointing for 31 foot wide pavements. Third point jointing, which eliminates the centerline joint, is frequently used for pavement narrower than 30 feet because of the narrower panel width and for 31 foot wide pavements with a depth greater than 8 inches. However, some jurisdictions desire a centerline joint and a gutterline joint, typically 3 to 3 1/2 feet from the back of curb. A gutterline joint should only be used if the pavement has depth of at least 9 inches or pavement widening is likely to occur.

The following examples depict jointing options for 26 foot and 31 foot wide pavements. The principles involved with jointing for these pavement widths can be extended to other pavement widths.

a. 26 Foot B-B Pavement: Three longitudinal joint options for 26 foot wide plain jointed concrete pavements are provided:

1) Third point jointing provides for a single 9 foot center panel with two joints, each 8 1/2 feet from the back of curb.
2) Quarter point jointing includes a centerline joint and two joints at the quarter points. This option is used when centerline crack control is desired.
3) Gutterline jointing provides two 10 foot lanes with a centerline joint and gutterline joints 3 feet from the back of curb. As stated above, care must be exercised with this option to prevent random cracking at the quarter point. This option is typically used for streets 9 inches or greater in thickness.

Figure 5G-2.04: 26 Foot B-B Pavement



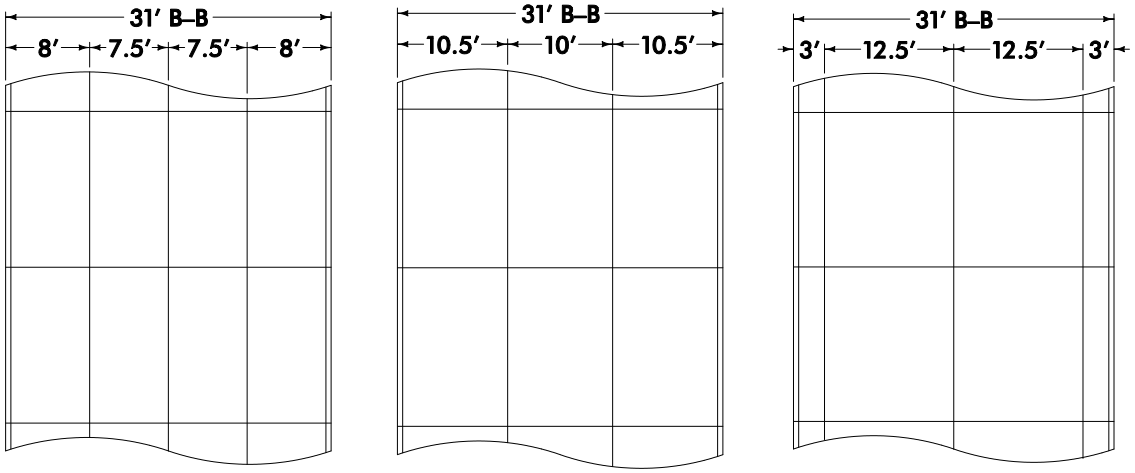
Third Point Jointing **Quarter Point Jointing** **Gutterline Jointing**

b. 31 Foot B-B Pavements: Three longitudinal joint options for 31 foot wide pavements are provided.

1) Quarter point jointing provides for a centerline longitudinal joint and two quarter point joints and is not intended to delineate driving lanes.
2) Third point jointing provides three nearly equally spaced panels, without a centerline joint. It typically is used as an option to quarter point jointing to minimize the number of longitudinal joints.
3) Gutterline jointing utilizes a centerline joint and gutterline joints 3 to 3 1/2 feet from the back of curb that delineate driving lanes.. This jointing pattern is typically used when the

pavement may be widened in the future, and the delineation of the lanes is desired. Care must be exercised with this option to prevent random cracking at the quarter point. Typically, gutterline jointing is used on streets with pavement thickness greater than or equal to 9 inches.

Figure 5G-2.05: 31 Foot B-B Pavements

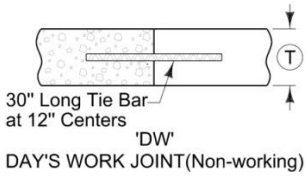
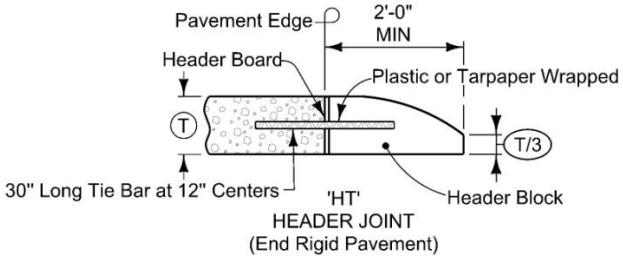
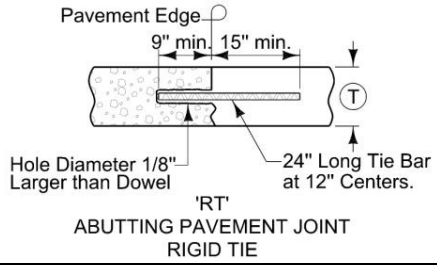
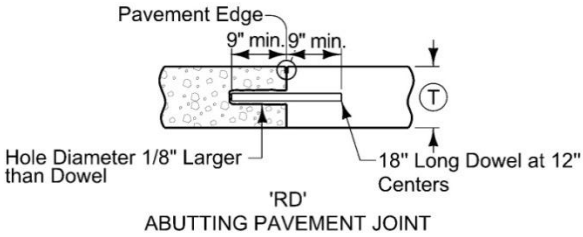
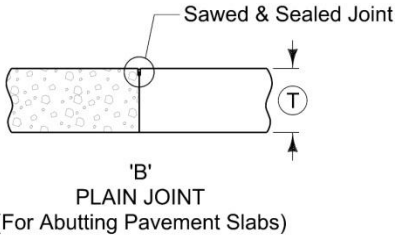


Quarter Point Jointing **Third Point Jointing** **Gutterline Jointing**

3. **Transverse and Longitudinal Construction Joints:** Construction joints are necessary for planned construction interruptions or widening/extending a pavement. Examples include construction of adjacent lanes at different times; box-outs for structures, radii, etc.; paving operation stoppages for over 30 minutes; and when a joint is needed between dissimilar materials. Construction joints are also used between an existing pavement and a new pavement. The joint is formed with the existing slab and is not sawed, except to accommodate joint sealing when required. Sawing and sealing of the joints are not required for those tied with deformed bars.

   a. **Transverse Construction Joints:** These types of joints are usually butt-type joints with deformed tie bars or dowels to provide load transfer and prevent vertical movement. When joint sealing is required, the depth of the saw cut (1 1/4 inches) is just deep enough to provide a reservoir for the joint sealant. The following are typical transverse construction joints.

Figure 5G-2.06: Transverse Construction Joints

	Used at emergency (unplanned) stopping point. Ideally, it should be located at mid-panel, but it should not be located less than 5 feet from a planned contraction joint.
	Used when the pavement ends and traffic will cross the joint. The header is removed when the pavement is extended.
	Typically used when an existing slab is extended.
	Functions as a CD joint when an existing slab is extended. Normally used when the pavement is 8 inches or greater in thickness.
	Typically used when two different pavement types or thicknesses abut or at the inside longitudinal edge of intake boxouts.

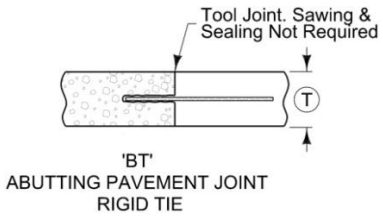
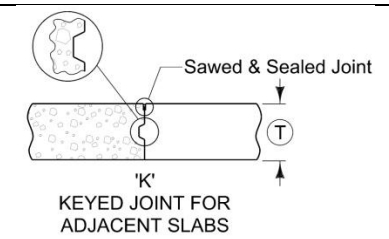
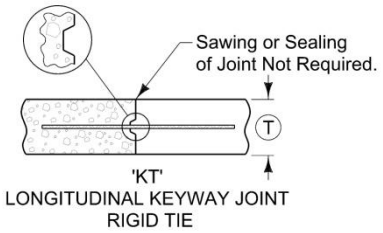
b. Longitudinal Construction Joints: These types of joints are used when adjacent lanes are constructed at different times. Tie-bars are primarily designed to resist horizontal movement but help with load transfer and vertical control. Under certain conditions, such as a drop in air temperature during the first night, longitudinal and transverse cracks may occur early. Early sawing of transverse joints is important when tied longitudinal construction joints are constructed in order to prevent the following two conditions from occurring.

1) Sympathy Transverse Cracking in New Lane Construction: When a new slab is longitudinally tied to an existing pavement, the existing transverse contraction joints can cause adjacent lane cracking in the new slab if early sawing of the transverse joints is not done. If there are transverse random cracks in an existing slab, the longitudinal

construction joint should be a plain butt joint or keyed joint (with no tie bars), if one exists in the old slab, to prevent sympathy cracks in the new pavement.

2) **Longitudinal Tie-bar Stress in Cooler Weather Conditions:** Care must be exercised to control cracking when utilizing longitudinal construction joints with tie bars, particularly in cool temperatures. For example, when a lane is constructed one day and the adjacent lane is constructed the following day or later, the existing lane could be expanding, particularly in the morning. If the new lane is in its final set (contracting) at the same time the existing pavement is expanding, stresses in the concrete at the tie bars can be significant. If the strength of the new concrete has not developed enough to resist the stresses, cracking could occur in the new concrete at the tie bars. During cooler weather conditions, care should be exercised when paving the new lane. Ideally, the new paving operation should take place at mid-day or later when the existing lane expansion is reduced.

Figure 5G-2.07: Longitudinal Construction Joints

	Used to tie existing and new parallel pavements together to prevent horizontal movements; and will provide some load transfer and resist vertical movement.
	Used when tie bars are not needed or desired and load transfer is required.
	Used in pavements under heavier traffic conditions and typically where the pavement thickness is 8 inches or greater. Also used at intersections under heavy turning movements where the pavement is less than 8 inches.

4. **Isolation Joints and Expansion Joints:** Expansion and isolation joints accommodate anticipated differential horizontal and vertical movements that occur between a pavement and structure. Their purpose is to allow movement without damaging adjacent structures or pavements. Contraction or control joints also absorb some movement; however, their main function is to control the location and geometry of the natural cracking pattern in the concrete slab. Because pavement performance can be significantly affected by the planned use and location of isolation and expansion joints, care should be taken in their design. Though the terms are sometimes used interchangeably, isolation joints are not expansion joints.

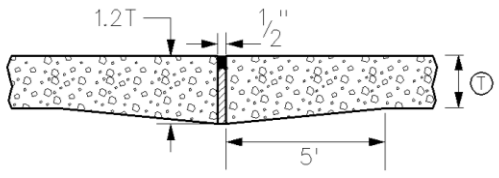
a. Isolation Joints: Isolation joints isolate the pavement from a structure, another paved area, or an immovable object. Isolation joints include full depth, full width joints found at bridge abutments, intersections, or between existing and new pavements. The term "isolation joint" also applies to joints around in-pavement structures such as drainage inlets, manholes, footings, and lighting structures. Isolation joints lessen compressive stresses that develop at T and unsymmetrical intersections, ramps, bridges, building foundations, drainage inlets, manholes, and anywhere differential movement between the pavement and a structure may take place. They are also placed adjacent to existing pavements, especially when it is not possible or desirable to match joint locations in the older pavement. Isolation joints should be 1/2 to 1 inch wide. Greater widths may cause excessive movement. They are filled with a pre-formed joint filler material to prevent infiltration of incompressibles.

At T-intersections, isolation joints should be used to isolate the T-intersecting street from the through street. Also, all legs of skewed streets should be isolated from the through street. Isolation joints used for this purpose should be placed one joint spacing back from the end of the intersection radii.

The joint filler material for expansion and isolation joints occupies the gap between the slabs and must be continuous from one pavement edge to the other and through curb and gutter sections. This filler material is usually a non-absorbent, non-reactive, non-extruding material typically made from either a closed-cell foam rubber or a bitumen-treated fiber board. No plug or sliver of concrete should extend over, under, through, around, or between sections of the filler, or it will cause spalling of the concrete. After the concrete hardens, the top of the filler may be recessed about 3/4 inch below the surface of the slab to allow space for the joint sealant to be placed later.

1) **Doweled Isolation Joints:** Isolation joints used at structures should have dowels to provide load transfer. The end of the dowel must be equipped with a closed-end expansion cap into which the dowel can move as the joint expands and contracts. The cap must be long enough to cover 2 inches of the dowel and have a suitable stop to hold the end of the cap at least the width of the isolation joint plus 1/4 inch away from the end of the dowel bar. The cap must fit the dowel bar tightly and be watertight. The half of the dowel with the capped end must be coated to prevent bonding and allow horizontal movement.
2) **Special Undoweled Isolation Joints:** Isolation joints at T and unsymmetrical intersections or ramps are not doweled so that horizontal movements can occur without damaging the abutting pavement. Undoweled isolation joints can be constructed with thickened edges to reduce the stresses developed at the slab bottom. The abutting edges of both pavements should be thickened by 20% starting with a taper 5 feet from the joint. The isolation filler material must extend completely through the entire thickened-edge slab.

Figure 5G-2.08: Thickened Edge Joint



a) Undoweled Isolation Joints for Boxouts: Isolation joints used at drainage inlets, manholes, and lighting structures do not have thickened edges or dowels.

b) Adjusting Isolation Joints for Utility Fixtures: After developing the jointing plan, plot any catch basins, manholes, or other fixtures that are within the intersection. Non-telescoping manholes will require a boxout or isolation joint to allow for vertical and horizontal slab movement. Consider using rounded boxouts to avoid crack-inducing corners. Also, for square boxouts, wire mesh or small-diameter reinforcing bars in the concrete around any interior corners will hold cracks tight should they develop. Telescoping manholes can be cast integrally within the concrete, and do not necessarily require a boxout. The multiple piece casting does not inhibit vertical movement and is less likely to create cracks within the pavement.

When a joint is within 5 feet of a fixture, it is desirable to adjust the joint so that it will pass through the fixture or the boxout surrounding the fixture. The following diagram shows several acceptable ways to skew or shift a joint to meet fixtures.

b. Expansion Joints: Expansion joints are defined as full depth, full width transverse joints placed at regular intervals of 50 to 500 feet (with contraction joints in between). This is an old practice that was used to relieve compressive forces in pavement. Unfortunately, this practice often caused other problems in the pavement such as spalling, pumping, faulting, and corner breaks.

Good design, construction, and maintenance of contraction joints has virtually eliminated the need for expansion joints, except under special conditions. In addition to the problems listed above, the improper use of expansion joints can lead to high construction and maintenance costs, opening of adjacent contraction joints, loss of aggregate interlock, sealant failure, joint infiltration, and pavement growth. By eliminating unnecessary expansion joints, these problems are removed and the pavement will provide better performance.

Pavement expansion joints are only needed when:

1) The pavement is divided into long panels (60 feet or more) without contraction joints in between to control transverse cracking.
2) The pavement is constructed while ambient temperatures are below 40°F.
3) The contraction joints are allowed to be infiltrated by large incompressible materials.
4) The pavement is constructed of materials that in the past have shown high expansion characteristics.

Under most normal concrete paving situations, these criteria do not apply. Therefore, expansion joints should not normally be used (PCA, 1992).

Figure 5G-2.09: Typical PCC Joint Layout at Manholes
(SUDAS Specifications Figure 7010.103)

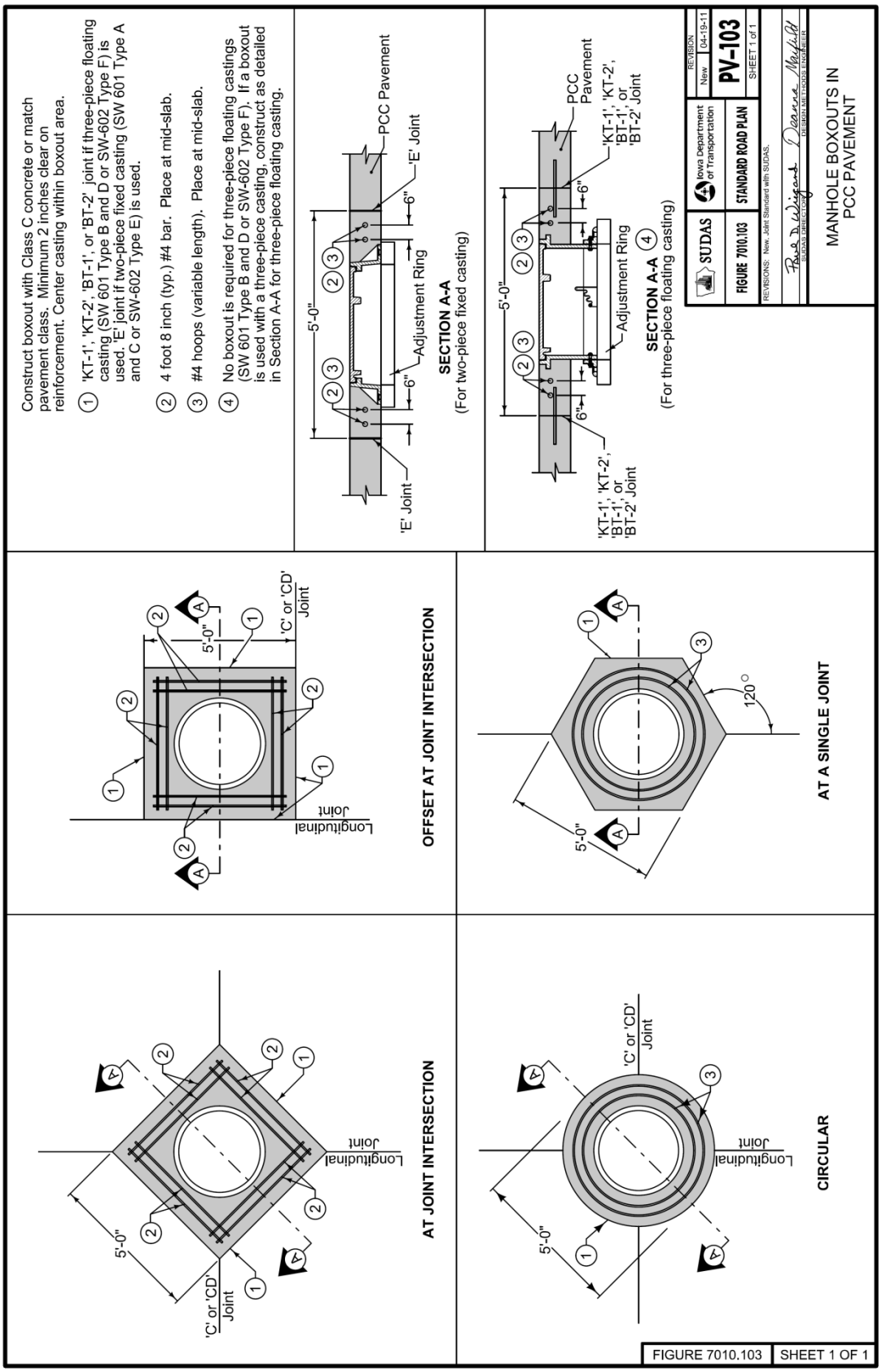
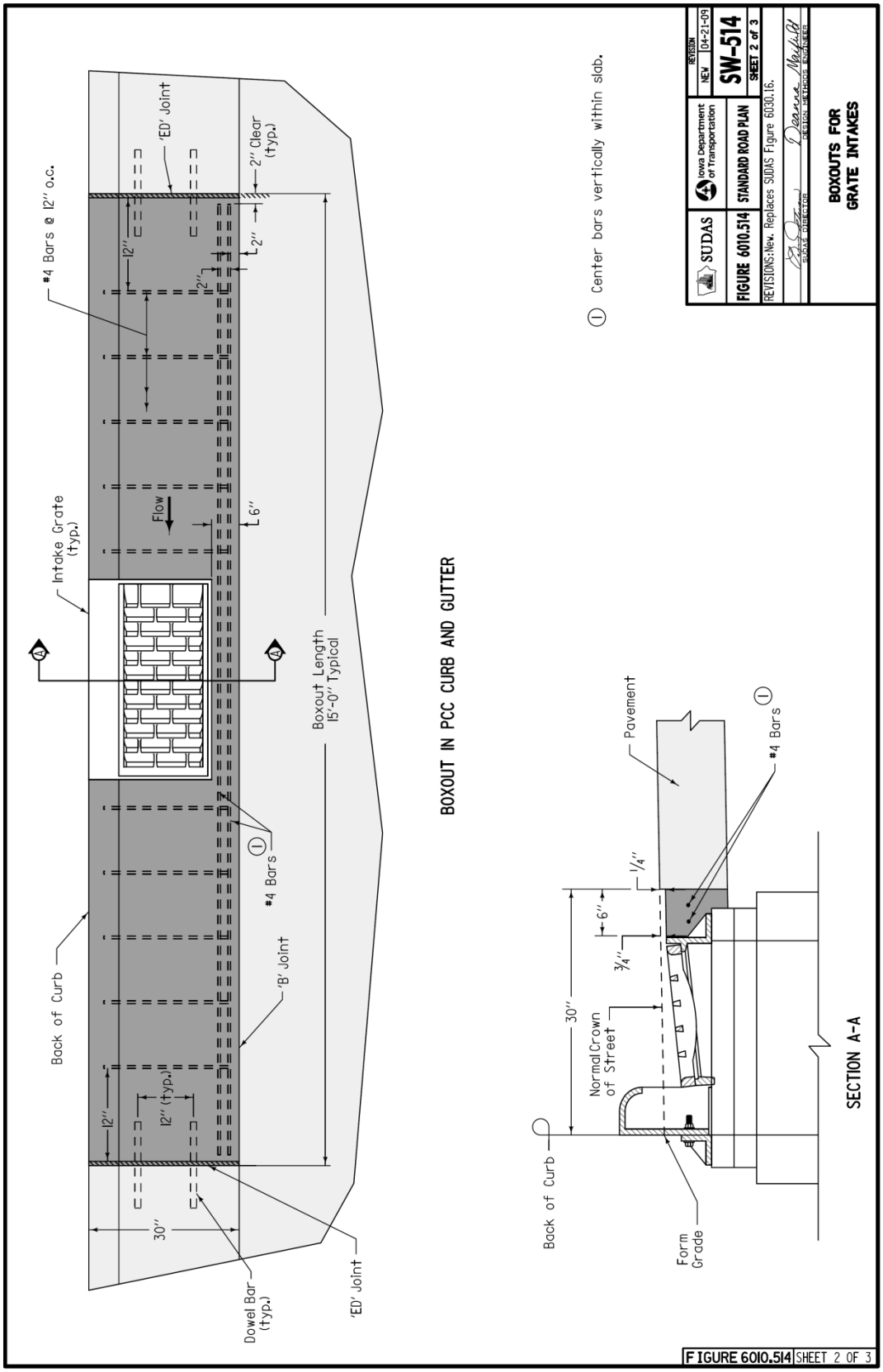
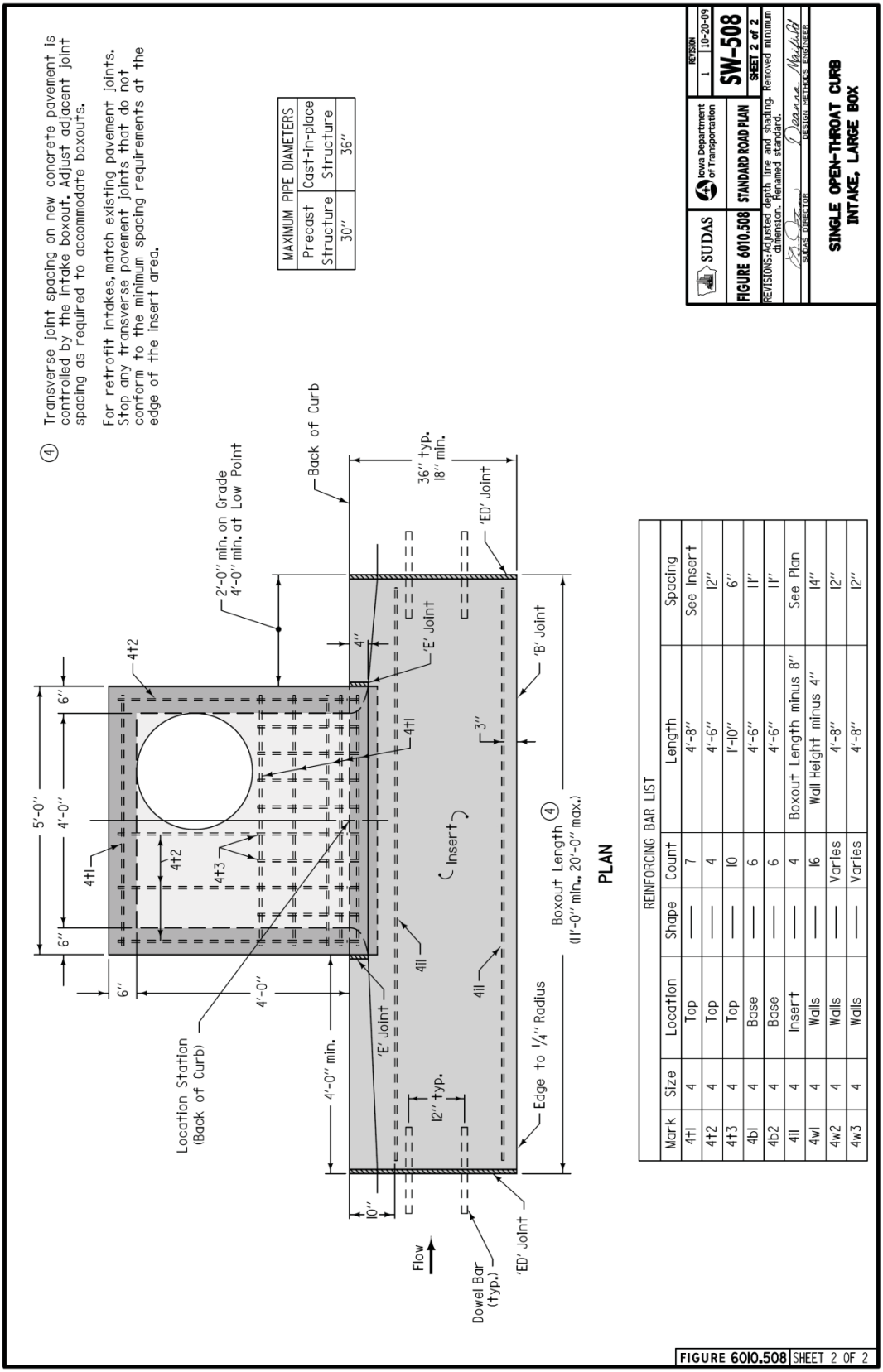


Figure 5G-2.10A: Typical PCC Joint Layout at Intakes - Boxout for Grate Intakes
(SUDAS Specifications Figure 6010.514, sheet 2*)



* SUDAS Specifications Figure 6010.514, sheets 1 and 3 include more boxout options.

Figure 5G-2.10B: Typical PCC Joint Layout at Intakes - Boxout for Open-throat Curb Intakes (SUDAS Specifications Figure 6010.508, sheet 2*)



* SUDAS Specifications Figure 6010.508, sheet 1 includes more information.

Table 5G-2.02: Summary of Joints
(Derived from the Iowa DOT Design Manual, Section 7A-2, Tables 1 and 2)

	Type			Method of Load Transfer				Thermal Movement				
Joint	Transverse	Longitudinal	Isolation/Expansion	Aggregate Interlock	Key	Tie Bar	Dowel Bar	Doweled to allow movement	Tied to prevent movement	Isolation/Expansion joint allows movement	Lack of reinforcing allows movement	**Comments**
B	x	x									x	Used between dissimilar materials or when other joints are not suitable.
C	x			x							x	Transverse joint used when T < 8 inches.
CD	x			x			x	x				Transverse joint used when T ≥ 8 inches.
CT	x			x		x			x			Specialty tied contraction joint.
DW	x					x			x			Used by contractor as a stopping point.
HT	x					x			x			Used at the end of rigid pavement prior to placement of second slab.
RD	x						x	x				Joint between new and existing pavements, dowels are used.
RT	x					x			x			Joint between new and existing pavements, tie bars are used.
BT-1		x							x			Longitudinal joint used when T < 8 inches, interchangeable with L-1 depending on paving sequence.
BT-2		x							x			Used when L-2 and the KT-2 are not possible, T ≥ 8 inches.
BT-3		x							x			Joint used between new and existing pavements. Tie bars are used when T ≥ 8 inches.
BT-4		x							x			Joint used between new and existing pavements. Tie bars are used when T ≥ 8 inches.
BT-5		x							x			Joint used between new and existing pavements. Tie bars are used when T < 8 inches.
K		x			x						x	T > 8 inches, minimal usage.
KS		x			x				x			Used in reinforced pavements.
KT-1		x			x				x			Longitudinal joint used when T < 8 inches, interchangeable with L-1 depending on paving sequence.
KT-2		x			x				x			Longitudinal joint used when T ≥ 8 inches, interchangeable with L-2 depending on paving sequence.
KT-3		x			x				x			Longitudinal joint used when T ≥ 8 inches, interchangeable with L-3 depending on paving sequence.
L-1		x		x					x			Longitudinal joint used when T < 8 inches, interchangeable with BT-1.
L-2		x		x					x			Longitudinal joint used when T ≥ 8 inches, interchangeable with KT-2 depending on paving sequence.
L-3		x		x					x			Longitudinal joint used with pavements of large width, interchangeable with KT-3 depending on paving sequence.
CF	x		x							x		4 inch expansion joint.
E	x	x	x							x		1 inch expansion joint.
ED	x		x				x	x		x		1 inch doweled expansion joint.
EE	x		x				x	x		x		2 inch doweled expansion joint.
EF	x		x				x	x		x		4 inch doweled expansion joint.
ES			x							x		Used in curb to match expansion joint in pavement.

D. Transverse Dowel Bar Size and Length

Table 5G-2.03 reflects the dowel bar size and length based on the pavement thickness. This information was obtained from the Portland Cement Association, the American Concrete Paving Association, and American Highway Technology. The SUDAS and Iowa DOT Specifications call for dowels when the slab is 8 inches or greater. Dowels are typically set at 12 inch spacing. The designer should note that a dowel bar that is too small induces high bearing stresses and causes the concrete matrix around the dowel to deteriorate or elongate. Elongation of the dowel bar hole then reduces the load transfer capabilities. Under special circumstances, smaller diameter and different shaped dowel bars may be used in thinner slabs.

Table 5G-2.03: Dowel Bar Size and Length

Pavement Thickness (inches)	Dowel Size (diameter in inches)	Dowel Length (inches)
8	1 1/4	18
9	1 1/4	18
10	1 1/2	18
11	1 1/2	18
12	1 1/2	18

E. Jointed Reinforced Concrete Pavements

Jointed reinforced concrete pavements (JRCP), sometimes referred to as distributed steel reinforcing, are not commonly used in Iowa jurisdictions. However, variations of JRCP are used effectively by several jurisdictions in Iowa. Therefore, the following is provided as an explanation of JRCP.

JRCPs utilize bar mats between transverse joints. Typically, the bar mats extend full width across the pavement, but with traditional JRCPs, they do not extend through the transverse joints. JRCPs use many of the same types of joints as jointed plain concrete pavements (JPCP), but the tie bars for longitudinal joints are replaced with the bar mats. Transverse joints, including doweled joints, are the same for both types of pavements since the bar mats of traditional JRCP do not extend through the transverse joints. Because of the bar mats, transverse joint spacing can be much longer than with JPCP, usually 27 feet to 45 feet. JRCP should not be confused with continuously reinforced pavement, which has very few or no joints.

JRCPs are used primarily to control cracking of concrete pavements, to provide for load transfer between joints, and to maintain the structural integrity of the slab between transverse joints. Just like JPCPs, random cracking of JRCPs may still occasionally occur even though the steel is present. The steel serves to hold the cracks close together, thus preventing the progressive opening of the cracks over time.

The added cost of the additional reinforcement for JRCPs is often offset by specifying a somewhat thinner slab. However, as pointed out by the American Concrete Institute (ACI), "the use of reinforcing steel will not add to the load-carrying capacity of the pavement nor compensate for poor subgrade preparation or poor construction practices." By holding random cracks tightly closed, it will maintain the shear resistance of the slab, and, consequently, will maintain its load carrying capacity. This improves the ride when the vertical displacement is controlled.

As mentioned previously, several jurisdictions in Iowa specify a variation of JRCP. The Iowa variations of JRCP typically include extending the longitudinal reinforcing bars through the 'C' plain transverse contraction joints. When 'CD' doweled transverse joints are specified, the longitudinal

reinforcement does not extend through the transverse joints. In addition, the transverse joint spacing is generally not lengthened as described for traditional JRCPs and follows the same guidelines as for JPCP. Figures 5G-2.11 and 5G-2.12 illustrate JRCP details typically used in Iowa.

Figure 5G-2.11: Iowa Jointed Reinforced Pavement Detail - 26' Back-To-Back Street (formerly SUDAS Specifications Figure 7010.1F)

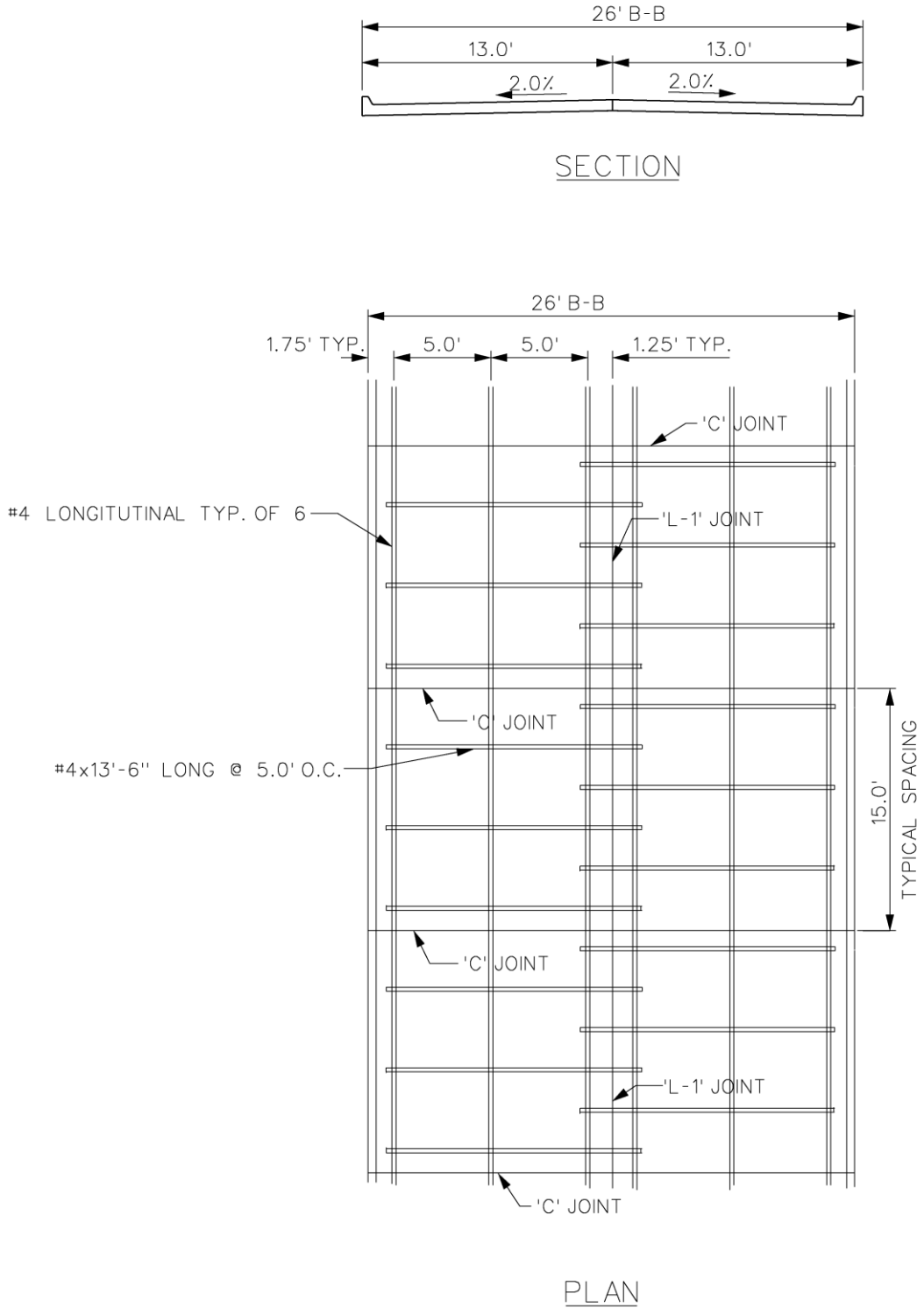
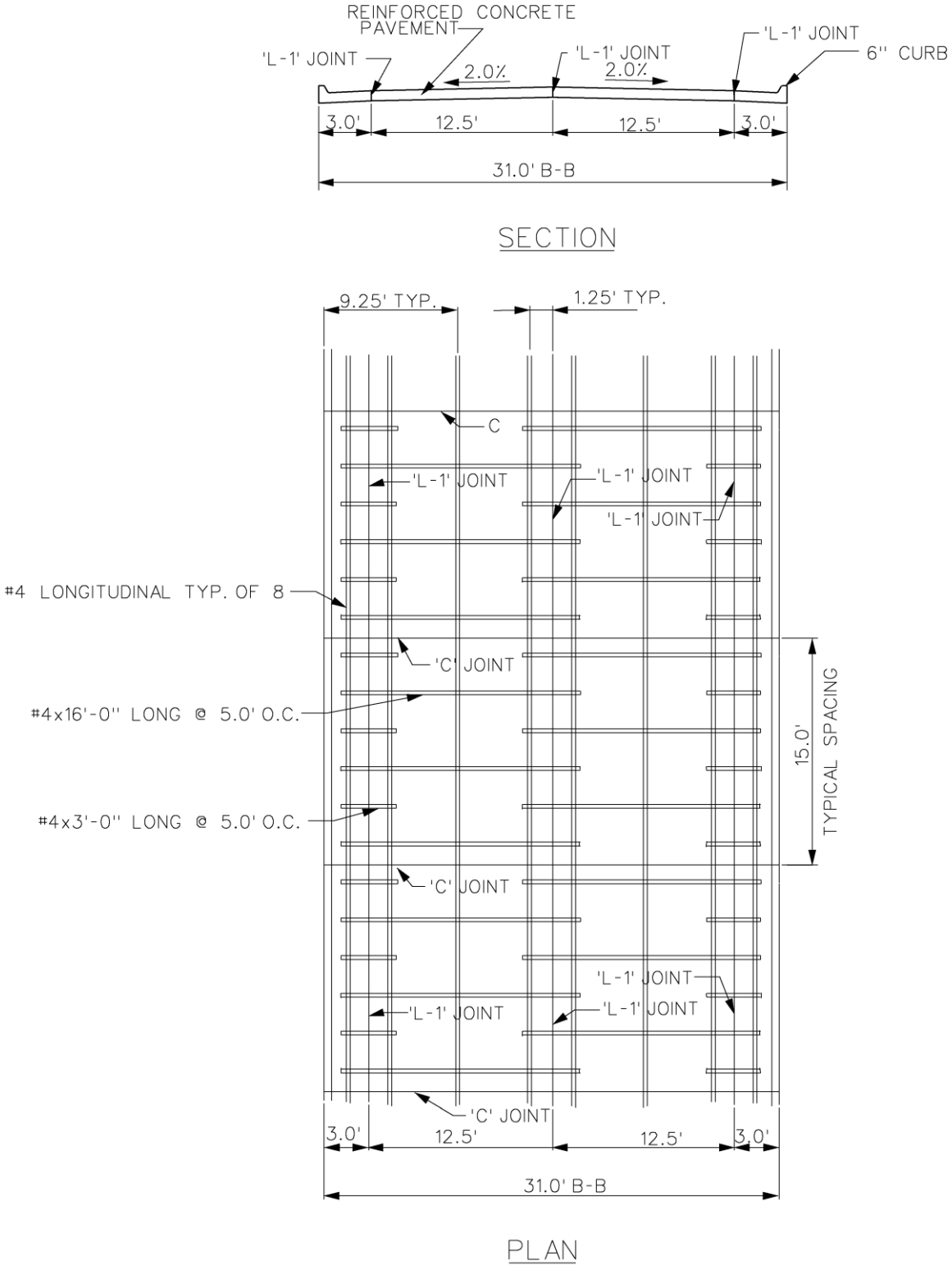
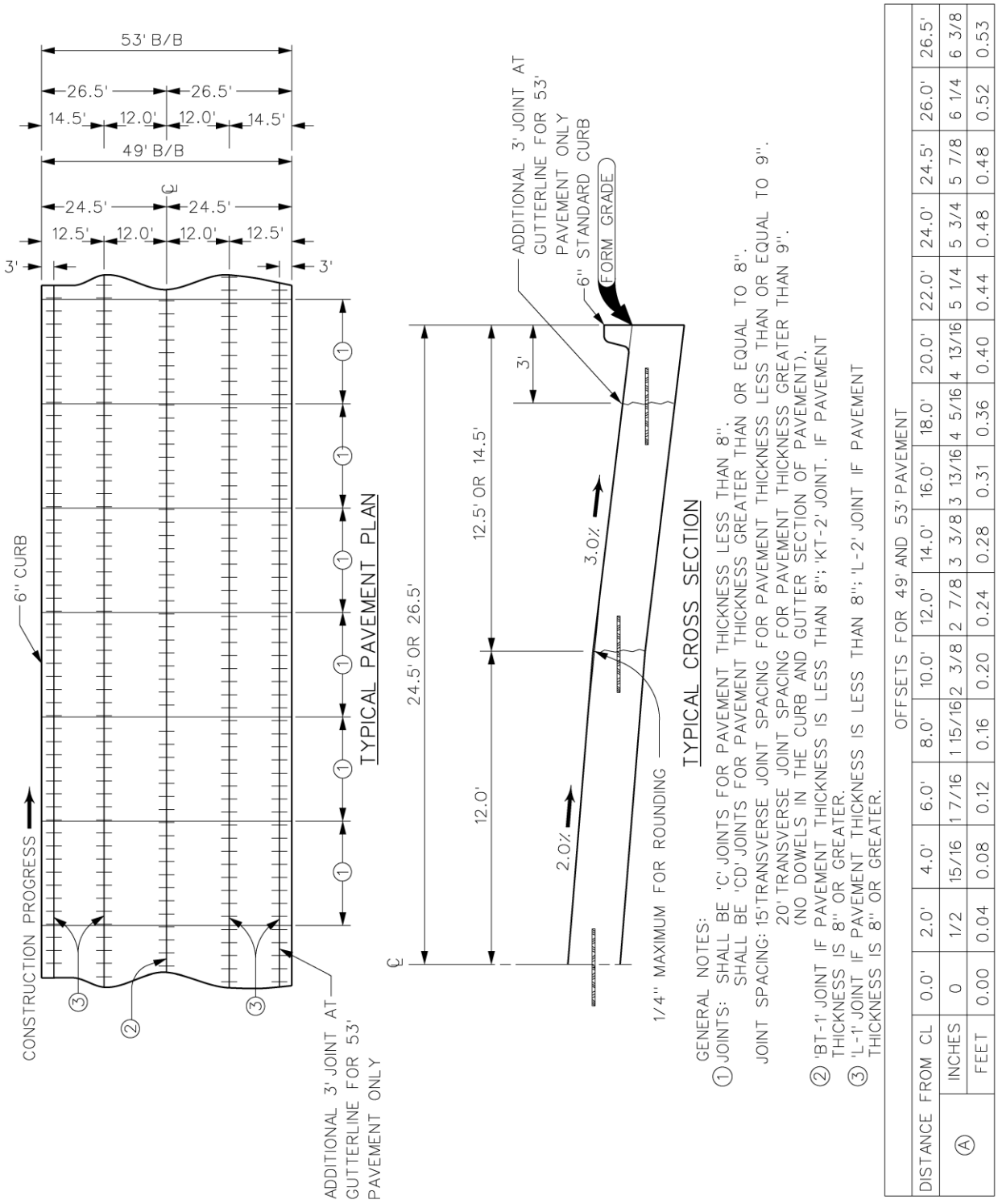


Figure 5G-2.12: Iowa Jointed Reinforced Pavement Detail - 31' Back-To-Back Street (formerly SUDAS Specifications Figure 7010.1G)



F. Miscellaneous PCC Pavement Jointing Figures

Figure 5G-2.13: 49' B/B and 53' B/B PCC Pavement Jointing and Crown Detail
(formerly SUDAS Specifications Figure 7010.1C)



GENERAL NOTES:
① JOINTS: SHALL BE 'C' JOINTS FOR PAVEMENT THICKNESS LESS THAN 8".
SHALL BE 'CD' JOINTS FOR PAVEMENT THICKNESS GREATER THAN OR EQUAL TO 8".
JOINT SPACING: 15'TRANSVERSE JOINT SPACING FOR PAVEMENT THICKNESS LESS THAN OR EQUAL TO 9".
20' TRANSVERSE JOINT SPACING FOR PAVEMENT THICKNESS GREATER THAN 9".
(NO DOWELS IN THE CURB AND GUTTER SECTION OF PAVEMENT).
② 'BT-1' JOINT IF PAVEMENT THICKNESS IS LESS THAN 8"; 'KT-2' JOINT. IF PAVEMENT THICKNESS IS 8" OR GREATER.
③ 'L-1' JOINT IF PAVEMENT THICKNESS IS LESS THAN 8"; 'L-2' JOINT IF PAVEMENT THICKNESS IS 8" OR GREATER.

OFFSETS FOR 49' AND 53' PAVEMENT

DISTANCE FROM CL		0.0'	2.0'	4.0'	6.0'	8.0'	10.0'	12.0'	14.0'	16.0'	18.0'	20.0'	22.0'	24.0'	24.5'	26.0'	26.5'
Ⓐ	INCHES	0	1/2	15/16	1 7/16	1 15/16	2 3/8	2 7/8	3 3/8	3 13/16	4 5/16	4 13/16	5 1/4	5 3/4	5 7/8	6 1/4	6 3/8
	FEET	0.00	0.04	0.08	0.12	0.16	0.20	0.24	0.28	0.31	0.36	0.40	0.44	0.48	0.48	0.52	0.53

Figure 5G-2.14: 49’ B/B and 53’ B/B C&G/HMA Pavement
(formerly SUDAS Specifications Figure 7020.2B)

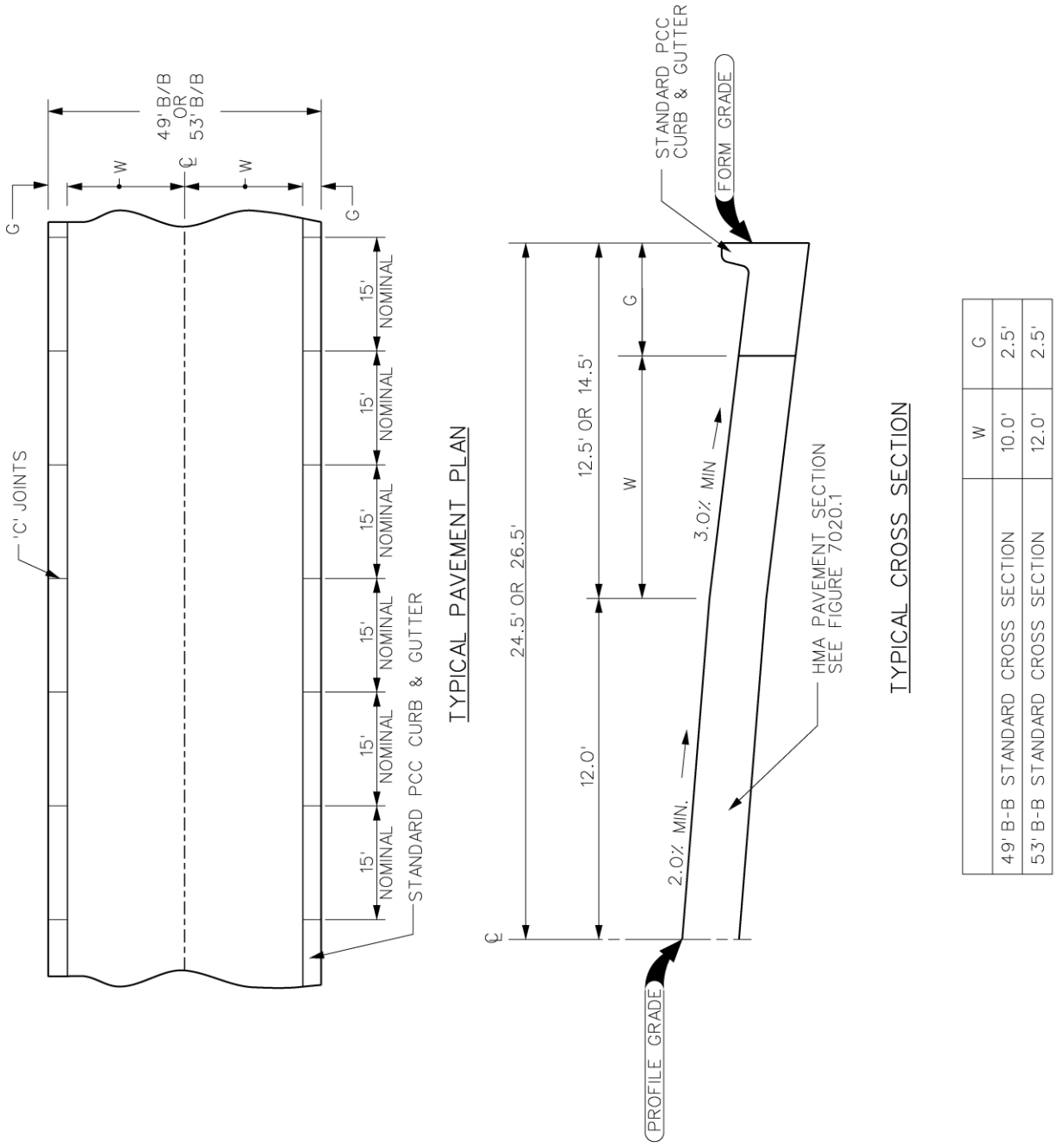
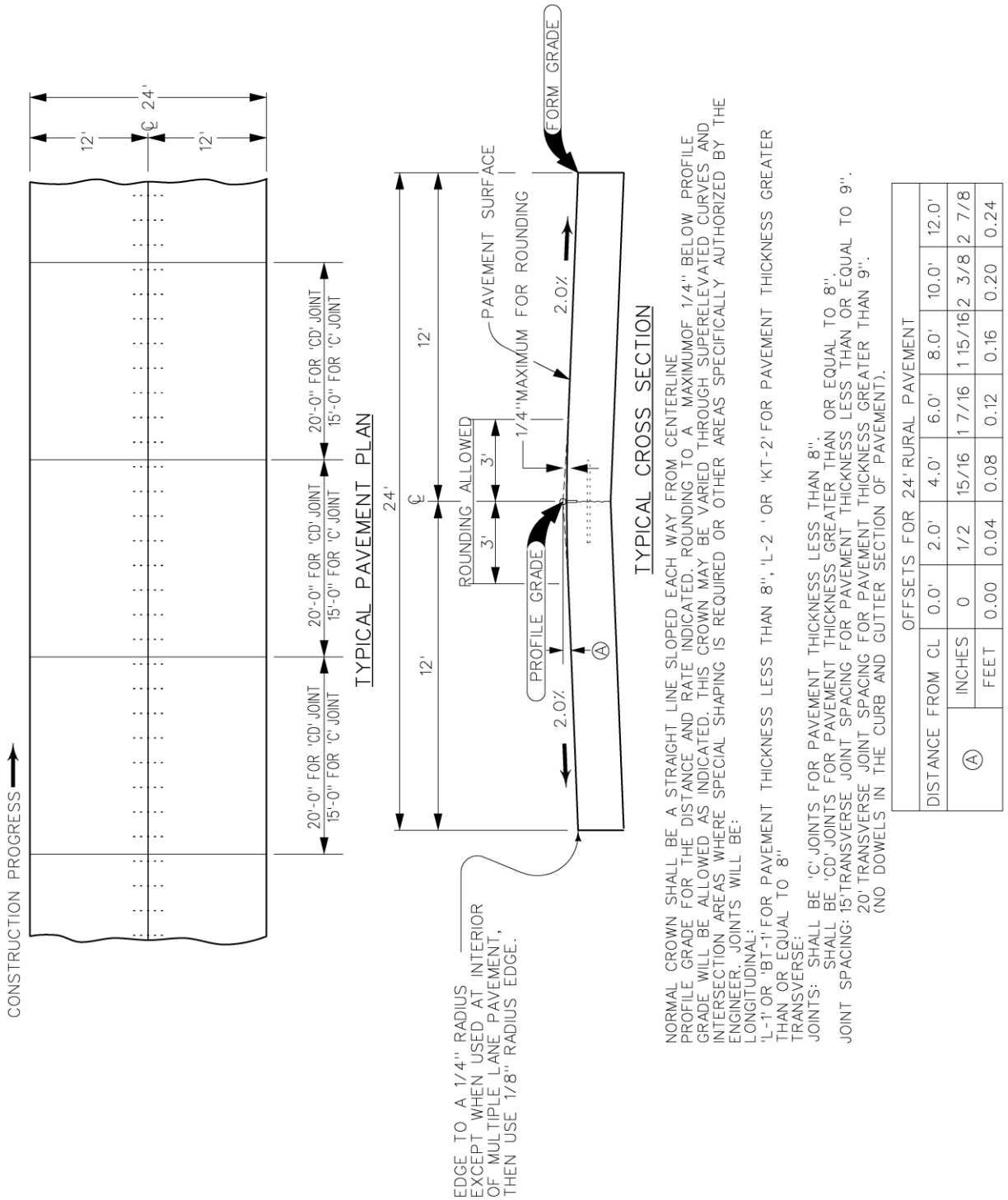
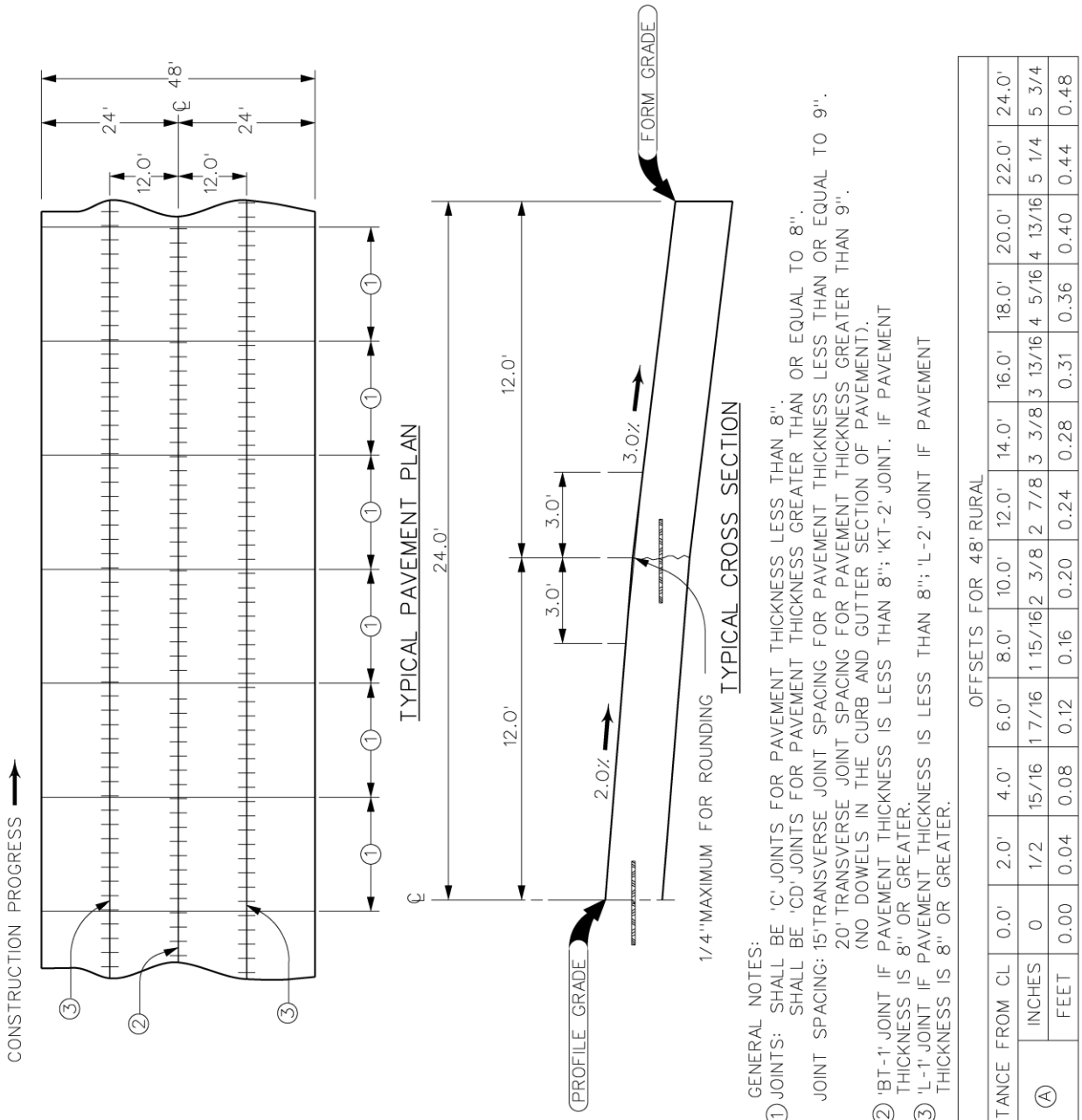


Figure 5G-2.15: 24' Rural PCC Pavement Jointing and Crown Detail
(formerly SUDAS Specifications Figure 7010.1D)



OFFSETS FOR 24' RURAL PAVEMENT								
DISTANCE FROM CL		0.0'	2.0'	4.0'	6.0'	8.0'	10.0'	12.0'
Ⓐ	INCHES	0	1/2	15/16	1 7/16	1 15/16	2 3/8	2 7/8
	FEET	0.00	0.04	0.08	0.12	0.16	0.20	0.24

Figure 5G-2.16: 48' Rural PCC Pavement Jointing and Crown Detail
(formerly SUDAS Specifications Figure 7010.1E)

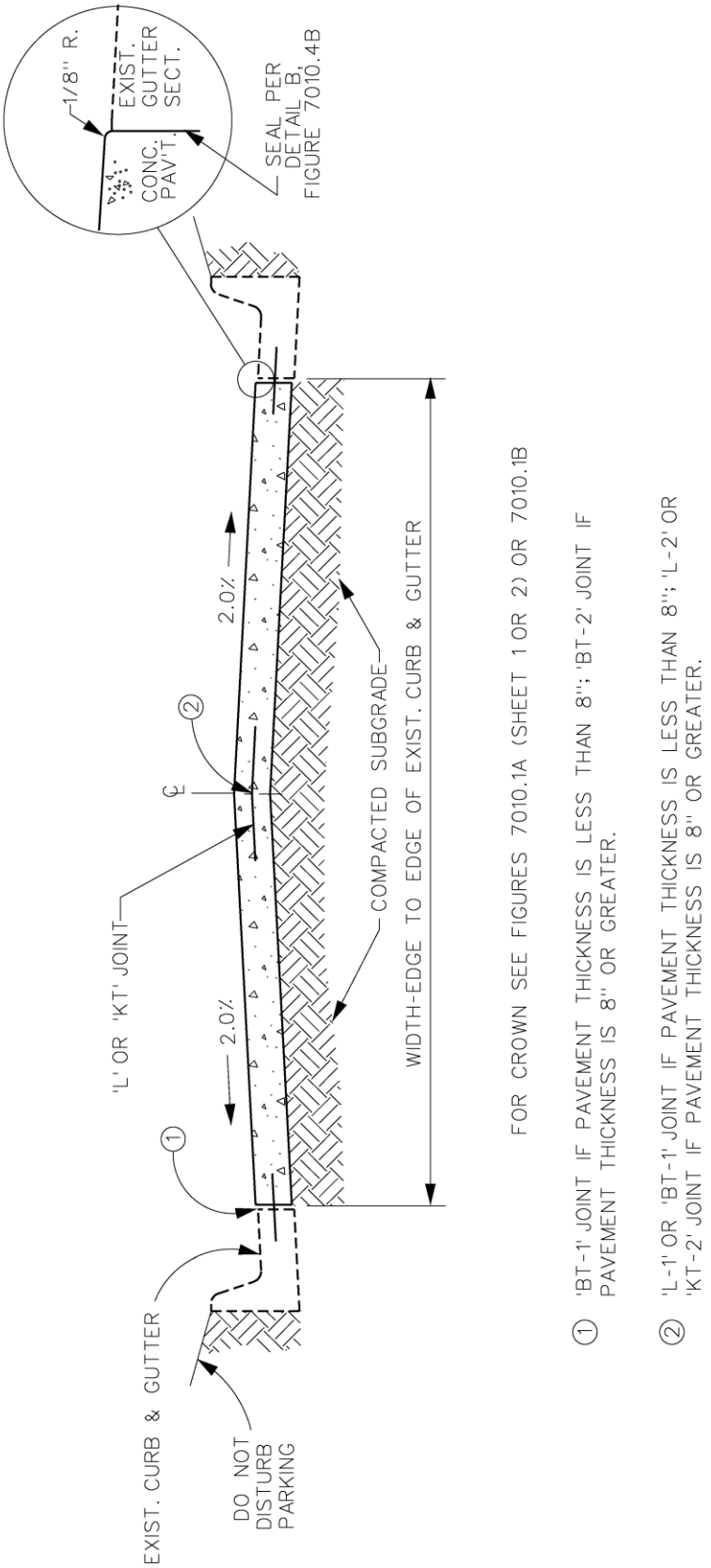


GENERAL NOTES:
① JOINTS: SHALL BE 'C' JOINTS FOR PAVEMENT THICKNESS LESS THAN 8".
SHALL BE 'CD' JOINTS FOR PAVEMENT THICKNESS GREATER THAN OR EQUAL TO 8".
JOINT SPACING: 15'TRANSVERSE JOINT SPACING FOR PAVEMENT THICKNESS LESS THAN OR EQUAL TO 9".
20'TRANSVERSE JOINT SPACING FOR PAVEMENT THICKNESS GREATER THAN 9".
(NO DOWELS IN THE CURB AND GUTTER SECTION OF PAVEMENT).
② 'BT-1' JOINT IF PAVEMENT THICKNESS IS LESS THAN 8"; 'KT-2' JOINT. IF PAVEMENT THICKNESS IS 8" OR GREATER.
③ 'L-1' JOINT IF PAVEMENT THICKNESS IS LESS THAN 8"; 'L-2' JOINT IF PAVEMENT THICKNESS IS 8" OR GREATER.

OFFSETS FOR 48' RURAL

DISTANCE FROM CL		0.0'	2.0'	4.0'	6.0'	8.0'	10.0'	12.0'	14.0'	16.0'	18.0'	20.0'	22.0'	24.0'
Ⓐ	INCHES	0	1/2	15/16	1 7/16	1 15/16	2 3/8	2 7/8	3 3/8	3 13/16	4 5/16	4 13/16	5 1/4	5 3/4
	FEET	0.00	0.04	0.08	0.12	0.16	0.20	0.24	0.28	0.31	0.36	0.40	0.44	0.48

Figure 5G-2.17: PCC Pavement Section Between Existing Curb and Gutter
(formerly SUDAS Specifications Figure 7010.5)



G. References

Portland Cement Association. *Portland Cement Association Manual*.1992.

Jointing Urban Intersections

This section describes examples on how to joint urban intersections. The process will be illustrated through examples of different types of streets, pavement thickness, and intakes. Even though not all urban intersections will be exactly like the one used in these examples, the process described is applicable to other layouts.

During construction, it is likely that location changes will be necessary for some joints within an intersection. The primary reason is to ensure that joints pass through fixtures like manholes or drainage inlets that are embedded in the pavement. As a result, it will be desirable for the construction crew to adjust the location of some joints so they coincide with the actual location of a nearby manhole. The designer should consider placing a note on the plans to give the field engineer and contractor the latitude to make appropriate adjustments.

It is common practice for some designers to leave intersection joint layout to the field engineer and contractor. These designers often justify this practice by citing the many field adjustments that occur during construction, which they contend negates the usefulness of a jointing plan. However, it is not desirable to eliminate the jointing plan except for very simple intersections. A jointing plan and appropriate field adjustments are both necessary for more complex intersections because islands, medians, and turning lanes complicate joint layout and require some forethought before construction. The jointing plan will also enable contractors to more accurately bid the project.

Example: This example is an intersection of a multi-lane street and a two-lane side street. The intersection is curbed, includes several intakes, and the pavement thickness is 10 inches.

Step 1: Set Joints with Predetermined Locations

Because the location of longitudinal joints for both streets is normally predetermined, these joints should be set first.

Within the intersection, the street that is paved first determines which joints are longitudinal and which are transverse. Generally, the mainline street will be paved prior to the side street. Therefore, the longitudinal joints running down the side street define the locations of the first transverse joints for the mainline (see Figure 5G-3.01).

To determine an appropriate longitudinal joint to use, refer to SUDAS Specifications Figure 7010.901. The type of joint used may depend on the pavement thickness. Since the pavement thickness is greater than 8 inches in this case, either a KT-2 or an L-2 joint is appropriate.

Step 2: Locate Difficult Joints

Intake locations and the boxouts at the corner radii of the intersection are addressed next. After joints have been placed at these locations, the rest of the joints can be worked in around them.

1. **Joints at Intakes:** The location of intakes is determined before the joints are laid out, so joints have to be worked in around them. To start out with, straddle the intake with two transverse joints spaced according to the standard joint length. These joints can be repositioned later if it

helps with the placement of other joints. In the final layout, the intake should be centered between the joints, and adjacent joints should be adjusted accordingly. See the appropriate intake boxout figure in SUDAS Specifications Section 6010 for boxout length requirements.

CD joints should be used on the mainline since the pavement thickness is greater than 8 inches. However, the CD joints straddling the intake do not extend all the way through the curb and gutter. The joints immediately surrounding the intake are specified on the detail plates and are shown in the example.

2. **Joints at Boxouts:** Before the mainline is paved, small areas near the corners are boxed-out. These boxed-out areas (shaded in Figure 5G-3.01) are poured later, after the mainline has been paved. If the paver were to proceed straight through this area, instead of using boxouts, the returns of the city street would narrow to a point where they meet the mainline. Pavement less than 2 feet in width is weak and cracks readily. By using boxouts, this situation can be avoided without the expense of stopping the paver at the intersection.

   Although the width of boxouts is normally the same as the roadway's gutter width, the size and shape of boxouts varies depending on where they are used. If placing joints around the boxout, remember to maintain intersecting angles greater than 70 degrees and joints at least 2 feet long. KT-2 or L-2 joints are used around the boxout. Figure 5G-3.01 illustrates joints properly placed, both around the boxouts and extending outward from them.

Step 3: Locate Remaining Joints

After the joints at intakes and boxouts are located, the remaining joints (generally transverse joints) are located in appropriate locations. The maximum spacing for CD joints is 20 feet (greater than 9 inch pavement) and the minimum spacing is typically 12 feet. Therefore, the remaining areas on the mainline that need transverse joints should have CD joints spaced within this range. Since the design year truck volume on the adjoining street is less than 200 vpd, C transverse joints are used there.

In Figure 5G-3.02, the C joints on the city street nearest the corners are skewed perpendicular to the free edge of the pavement. If this joint were carried straight through, instead of skewed, the acute angle between the joint and the free edge of the pavement would be less than 70 degrees, which is not acceptable.

After all joints are located, the layout should be checked to ensure that all joint spacings and angles are acceptable. Figure 5G-3.02 shows all of the transverse joints appropriately located.

Step 4: Label Joints

The completed jointing layout of the intersection is shown in Figure 5G-3.02. For pavements 8 inches or greater, the L-2 and KT-2 joints may be used interchangeably, at the contractor's discretion, depending on the paving sequence. Therefore, the designer may identify the longitudinal joints as either L-2 or KT-2 on the jointing layout.

It is not necessary to identify every joint on the jointing layout. A few key joints on the diagram should be identified and whenever a series of joints changes to a different type of joint, the joint at the location of the change should be identified. Also, any joint that may be a source of confusion should be identified.

Joint lengths are also shown on the jointing layout, normally rounded to the nearest foot. Similar to labeling joint types, not every length needs to be identified. However, any length that cannot be inferred from the diagram should be labeled.

Figure 5G-3.01: Locating Predetermined and Difficult Joints

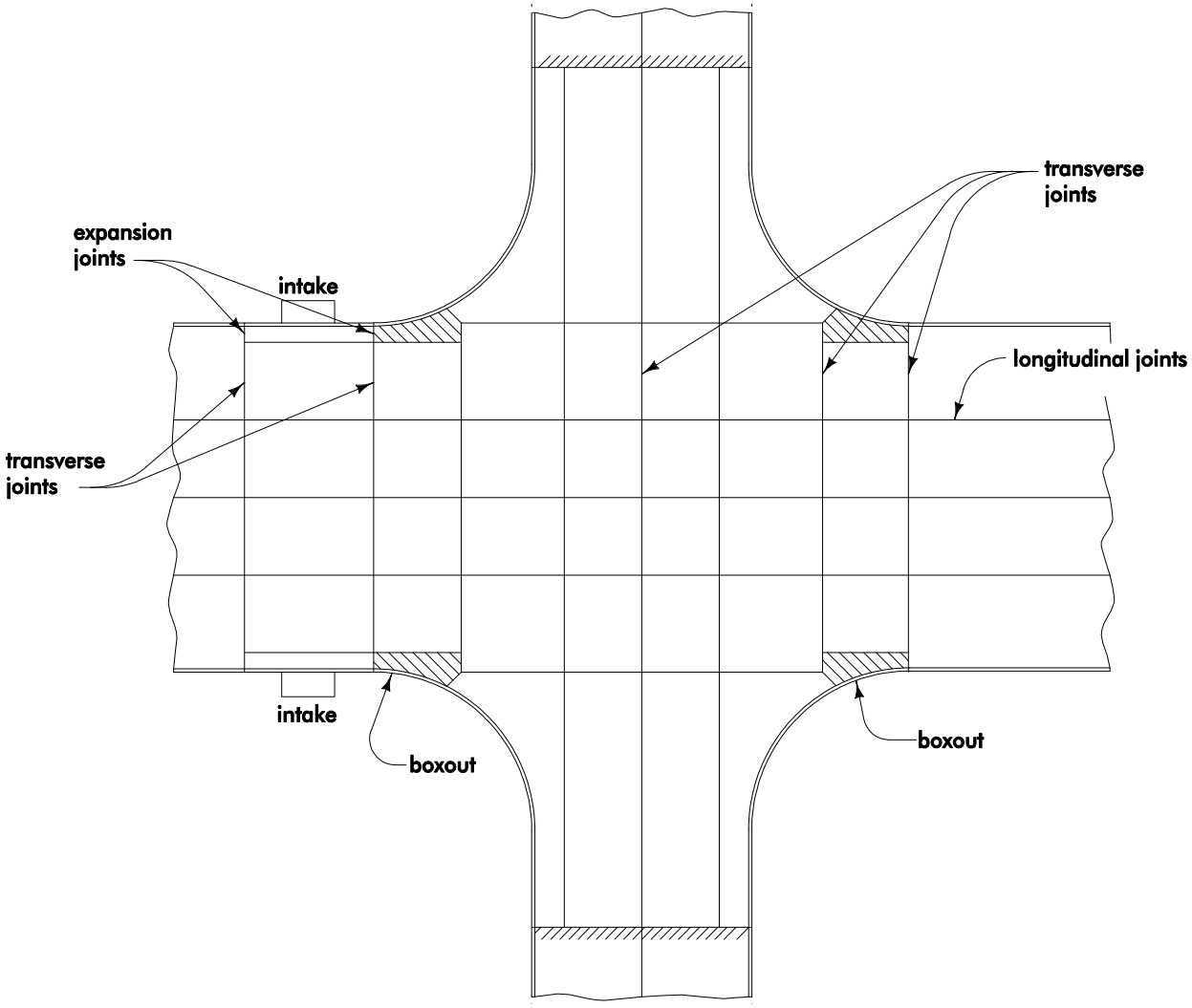
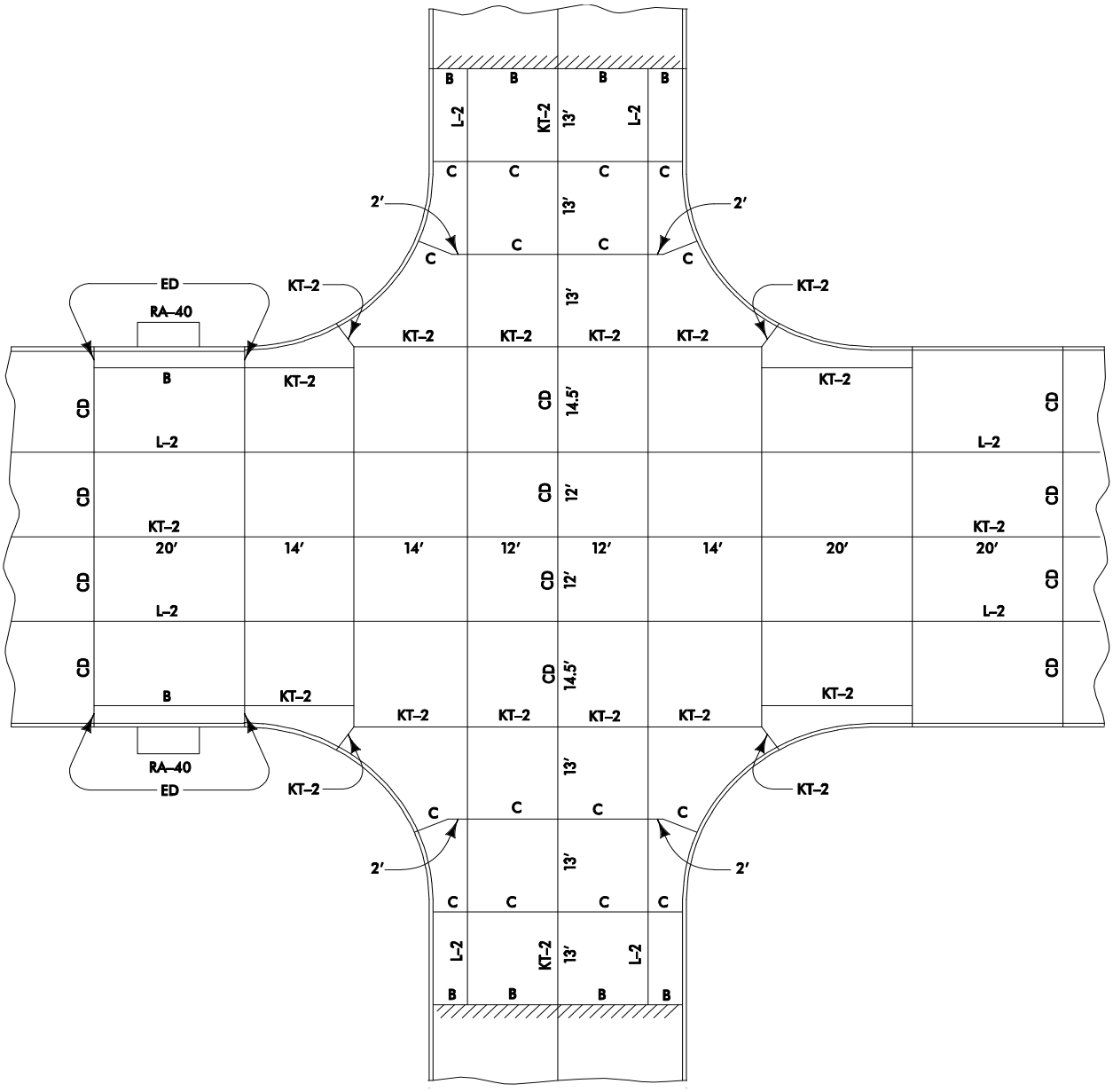
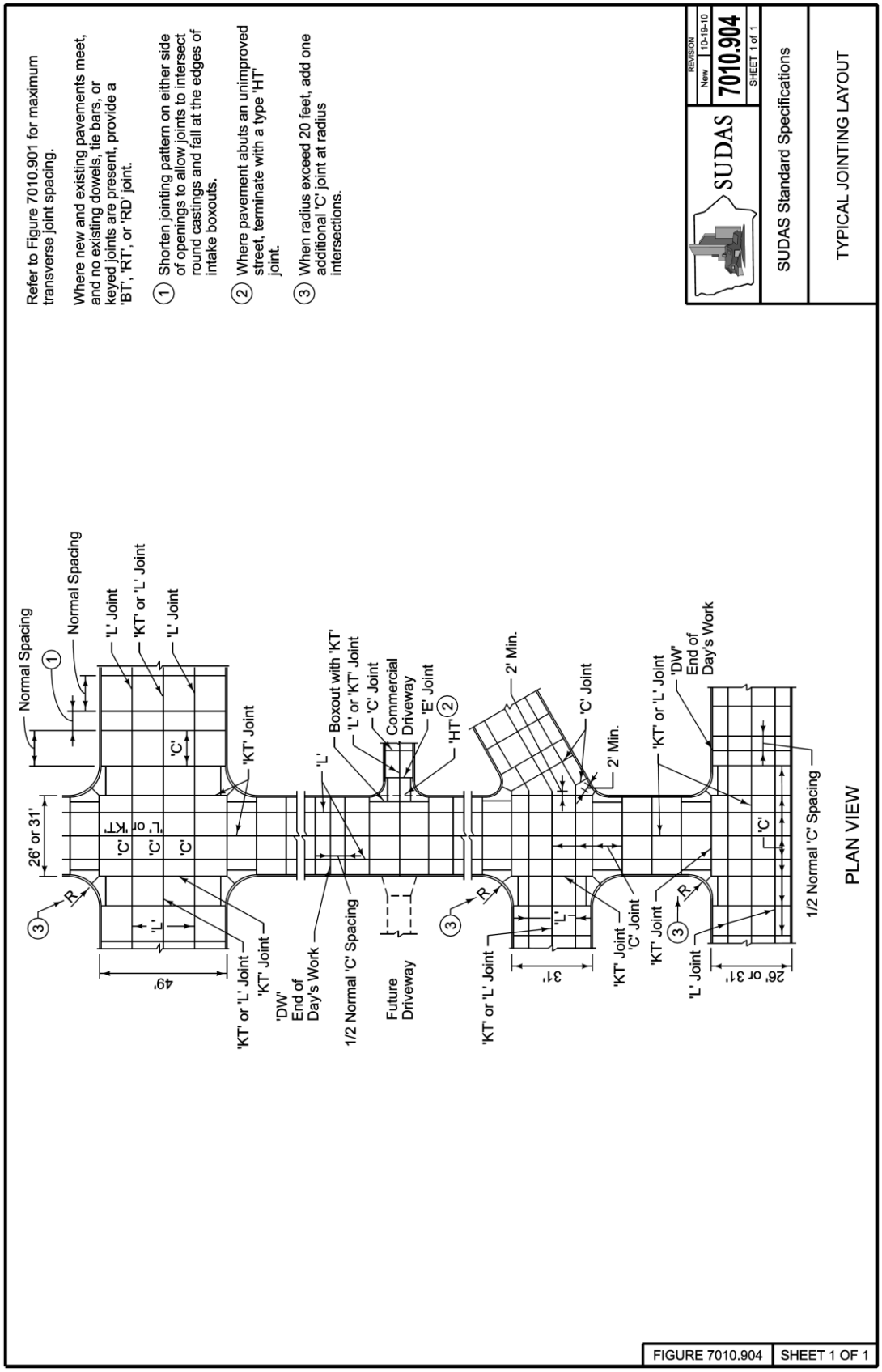


Figure 5G-3.02: Final Jointing Layout



Note: All longitudinal joints will be either KT-2 or L-2 unless otherwise indicated.

Figure 5G-3.03: Typical Municipal Jointing at Multiple Intersection Locations
(SUDAS Specifications Figure 7010.904)



A. Jointing Urban Transition Areas

This section provides examples of how to joint transition areas, such as approaches, to intersections. Many times, approaches to intersections are wider than the street and thus require a transition section.

The importance of considering constructability when developing jointing layouts for transition areas cannot be overstated. As previously noted, lane delineation with jointing should not be the predominate factor in joint layouts, particularly in urban areas. Critical lane delineation can be handled with other methods, such as pavement markings and a raised island.

Therefore, adequate jointing should be governed by the function of the joint, proper load transfer, and constructability.

Two basic widening types (with and without medians) are shown in the following figures. There are:

1. **Two-lane to Three-lane:** (i.e. 31 foot to 41 foot)
   - Quarter-point jointing
     - Concentric widening (Figure 5G-3.04)
     - One side widening (Figure 5G-3.05)
   - Third-point jointing
     - Concentric widening (Figure 5G-3.06)
     - One side widening (Figure 5G-3.07)
   - Gutterline jointing
     - Concentric widening (Figure 5G-3.08)
     - One side widening (Figure 5G-3.09)

2. **Four-lane to Five-lane:**
   - Concentric widening (Figure 5G-3.10)
   - Widening one side (Figure 5G-3.11)

Figure 5G-3.04: Quarter-Point Jointing - Concentric Widening (31 Foot to 41 Foot)

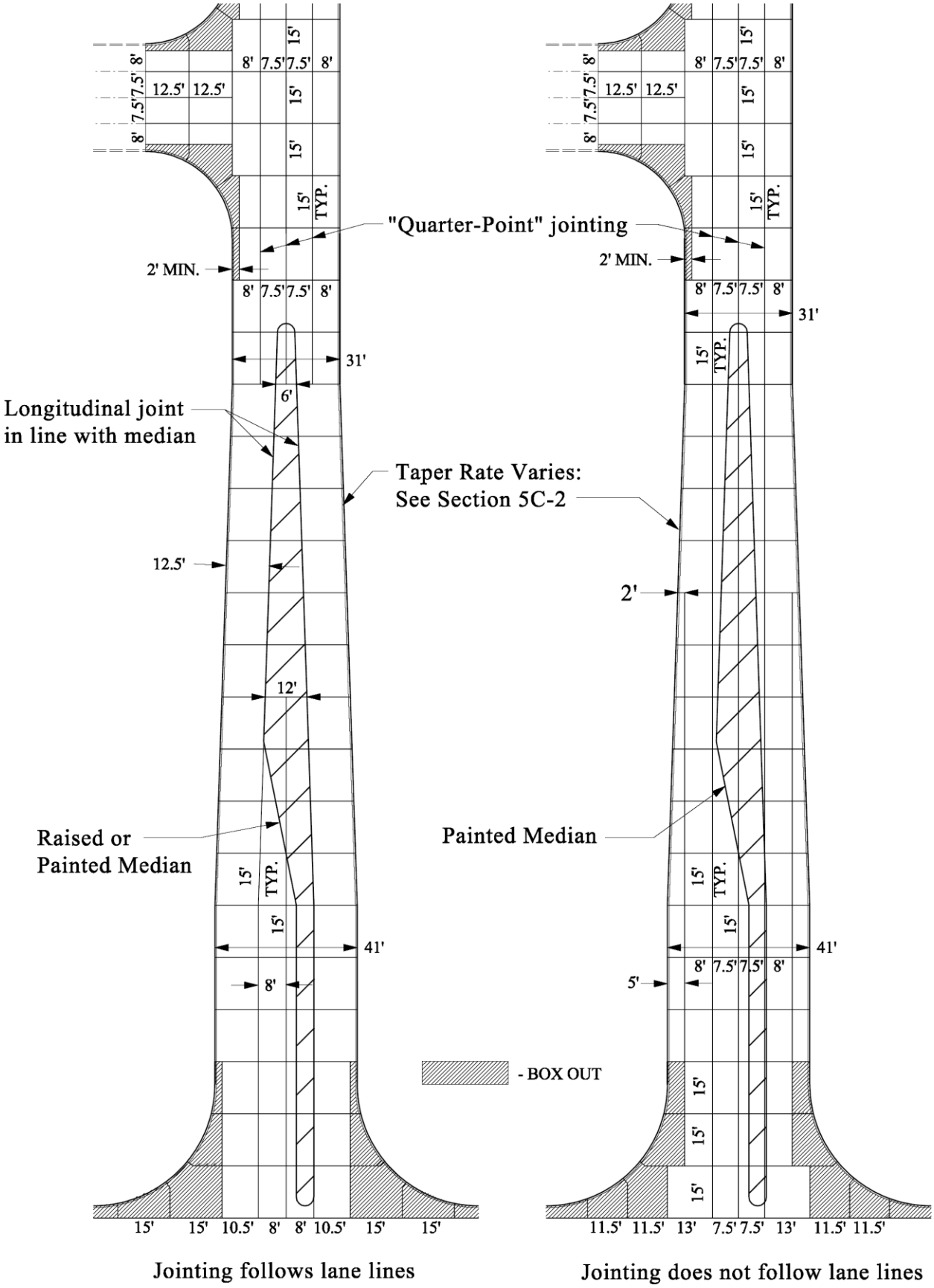


Figure 5G-3.05: Quarter-Point Jointing - Widening One Side (31 Foot to 41 Foot)

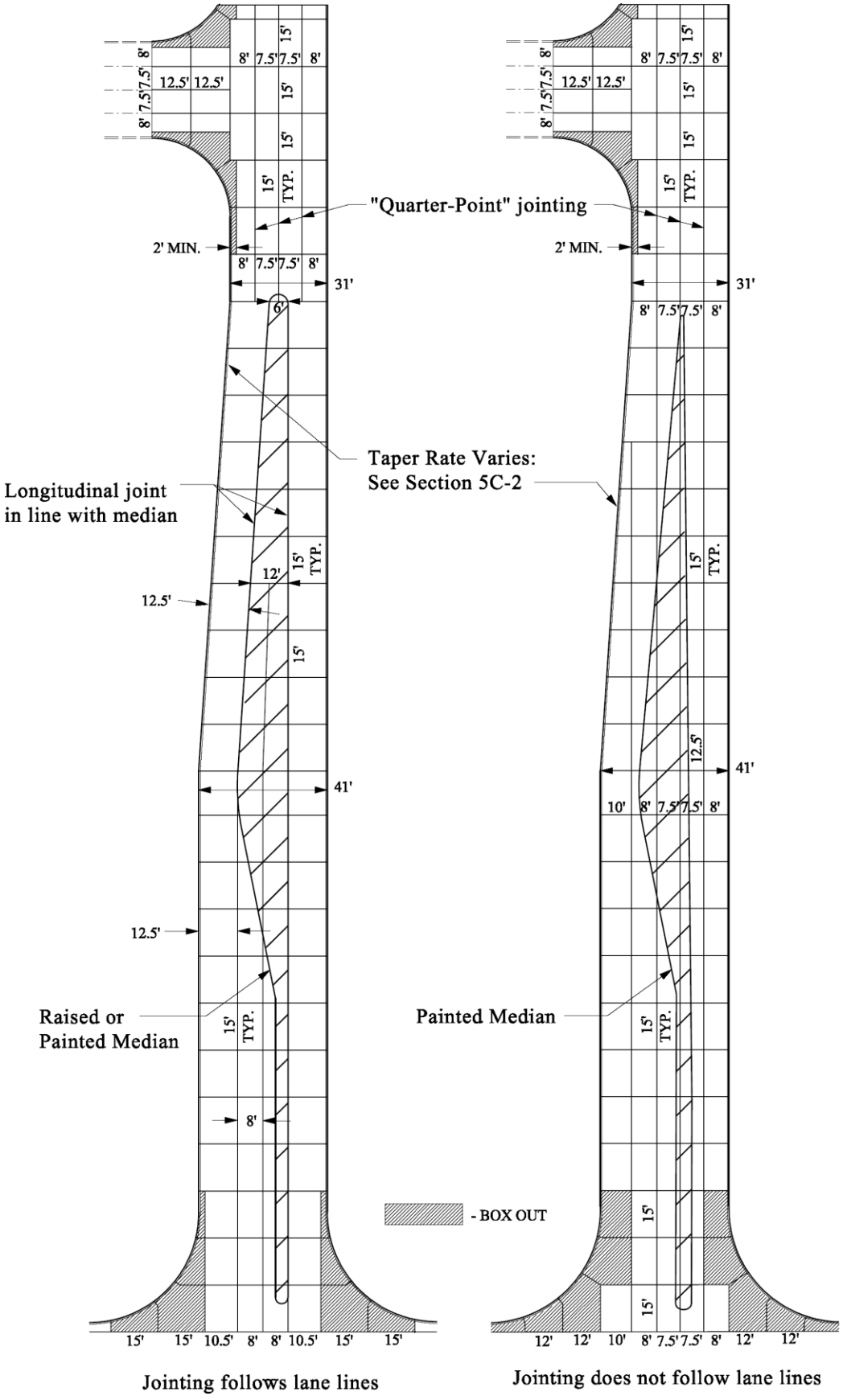


Figure 5G-3.06: Third-Point Jointing - Concentric Widening (31 Foot to 41 Foot)

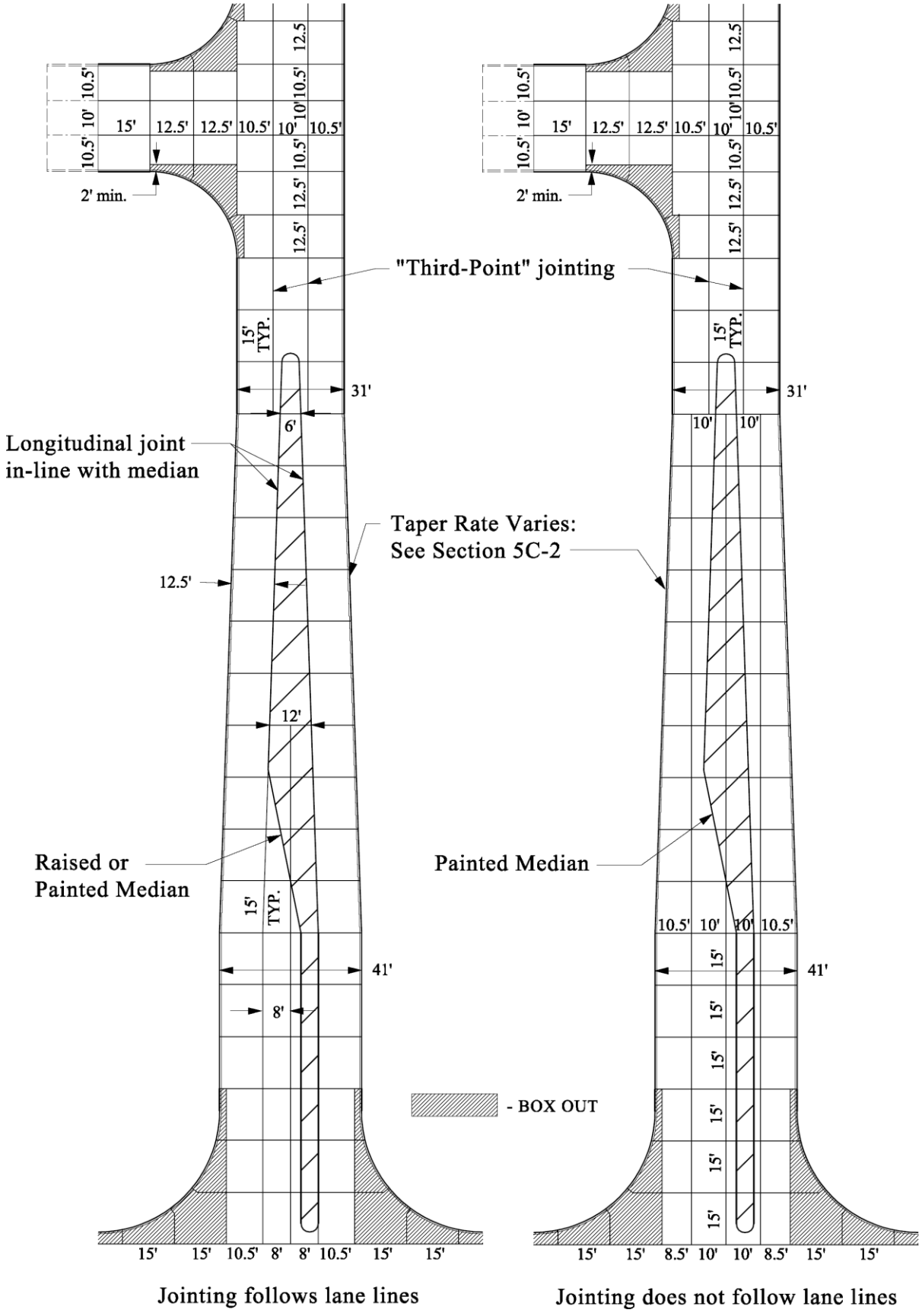


Figure 5G-3.07: Third-Point Jointing - Widening One Side (31 Foot to 41 Foot)

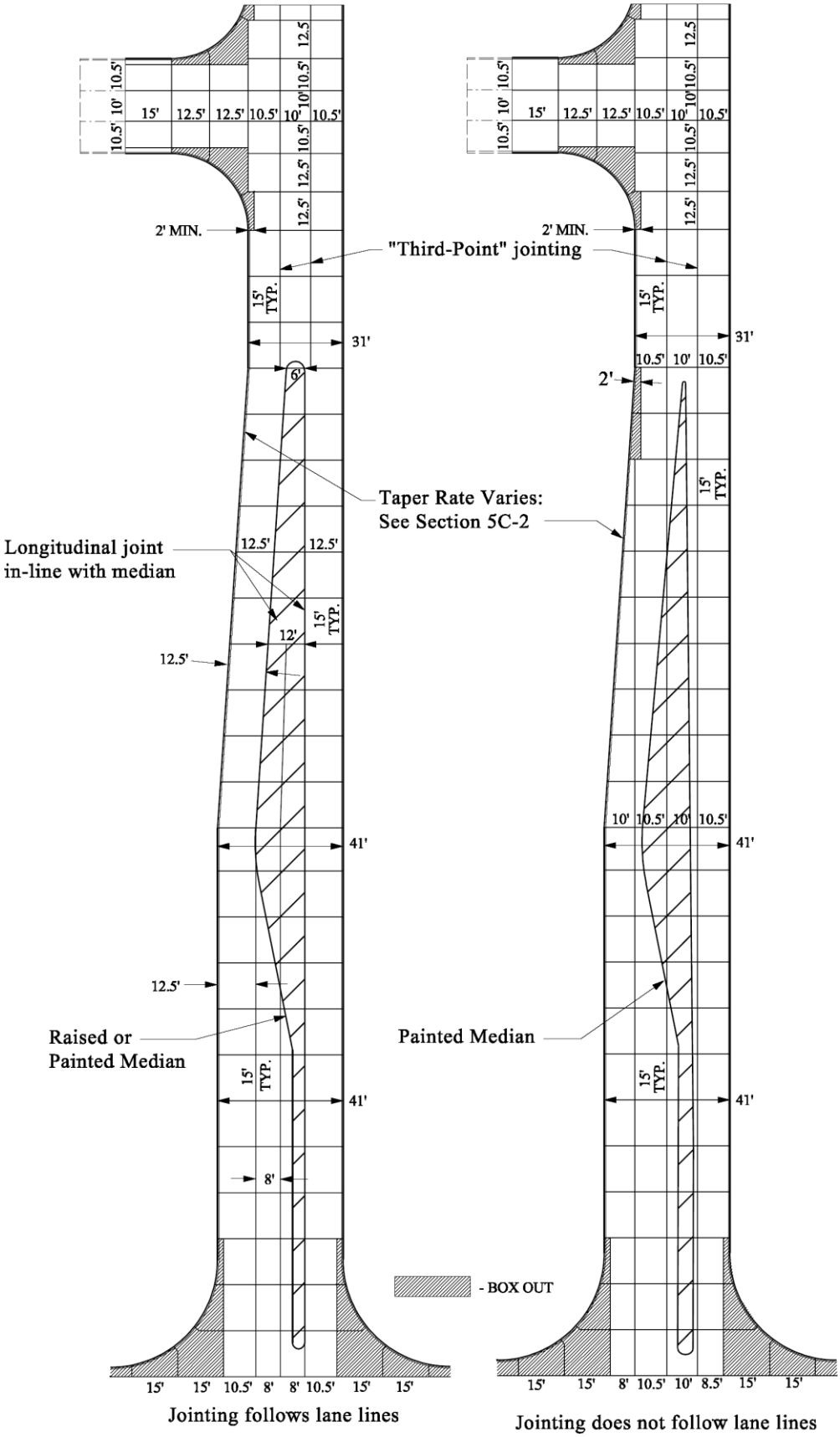


Figure 5G-3.08: Gutterline Jointing - Concentric Widening (31 Foot to 41 Foot)

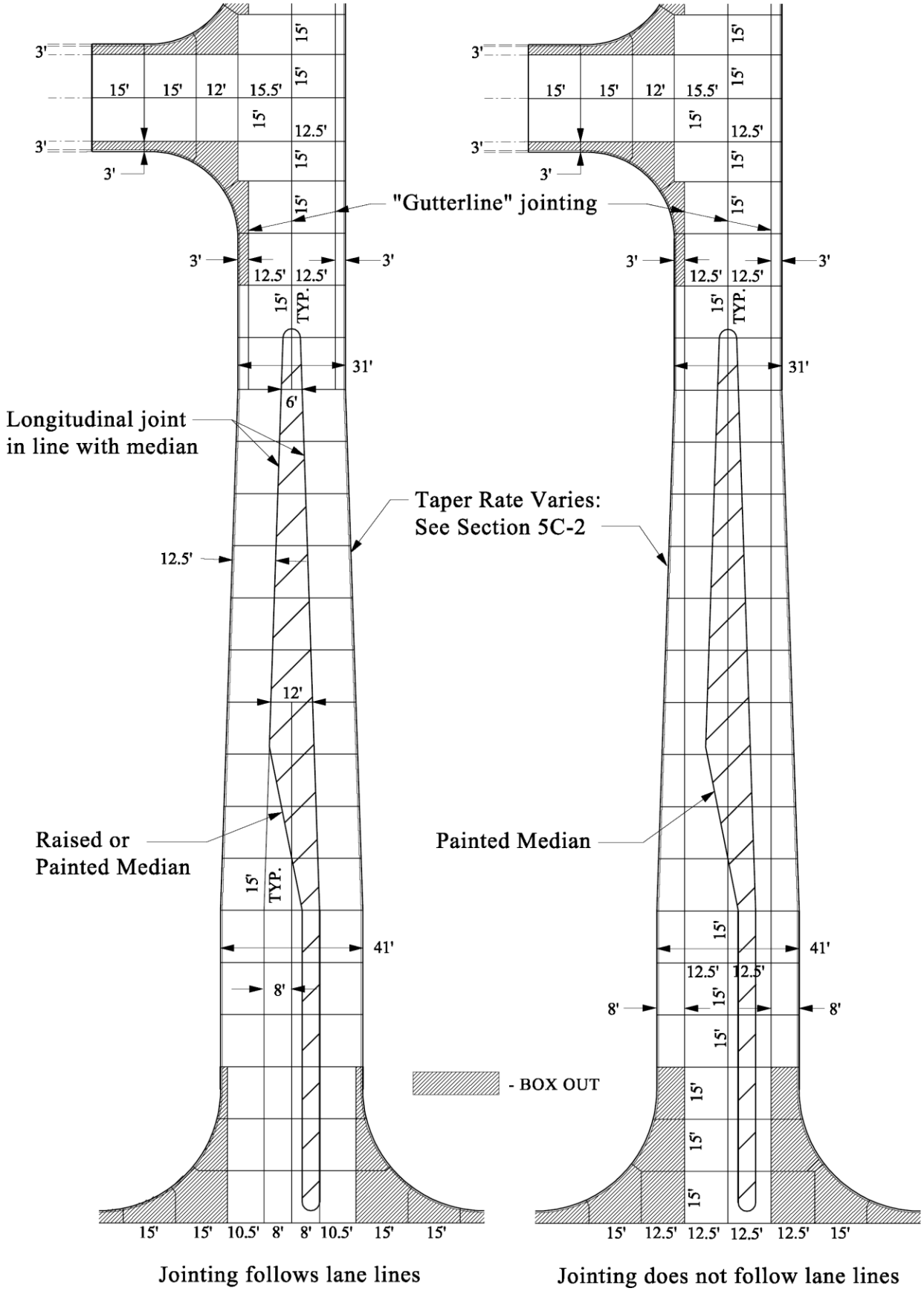


Figure 5G-3.09: Gutterline Jointing - Widening One Side (31 Foot to 41 Foot)

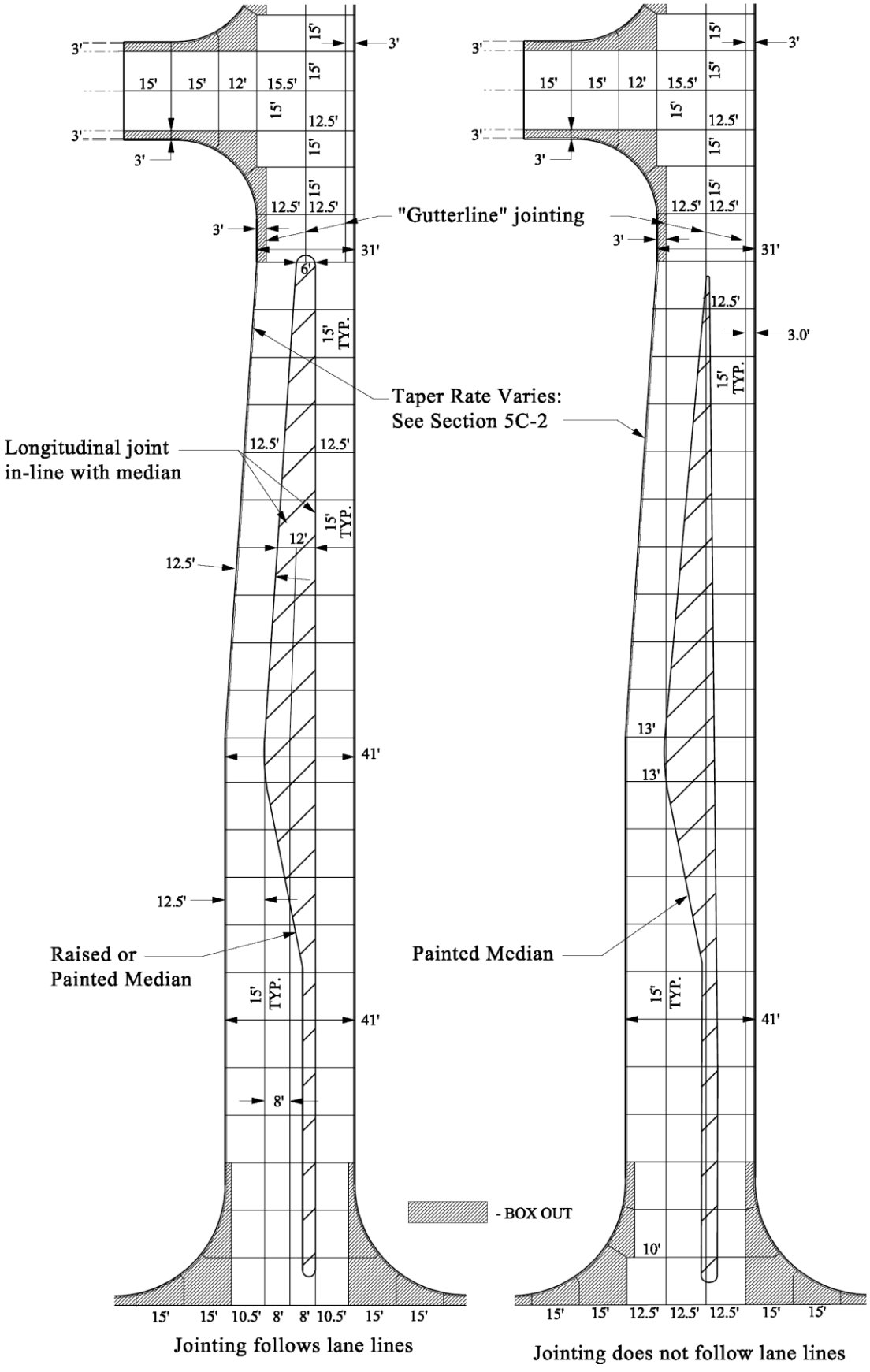


Figure 5G-3.10: Concentric Widening - Four Lane to Five Lane

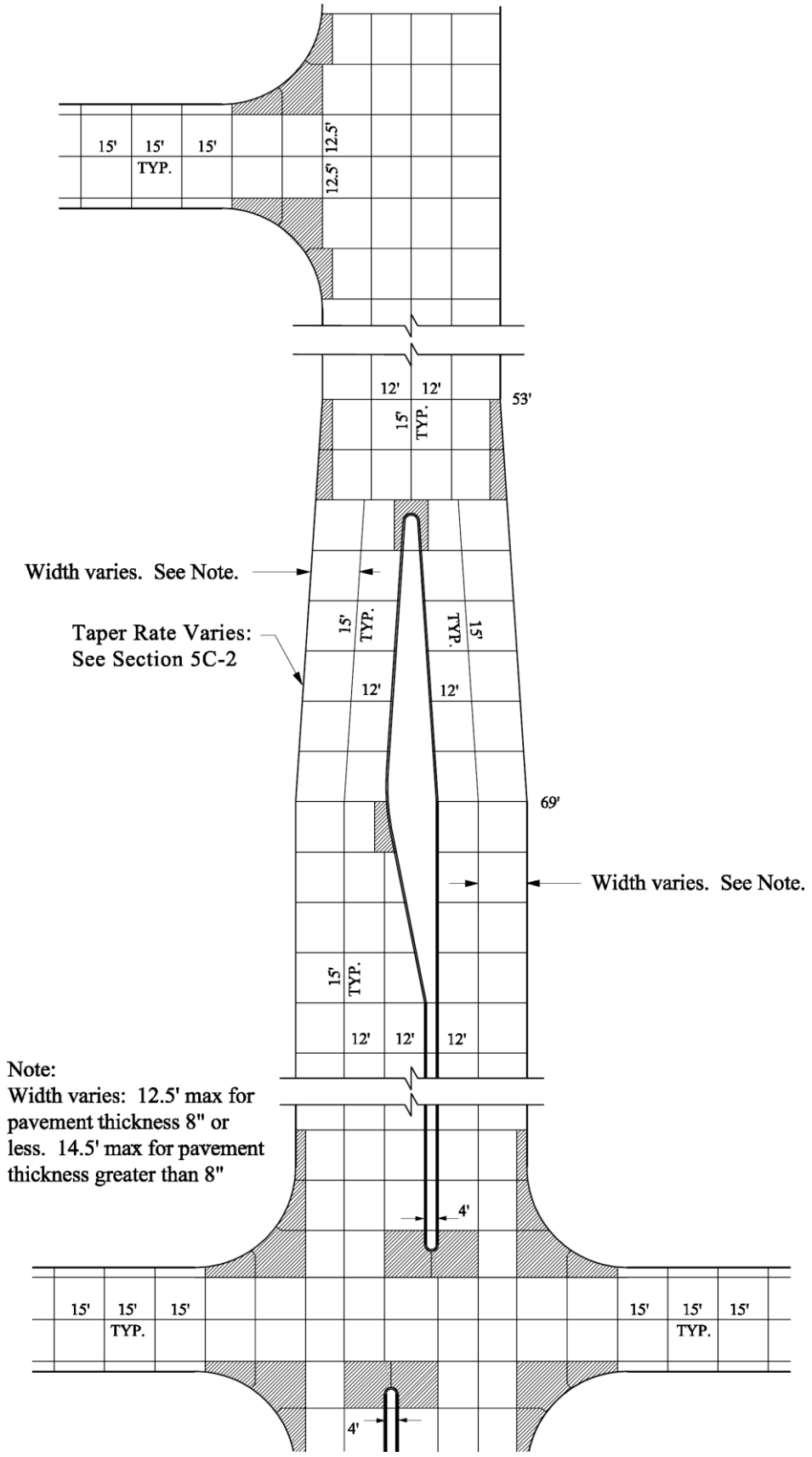
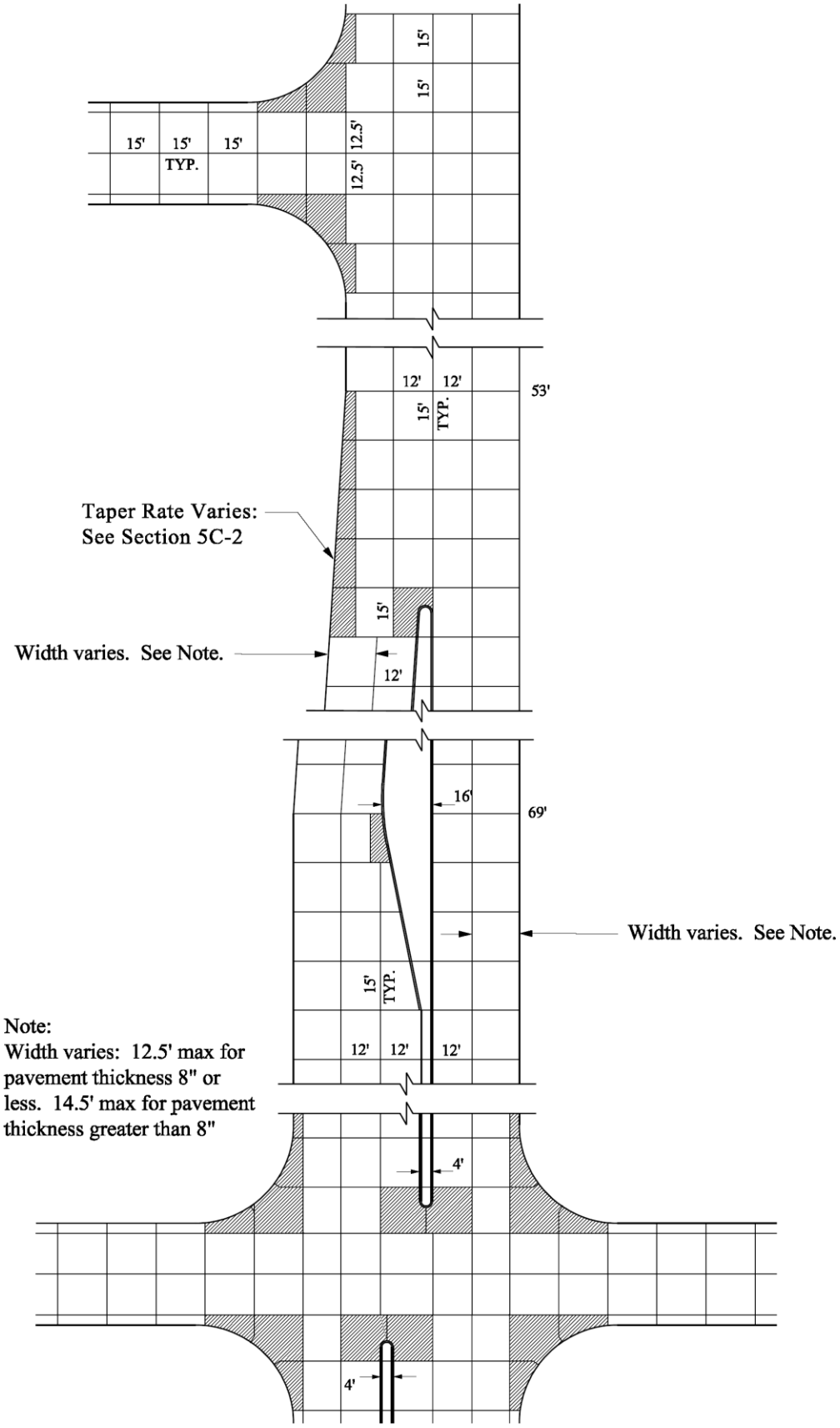


Figure 5G-3.11: Widening One Side - Four Lane to Five Lane



B. Jointing Cul-de-sacs

This section describes how to joint a cul-de-sac. The process is illustrated through an example of a street that is terminated with a cul-de-sac. Assume the pavement thickness is 7 inches.

Step 1: Locate Longitudinal Joints

The longitudinal joints running down the street should be extended into the cul-de-sac. The remaining longitudinal joints in the cul-de-sac should be placed roughly a lane width apart - somewhere in the range of 8 to 12 feet is acceptable.

A BT-1 or L-1 is an appropriate longitudinal joint, since the pavement thickness is less than 8 inches.

Step 2: Locate Transverse Joints

The next step is to place the transverse joints. The maximum spacing for transverse joints is 15 feet and the minimum spacing is 12 feet. Therefore, the joints within the cul-de-sac should be spaced within this range (see Figure 5G-3.12).

A C joint is the appropriate joint to use since the pavement thickness is less than 8 inches.

Step 3: Extend Joints Through the Free Edge of the Pavement

When extending the previously placed joints through the free edge of the pavement, the acute angle between the joint and the pavement edge (and between the joint and other joints) must be greater than or equal to 70 degrees. Also, all joints should be at least two feet long. Details A, B, and C in Figure 5G-3.13 illustrate how this can be accomplished.

- Detail A shows a transverse joint that is extended through the free edge of the pavement unaltered. These are acceptable because all angles between the transverse joint and the longitudinal joints and between the transverse joint and the free edge of the pavement are greater than 70 degrees.
- Detail B uses a dashed line to show the original position of a transverse joint whose angle, with the free edge of the pavement, is less than 70 degrees. This joint should be skewed to make it perpendicular to the free edge of the pavement, as shown by the solid line.
- Detail C illustrates a situation where skewing the joint to make it perpendicular to the free edge of the pavement would cause the angle between the joint and a longitudinal joint to be less than 70 degrees (shown by the dashed line). When this situation occurs, the joint is extended a minimum of two feet beyond the longitudinal joint, and then it is skewed to make it perpendicular to the free edge of the pavement. Both segments of the joint should be at least two feet long.

Step 4: Label Joints

The completed jointing layout for the cul-de-sac is shown in the figures that follow. The L-1 and BT-1 joints may be used interchangeably, at the contractor's discretion, depending on the paving sequence. Therefore, the designer may identify the longitudinal joints as either L-1 or BT-1 on the jointing layout.

Because the majority of the joints are either the C or the BT-1 or L-1, it is not necessary to identify every joint on the jointing layout. A note on the plan describing the transverse joints as C and longitudinal joints as L-1 or BT-1 except as noted otherwise is sufficient, provided that a few key joints on the diagram are identified. Whenever a series of joints changes to a different type of joint, the joint at the location of change is identified. Any joint that may be a source of confusion should also be labeled.

Joint lengths are also shown on the jointing layout, normally rounded to the nearest foot. Similar to labeling joint types, not every length needs to be indicated. However, any length that cannot be inferred from the diagram should be labeled.

Figure 5G-3.12: Placement of Longitudinal and Transverse Joints

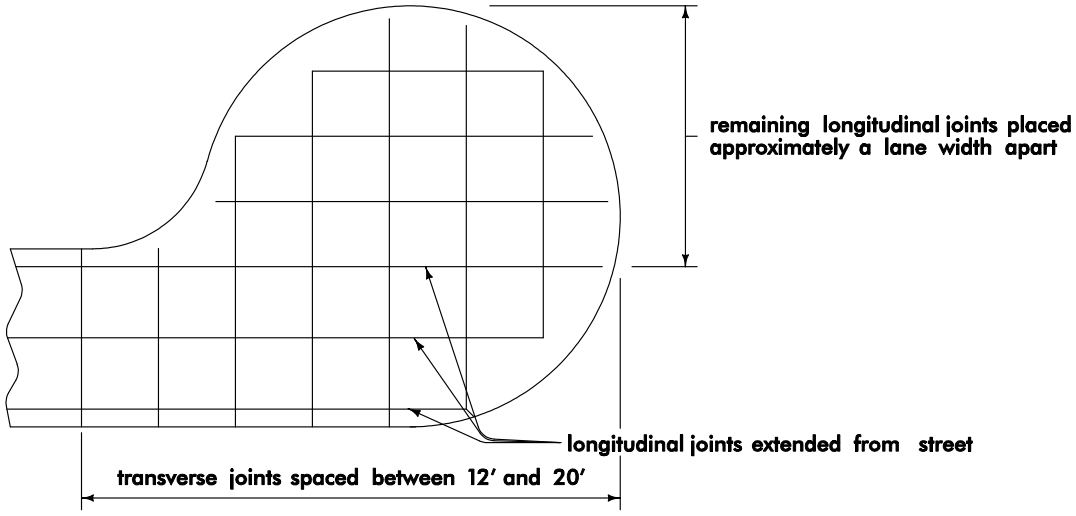
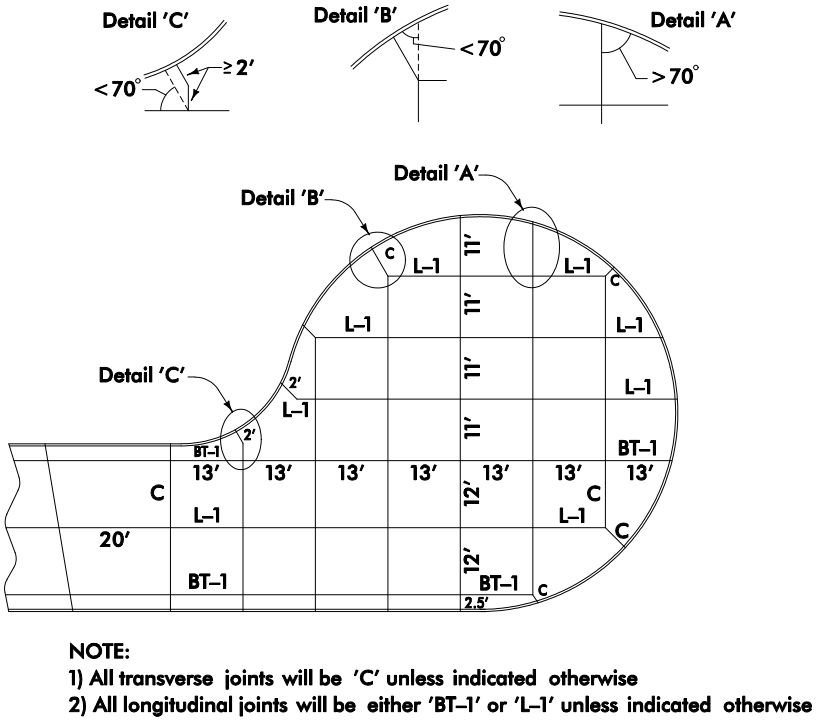


Figure 5G-3.13: Final Jointing Layout - Gutterline Jointing Examples



NOTE:
1) All transverse joints will be 'C' unless indicated otherwise
2) All longitudinal joints will be either 'BT–1' or 'L–1' unless indicated otherwise

Figure 5G-3.14: Cul-de-sac Joint Locations - Quarter-point Jointing Examples
(SUDAS Specifications Figure 7010.905, sheet 1)

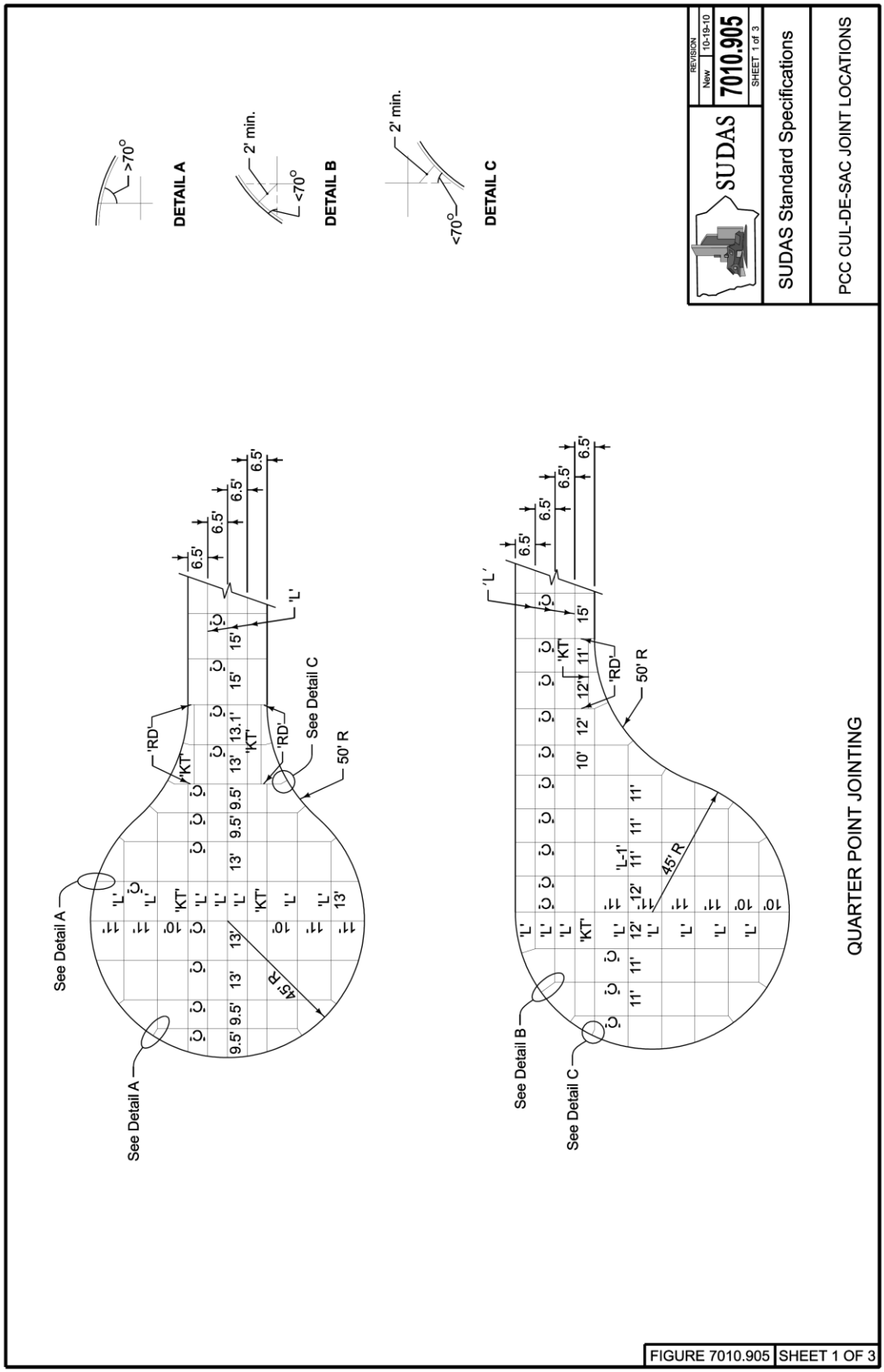


Figure 5G-3.15: Cul-de-sac Joint Locations - Third-point Jointing Examples
(SUDAS Specifications Figure 7010.905, sheet 2)

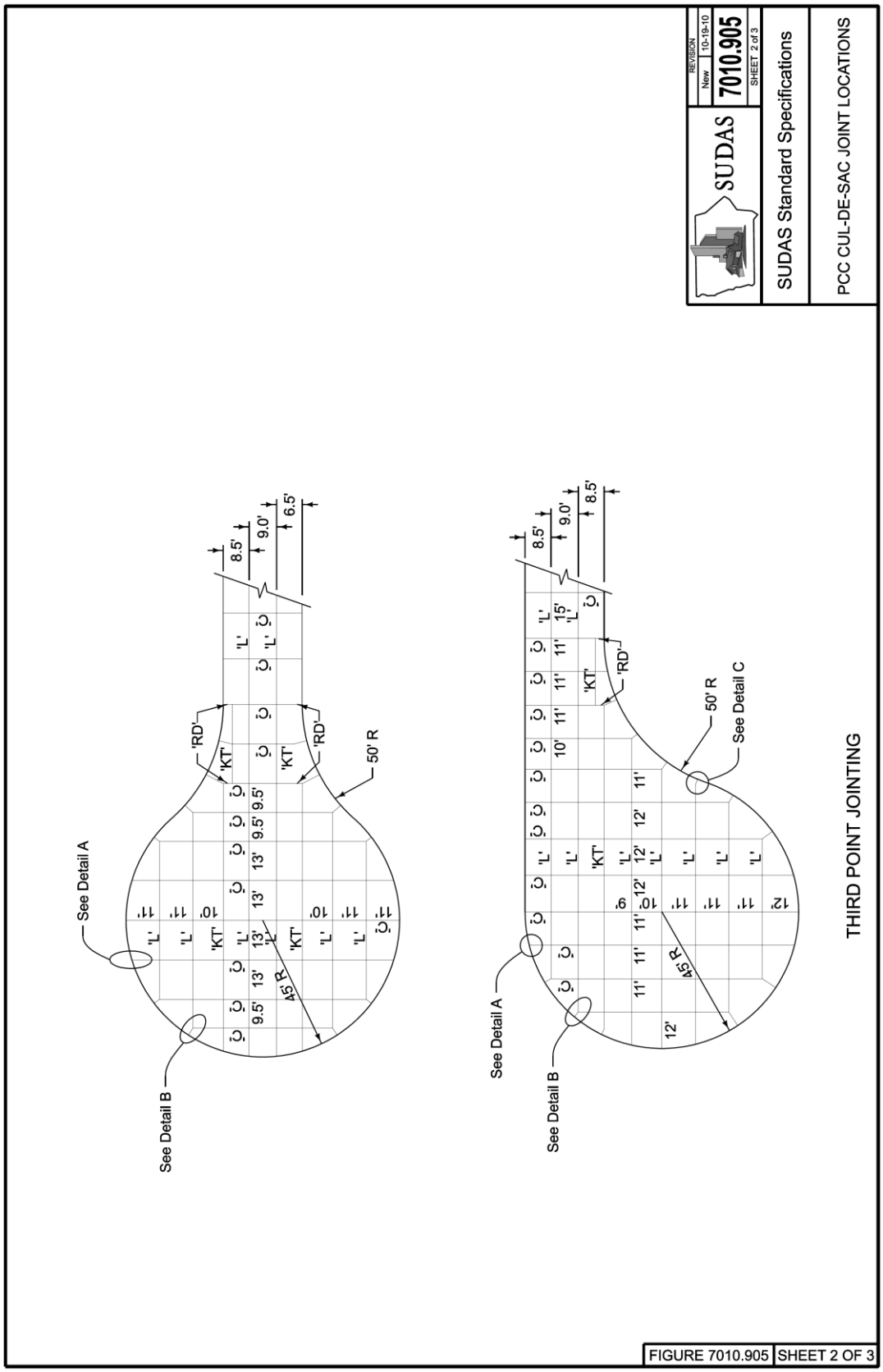
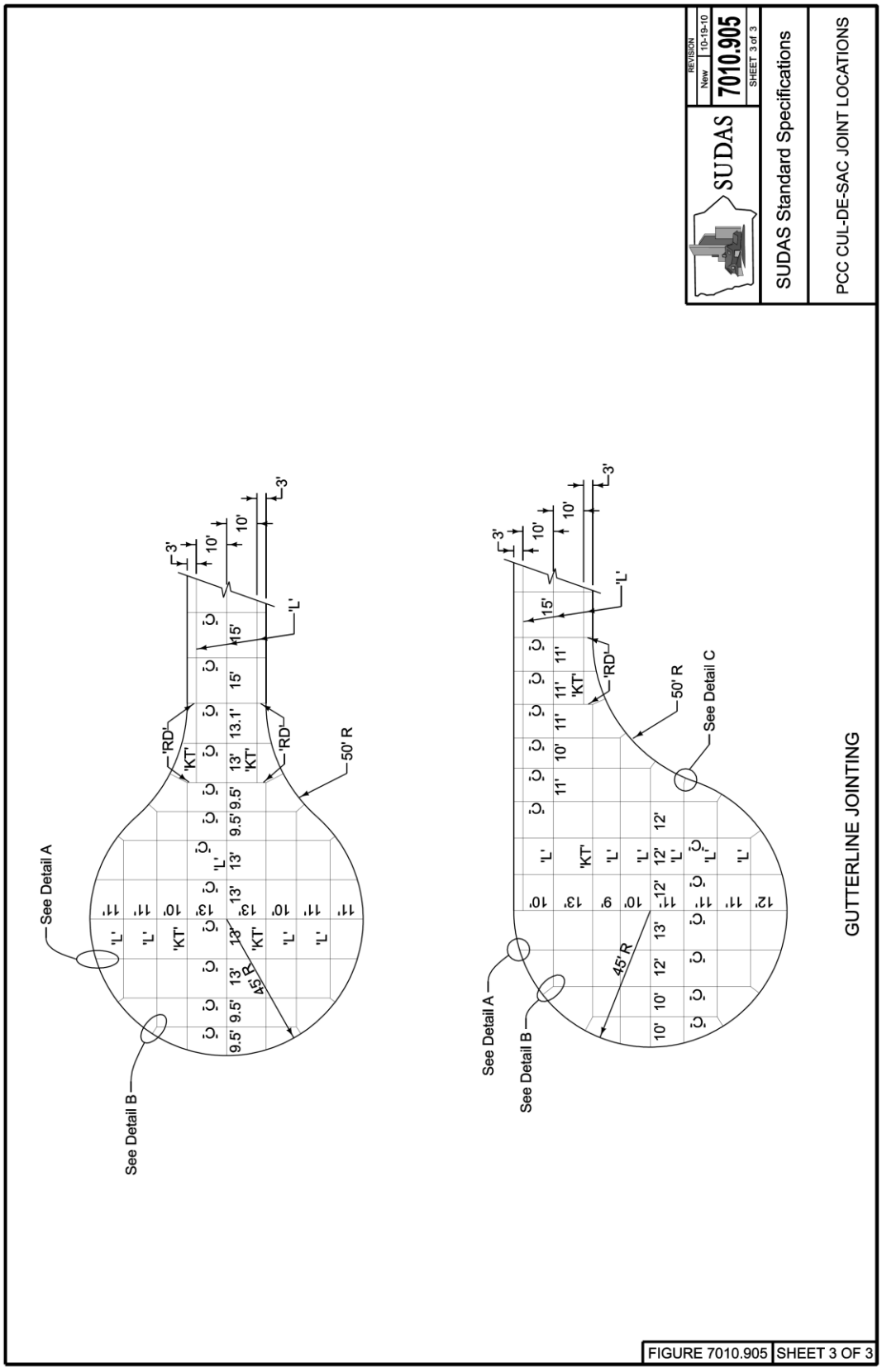


Figure 5G-3.16: Cul-de-sac Joint Locations - Gutterline Jointing Examples
(SUDAS Specifications Figure 7010.905, sheet 3)



Jointing Rural Intersections

This section describes how to joint rural intersections by following the guidelines outlined in Iowa DOT Design Manual Section 7A-3. The first example illustrates a step-by-step process for jointing a T-intersection. The second example discusses the jointing of an intersection at a divided highway. Even though not all rural intersections will be exactly like the ones in these examples, the process described is applicable to other layouts.

A. Example 1: T-Intersection

The first example is a T-intersection of a rural two-lane highway and a paved sideroad. The intersection has returns on each side (see Figure 5G-4.01) and the pavement thickness is 10 inches. The design year truck volume on the sideroad is 250 vpd.

Step 1: Place Joints with Predetermined Locations

1. **Longitudinal Joints:** Because the location of longitudinal joints for both the mainline and the sideroad are predetermined by the lane pavement width, these joints should be placed first. Within the intersection, the road that is paved first, or already exists, determines which joints are longitudinal and which are transverse. In this example, assume that the mainline is paved first. Since the mainline is a rural two lane highway, the longitudinal joints are spaced at the lane pavement width. The longitudinal joints running down the centerline and edges of the sideroad define the locations of the first transverse joints for the mainline (see Figure 5G-4.01).

   To determine an appropriate longitudinal joint to use, refer to SUDAS Specifications Figure 7010.101. Normally, the type of joint used depends on the pavement thickness. Since the pavement thickness is greater than 8 inches in this case, either a KT-2 or an L-2 joint is appropriate.

2. **Joints at End-of-taper:** The only other joints with predetermined locations are the transverse joints that are placed where the end-of-taper sections terminate. End-of-taper sections are 2 foot wide sections placed at the ends of an intersection return (see Figure 5G-4.01). They are used to prevent the return from narrowing to a point as it intersects with the pavement. Concrete less than 2 feet in width is weak and cracks readily.

   As Figure 5G-4.01 shows, normal practice is to place a transverse joint in the mainline or sideroad pavement where the end-of-taper section terminates. Figure 5G-2.02 in Section 5G-2 indicates a CD joint should be used on the mainline if the pavement thickness is greater than or equal to 8 inches. On the sideroad, CD joints are also used since the design year truck volume is greater than 200 vpd (C joints could be used on the sideroad if the design year truck volume was less than 200 vpd).

   Note that the transverse joints within the intersection are not skewed.

Step 2: Locating Difficult Joints

Difficult locations to joint, such as intersection returns and traffic islands, are addressed next. After joints have been placed in these locations, the rest of the joints can be worked in around them.

1. **Intersection Returns:** The two intersection returns are shaded in Figure 5G-4.01. To help vehicles negotiate the turn, a curved longitudinal joint (normally offset 12 feet from the free edge of the pavement) is placed in the intersection return to delineate the turning path. A second curved longitudinal joint (normally offset 24 feet from the free edge of the pavement) is placed if enough area is available.

2. **Traffic Islands:** Joint design at the traffic islands is not an exact process. It is done by trial-and-error until satisfactory results are achieved.

   The first thought may be to place CD transverse joints at every radius point of the island (see Figure 5G-4.01, Detail A). However, with this layout, the 20 foot maximum and 12 foot minimum spacings for a CD joint are violated.

   Detail B shows joints at the desired 20 foot interval. Although the spacing of this placement is correct, an awkward area of pavement is formed and a crack is likely to develop as shown in Detail B.

   Detail C illustrates a combination of the methods used in the first two details. No rules of spacing are violated and no awkward areas of pavement exist.

   The transverse joints attached to the island are extended across the sideroad and mainline pavements and across the intersection return adjacent to the island, as shown in Figure 5G-4.01. The joints used in one area must also be acceptable for any other areas into which they are extended. If the extended joints do not satisfy spacing or other criteria in any adjacent areas, they must be redesigned in the original area.

Step 3: Locating Remaining Joints

After the joints at difficult locations are located, the remaining joints (generally transverse joints) are placed in appropriate locations. As noted in Step 1, the appropriate transverse joint for both the mainline and the sideroad is the CD joint. The maximum spacing for CD joints is 20 feet and the minimum spacing is 12 feet. Therefore, the remaining areas that need transverse joints should have CD joints spaced within this range.

1. **Mainline and Sideroad:** The location of the remaining transverse joints on the mainline and sideroad is largely determined by the location of joints already placed in Steps 1 and 2 (see Figure 5G-4.01). The remaining joints are spaced between 12 and 20 feet between these already-placed joints. However, you must also consider how these joints will be extended into the returns (described below).

2. **Intersection Returns:** After the transverse joints have been located in the mainline and the sideroad, they are extended into the intersection returns to be used as transverse joints for those areas as well. As with other transverse joints, those in intersection returns must intersect with the free edge of the pavement. However, the acute angle between the joint and the pavement edge (and between the joint and other joints) must be greater than or equal to 70 degrees. Details A, B, C, and D in Figure 5G-4.02 illustrate how to intersect joints with the free edge of the pavement (and with other joints) under various conditions.

- Detail A shows a transverse joint that intersects with the free edge of the pavement unaltered. This is acceptable because all angles between the transverse joint and the longitudinal joints and between the transverse joint and the free edge of the pavement are greater than 70 degrees.
- Detail B uses a dashed line to show the original position of a transverse joint whose angle with the free edge of the pavement is less than 70 degrees. This joint should be skewed to make it perpendicular to the free edge of the pavement, as shown by the solid line.
- Detail C illustrates a situation where skewing the joint to make it perpendicular to the free edge of the pavement causes the angle between the joint and the edge of the mainline to be less than 70 degrees. When this situation occurs, the joint is extended a minimum of 2 feet beyond the edge of the mainline or sideroad, and then it is skewed to make it perpendicular to the free edge of the pavement.
- Detail D shows the curved longitudinal joints that were placed in the intersection return in Step 2. Each of these joints terminates at an intersection with a transverse joint. The intersection of these joints is required to be at least 2 feet from the edge of the mainline or sideroad. This requirement determines the appropriate transverse joint at which the longitudinal joint terminates. The dashed line in the detail indicates the position of the longitudinal joint if it is extended too far. Because the intersection with the transverse joint is less than 2 feet from the pavement edge, the longitudinal joint is terminated at the previous transverse joint.

After all joints are placed, the layout should be checked to ensure that all joint spacings and angles are acceptable. If they are not, the spacing of the mainline or sideroad joints may need to be changed, one or more joints may be added, or joints within the returns may be modified. Figure 5G-4.02 shows all of the transverse joints appropriately placed.

Step 4: Label Joints

The completed jointing layout of the T-intersection is shown in Figure 5G-4.03. As stated on SUDAS Specifications Figure 7010.101, the L-2 and KT-2 joints may be used interchangeably at the contractor's discretion, depending on the paving sequence. Therefore, the designer may identify the longitudinal joints as either L-2 or KT-2 on the jointing layout. The transverse joints in the end-of-taper sections are C joints because they are only 2 feet long, which are not long enough to use a doweled transverse joint like the CD. The joints on the right side of the traffic island are also C joints.

It is not necessary to identify every joint on the jointing layout. A few key joints on the diagram should be identified and whenever a series of joints changes to a different type of joint, the joint at the location of the change should be identified. Also, any joint that may be a source of confusion should be identified.

Joint lengths are also shown on the jointing layout, normally rounded to the nearest foot. Similar to labeling joint types, not every length needs to be indicated. However, any length that cannot be inferred from the diagram should be labeled. For example, the distance the mainline or sideroad transverse joints extend into the intersection returns before being skewed perpendicular to the free edge of the pavement, should be dimensioned (see Figure 5G-4.03).

B. Example 2: Intersection at a Divided Highway

The jointing design process for a four-way intersection at a divided highway is basically the same as the T-intersection, except that there is also a paved median opening to deal with.

As with the T-intersection, start out by placing the longitudinal joints that are predetermined by the lane pavement width. After doing this, place longitudinal joints through the opening (see Figure 5G-4.04). The edges of the left-turn lanes define the location of two of these joints. The remaining longitudinal joints in the opening are spaced roughly a lane width apart - somewhere in the range of 10 to 16 feet is acceptable.

After this, the process is basically the same as the T-intersection:

- Place the transverse joints at the end-of-taper sections.
- Place the curved longitudinal joints in the return.
- Place the transverse joints around the islands. Figure 5G-4.04 illustrates the design through this point.
- Place the remaining transverse joints and extend them into the returns and into the median opening. Refer back to the T-intersection example for details on how the joints should intersect with the free edge of the pavement and with other joints.
- Label the joints.

Figure 5G-4.05 illustrates the final jointing layout.

Figure 5G-4.01: Placement of Predetermined and Difficult Joints

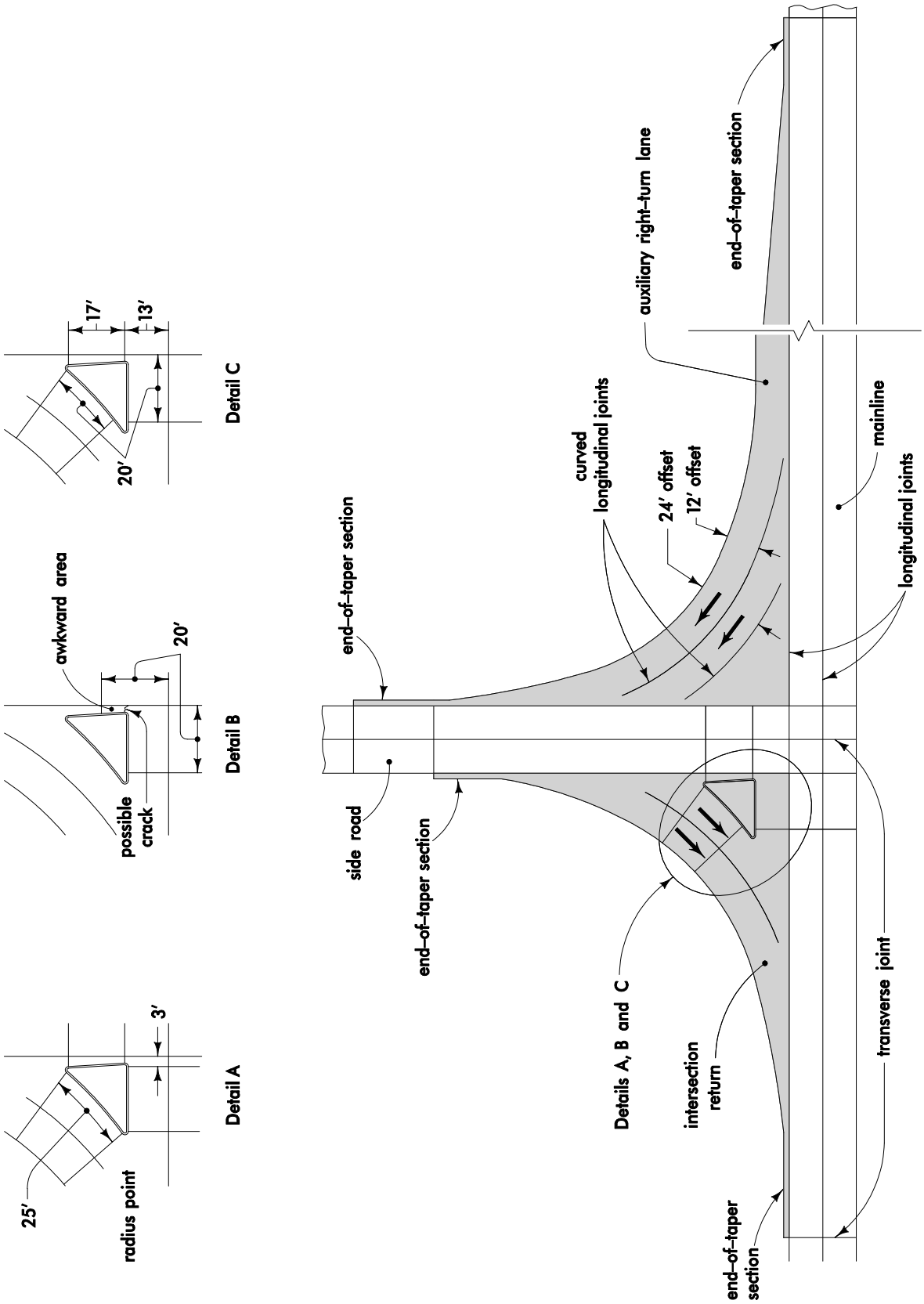


Figure 5G-4.02: Placement of Remaining Joints

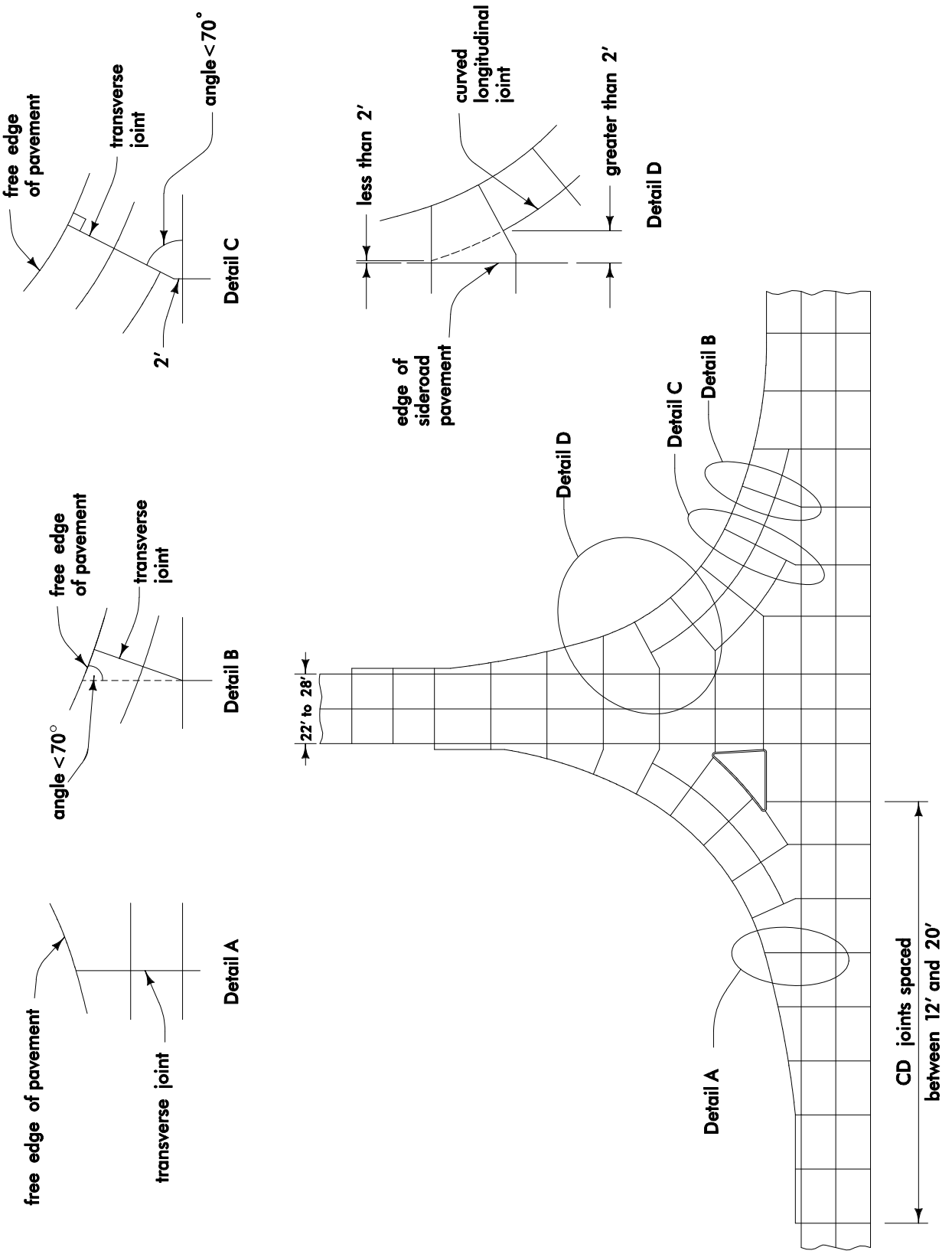


Figure 5G-4.03: Final Joint Layout

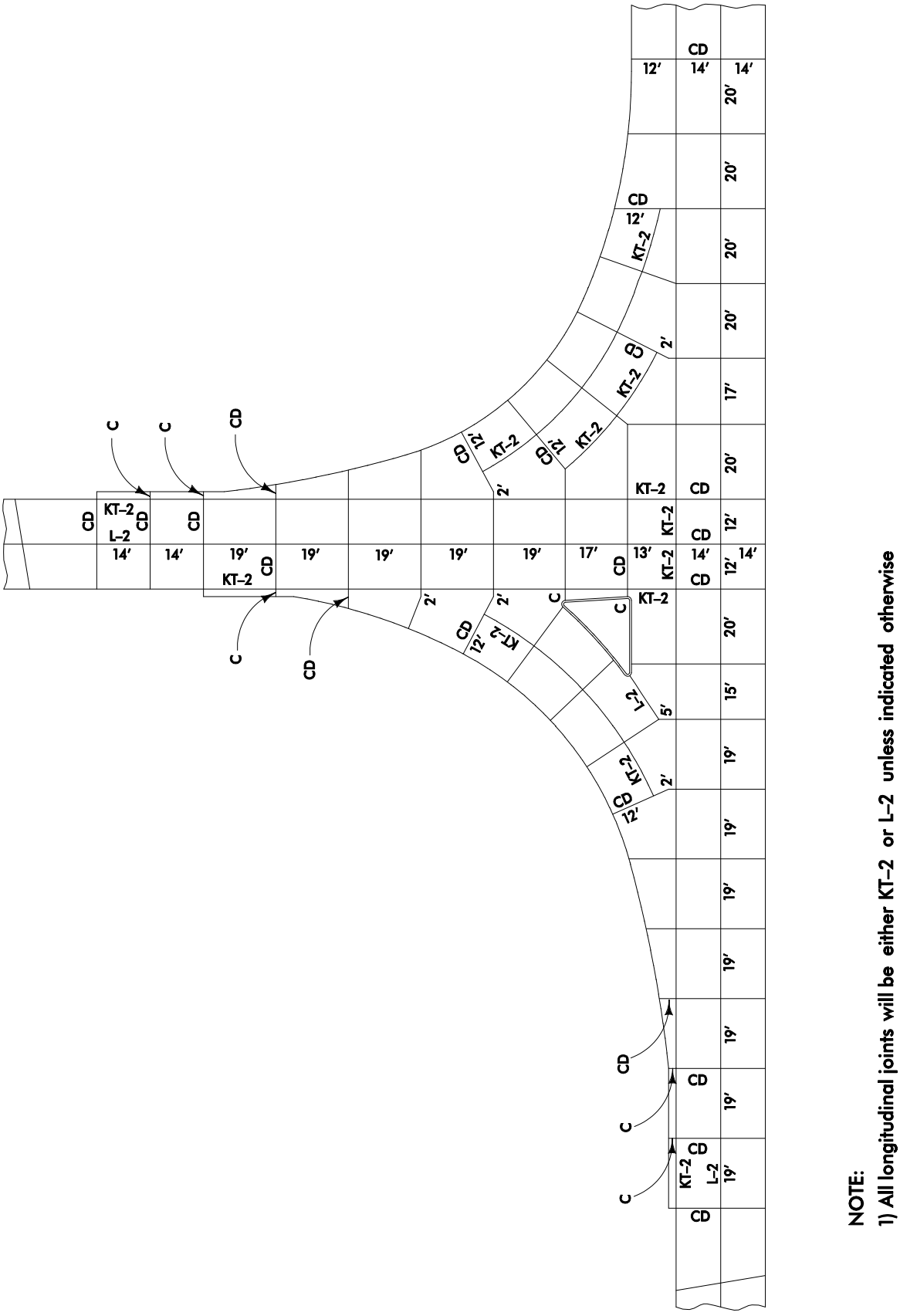


Figure 5G-4.04: Placement of Predetermined and Difficult Joints

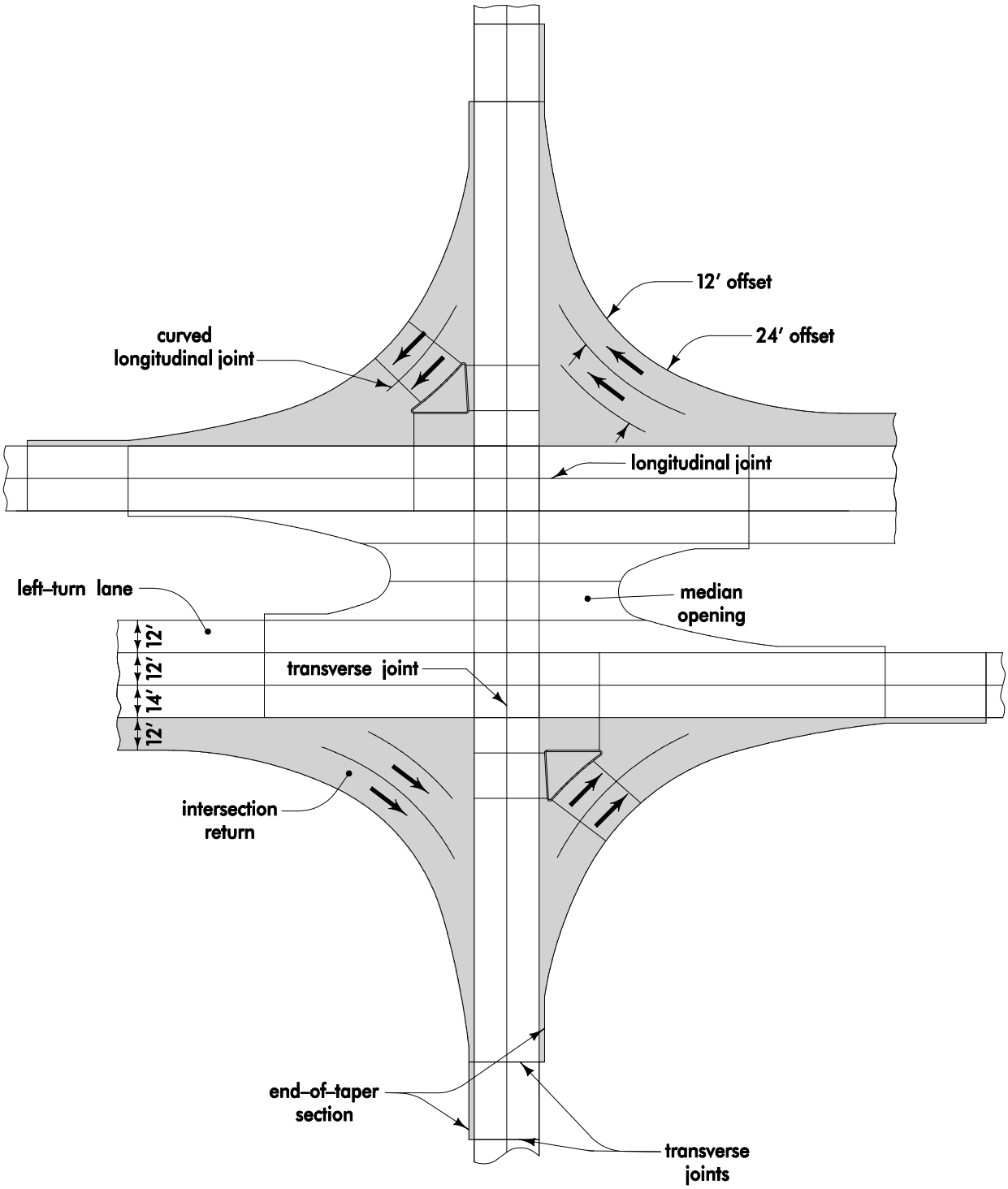
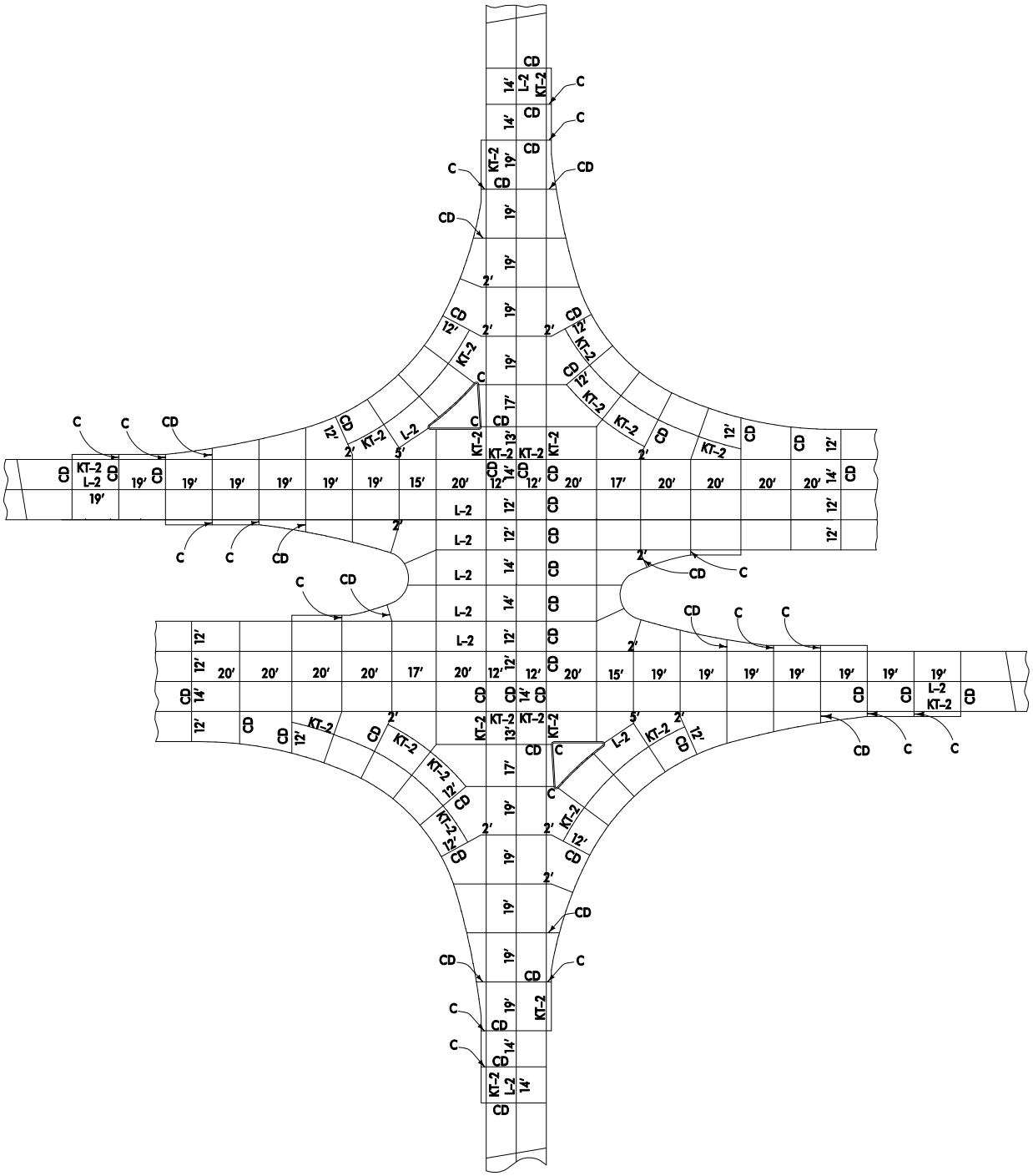


Figure 5G-4.05: Final Jointing Layout



Note:
1) All longitudinal joints will be either KT–2 or L–2 unless indicated otherwise.

Jointing Concrete Overlays

A. General Information

Bonded and unbonded concrete overlays can be placed on existing concrete pavements, asphalt pavements, and composite pavements. Although some normal joint design criteria apply to all concrete overlays, the various overlay options require special design considerations. For the purposes of this section dealing only with jointing guidance, guidelines for concrete overlays over asphalt and composite pavements are combined because of their similarity.

B. Bonded Concrete Overlays

1. Bonded Concrete Overlays of Concrete Pavements:

a. Joint Design: The bonded overlay joint type, location, and width must match those of the existing concrete pavement in order to create a monolithic structure. Matched joints eliminate reflective cracking and ensure that the two layers of the pavement structure move together, helping maintain bonding. To minimize curling and warping stresses, some agencies have successfully created smaller overlay panels by sawing additional transverse and longitudinal joints in the overlay between the matched joints. An important element in transverse joint design is joint dimensions. The depth of transverse joints should be full depth plus 0.50 inch. To prevent debonding, the width of the transverse joints should be equal to or greater than the width of the underlying joint or crack in the existing pavement. If the pavement system experiences expansion and the overlay pushes against itself because the width of the transverse overlay joint is less than the width of the underlying existing pavement crack, debonding may occur. The width of the existing underlying pavement crack may be determined by spot excavating along the pavement edge. Longitudinal joints should be sawed at least one-half the thickness of the overlay. Tie bars, dowel bars, or other embedded steel products are not used in bonded concrete overlays to minimize restraint forces in the bond.

b. Joint Sawing: Timely joint sawing is necessary to prevent random cracking. Sawing should begin as soon as the concrete is strong enough that joints can be cut without significant raveling or chipping. Lightweight early-entry saws allow the sawing crew to get on the pavement as soon as possible. To help match transverse joint locations, place guide nails on each edge of the existing pavement at the joints; after the overlay is placed, mark the joint with a chalk line connecting the guide nails.

2. Bonded Concrete Overlays of Asphalt and Composite Pavements:

a. Joint Design: The recommended joint pattern for bonded overlays of asphalt is small square panels, typically in the range of 3 to 8 feet, to reduce curling and warping stresses. It is recommended the length and width of joint squares, in feet, be limited to 1.5 times the overlay thickness in inches. In addition, if possible, longitudinal joints should be arranged so that they are not in the wheel path. The use of tie bars or dowels is not necessary because of the small panel spacings.

b. Joint Sawing: Timely joint sawing is necessary to prevent random cracking. Joint sawing should commence as soon as the concrete has developed sufficient strength so that joints can be cut without significant raveling or chipping, typically within 3 to 6 hours of concrete placement. Lightweight early-entry saws with 1/8 inch wide blades may be used to allow the sawing as soon as possible. Transverse joints can be sawed with conventional saws to a depth of T/4. Transverse joint sawcut depths for early-entry sawing should not be less than 1.25 inch. Longitudinal joints should be sawed to a depth of T/3. Joint sealing is not required.

C. Unbonded Concrete Overlays

1. Unbonded Concrete Overlays of Concrete Pavements:

a. Joint Design: Load transfer is better in unbonded overlays of concrete pavements than in new JPCPs because of the load transfer provided by the underlying pavement. Doweled joints are used for unbonded overlays of pavements that will experience significant truck traffic, typically pavements 8 inches and thicker. Joints are typically mismatched to maximize load transfer from the underlying pavement. Shorter joint spacing should be used to reduce the risk of early cracking due to enhanced curling caused by the stiff support provided by the underlying pavement (see Table 5G-5.01). Using lane tie bars may be appropriate in open-ditch (or shoulder) sections of unbonded overlays if the overlay is 5 inches or greater. In this category, a #4 tie bar (0.50 inch) may be appropriate. The use of tie bars in confined curb and gutter sections should be considered if the overlay is 6 inches or greater.

Table 5G-5.01: Typical Transverse Joint Spacing

Unbonded Resurfacing Thickness	Maximum Transverse Joint Spacing
< 5 inches	6 x 6 foot panels
5 to 7 inches	Spacing in feet = 2 times thickness in inches
> 7 inches	15 feet

Source: Harrington, 2008.

b. Joint Sawing: Timely joint sawing is necessary to prevent random cracking. Transverse joints can be sawed with conventional saws to a depth of between T/4 (minimum) and T/3 (maximum), but not less than 1.25 inch. Transverse joint sawcut depths for early entry sawing should not be less than 1.25 inch. Saw longitudinal joints to a depth of T/3.

2. Unbonded Concrete Overlays of Asphalt and Composite Pavements:

a. Joint Design: The load transfer design is the same as for new concrete pavements. Doweled joints are used for unbonded overlays of pavements that will experience significant truck traffic, typically pavements 8 inches and thicker. For pavements less than 6 inches thick, the maximum spacing in feet is 1.5 times the slab thickness in inches. For pavements 6 inches thick or greater, a maximum joint spacing in feet of two times the slab thickness in inches is often recommended for unbonded overlays. A 6 inch overlay would thus receive a maximum 12 foot joint spacing. The maximum recommended spacing is typically 15 feet. The use of tie bars for unbonded overlays should follow conventional use for pavements 5 inches thick or more. Using lane tie bars may be appropriate in open-ditch (or shoulder) sections of unbonded overlays if the overlay is 5 inches or greater. In this category, a # 4 tie bar (0.50

inch) may be appropriate. The use of tie bars in confined curb and gutter sections should be considered if the overlay is 6 inches or greater.

b. Joint Sawing: Timely joint sawing is necessary to prevent random cracking. Transverse joints can be sawed with conventional saws to a depth of between T/4 (minimum) and T/3 (maximum). When there is evidence of some wheel rutting on the existing asphalt pavement, sawcut depth is of particular concern for unbonded overlays because the distortions in the underlying asphalt pavement can effectively increase the slab thickness. Transverse joint sawcut depths for early-entry sawing should not be less than 1.25 inch. Longitudinal joints should be sawed to a depth of T/3. Always match overlay joints to the joints in any concrete patches in the existing pavement and cut the joints full depth.

Table 5G-5.02: General Jointing Practices for PCC Overlays

Construction Consideration of Joints	Bonded Overlays of Concrete	Bonded Overlays of Asphalt or Composite	Unbonded Overlays of Concrete	Unbonded Overlays of Asphalt or Composite
Typical Thickness:	***3 to 4 inch***	***3 to 4 inch***	***5 to 12 inch***	***5 to 12 inch***
Joint spacing for concrete overlays requires special consideration for each type:				
• Joints are to be matched with underlying concrete to prevent cracking.	X			
• Joints are typically mismatched to maximize load transfer from the underlying pavement.			X	
• Recommended length and width of panels in feet should be limited to 1.5 times the overlay thickness in inches.	X	X		
• Because of the potential for higher curling and warping stress from a rigid underlying pavement, shorter than normal spacing is typical.			X	X
Joint sawing:				
• The timing of sawing is critical. Sawing joints too early can cause excess raveling.	X	X	X	X
• Sawing must be completed before curl stresses exceed the bond strength developed.	X	X		
• Sawing too late can cause excess stresses, leading to uncontrolled random cracking.	X	X	X	X
• Transverse joint saw-cut depth for conventional saws.	Full Depth + 0.50 inch	T/4	T/4 min. - T/3 max.	T/4 min. - T/3 max.
• Transverse joint saw-cut for early-entry saws.	Full Depth + 0.50 inch	Not < 1.25 inch	Not < 1.25 inch	Not < 1.25 inch
• Longitudinal joint saw-cut depth.	T/2 (at least)	T/4 – T/3	T/4 – T/3	T/4 – T/3
• Transverse joint width must be equal to or greater than the underlying crack width at the bottom of the existing transverse joint.	X			
• Joint type, location, and width must match those of the existing pavement to create a monolithic structure.	X			
• Recommended joint pattern is square panels		X		
Sealing:				
• Seal joint using low-modulus hot-pour sealant with narrow joint.	X	**	*	*
Other considerations:				
• Doweled joints are used for pavements that experience heavy truck traffic, 8 inch pavements and thicker.			X	X
• Lane tie bars may be appropriate in open ditch (or shoulder) sections of unbonded overlays if the overlay is 5 inches or greater.			X	X
• Tie bar use in curb and gutter sections should be considered if the overlay is 6 inches or greater.			X	X

* Some states have experienced problems with asphalt stripping of the separation layer, particularly under heavy truck traffic and high speeds. Therefore, sealing is important in these conditions. On lower speed roadways without a heavy traffic loading, some states successfully do not seal.

** Joint between overlay and non-integral curb and gutter.

Source: Harrington, 2008.

D. References

Harrington, D. *Guide to Concrete Overlays: Sustainable Solutions for Resurfacing and Rehabilitating Existing Pavements*. Second Ed. National Concrete Pavement Technology Center, Iowa State University. ACPA publication no. TB021.02P. 2008.

Railroad Crossings

A. Railroad Crossing Improvements

Improvements to railroad crossings can take several forms. These include closing of an existing crossing, improvements to the existing crossing, and separating the roadway from the railroad tracks. Potential improvements to existing crossings include installation of adequate signage, signals, and signals with gate arms.

The local jurisdiction must use judgment in the selection process for crossing improvements. Several factors weigh into the selection process including the amount and speed of traffic on the roadway and railroad, available sight distance, and safety benefits. Traffic control systems for railroad-highway grade crossings must comply with the Manual on Uniform Traffic Control Devices (MUTCD).

The Jurisdiction should contact the offices of Rail Transportation and Local Systems at the Iowa DOT for any agreements and requirements that must be followed.

B. Railroad Crossing Construction

When railroad crossings are required on streets subject to heavy loads, an approved high quality grade crossing material should be installed. Some railroads may require an asphalt separation between the header and the crossing to allow for easier railroad maintenance of the crossing. Some railroads may require that the crossing material be installed by their own forces, with the costs borne by or shared with the local jurisdiction. Example railroad crossing approaches are included in Figures 1 and 2. In all cases, the railroad should be contacted for their specific crossing requirements.

C. Working with a Railroad

Working with a railroad company requires coordination at numerous steps along the planning, design, and construction process. A list of potential coordination steps follows; however, these requirements differ for each company and should be verified early in the planning process.

Phase	Possible Coordination Required
Planning	Right of entry permit for survey Coordination regarding potential modifications/improvements
Design	Right of Entry Permit for Survey Utility Accommodation Permit Maintenance Consent Agreement Coordination regarding crossing material and safety elements
Construction	Railroad Protective Liability Insurance Right of Entry for Construction Railroad Flaggers

D. Railroad Related Agencies in Iowa

Two governmental agencies are involved in regulating railroad activities within the State of Iowa. Additional information about these organizations is available at their respective websites:

Iowa DOT, Rail Transportation	http://www.iowadot.gov/iowarail/index.htm 800 Lincoln Way Ames, Iowa 50010 515-239-1140
Federal Railroad Administration Region 6 Office	http://www.fra.dot.gov/ 901 Locust Street – Suite 464 Kansas City, MO 64106 816-328-3840

E. Railroad Companies in Iowa

Currently there are nineteen railroads operating within the State of Iowa. These include three Class I railroad companies, Amtrak, and several regional and local railroads. The Iowa DOT maintains a website with links to the websites of the freight railroads operating within the State (http://www.iowadot.gov/iowarail/railroads/industry/profiles.aspx#).

Figure 5H-1.01: PCC Railroad Crossing Approach
(SUDAS Specifications Figure 7010.903)

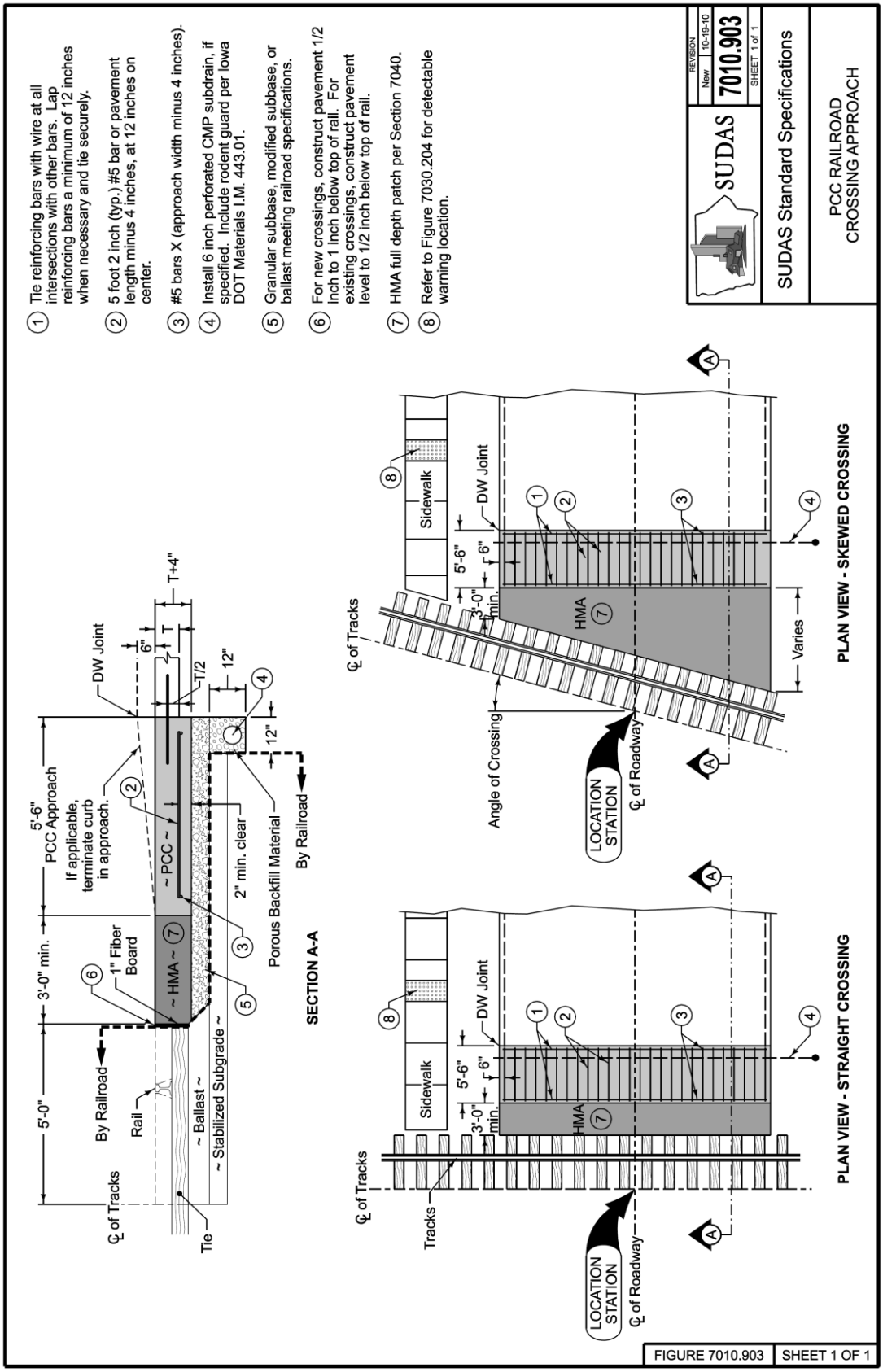
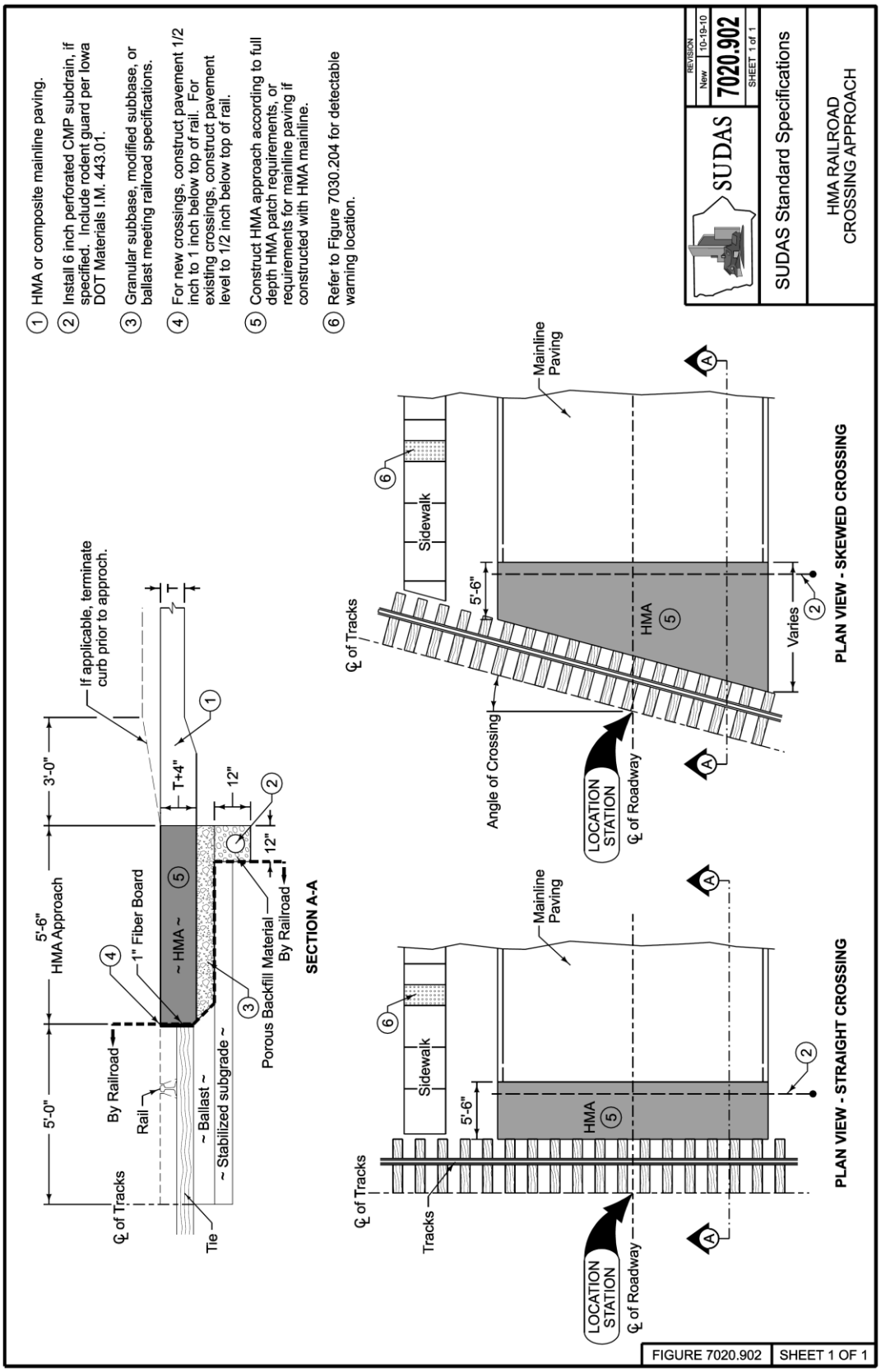


Figure 5H-1.02: HMA Railroad Crossing Approach
(SUDAS Specifications Figure 7020.902)



General Access Management

A. General Information

The efficiency and safety of a street or highway depends largely upon the amount and character of interruptions to the movement of traffic. The primary cause of these interruptions is vehicular movements to and from businesses, residences, and other developments along the street or highway. Regulation and overall control of access is necessary to provide efficient and safe highway operation and to utilize the full potential of the highway investment.

The Jurisdictions reserve the right to make exceptions to the criteria where the exercise of sound and reasonable engineering judgment indicates that the literal enforcement of the criteria would cause an undue hardship to any interested party.

B. Access Permit Procedure

An access permit may be required for any public or private access constructed to a public street. The Jurisdictional Engineer will stipulate the information required and the permit form to use. Access to streets or highways under the jurisdiction of the Iowa DOT will be governed by requirements of the Iowa DOT with Jurisdictional review (See Section 5J-1).

In addition to specific details, the following general criteria will be used by the Jurisdiction when reviewing an access request:

1. Safety to the traveling public
2. Preservation of the traffic-carrying capacity of the highway
3. The impact upon the economy of the area
4. Protection of the rights of the traveling public and of property owners, including the rights of abutting property owners

C. Definitions

Access management definitions can be found in the following resources:

1. Iowa Department of Transportation - "Iowa Primary Road Access Management Policy."
2. Transportation Research Board - "Access Management Manual."

D. Entrance Type

1. **Major:** An entrance developed to carry sporadic or continuous heavy concentrations of traffic. Generally, a major entrance carries in excess of 150 vehicles per hour. An entrance of this type would normally consist of multiple approach lanes and may incorporate a median. Possible examples include racetracks, large industrial plants, shopping centers, subdivisions, or amusement parks.

2. **Commercial/Industrial:** An entrance developed to serve moderate traffic volumes. Generally, a commercial/industrial entrance carries at least 20 vehicles per hour but less than 150 vehicles per hour. An entrance of this type would normally consist of one inbound and one outbound traffic lane. Possible examples include service stations, small businesses, drive-in banks, or light industrial plants.

3. **Residential:** An entrance developed to serve light traffic volumes. Generally, a residential entrance carries less than 20 vehicles per hour. An entrance of this type would not normally accommodate simultaneous inbound and outbound vehicles. Possible examples include single-family residence, farm, or field entrances.

E. Access Management Principles

A variety of access management, location, and design practices and policies can be used to improve the safety and operations of the roadway within a state's, city's, or county's jurisdiction.

Following are the 10 Principles of Access Management identified by the TRB:

1. **Provide a Specialized Roadway System:** Different types of roadways serve different functions. It is important to design and manage roadways according to the primary functions that they are expected to serve.

2. **Limit Direct Access to Major Roadways:** Roadways the serve higher volumes of regional through traffic need more access control to preserve their traffic function. Frequent and direct property access is more compatible with the function of local and collector roadways.

3. **Promote Intersection Hierarchy:** An efficient transportation network provides appropriate transitions from one classification of roadway to another.

4. **Locate Signals to Favor through Movements:** Long uniform spacing of intersections and signals on major roadways enhances the ability to coordinate signals and ensure continuous movement of traffic at the desired speed.

5. **Preserve the Functional Area of Intersections and Interchanges:** The functional area of an intersection or interchange is the area that is critical to its safe and efficient operation. This is the area where motorists are responding to the intersection or interchange, decelerating, and maneuvering into the appropriate lane to stop or complete a turn.

6. **Limit the Number of Conflict Points:** Drivers make more mistakes and are more likely to have collisions when they are presented with the complex driving situations created by numerous conflict points.

7. **Separate Conflict Areas:** Drivers need sufficient time to address one potential set of conflicts before facing another. The necessary spacing between conflict areas increases as travel speed increases, to provide drivers adequate perception and reaction time.

8. **Remove Turning Vehicles from Through-traffic Lanes:** Turning lanes allow drivers to decelerate gradually out of the through lane and wait in a protected area for an opportunity to complete a turn. This reduces the severity and duration of conflict between turning vehicles and through traffic, and improves the safety and efficiency of roadway intersections.

9. **Use Nontraversable Medians to Manage Left Turn Movements:** Medians channel turning movements on major roadways to controlled locations. Nontraversable medians and other techniques that minimize left turns or reduce driver workload can be especially effective in improving roadway safety.

10. **Provide a Supporting Street and Circulation System:** Provide a supporting network of local and collector streets to accommodate development, as well as unified property access and circulation systems.

F. References

American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets*. 2004.

Institute of Transportation Engineers. *Transportation and Land Development*. 2002.

Iowa Department of Transportation. *Iowa Primary Road Access Management Policy.* 2012.

Transportation Research Board. *Access Management Manual*. 2003.

Transportation Research Board. *NCHRP Report 420: Impacts of Access Management Techniques.* 1999.

Transportation Research Board. *NCHRP Report 457: Evaluating Intersection Improvements: An Engineering Study Guide*. 2001.

Transportation Research Board. *NCHRP Report 659: Guide for the Geometric Design of Driveways.* 2010.

Transportation System Considerations

This section addresses transportation system considerations in access management, including TRB Principles of Access Management 1 through 4 and 10:

A. Provide a Specialized Roadway System (Principle 1)

The primary function of major arterial roadways is to safely and efficiently accommodate through traffic. The primary function of local streets is to provide access to adjacent properties. Minor arterials and collectors provide a blend of the mobility and access functions. Design and management of transportation facilities, including access management, must consider the classification and intended function of roadways.

B. Limit Direct Access to Major Roadways (Principle 2)

Providing direct property access to major roadways can significantly affect corridor operations and safety, and is not consistent with the function of the major roadway. Higher levels of access control become more necessary as major road through traffic volumes and speeds increase.

C. Promote Intersection Hierarchy (Principle 3)

Provide appropriate transitions from one roadway classification to the next.

- Freeways intersect arterials with interchanges.
- Arterials intersect collectors.
- Collectors intersect local streets.
- Local streets provide connections to private accesses.

D. Locate Signals to Favor through Movements (Principle 4)

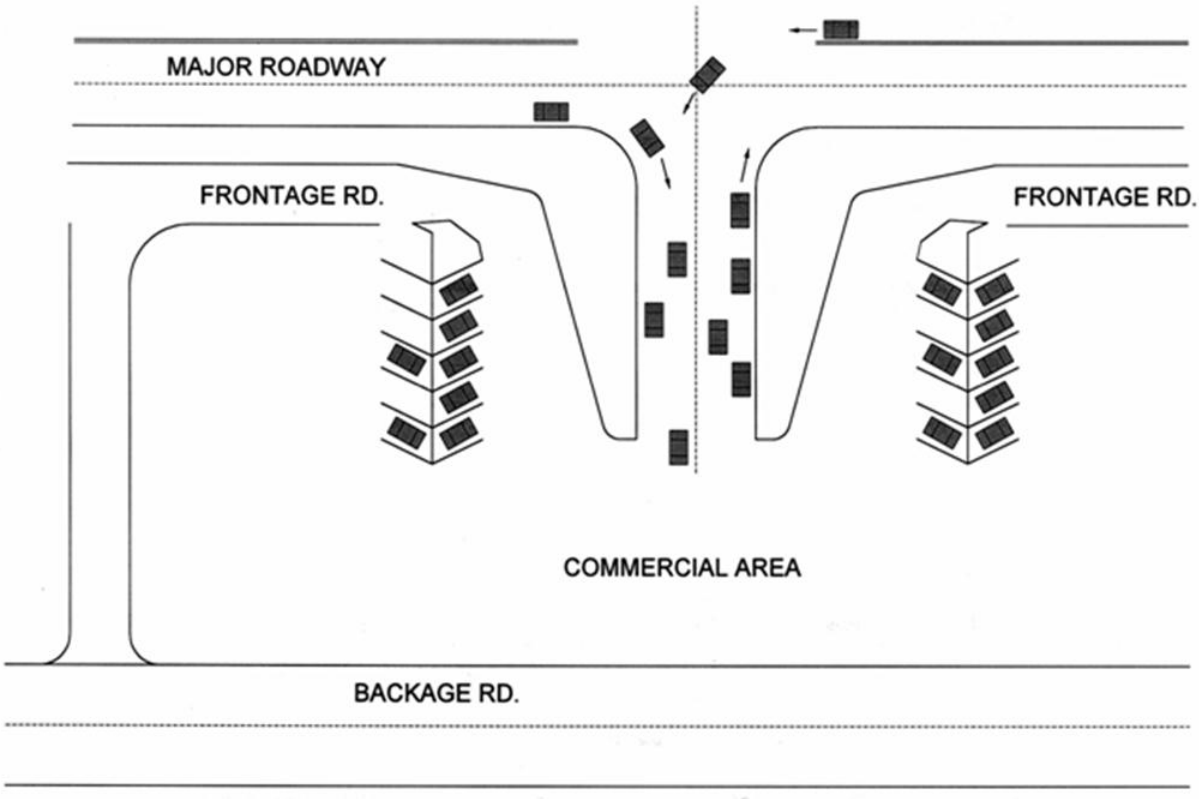
All major arterials, minor arterials, and major collectors within urbanized areas, the urban fringe or areas that may ultimately be subject to urban growth should have long, uniform traffic signal spacing.

- Provides the flexibility to use timing plans that can provide efficient traffic progression over a wide range of speeds and cycle lengths.
- Use a minimum of 1/2 mile spacings on major suburban/urban arterials.
- Use a minimum of 1/4 mile spacings on minor arterials and major collectors where traffic progression is less important than on major arterials.
- Locate cross-roads and full median openings only at locations that conform to the selected spacing interval so that the intersection may be signalized when conditions warrant.
- Where signal location does not conform to recommended spacing, reduce the cross-street green and increase the major street green so as to maintain progression on the major street.

E. Provide a Supporting Street and Circulation System (Principle 10)

- Provide local and collector streets to accommodate access to development.
- Provide access connections between adjacent parcels.
- Require adequate internal circulation for development.
- Provide alternate access from minor roads.
- Provide frontage and backage roads (see Figure 5I-2.01).

Figure 5I-2.01: Frontage and Backage Roads with Adequate Vehicle Queue Storage



Access Location, Spacing, Turn Lanes, and Medians

This section addresses access location, spacing, turn lane and median needs, including TRB Principles of Access Management 5-9:

A. Preserve the Functional Area of Intersections and Interchanges (Principle 5)

AASHTO states, "Ideally, driveways should not be located within the functional area of an intersection or in the influence area of an adjacent driveway. The functional area extends both upstream and downstream from the physical intersection area and includes the longitudinal limits of auxiliary lanes."

1. **Upstream Functional Distance:** The upstream functional distance of the intersection can be further defined as the approach distance to an intersection that is required for the driver to change speeds in order to complete a movement, such as entering an auxiliary lane or slowing down for a turn or signal. The upstream functional distance includes the sum of:

   d_1, distance traveled during driver's perception - reaction time

   d_2, deceleration distance while the driver maneuvers to a stop

   d_3, queue storage length required (50 foot minimum)

Table 5I-3.01: Distance Traveled During Driver's Perception-reaction, (d_1)

Speed (mph)	Rural (feet)	Urban/ Suburban (feet)
20	75	45
30	110	65
40	145	90
50	185	110
60	220	135
70	255	155

Source: TRB Access Management Manual

Table 5I-3.02: Desirable Maneuver Distances, (d_2)

Speed (mph)	Distance (feet)
20	70
30	160
40	275
50	425
60	605
70	820

Source: TRB Access Management Manual

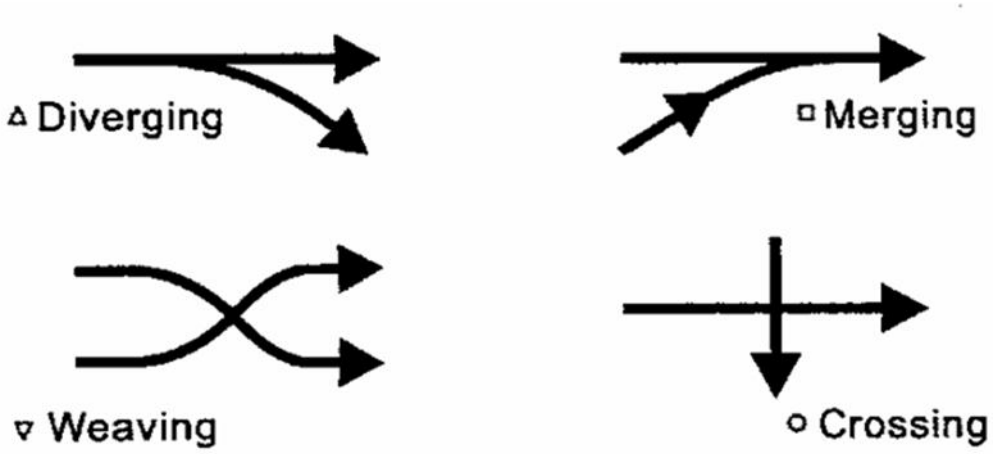
For example, at an urban intersection approach with a 30 mph speed and minimal queuing, the upstream functional distance would be 275 feet (65 feet + 160 feet + 50 feet).

2. **Downstream Functional Distance:** The downstream functional distance from an intersection should be based on upstream functional distance for the proposed adjacent access point. Minimum separation should be no less than the AASHTO stopping sight distance.

B. Limit the Number of Conflict Points (Principle 6)

Traffic conflicts occur where the paths of traffic movements cross. Eliminating or reducing conflict points will simplify the driving task, contributing to improved traffic operations and fewer collisions.

Figure 5I-3.01: Types of Vehicular Conflicts



C. Separate Conflict Areas (Principle 7)

Separating conflict areas allows drivers to address one potential set of conflicts at a time. The higher the speed, the longer the distance a vehicle will travel during a given perception-reaction time. Also, drivers need more time to react to complex conflict areas. Hence minimum separation distances are a function of both the speed of traffic on a given section of roadway and the complexity of the decision with which the driver may be presented. The complexity of the problem, in turn, increases with both the number and type of conflicts and the volume of traffic.

Various methods that can be utilized to separate conflict areas include the following:

- Minimum access spacing
- Minimum corner clearance
- Minimum property line clearance
- Limit the number of accesses per property
- Designate the access for each property

Figure 5I-3.02: Two Lane Undivided Roadway (Single Entrance)

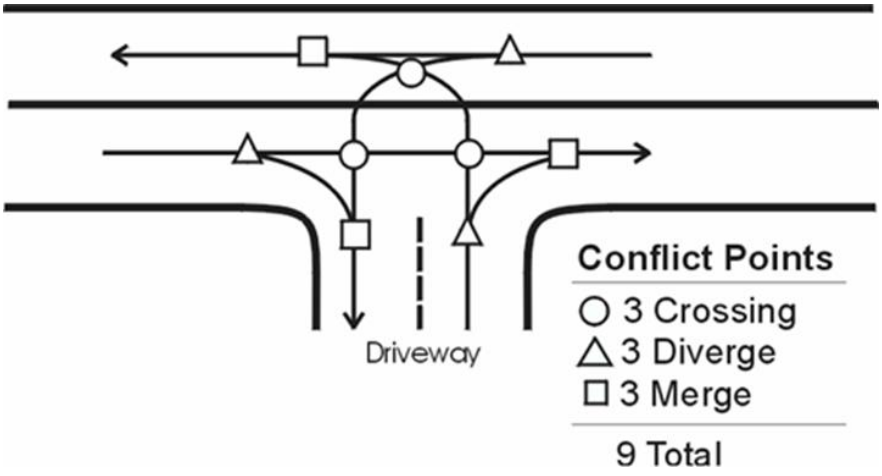
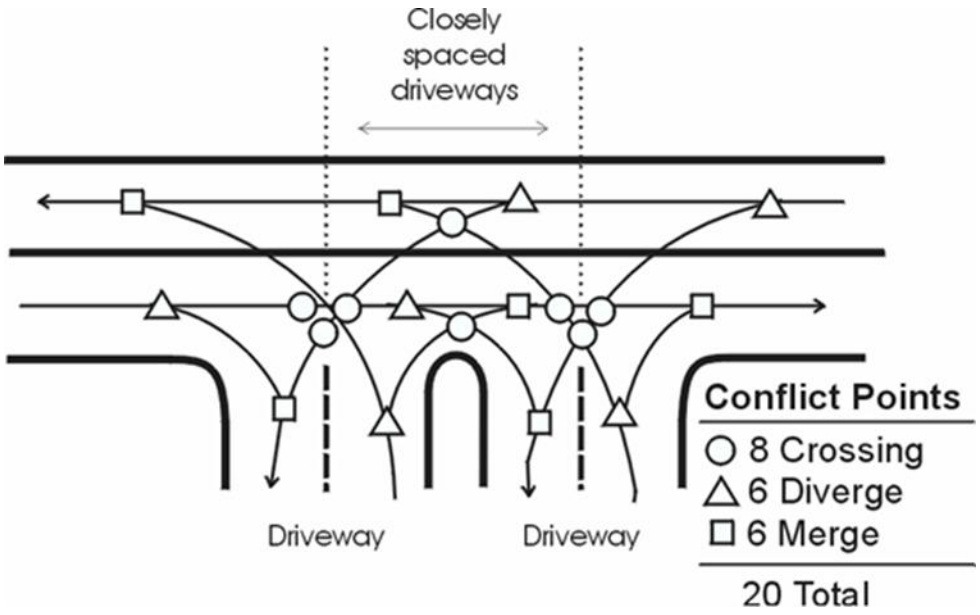


Figure 5I-3.03: Two Lane Undivided Roadway (Closely Spaced Entrances)



1. **Driveway Density:** The number of driveways per block or per mile significantly affects the safety of the corridor. Crash rates increase very quickly as the number of access points increases on arterial and collector roadways.

Table 5I-3.03: Crash Rates (crashes per million vehicle miles traveled) vs. Access Point Density

Access Points per Mile	Approximate Accesses per 500 feet	Representative Crash Rate for an Undivided Roadway	Increase in Crashes Associated with More Access Density
Under 20	Under 2	3.8	----
20 to 40	2 to 4	7.3	+92%
40 to 60	4 to 6	9.4	+147%
Over 60	Over 6	10.6	+179%

Source: National Cooperative Highway Research Program Report 420.

2. **Access Spacing for Major Arterials:** Provide separation between access connections so that drivers can assess potential conflict locations one-at-a-time. Applicable spacing criteria may include:
   - Functional area (Section 5I-2)
   - AASHTO stopping sight distance
   - Preventing right turn overlap (see below)
   - Other criteria as established by the Jurisdiction

   Right turn overlap occurs when a through vehicle must monitor two egress right turning vehicles at once while still performing other driving tasks. By separating access points a proper distance, the overlap does not occur, and the through driver has only one egress right turning vehicle to monitor. Recommended minimum access spacings to avoid right turn overlap shown in Table 5I-3.04 are comparable to AASHTO stopping sight distances.

Table 5I-3.04: Minimum Access Spacing to Prevent Right Turn Overlap

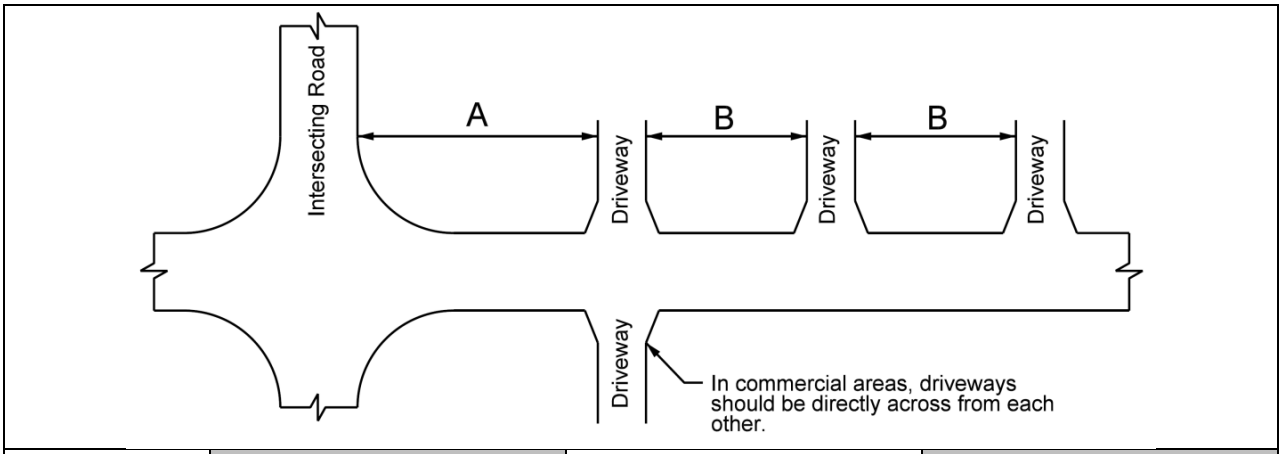
Speed (mph)	Recommended Minimum (feet)[1]
25	120
30	185
35	245
40	300
45	350

[1] Intersection clearance should be the same as driveway spacings or at least as long as stopping sight distance.

Source: Transportation Research Board Record 644, 1977.

3. **Access Spacing for Minor Arterials, Collectors, and Local Streets in Urban/Suburban Areas:** For minor arterials and major collectors, direct access from individual properties should be avoided wherever possible. Property access should be provided from minor collectors, local streets, frontage roads and backage roads. Major arterial access spacing criteria should be used for minor arterials and major collectors when possible.

Table 5I-3.05: Minimum Distance between Driveways or from Intersecting Streets



	Minor Arterial			Collector			Local		
	Res. Area	***C/ I Area***	***Ag Area***	***Res. Area[3]***	***C/I Area***	***Ag Area***	***Res. Area[3]***	***C/I Area***	***Ag Area***
A. Minimum intersection clearance[1]	145’	170’	300’	100’	100’	300’	75’	75’	150’
B. Minimum driveway spacing[2]	100’	200’	300’	75’	100’	300’	---[4]	---[4]	150’

Res = Residential, C/I = Commercial/Industrial
[1] Values are measured from the back of the curb, intersecting road to the adjacent driveway near edge.
[2] Values are measured between driveway edges.
[3] One access drive allowed per lot. Depending on lot size, an additional drive may be allowed upon approval of the Jurisdiction.
[4] See Jurisdictional Engineer for local requirements.

4. **Access Spacing for State Primary Roads:** In rural areas, travel speeds are usually 55 mile per hour and above. This means that driveway spacing in rural areas must be longer to provide for a safe driving environment. On state highways, spacing is also longer because the routes are primarily designed to carry through traffic rather than to serve as property access routes. The more important a route is for through traffic and commerce, the longer the spacing between driveways. The following table shows the State of Iowa's standards for its highway system.

Table 5I-3.06: Iowa DOT Access Control - Minimum Spacings

State Highway Priority	Minimum Spacing Between Driveways	Number of Driveways Per Mile
Priority I (Full Access Control)	Interchanges at roads	N/A
Priority II (Four Lane Divided)	2,640' (minimum)[1] 5,280' (preferred)[1]	2 2
Priority III	1,000' rural (minimum)[1] 1,320' rural (preferred)[1]	4 4
Priority IV(a) Priority IV(b)	600' rural (≥ 45 mph) 300' urban (≤ 40 mph)	8 16
Priority V (Access Right Acquired Between 1956 to 1966)	1 access per 1,000' of frontage not exceeding 2,000'	2 to 5
Priority VI	Safety and need	Varies

[1] Access allowed only at interchanges and selected at-grade locations

5. **Access Spacing for County Roads:** On county roads, the spacing standard should also depend on the nature of the road, e.g. how important the road is for through traffic. Even on the lowest functional levels, some sort of driveway spacing standard is important for traffic safety.

Table 5I-3.07: County Road Minimum Access Spacings

County Road Route Type	Minimum Spacing Between Driveways	Number of Driveways Per Mile
Minor arterials	600'	9
Collectors	300'	18
Local traffic service	150'	36

6. **Additional Access Spacing Considerations:**
   - At a minimum, the upstream corner clearance should be longer than the longest expected queue at the adjacent intersection.
   - High speed, high volume roadways need longer corner clearances whereas the corner clearance on a local street can be much shorter.
   - Residential streets - driveways on corner lots should be located on the lesser street and near the property line most distant from the intersection.
   - Typically, all elements of an access drive, including the radii should be within a property frontage.
   - At a minimum, all driveway geometrics should be along the frontage of the property served by the driveway.
   - On major roadways, the corner clearance should be at least as long as the stopping sight distance so that vehicles turning corners can make safe stops when encountering entering traffic.
   - Encourage owners of adjacent properties to construct joint-use driveways in lieu of separate driveways.
   - Encourage a property owner to replace two or more driveways with a single driveway (or fewer driveways).
   - For adjacent properties, locate joint access on the property line. Reciprocal easements must be executed.

D. Remove Turning Traffic from Through-traffic Lanes (Principle 8)

All driveway and intersection geometrics require that turns be made at very slow speeds and hence result in high speed differentials. Providing auxiliary lanes (left-turn and right-turn bays) is the most effective means of limiting the speed differential. This is especially important on high volume and high speed roadways.

The several methods by which turning vehicles can be removed from through traffic lanes are:

- Install isolated left-turn bay
- Install a nontraversable median with left-turn bays
- Install right-turn deceleration bay
- Install right-turn lane
- Install a continuous two-way left-turn lane (TWLTL)

1. **Turn Lane Warrants for Urban/Suburban Areas (Unsignalized):** Providing left and/or right turn lanes can significantly improve the operation and safety of an intersection. They allow turning vehicles to exit the through traffic lane with reduced speed differential and provide queue storage without interference with through traffic. Rear-end and side-swipe collisions are greatly reduced. Capacity is increased and delay decreased.

   Although much research has been done on the topic of turn lane warrants, there are no universally accepted guidelines regarding traffic conditions that warrant left and right turn lanes. However, NCHRP Report 457 provides turn lane warrant criteria that should be considered. This report also includes links to spreadsheet tools that can be used to evaluate the need for turn lanes.

   In general, the decision to provide turn lanes should be based on safety rather than just capacity. Where practical, left turn lanes should be provided at median openings on divided roads, regardless of projected traffic volumes.

2. **Rural Turn Lane Warrants and Right Turn Deceleration Length (Unsignalized):** See Iowa Department of Transportation's Design Manual, Chapter 6 - Geometric Design.

3. **Three Lanes with TWLTL:** Three lane roadway designs can be effectively used in situations where there are low to moderate levels of through traffic, yet there are concerns about conflict points and crashes caused by left-turning traffic. The upper limit for using a three lane design is about 17,000 vehicles per day of traffic. Three lane designs are ideal where right-of-way width is limited due to existing land development or other constraints. Three lane roads can either be designed that way originally or can be created by widening an existing two lane route or by modifying an existing four lane undivided route.

4. **Five lanes with TWLTL:** When the average daily traffic (ADT) on a street exceeds about 17,000 vehicles per day, four lane roadways with raised medians or five lane roadways with TWLTL are more appropriate designs. The limit for five lane roadway (with TWLTL) is approximately 24,000 ADT. TWLTL should generally not be used in situations where there are more than four total through lanes.

E. Use Nontraversable Medians to Manage Left Turn Movements (Principle 9)

The majority of access-related crashes involve left turns. Providing nontraversable medians limits and defines locations of left turns, thereby improving safety. Full access median openings that allow left turns from all directions are best provided at signalized intersections and unsignalized junctions of arterial and collector streets. Providing median closures or partial access medians at other intersections and access points reduces the number and types of conflicts.

1. **Median Closures:** Median openings should be considered for closure where:
   - A safety or operational problem is evident and an appropriate retrofit cannot be made.
   - Median width is less than 11 feet, thereby not allowing for construction of left turn lanes.
   - The left-turn bay of a nearby signalized intersection needs to be extended.
   - A pattern of left-turn crashes is evident.
   - Heavy pedestrian use is predicted or crashes involving pedestrians have occurred at the intersection.

   Implementation of a median closure involves providing a section of median of the same design as existing on either side of the opening. The following should be considered during design:
   - Tree lines, building lines, and lighting may head drivers into believing the median can be crossed.
   - Visual cues should be provided to clearly inform drivers that the opening has been closed.
   - The need for visual cues is especially critical during nighttime hours where a four way intersection previously existed or there are access drives directly opposite each other.
   - Minimum 4 feet median width face-to-face of curbs is recommended.
   - Select and locate landscaping materials to delineate the median while considering potential sight distance obstructions.

Figure 5I-3.04: Two Lane Roadway Conflict Points at Typical Three Way Intersection or Driveway

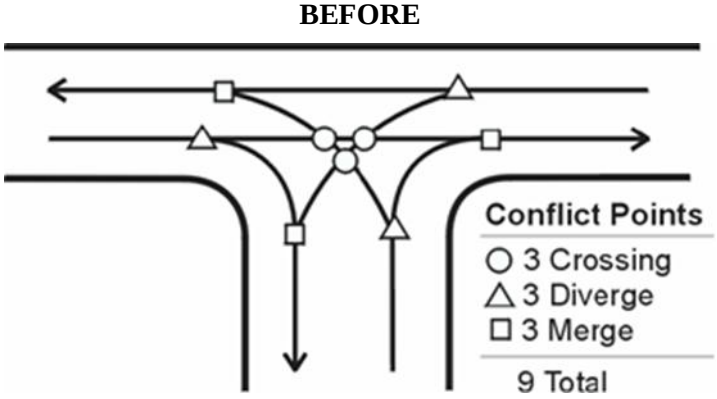


Figure 5I-3.05: Two Lane Roadway with Raised Median Closure (right-in/right-out only access)

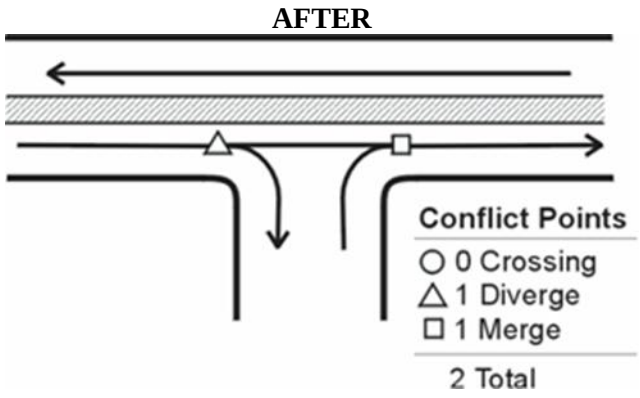
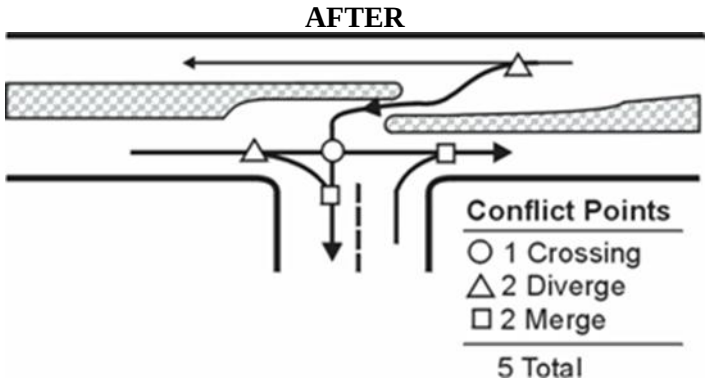


Figure 5I-3.06: Two Lane Roadway with Raised Median (left turn ingress allowed into driveway)



2. Raised Medians vs. Two Way Left Turn Lanes:

- Because they are the most restrictive access management treatment, constructing raised center medians along arterials is often very controversial among business and property owners. Two way left turn lanes (TWLTL) are usually much less controversial. Business persons and property owners feel that installation of raised medians will have a large, negative impact on their customers, sales, and property values. Therefore, TWLTL are often suggested as compromise solutions.
- Arterial roadways with raised medians are statistically safer and operate better than any other configuration. Research indicates that raised median roadways are significantly safer than undivided roadways in urban areas. When traffic volume on an arterial roadway is projected to exceed about 24,000 average annual daily traffic (AADT) during the next 20 years, including a raised median is prudent.
- In general, TWLTL projects function well when traffic levels are moderate, when the percentage of vehicles turning as opposed to traveling through is high, and when the density of commercial driveways is low. TWLTL will function very well on most arterials where AADT is in the range of 10,000 to 24,000 AADT (five lane TWLTL).
- TWLTL projects can also work very well in places where the number of driveways per block or mile is high, but the land use is such that not many turning movements are generated per hour. An example would be an arterial street passing through a predominately residential area.

- TWLTL are much less effective in situations where commercial driveway densities are high and these driveways are spaced close together. In such a situation, the number of conflict points is high, and this will be reflected in crash experience. Research from many states indicates that raised median roadways are always safer than TWLTL roadways. If TWLTL are considered, driveway density and driveway spacing must be managed very aggressively.

Table 5I-3.08: Crash Rates (crashes per million vehicle miles traveled) vs. Median Type

Access Points Per Mile	Undivided (Painted Centerline) Crash Rate	TWLTL Crash Rate	Raised Median Crash Rate	Rate Reduction Raised Median Versus TWLTL
Less than 20	3.8	3.4	2.9	-0.5 (15%)
20 to 40	7.3	5.9	5.1	-0.8 (14%)
40 to 60	9.4	7.4	6.5	-0.9 (12%)
Over 60	10.6	9.2	8.2	-1.0 (11%)

Source: National Cooperative Highway Research Program Report 420

F. References

Transportation Research Board - National Cooperative Highway Research Program (NCHRP). *NCHRP Report 420: Impacts of Access Management Techniques*. National Academy Press. Washington, DC. 1999.

Transportation Research Board - National Cooperative Highway Research Program (NCHRP). *NCHRP Report 644: Guidelines for Conducting a Disparity and Availability Study for Federal DBE Program*. National Academy Press. Washington, DC. 1977.

Driveway Design Criteria

A. General

For efficient and safe operations, access drives and minor public street intersections can be improved by the following:

- Smooth vertical geometrics
- Adequate driveway throat width and curb return radii
- Provide adequate sight distance
- Additional egress lane
- Quality driveway construction
- Define the ingress and egress sides of the access drive

Refer to NCHRP Report 659 - Guide for the Geometric Design of Driveways for supplemental information.

B. Width Measurement

1. The width of an entrance with a radius return or with a flared taper that connects to a curb and gutter roadway is measured at a point 10 feet back from the roadway curb. The curb opening may exceed the maximum allowable width of the entrance to accommodate the allowable radius or taper.

2. The width of an entrance that connects to a rural roadway (no curb and gutter) is measured across the top of the entrance at the culvert line or at the location where a culvert would normally be placed.

C. Dimensions

Figure 5I-4.01: Entrance Dimensions

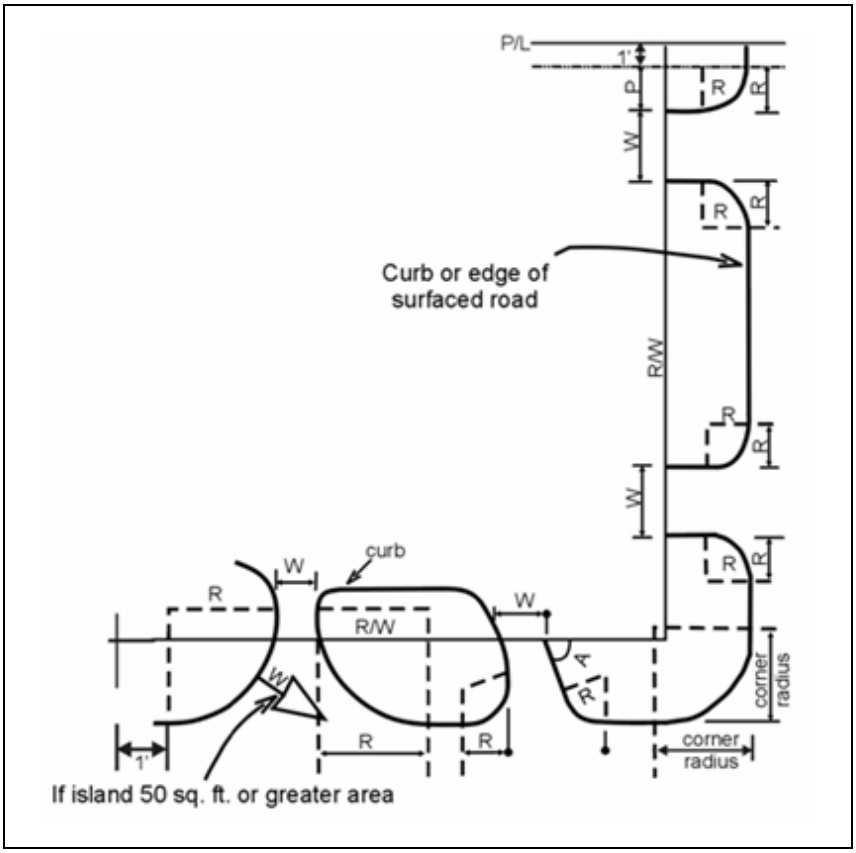


Table 5I-4.01: Driveway Dimensions[1]
(all dimensions are in feet)

Dimension Reference (See Figure 5I-4.01)		Major Arterial Street				Minor Arterial Street				Collector (Major and Minor)				Local Street			
		Residential	Commercial	Industrial	Agricultural	Residential	Commercial	Industrial	Agricultural	Residential	Commercial	Industrial	Agricultural	Residential	Commercial	Industrial	Agricultural
Width																	
Minimum	W	15	24	24	20	15	24	24	20	10	24	24	20	10	24	24	20
Maximum		30	45	45	30	30	45	45	30	24	40	45	30	24	32	40	30
Right-turn Radius[2]																	
Minimum	R	10	10	25	25	10	10	25	25	10	10	25	25	10	10	10	20
Maximum		25	35	50	35	25	35	50	35	25	35	50	35	15	20	30	35
Min. Acute Angle[3]	A	60°	70°	70°	70°	60°	70°	70°	70°	60°	70°	70°	70°	60°	70°	70°	70°
Pref. Acute Angle		90°	90°	90°	90°	90°	90°	90°	90°	90°	90°	90°	90°	90°	90°	90°	90°
Min. Pavement Thickness (inches)	T	6/8	7/9	*	6	6	7	*	6	6	7	*	6	6	7	*	6

[1] Major entrances require special design.
[2] 3 foot flares (F) may be used for residential and agricultural entrances.
[3] Any variation from 90° will be evaluated on a case by case basis. The minimum acute angle (measured from the edge of the pavement) is 60°.
* Requires special design.

1. The width (W) shown applies to rural routes and city streets including neighborhood business, residential, and industrial streets. For joint entrances centered on property lines, the entrance width may increase 5 feet rounded to the nearest 5 foot interval but should not exceed 45 feet. In rural areas (open ditch roadways) widths for paved entrances should include an additional 4 feet for shoulders (Minimum 2 feet shoulders each side).

2. The radius (R) for agricultural uses will vary according to the following intersecting acute angles:

Table 5I-4.02: Agricultural Acute Angle and Radius

Acute Angle	Acute Radius Decrease (feet)	Obtuse Radius Increase (feet)
85° to 90°	0	0
75° to 85°	5 feet	5 feet
65° to 75°	5 feet	10 feet
60° to 65°	10 feet	15 feet

Where the entrance radius specified is greater than the distance between the back of curb and the front edge of the sidewalk the radius may be reduced to meet the available space but should be no less than 10 feet. An option to the radius under this condition is the use of flared entrances. When a flare is used, it should be 3 feet wide and should be constructed from the back of curb to the sidewalk. If no sidewalk exists, flares should be 10 feet long.

3. For individual properties, the number of entrances should be as follows:

   a. Single Family (SF) Residential: Each SF residential property is limited to one access point. However, where houses are located on corner lots, have extra wide frontage, or on heavy traveled roadway more than one access point may be allowed to eliminate backing out on a heavily traveled roadway.

   b. Multi-family (MF) Residential: Access is determined by information provided by the Owner/Developer in a Traffic Impact Report and by comments generated during the Jurisdiction Engineer's review and acceptance of that report.

   c. Commercial: Commercial property having less than 150 feet of frontage and located mid-block is limited to one access point to the street. An exception to this rule may be where a building is constructed in the middle of a lot and parking is provided for each side of the building. A second access point may be allowed for commercial property having more than 150 feet of frontage. For commercial property located on a corner, one access to each street may be allowed, provided dimensions are adequate from the intersecting street to the proposed entrance. (See Section 5I-3 - Access Location, Spacing, Turn Lanes, and Medians).

   d. Industrial: Access is determined on a case-by-case basis. The Jurisdiction will consider good traffic engineering practice and may require information to be provided by the applicant in a Traffic Impact Report. (See Section 5I-3 - Access Location, Spacing, Turn Lanes, and Medians).

   e. Agricultural: Access with adequate frontage may be authorized with more than two accesses at not less than 300 feet intervals provided a minimum distance of 30 feet is maintained from the inlet and outlet of two adjacent culverts.

   In all cases, the location of the access will be such that the taper or radius does not extend beyond the extension of the property line. In general, all construction must occur only on the

property owner's frontage.

4. Minimum acute angle (A) is measured from the edge of pavement and is generally based on one-way operation. For two-way driveways, and in high pedestrian activity areas, the minimum angle should be 70 degrees. Entrances should be placed at 90 degrees whenever possible.

5. The entrance pavement thickness (T) is based on the following:

   PCC - Class "A" or "C" - 4,000 psi
   HMA - Greater than or equal to 100K ESAL (optional for rural area).

   For those entrances not paved, 6 inches (min.) of Class "A" gravel should be required.

D. Sight Distance

1. Sight distance is based upon AASHTO stopping sight distance criteria. However, the height of an object is increased from 2.0 feet to 3.5 feet to acknowledge an approaching vehicle as the "object" of concern. Therefore, sight distance at an access location is measured from the driver's height of eye (3.5 feet) to the height of approaching vehicle (3.5 feet).

2. An access location should be established where desirable sight distance is available, as shown below.

Table 5I-4.03: Desirable Sight Distances

Design Speed (mph)	**Intersection Sight Distance (feet)**	
	Left Turn from Stop	***Right Turn from Stop and Crossing Maneuver***
55	610	530
50	555	480
45	500	430
40	445	385
35	390	335
30	335	290
25	280	240

Note: the sight distances shown above are for a stopped passenger car to turn onto or cross a two lane roadway with no median and grades of 3% or less. For conditions other than those stated, refer to the 2004 AASHTO "Green Book" for additional information.

Source: Based on Exhibit 9-55 and Exhibit 9-58 of the 2004 AASHTO "Green Book."

3. On a four lane divided primary highway where access is proposed at a location not to be served by a median crossover, sight distance is required only in the direction of the flow of traffic.

E. Driveway Grades

1. **Slopes vs. Speed Differential:** Driveway slope is important due to speed differential. Turning vehicles must slow appreciably to enter a driveway. The steeper the driveway, the more vehicles must slow in order to prevent "bottoming out", increasing the speed differential with through traffic and increasing the possibility of rear-end collisions.

Table 5I-4.04: Driveway Slope and Entry Speed

Driveway Slope	Typical Driveway Entry Speed
Greater than 15%	Less than 8 mph
14 to 15%	8 mph
12 to 13%	9 mph
10 to 11%	10 mph
8 to 9%	11 mph
6 to 7%	12 mph
4 to 5%	13 mph
2 to 3%	14 mph
0 to 2%	About 15 mph

Source: Oregon State University, 1998

A speed differential much above 20 miles per hour begins to present safety concerns. When the speed differential becomes very large (say, 30 to 35 miles per hour), the likelihood of traffic crashes involving fast-moving through vehicles colliding with turning vehicles increases very quickly. Rear-end collisions are very common on roads and streets when large speed differentials exist and the density of commercial driveways is high. When the speed differential is high, it is also more likely that when crashes do occur they will be more severe, causing greater property damage and a greater chance of injury or fatalities. Keeping the speed differential low is very important for safety reasons, as the table below indicates.

Table 5I-4.05: Speed Differential and Crashes

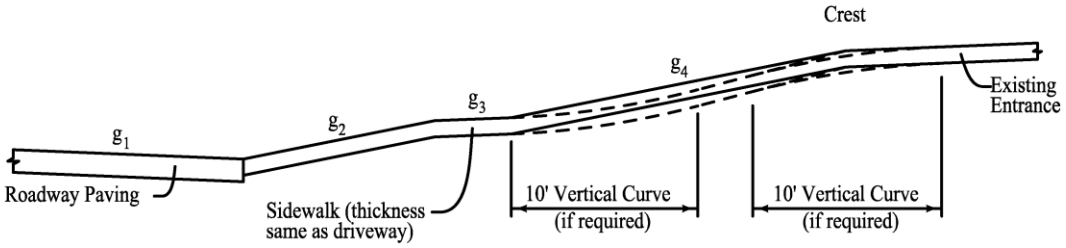
When the Speed Differential Between Turning and Through Traffic Is:	The Likelihood of Crashes Is:
10 mph	Low
20 mph	3 times greater than at 10 mph
30 mph	23 times greater than at 10 mph
35 mph	90 times greater than at 10 mph

Source: Oregon State University, 1998

2. **Vertical Profile:** A driveway's vertical profile should allow a smooth transition to and from the roadway. The National Highway Institute's course workbook on Access Management recommends the following maximum driveway slopes for urban/suburban streets:
   - Arterial 3 to 4%
   - Collector 5 to 6%
   - Local Less than 8% (may use 9% in special areas)

   These slopes were chosen to keep the speed differential at or below 20 miles per hour. See Figures 5I-4.02A and 5I-4.02B.

Figure 5I-4.02A: Typical Section - Commercial/Industrial and Residential Entrance



1. Algebraic Difference Between g1 and g2:
   a. Commercial/Industrial: Not to exceed 9%
   b. Residential: Not to exceed 12%

2. Algebraic Difference Between g2 and g3:
   a. Commercial/Industrial: Not to exceed 6%
   b. Residential: Not to exceed 8%

3. Maximum Slope of g3 = 2% (ADA compliance)

4. Algebraic Difference g3 to g4:
   a. Commercial/Industrial: Not to exceed 5%
   b. Residential: Not to exceed 8%
   c. 10 foot vertical curve required for change in grade exceeding 5%

5. Maximum Slope of g4:
   a. Commercial/Industrial: 7%
   b. Residential: 10%

6. 10 foot vertical curve required for change in grade from g4 to existing exceeding 5%

7. If the above grade restrictions require a depressed sidewalk through the driveway, a transition section should be provided between the normal sidewalk grade and the depressed section. As a general rule, use the following transition lengths:

Elevation Difference from Normal Sidewalk Grade (inches)	**Transition Distance (feet)**
1 to 2	8
2 to 4	12
4 to 6	16
Greater than 6	Desirable max. slope is 16:1 Absolute max. slope is 12:1

3. Non-curb and Gutter Roadways:

a. Private drive access to local, collector, or arterial streets that have no curb and/or gutter improvements should be constructed with grades and dimensions as shown in Figure 5I-4.03. Heavily used driveways connected to existing gravel roadways may require an 8 inch deep compacted Class "A" crushed stone base material. The driveway pavement should be extended to the proposed roadway pavement width, if known, or 15.5 feet from the centerline, if not known. A culvert properly sized for the ditch flow should be installed at the established roadside ditch flowline beneath the private drive access. Culvert should be 15 inches minimum and 18 inches desirable. The culvert should be either corrugated metal or reinforced concrete pipe with minimum of 1 foot of cover over the pipe per the Jurisdiction's requirements.

b. For Farm to Market (FM) roads, when grading on new construction, or complete reconstruction projects on paved (or to be paved) FM roads, the following will apply:
   1) When a culvert is not required, the following slopes will apply.
      - 10:1 slope of flatter from shoulder line to ditch bottom in clear zone area.
      - 6:1 slope or flatter from clear zone area to the right-of-way line.
      - 10:1 to 6:1 transition zone.
   2) When a culvert is required, the following slopes will apply.
      - 8:1 slope or flatter from shoulder line to normal placement of a culvert.
      - 6:1 slope or flatter from culvert area to the right-of-way line.
      - 8:1 to 6:1 transition zone.

      For remaining open ditch roadways (paved or non-paved), the sideslopes will be 6:1 for posted speeds of 40 mph or greater, and 4:1 for posted speeds of less than 40 mph.

F. Other Criteria

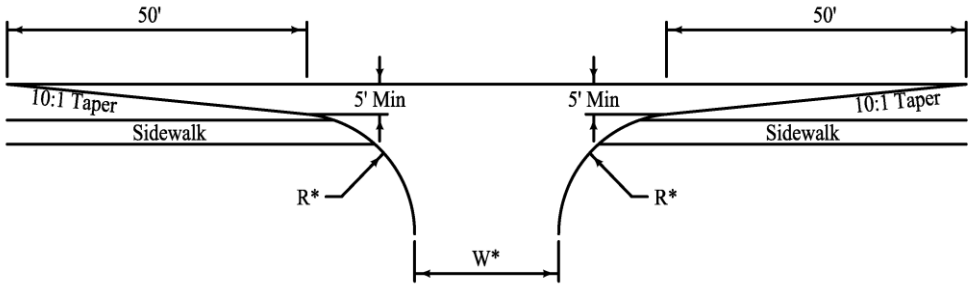
1. Utility Conflicts: Any adjustments made to utility poles, street light standards, fire hydrants, catch basins or intakes, traffic signs and signals, or other public improvements or installations, which are necessary as the result of the curb openings or driveways, should be accomplished with no additional cost to the Jurisdiction.

2. Access Signs: Driveway approaches, whereby the driveway is to serve as an entrance only or as an exit only, should be appropriately signed by, and at the expense of, the property owner subject to approval of the Jurisdiction Engineer.

3. Abandoned Driveways: Any curb opening or driveway that has been abandoned should be restored by the property owner.

4. Offset Radius and Driveway Tapers: For driveways without a right turn lane on the street approach, providing an offset radius and driveway taper can help reduce speed differential between turning and through traffic, reducing the possibility of rear-end crashes. Figure 5I-4.03 shows a typical taper system that can be effectively used. The downstream taper for right turns from the driveway may be considered optional. Right-of-way restrictions may limit the use of this method.

Figure 5I-4.03: Offset Radius and Driveway Tapers



*Driveway radii and widths vary depending on entrance type, street classification, and zoning.

5. **Sidewalks:** For driveways that intersect pedestrian circulation paths and pedestrian access routes (sidewalks and shared use paths), all ADA requirements must be met. See Chapter 12 - Sidewalks and Bicycle Facilities.

G. References

Institute of Traffic Engineers. *Transportation and Land Development*. 1988.

Oregon Department of Transportation. *Driveway Profile Study - Summary of Results*. 1998.

Transportation Research Board - National Cooperative Highway Research Program (NCHRP). *NCHRP Report 659: Guide for the Geometric Design of Driveways for Supplemental Information*. National Academy Press. Washington, DC. 2010.

Traffic Impact Studies

A. General

A traffic impact study may be required for commercial, industrial, or residential developments in obtaining site plan, rezoning, or access permit approval. The Jurisdictional Engineer must be contacted to determine if a traffic impact study is required. If a study is required, the study scope (study limits, analysis years, scenarios, etc.) should be determined through discussion with the Jurisdictional Engineer.

B. Study Process

Traffic impact studies typically include the following elements. Specific tasks, level of analysis, and documentation requirements will depend on the specific needs of the study and Jurisdictional requirements.

1. **Data Collection:** Gather and review needed information regarding existing and proposed conditions, possibly including:
   - Current and historic daily and hourly traffic volume counts.
   - Recent intersection turning movement counts.
   - Projected volumes from previous studies, travel demand models, or area transportation plans.
   - Current land uses, densities, and occupancy near the site.
   - Preliminary site plan for proposed development with land uses, building areas, phasing and completion dates, and proposed access locations identified.
   - Other approved projects and anticipated development near the site.
   - Land use and zoning plans near the site.
   - Current street system information (functional classifications, lane configurations, speed limits, access locations, traffic control, parking)
   - Traffic signal locations, phasing, timing, and coordination.
   - Planned or proposed transportation improvement projects in the area.
   - Crash history (3 to 5 years), if safety concerns have been identified.

2. **Background Traffic:** Determine estimated background traffic for analysis years and scenarios. For simple studies with a short-term analysis year, this may simply be current traffic count data. For more complex studies or longer-range analysis years, background traffic may also include trip generation from proposed area development or land uses, annual traffic growth rates, and/or area travel demand model traffic forecasts.

3. **Site Traffic:** If available, local data should be used in determining estimated daily and peak hour trip generation for the site. If local data is not available, the latest edition of *ITE Trip Generation* or other national data should be used as a basis for estimating trip generation for the site. Sound judgment must be used in reviewing, adjusting, and applying published trip generation data. Depending on specific site characteristics, generated trips may need to be adjusted for mixed-use developments (internal or multi-purpose trips) or unique pedestrian, bicycle, or transit usage. After site-generated trips are prepared, they are distributed and assigned to the study area roadway system, considering the following:

- Type of proposed development and area from which trips will be attracted
- Size of proposed development
- Surrounding land uses and population density
- Proposed site access locations and configurations
- Proposed or anticipated traffic control at access points
- Conditions of surrounding street system
- Competing developments, where applicable

Site traffic is normally distributed in terms of a percentage of inbound or outbound traffic at each study access point, intersection, or ramp junction for each analysis period. These distributions are then used to calculate assigned peak hour traffic turning movement volumes. Assigned traffic is combined with background traffic to determine total traffic for each analysis scenario to be analyzed. Depending on the type of development, the total traffic is often adjusted for pass-by trips. Pass-by trips (or diverted trips) are those trips already on the adjacent street network (background traffic) that will enter and exit the site.

4. **Analysis:** Total peak hour traffic for each study access point, intersection, and ramp junction is analyzed for each analysis scenario according to current *Highway Capacity Manual* (HCM) procedures. Analyses will determine projected vehicle delays, volume/capacity ratios, levels of service, and vehicle queuing. Analysis software such as Highway Capacity Software (HCS) or *Synchro* is typically used. For complicated roadway systems or conditions, additional simulation analysis may also be necessary. In addition to capacity analysis results, several other factors should be considered in evaluating traffic operations for the study, including the following:
   - Crash history, crash rates, predominate crash types, and safety concerns
   - Traffic control needs, including MUTCD traffic signal warrant analysis
   - Anticipated impacts and vehicle queuing and access point/intersection spacing
   - On-site parking, circulation, and potential impacts on adjacent street system
   - Pedestrian, bicycle, and transit needs
   - Service and delivery vehicle access

5. **Improvement Needs:** Based on analysis results, identify access and/or street network improvement needs necessary to provide acceptable operations. Perform capacity analyses with proposed improvements to evaluate expected operations. Typical improvement needs may include:
   - Adding or lengthening intersection turn lanes
   - Widening, reconstructing, or reconfiguring streets to provide needed lanes and geometry
   - Constructing new street connections for access or through traffic
   - Interchange modifications
   - Changes to traffic control or intersection type (such as all-way STOP, signalization, right-in/ right-out only access, or roundabout)
   - Changes to traffic signal phasing, timing, and coordination
   - Access management (combining, eliminating, adding, or improving spacing of access points)
   - Revising site circulation or on-site queue storage
   - Signing or pavement marking modifications

6. **Report:** Prepare and submit to Jurisdictional Engineer a draft traffic impact study report summarizing data collected, analyses performed, and recommendations. Include appropriate tables and graphics. Finalize the report based on comments or concurrence received from the Jurisdictional Engineer.

C. Iowa DOT Access Permits

If a new or modified access is proposed from a highway under the jurisdiction of the Iowa DOT, the applicable District office should be contacted early in the project development to determine access requirements, limitations, and documentation needed. Guidance is provided in the Iowa DOT *Iowa Primary Road Access Management Policy*. Analyses and documentation required will depend on the proposed type and size of development, current access provided, and priority type of the highway. For proposed Type "A" entrances, detailed geometric plans, opening year and full-build year traffic data, and proposed site data are required. Capacity analysis and MUTCD traffic signal warrant analysis may also be required.

D. References

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General Information for Pavement Preservation Program

A. Concept

As effective financial resources for management of infrastructure systems continue to decline, it is necessary to explore different techniques to meet the needs of the public. This is particularly the case for pavements. One such technique is to develop a program of pavement preservation. Pavement preservation techniques have been in use for many years, but most transportation agencies have not developed a complete long-term pavement preservation program that is a part of an overall asset management program. The implementation of a pavement preservation program focuses on maximizing the condition and life of a network of pavements while minimizing the network's life-cycle cost.

The concept of a long-term pavement preservation program is a different approach than has been done in the past; the biggest difference is preservation focuses on being proactive as opposed to reactive. The concept of adopting a proactive preservation approach enables agencies to reduce the frequency of costly, time-consuming rehabilitation and reconstruction projects. It is important to understand, that pavement preservation activities do not include any structural upgrades to the pavement. All treatments are non-structural in nature.

The process to establish a long-term pavement preservation program involves data gathering for each particular pavement section concerning its construction and previous maintenance history, examining the current condition of the pavement, and then determining the appropriate treatment technique to preserve and extend the pavement life.

Customer satisfaction is the greatest benefit of a successful pavement preservation program. From project selection to treatment selection to construction, a good pavement preservation program will benefit users. Safer roads, faster repairs, and a pavement network in better condition that needs fewer repairs are the logical outcomes of a preventative maintenance program.

Transitioning from the traditional rehabilitation and reconstruction activities to a greater emphasis on pavement preservation can be difficult and will require an active educational program. Government officials and the public must be convinced that it is best to provide for continuation of proactive activities to maintain a roadway in good condition as opposed to delaying preservation until major work is required. NCHRP Repot 742 *Communicating the Value of Preservation: A Playbook* is a good source of educational program examples.

B. Definitions

Minor Rehabilitation: Non-structural enhancements made to existing pavements to eliminate age related top-down surface deterioration.

Pavement Preservation: A program employing a network level, long-term strategy that enhances pavement performance by using an integrated, cost-effective set of practices that extends pavement life, improves safety, and meets motorist expectations (FHWA, 2005). A pavement preservation

program is achieved through the application of routine maintenance, preventative maintenance, and minor non-structural rehabilitation.

Preventative Maintenance: A planned strategy of cost effective treatments to an existing roadway system that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system without significantly increasing the structural capacity. (AASHTO, 1997) Preventive maintenance treatments are applied to pavements in good condition and that have significant remaining service life. Preventative maintenance activities are often undertaken by contract, although in-house staff can accomplish them as well.

Routine Maintenance: Day-to-day activities that are scheduled by maintenance personnel to maintain and preserve the condition of the roadways at a satisfactory level of service. Routine maintenance activities are traditionally completed by in-house forces and not contracted out.

C. Benefits

Preventive maintenance activities are the backbone of a pavement preservation program. Simply constructing a roadway and allowing it to deteriorate over its design life is not acceptable. Some of the benefits of a strong preventative maintenance program, besides higher customer satisfaction, include:

- Better informed decisions
- Improved pavement condition
- Cost savings
- Improved strategies and techniques
- Improved safety
- Extended pavement life

An essential part of the pavement preservation process is utilizing all forms of information about a pavement section as the appropriate treatment type is selected. Once the history of a pavement section and the current condition are known, the type of treatment can be selected and the timing of the treatment can be established.

Through the implementation of a pavement preservation program, pavement conditions stabilize because pavements in good condition are maintained in good condition longer. Although it is difficult to prove, especially in the early stages of a pavement preservation program when there are a number of pavements that require major rehabilitation or reconstruction, many agencies have shown that the cost to maintain their roadway system is reduced because of less expensive treatments and extended service lives.

As pavement preservation decisions are being made, agencies are examining different materials and processes that will improve performance of the treatments. The use of high quality materials and quality control are increasingly important.

To users, safety is one of the fundamental expectations they have as they travel. Most pavement preservation treatments will improve the surface characteristics of smoothness and friction. Other surface defects are also addressed. In addition, pavement preservation treatments are less disruptive to traffic movements and extend the time when reconstruction with its extensive impact to traffic flow is needed.

Pavement Preservation Process

A. Pavement Deterioration

The concept behind pavement preservation is to treat pavements while they are still in good condition and without serious structural damage. Successive, systematic treatments will extend the service life and delay the more expensive major reconstruction project. In order to apply the appropriate preventative maintenance treatment at the optimum time, the history and the current condition of a pavement section must be evaluated. Preventative maintenance techniques address the pavement surface condition and do not impact the structural capacity of the pavement. If the structural carrying capacity is affecting the condition of the roadway, it is probably not a candidate for preventative maintenance and it is best to program as a reconstruction project. The causes of any pavement deterioration for various types of pavements must be accurately determined. Typical causes of deterioration for each pavement type include the following:

1. **Flexible Pavements:** Flexible pavements, hot mix asphalt, or other bituminous pavements are affected by traffic, environmental/aging, material problems, and moisture intrusion. These elements impact the pavement in different ways:

    a. **Traffic:** Traffic can lead to load related distresses, such as rutting or fatigue cracking in the wheel paths. Fatigue can lead to development of potholes. Also polishing of the surface leads to friction loss.

    b. **Environmental/Aging:** The environment and aging can lead to oxidation of the asphalt, block cracking, and raveling. Environmental elements can also cause the development of thermal cracks, which are seen as regularly spaced transverse cracks.

    c. **Material Problems:** Material problems include bleeding, shoving, stripping, and surface deformation.

    d. **Moisture Infiltration:** Moisture infiltration can cause further breakdown of existing cracks and thus increased roughness.

2. **Rigid Pavements:** For rigid PCC pavements, the general causes of deterioration include traffic loading, environmental factors, material problems, construction problems, joint deterioration, and moisture infiltration.

    a. **Traffic:** Traffic can lead to load related distress, such as mid-slab cracking, pumping, faulting, and corner breaks. Polishing and the subsequent loss of friction is also traffic related.

    b. **Environmental and Materials:** D-cracking and alkali-silica reactivity (ASR) are material problems. Freeze-thaw action and poor entrained air can affect joint stability.

    c. **Construction Problems:** Construction quality can cause cracking and surface defects in the form of map cracking and spalls.

d. **Joint Deterioration:** Incompressible materials in the joint from poor joint seal maintenance can cause joint spalls.

e. **Moisture Infiltration:** Moisture can lead to further breakdown of cracks and spalls and increased roughness. It can also contribute to pumping, transverse joint faulting, and corner breaks.

B. Evaluating Pavement Conditions

Numerous pieces of information need to be examined in order to determine if the pavement section is a candidate for preventative maintenance and the selection of the type of treatment that best meets that pavement section's needs. The extent of the evaluation process will vary depending on the roadway classification and the type of project. In each case, once the information is compiled, engineering judgment must be applied to determine the correct treatment to use to address the distresses exhibited by each section of pavement.

1. **Background Data:** Obtain data from project files, such as original design parameters, construction information regarding materials, subgrade/subbase information, current traffic data, and maintenance activities undertaken on that roadway section. This information can sometimes be difficult to locate if a good record system has not been established, but as much information as possible should be compiled. Discussions with agency engineering and maintenance staff members can potentially fill in gaps in records.

2. **Existing Condition:** Undertake a visual site inspection to obtain information about the condition of the pavement. Ascertain information on what types of distress are exhibited by the pavement section. Note any restrictions such as right-of-way limitations, presences of bridges, drainage problems, and obstructions.

   The specific severity and extent of each type of pavement distress should be examined closely. Additional field testing such as falling weight deflectometer (FWD), pavement cores, friction testing, splash and spray, and materials testing may be necessary. The extent of the additional testing may be dependent on the roadway classification. Much of this information should be contained in a pavement management system (PMS). The PMS can be in many forms including a sophisticated computer program, a relatively simple spreadsheet, or notes accumulated by maintenance personnel.

   The Iowa DOT has a program of data collection on all roads in the state and is one source of pavement condition information. The program in administered by the CTRE program at The Institute for Transportation at Iowa State University. Information can be accessed at http://www.ctre.iastate.edu/ipmp/services/. It is necessary to undertake additional effort to convert the raw data to useable information.

3. **Future Projections:** It is also important to evaluate any future changes that may be expected for each roadway section. Changes in adjacent land use or improvements to area roadways could impact the traffic volume and the vehicle mix of a roadway section. Long-range transportation planning documents should provide this information. This information is critical to understanding the service life expectancy of the existing pavement and then the subsequent preventative maintenance treatments to match that service life.

Preventative Maintenance Treatment Type Selection

A. Introduction

Once all of the background, existing pavement condition, and future changes have been determined for a pavement section, the appropriate preventative maintenance treatment or treatments can be selected. Professional engineering judgment is critical in order to analyze the available data and select the most effective treatment. The selection of the most appropriate treatment must also take into consideration the availability of qualified contractors and the availability of quality materials to accomplish the work. In some instances, a combination of treatments may be needed to maintain the pavement in good condition.

In addition to the technical analysis, it is important to complete a financial review that will compare the various treatment types, their expected service life, and the associated costs. Comparisons can be made by calculating a simplified annualized cost through dividing the estimated cost of the treatment by the expected service life of each treatment type.

B. Flexible Pavement Treatment Types

Several traditional preventative maintenance treatments are available for flexible pavements. These include:

- Crack filling
- Crack sealing
- Full/partial depth patches
- Fog seals
- Slurry seals
- Microsurfacing
- Bituminous seal coats
- Milling
- Thin overlays

The above treatments will be described in greater detail. Additional treatments are available, but generally involve use of proprietary materials or processes or are not included in this manual. If appropriate, designers should include some of these other treatments in their analyses. These treatments are only effective if there are no structural problems with the pavement or the supporting subbase/subgrade.

1. **Crack Filling:** Crack filling is a good treatment method for reducing intrusion of moisture through the pavement slab. It will assist in reducing further crack deterioration, associated roughness, and rutting. Crack filling will traditionally involve minimal preparation and use of lower quality bituminous materials. Treatment should occur during cool, dry weather, which will provide for wider crack widths. Proper cleaning and a dry condition are the key to achieving good performance and maximizing service life. Cracks should be cleaned to a depth of 3 inches. Crack filling material is generally an asphalt emulsion since actually sealing of the crack is not

expected. Crack filling is appropriate for non-working cracks between 1/4 inch and 1 inch wide. The potential exists for increased roughness and loss of surface friction if the joint is overfilled. See SUDAS Specifications Section 7040, 3.07.Service life is from 2 to 4 years.

2. **Crack Sealing:** Crack sealing is effective at reducing moisture intrusion in the pavement as well as minimizing the amount of incompressible materials in the cracks. It differs from crack filling in that it is used on working cracks and involves crack routing, substantial crack preparation, and higher quality sealant material. Crack sealing is appropriate for cracks between 1/4 inch and 3/4 inch wide. Use on longitudinal or transverse cracks with little or no secondary cracking or raveling at the crack face. Proper crack preparation and cleaning are essential to optimal performance. Saw or rout cracks to a minimum 3/8 inch width and a depth of 1/2 inch. The width and depth may be adjusted depending on the sealant to be used. Clean cracks of existing joint filler material, vegetation, dirt, or other foreign material. See SUDAS Specifications Section 7040, 3.06. Service life is from 2 to 8 years.

3. **Full/Partial Depth Patches:** Patches restore a pavement's structural integrity and improve its ride. Partial depth patches address distress in the upper one-third of the pavement slab. Slab removal may be accomplished by sawing and jackhammer or by milling. Minimum partial patch depth is 2 inches and maximum depth is 1/2 of the slab thickness. Prior to placement of patch material, clean partial depth patch area and ensure it is dry. Cover entire patch area with tack coat. Lifts should not exceed 3 inches in thickness with the top lift 2 inches or less. Ensure the final compacted surface is level with or not more than 1/8 inch above the surrounding pavement. Full depth patches will address various types of more structural distress, such as broken down thermal cracks. Apply tack coat to all vertical edges. Maximum lift thickness is 3 inches with the top lift being 2 inches or less. Compact intermediate lifts with a roller or vibratory compactor, depending on patch size. Compact final lift with steel-wheeled finish roller. Ensure final compacted surface is level with or not more than 1/8 inch above the surrounding pavement. See SUDAS Specifications Section 7040, 3.02 and 3.03. Patches are often completed in advance of a surface treatment. Service life is from 3 to 15 years.

4. **Fog Seals:** Fog seals are applications of diluted emulsion without a cover aggregate and are used to seal the pavement, inhibit raveling, and slightly enrich hardened or oxidized asphalt. Application rates vary from 0.05 to 0.10 gallons per square yard. If necessary, vegetation control should be completed in advance of the treatment. Ensure pavement is clean and dry prior to application. See Section 5K-4 for additional information. Fog seals can have a negative effect on friction and stripping in susceptible asphalts. Service life is from 1 to 3 years.

5. **Slurry Seals:** Slurry seals are effective at sealing low-severity cracks, waterproofing the pavement, and restoring friction. Slurry seals also address raveling, oxidation, and hardening of asphalt. They are a mixture of crushed, well-graded aggregate, a mineral filler, and asphalt emulsion that is spread across the full width of the pavement or it can be used as a strip treatment for low areas and cracks. Thickness is generally less than 1/2 inch. The slurry is basically placed one aggregate layer thick. Allow a minimum of 7 days cure time before applying permanent pavement markings. See Section 5K-4 and SUDAS Specifications Section 7070. Service life is 3 to 6 years.

6. **Microsurfacing:** Microsurfacing corrects or inhibits raveling and oxidation of the pavement, improves surface friction, reduces moisture infiltration, addresses low to medium severity bleeding, and can be used to fill surface irregularities and ruts up to 1 1/4 inch deep. Microsurfacing materials are similar to slurry seals except that microsurfacing uses latex modified asphalts versus an emulsified asphalt. Application of the microsurfacing is by specialized equipment using an augured screed. Microsurfacing typically breaks within a few

minutes of placement and can carry traffic after about an hour. See Section 5K-4. Service life is 4 to 7 years.

7. **Bituminous Seals Coats:** Seal coats, also sometimes known as chip seals, are effective at improving surface friction, inhibiting raveling, correcting minor roughness and bleeding, and sealing the pavement surface. Bituminous seal coats are also used to address longitudinal, transverse, and block cracking, as well as sealing medium severity fatigue cracks. Seal coats can be applied in multiple layers to address more serious problems. Asphalt emulsion is applied directly to the pavement surface and is followed by the application of aggregate chips that are immediately rolled to embed them into the emulsion. Application rates depend upon the aggregate gradation and maximum size. Loose chips may be a problem on higher speed roadways. Fog seals may be used in conjunction with seal coats to provide a greater degree of binding for the aggregates. See Section 5K-4 and SUDAS Specifications Section 7060. Single layer service life is 4 to 6 years.

8. **Milling:** Milling is used to reduce pavement irregularities and to produce a uniform surface. Milling should be considered if rutting is at a level of 1/4 inch or more. Milling is used in conjunction with other surface treatments, such as slurry seals and microsurfacing in addition to thin asphalt overlays, and is not suggested to be used as a final stand-alone treatment. It can be used to restore proper grades and pavement cross-slopes. For best results, the milling depth should match the lift thickness of the exiting pavement. See Section 5K-4 and SUDAS Specifications Section 7040, 3.05.

9. **Thin Overlays:** Thin overlays are placed in a single lift less than 1 1/2 inches thick. The overlay is expected to improve rideability, surface friction, profile, crown, and cross slope. In addition, specific distress types of low severity cracking, raveling, roughness, low severity bleeding, and low severity block cracking are improved. Thin overlays dissipate heat rapidly and rely on timely compaction to be successful. Dense-graded, open-graded, and stone-matrix mixes may be used. See SUDAS Specifications Section 7020. Service life is 7 to 10 years.

C. Rigid Pavement Treatment Types

Several preventative maintenance treatment types are available to address pavement distresses in PCC pavements. These include:

- Crack sealing
- Joint resealing
- Partial depth patches
- Full depth patches
- Dowel bar retrofit
- Diamond grinding
- Pavement undersealing/stabilization
- Pavement slab jacking
- Concrete overlays

These are the traditional preventative maintenance treatment types. Other less frequently used treatments are available to address specific distress needs.

1. **Crack Sealing:** Crack sealing is accomplished to reduce moisture intrusion and retard the rate of deterioration of the cracks. It is accomplished by thorough preparation and placement of high quality materials. It is used on random transverse and longitudinal cracks of low to medium severity where the crack width is less than 1/2 inch. Proper preparation of the crack and placement of the sealing material are critical for attainment of the expected 4 to 8 year service

life. The sealant material is critical to the success of the operation. Thermoplastic (rubberized asphalt) and thermosetting (silicone) sealants are the usual materials. The crack should be routed to 3/8 inch wide and 1/2 inch deep. The crack should be thoroughly cleaned and dried prior to application of the sealant. Refacing the sides of the crack with sandblasting is recommended. See SUDAS Specifications Section 7040, 3.06.

2. **Joint Resealing:** Joint resealing is important to minimize moisture in the joint and the subgrade/subbase, in addition to minimizing the intrusion of incompressible materials into the joint. Proper resealing of joints will reduce faulting, pumping, and spalling. Removal of the old sealant material and cleaning of the joint prior to resealing are critical. Removal of the old joint material can be accomplished by using a rectangular joint plow, diamond saw, or high-pressure water blast. Following refacing of the joint with a diamond bladed saw, the joint should be cleaned with high pressure air or water. Immediately prior to sealant application, the joint should be blown again with high pressure air to remove any sand, dust, or other incompressible that may remain in the joint. The joint must be dry and clean as joint sealant material is applied. See SUDAS Specifications Section 7040, 3.06. Service life is 4 to 8 years.

3. **Partial Depth Patches:** Partial depth patches are used to address spalling and surface scaling, as well as other problems in the top one-third of the pavement slab. Repair materials are selected based on available curing time, ambient temperature, size and depth of the repair, and cost. The materials are generally classified as cementitious, polymers, or bituminous. Rapid cure and high strength proprietary products are also available. It is critical to identify the limits of the weakened concrete so the patch can connect to sound concrete. The actual extent of the deterioration is often greater than what is visible at the surface. The removal area should extend a minimum of 3 inches beyond the deteriorated area in all directions. The patch area can be prepared by chipping with a lightweight jackhammer, milling with a carbon tipped milling machine, and sawing the edges of the patch and removal with a lightweight jackhammer. The patch area should be square or rectangular in shape and in line with existing joint patterns. The repair area must be swept, sandblasted, and air blasted to ensure a clean, dry patch area. Sandblasting is very effective at removing any dirt, oil, thin layers of unsound concrete, and laitance. Bonding agents are generally required for the patch materials. Sand-cements grouts consisting of one part sand and one part Type III cement with sufficient water to create a thick, creamy consistency have proven successful. Epoxy bonding agents can also be used with PCC and proprietary patching materials. Compressible joint materials must be used against the adjoining slab or to extend an existing joint through the patch area. The compressible material should extend 1 inch below and 3 inches beyond the repair boundaries. It may be possible to saw the joint through the patch, but timing is very critical. Since partial depth patches have large surface areas compared to their volume, it is very important to apply a curing compound as soon as the water has evaporated from the surface. The curing compound should be applied at 1.5 to 2 times the normal rate. The final step is resealing of the joint. See SUDAS Specifications Section 7040, 3.03. Service life of a well done partial depth patch is 5 to 15 years.

4. **Full Depth Patches:** Typical PCC pavement distresses that can be addressed by full depth repairs include transverse cracking, corner breaks, deteriorated joints, and blowups. Full depth repairs are an effective means for restoring the rideability and structural integrity of deteriorated PCC pavements. Long lasting full depth repairs are dependent upon selecting appropriate locations, effective load transfer design, and correct construction procedures, including finishing, texturing, and curing the patch. If the pavement exhibits a materials related deficiency, such as D-cracking, the service life of the patch will be short. Sizing the patch is critical to its success. Distressed areas should be identified and marked. Extent of the patch area may have to be adjusted if a period of time passes between initial identification and actual work activity. It may be necessary to do coring and deflection studies to identify the extent of deterioration below the slab surface. Full depth patches should be a minimum of 6 feet long and a full lane wide. All

joints through or adjacent to full depth patches must be re-established. Connect patches to make one large patch if the patches are 8 to 10 feet from each other in a single lane. The load transfer technique used in the patch should match the load transfer technique in the existing slab. Full depth repairs could be used in conjunction with diamond grinding to correct any roughness problems. See SUDAS Specifications Section 7040, 3.02. Service life is expected to be from 10 to 15 years.

5. **Dowel Bar Retrofit:** Dowel bar retrofit (DBR) is a method of load transfer restoration. It is used on non-doweled plain jointed concrete pavements. A successful dowel bar retrofit project will enhance pavement performance by reducing pumping, faulting, and corner breaks. Pavements with structurally adequate slab thickness, but exhibiting significant loss of load transfer due to poor aggregate interlock or base/subbase/subgrade erosion, are good candidates for DBR. It will also retard deterioration of transverse joints and cracks. Typical design includes three or four dowels inserted into the pavement at joints in each wheel path. The size of the dowel bar varies from 1 inch to 1 1/2 inches in diameter according to the slab thickness. See SUDAS Figure 7010.101. The slots are generally 3 feet long, centered on the joint or crack. The slot must be long enough to allow the dowel to lie flat in the slot without hitting the curve of the saw cut. The width of the slot should be 2.5 inches and the depth sufficient to position the center of the dowel at the mid-depth of the slab. The slot must be parallel to the centerline of the pavement slab so the dowels do not lock up pavement movements. The dowel assembly will have end caps to facilitate movement and a compressible insert to form the joint across the slot. The slot filler materials are the critical element to a successful installation. Desirable properties include little or no shrinkage, similar coefficient of thermal expansion as the existing concrete, good bond strength, and the ability to gain strength rapidly. Concrete with Type III cement, sand, and 3/8 inch maximum sized aggregate can be used or there are proprietary products available. Dowel bar retrofit projects often include following up with diamond grinding. All transverse joints should be re-established by sawing over the joint and through the fill board. The joint should then be prepared and sealed. Dowel bar retrofit projects will allow the original service life of the pavement to be restored.

6. **Diamond Grinding:** Diamond grinding is the removal of a thin layer of pavement surface using closely spaced diamond saw blades. It is used to improve ride quality by eliminating joint and crack faulting. In addition, surface friction, transverse cross slope, and tire/pavement noise are improved. It does not address structural problems or material related distress. Structural problems, such as pumping, corner breaks, and working transverse cracks, must be addressed before grinding. If joint/crack faulting exceeds 1/4 inch, the project may not be a candidate for diamond grinding. The blade spacing and width of groove are dependent on the hardness of the aggregate. As the aggregates get softer, the width of the land area and groove get larger. The depth of cut should be set so that 95% of the area is ground. The surface distresses will redevelop if the root cause of the distress is not corrected prior to diamond grinding. Thus, it may be necessary to complete full and partial depth patches, load transfer restoration, and slab stabilization prior to grinding. See SUDAS Specifications Section 7040, 3.04. Service life varies from 5 to 15 years, depending on the hardness of the aggregates and the level of structural distress correction completed prior to grinding.

7. **Pavement Undersealing/Stabilization:** Slab stabilization is pressure insertion of a flowable material to restore support beneath PCC slabs. It fills existing voids but does not lift the slab. Pavement stabilization restores pavement support, reduces pavement deflections, and reduces progression of pumping, faulting, and corner breaks. Slab stabilization must be completed prior to significant pavement damage. The main issue with slab stabilization is identifying where the voids are located and the extent of the voids. Distress surveys and deflection testing are necessary. Deflections may be measured using a FWD or by using a loaded truck with gauges placed at the corners of the slab. Other methods, such as ground penetrating radar or

thermography, are also available. Pozzolan-cement grout and polyurethane are the most common materials used for slab stabilization. Other proprietary products are available. It is important to only apply the material at locations where voids exist. If it is placed in areas without voids, the material can induce pressure points and actually increase the pavement deterioration. Once the area of the void is determined, the grout insertion holes can be drilled. Holes should be placed as far as possible from cracks and joints. Holes should be placed close enough to achieve flow from one insertion hole to another. Service life is from 5 to 10 years, depending on the level of truck traffic.

8. **Pavement Slab Jacking:** Slab jacking consists of the pressure insertion of a grout or polyurethane material beneath the PCC slab as a means of raising the slab to a smoother profile. Slab jacking is normally used to correct localized settlement areas, such as over culverts or at bridge approaches. It should not be used to correct faulted joints. Grout insertion holes should be a minimum of 12 inches from a transverse joint or the edge of the slab. Holes should be spaced 6 feet or less center-to-center. It is critical to monitor the amount of lift performed at each location. The slab should not be lifted more than 1/4 inch at a time so that excessive stresses are prevented and slab cracking minimized. Uniform positioning of the grout holes is also important. Work should start from the lowest point of the section being raised and proceed out to the edges of the settled area in a repeating pattern. Materials for slab jacking are typically stiffer than those used for slab stabilization. Cement grout and polyurethanes are typically used.

9. **Overlay:** Concrete overlays exist for all types of pavements, including concrete, asphalt, and composite. Thickness for preservation projects are generally between 3 to 4 inches. Similar to other concrete pavements, overlays require uniform support and effective management of movement. The overlay type can be bonded or unbonded. Bonded overlays are used to eliminate surface distresses when the existing pavement is in good structural conditions. Bonded overlays utilize the existing pavement as an integral part of the new monolithic system and thus thorough surface preparation is critical. Unbonded overlays are essentially a new pavement over a stabilized base (the old pavement). A bond breaker, such as a thin asphalt layer or a layer of non-woven geotextile, is needed between the existing pavement and the overlay. Typically, overlays are constructed using standard concrete mixes and standard construction techniques. Fibers may be added to the concrete mix for additional strength. Joints in bonded concrete overlays must match those in the existing pavement. Service life of concrete overlays is 15 to 20 years. Visit the National Concrete Pavement Technology Center's website (www.cptechcenter.org) for more publications on concrete overlays.

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Thin Maintenance Surfaces

A. General

Seal coats, slurry seals, microsurfacing, and fog seals are termed thin maintenance surfaces or TMS. These thin maintenance surfaces can be a cost effective approach to maintaining flexible pavements. Studies have shown that agencies can maintain a city street or county road network in better condition at lower costs through the use of TMS. Project selection, treatment selection, and timing are critical to the use of TMS.

Since TMS do not involve increasing the structural carrying capacity of a street, it is vitally important to apply the appropriate treatment prior to the start of pavement deterioration. Pavement condition, traffic volumes, materials availability, roadway classification, and local preference must be evaluated before determining the type of TMS to use. General uses for TMS are noted in the following table:

Criteria	Seal Coat	Slurry Seal	Microsurfacing
Traffic Volume:			
Low (< 2,000 vpd)	Recommended	Recommended	Recommended
Medium (2,000 to 5,000 vpd)	Marginal	Marginal	Recommended
High (> 5,000 vpd)	Not Recommended	Not Recommended	Recommended
Bleeding	Recommended	Recommended	Recommended
Rutting	Not Recommended	Recommended	Recommended
Raveling	Recommended	Recommended	Recommended
Cracking			
Slight	Recommended	Recommended	Recommended
Moderate	Recommended	Not Recommended	Not Recommended
Low Friction	May improve	May Improve	May Improve
Snowplow Damage	Most susceptible	Moderately susceptible	Least susceptible

Source: Jahren, 2003

Design of these TMS treatments must take into account the type of pavement distress that is being addressed with the proposed project. It may be necessary to complete crack filling, patching, or other maintenance activities prior to implementing the TMS.

B. Seal Coat

A seal coat is a single layer of asphalt binder that is covered by embedded aggregate with its primary purpose to seal fine cracks in the underlying pavement and retard water intrusion into the pavement and subgrade/subbase. The aggregate protects the asphalt binder layer and provides macrotexture for improved skid resistance. Seal coating is also a cost effective way to address bleeding and raveling. Most often, the asphalt binder is an emulsion. Cutback asphalts may be used as well. Emulsified asphalt is a mixture of liquid asphalt and water. A cutback is a mixture of liquid asphalt and a distillate, such as kerosene or fuel oil. The aggregates are typically less than 1/2 inch in size.

One of the most critical factors in the design is to determine the quantities of asphalt binder and aggregate. The goal should be to have the single layer of stone 70% into the asphalt binder layer with

little or no stones to clean up. In order to attain that goal, the designer must take into account the traffic volume; the absorption of the binder into the cover aggregate; the texture of the existing pavement; and size, shape, and gradation of the aggregate. Seal coat projects have an expected life span of 4 to 6 years.

Seal coating is recommended for low and medium volume roadways with low speeds due to the increased chance for insurance claims for vehicle damage from the loose rock as traffic volumes and speed increases. In addition, the impact to the public is compounded on high volume roadways due to the time the facility is out of service, generally 24 hours. As traffic volumes increase, it becomes more critical to include very high quality, durable aggregates in the mix design.

Selection of the asphalt binder is important to the success of the project. Although cutback asphalts can be used, their use has rapidly declined over the years due to the costly and harmful solvents used. Typically, asphalt emulsions are used. They are made up of asphalt cement, water, and an emulsifying agent (surfactant). The asphalt cement is typically in the same range as is used for hot mix production and makes up about 2/3 of the volume of the binder. Water provides the medium to keep the asphalt in suspension. The surfactant (usually soap) causes the asphalt particles to form tiny droplets that remain in suspension in the water, and it determines the electrical charge of the emulsion. It is important that the emulsion and the aggregate have opposite electrical charges in order to maximize the bond between the emulsion and the aggregate. Since most aggregates have a negative charge, emulsions such as CRS-2P with a positive (cationic) charge are used.

Cover aggregate should be clean and dust free to maximize adherence. A uniform gradation of hard, durable aggregate will increase the resistance to impact from traffic and snowplows. Aggregate application needs to follow binder application very closely. The cover aggregate should be applied so it is only one layer thick. Excess aggregate increases the chance for dislodging properly embedded aggregate during the cleanup operation, as well as increasing the potential for vehicle damage. The aggregate may be gravel, crushed stone, or a mixture. Cubical shaped aggregate is preferable to flat aggregate. Flat and elongated aggregates can be susceptible to bleeding due to traffic causing the flat chips to lie on their flattest side. If flat aggregate is used and the binder is applied too thick, the pavement will bleed; if it is too thin, the pavement will ravel. Angular aggregate is preferable to round aggregate because angular aggregate chips tend to lock together.

One of the problems with seal coats is the generation of dust from the aggregate. One way to address the dust problem is to pre-coat the aggregate. Pre-coating involves applying either a film of paving grade asphalt or a specially formulated pre-coating bitumen to the aggregate. The use of pre-coated aggregate improves aggregate bonding properties, as well as reducing dust. It also shortens the required curing time and vehicle damage from loose aggregate. Fog seals may also be used to address dust problems and to cover the "gravel road" appearance of seal coat. Fog seals are generally a 50-50 mix of emulsion and water. It is important to recognize that skid resistance may be compromised with the use of fog seals.

Many design tools are available. One of the most often used is the Minnesota Seal Coat Handbook. It can be found at: http://www.lrrb.org/media/reports/200634.pdf. Another source is the Thin Maintenance Surfaces Manual developed by the Institute for Transportation at Iowa State University. It can be found at: http://www.intrans.iastate.edu/publications/_documents/handbooks-manuals/thin-maintenance-surfaces/thin_maint_surf.pdf.

C. Slurry Seal

Slurry seal is a mixture of emulsified asphalt oil, aggregates, water, and additives. It is pre-mixed and placed as a slurry onto the pavement. Slurry seals are commonly recommended for use on low and medium volume roadways. They are used to treat low to medium levels of raveling, oxidation, and rutting. Applications of slurry seals will improve skid resistance. Slurry seals are often described as the most economical, versatile TMS for low to medium volume roadways.

Aggregates commonly used for slurry seal applications consist of a combination of crushed stone and additives, such as Portland cement, lime, and aluminum sulfate. The additives are used to modify curing time. Aluminum sulfate retards curing time and Portland cement and lime shortens curing time. The aggregate gradations are described as fine and coarse. Coarse gradations have greater stability and are preferred for rut filling and scratch (bottom) courses. The additives make up less than 2% of the mixture and the aggregates are about 75% to 80%. Higher quality aggregates such as granite and quartzite will provide for a more durable application of slurry seal. Smaller aggregate gradations are used for maximum crack sealing, while coarse gradations are used when the project goal is to improve skid resistance.

The asphalt binder is an asphalt emulsion. The usual grades are CSS-1h or SS-1h, which are cationic and anionic slow setting emulsions, respectively. The emulsion is formulated with relatively stiff base asphalt (the suffix h = hard) for use in warm climates. The emulsion will make up about 7% to 14% of the mixture. Water is the remaining element of the mixture.

Temperature and humidity are critical to the cure time of the slurry seal. Temperatures must be 50°F and rising before application can begin. Slurry seals should not be placed at night. Slurry seals can be used to address slight rutting distress as well as to fill open joints by a strip treatment.

Mix design is generally completed by a laboratory certified by the International Slurry Seal Association (ISSA). Compatibility of the emulsion, aggregates, water mineral filler additives, and any other elements needs to be checked using materials that will be incorporated into the project.

D. Microsurfacing

Microsurfacing is a mixture of polymer-modified asphalt emulsion, graded aggregates, mineral filler, water, and other additives. It is mixed in a pug mill and evenly spread over the pavement. It is used to address oxidation, raveling, rutting, and skid resistance problems. Microsurfacing can be applied to higher speed, higher volume roadways than slurry seals and it can be used on both asphalt and concrete roadways. It can be placed at a thickness that is two or three times the size of the largest aggregate; however, trying to lay it too thick may result in rippling, displacement, and segregation. Multiple lifts are used if thicker application rates are needed.

Microsurfacing differs from slurry sealing in four main areas. They are:

- Microsurfacing can be placed in layers thicker than a single aggregate size.
- Microsurfacing always contains polymer modifiers.
- It cures through a chemical reaction versus evaporation.
- Higher quality aggregates are used.

Microsurfacing can also be accomplished at night to potentially minimize traffic impacts. If specifically designed for rapid opening, a microsurfacing project may be returned to traffic in as little as 1 hour.

Design of the microsurfacing mix is generally included in the contract to be completed by the

contractor and/or the emulsion supplier. It is critical that all elements of the mix be compatible with each other in order to develop a mix that will address the project conditions. Publication A143 from the ISSA is used as a guide for development of the mix design.

Generally, a single emulsion that works for the climatic conditions and traffic volumes is selected. Cationic emulsions, such as CSS-1h, are typically used, but CQS-1h can be used if it is important to minimize traffic delays due to construction activities. Polymers are added to the emulsions to reduce thermal susceptibility, improve thermal crack resistance, and improve aggregate retention.

Aggregates must be high quality with fractured faces to form higher bonds with the emulsions. Freshly crushed aggregates, as opposed to weathered aggregates, have a higher electrical charge and improve the bond as well. Washing the aggregate to remove clay, silt, and dust is important to ensure proper cohesion.

Mineral fillers are used to aid in the mixing process and spreading of the mixture. Typical mineral fillers are Portland cement, hydrated lime, fly ash, kiln dust, limestone dust, and baghouse fines.

Additives may also be included in the mix. Aluminum sulfate, aluminum chloride, and borax are typically used. These additives allow the contractor to control breaking and curing times.

Properly designed and applied microsurfacing projects have a service life of up to 7 years.

E. Fog Seal

A fog seal is an application of diluted asphalt emulsion without a cover aggregate. It is used to seal and enrich the asphalt surface, seal minor cracks, and provide shoulder delineation. Fog seals are used on low and high volume roads. Its primary use on high volume roads has been to prevent raveling of open-graded friction courses in addition to delineating between the mainline and shoulder.

A fog seal is designed to coat, protect, and/or rejuvenate the existing asphalt binder. Fog seal use on mainline pavements should generally be restricted to only those locations having an open surface texture. This includes chip seals, heavily aged dense graded pavements, and open graded pavements. The fog seal emulsion must fill the voids in the surface of the pavement. A slow setting emulsion such as CSS-1 or SS-1, diluted to one part asphalt emulsion to four parts water is used. Emulsions that are not correctly diluted may not properly penetrate the surface voids and a slippery surface may be the result.

Before placing the fog seal, the pavement must be dry and clean and all pavement repairs accomplished. The diluted asphalt emulsion should be applied at 0.12 gallons per square yard. Success of application is impacted by temperature so summertime application is required. Generally, no application past August 31 is allowed. Pavement and air temperatures must be greater than 60°F to apply the fog seal.

The service life of a fog seal is fairly short, ranging from 1 to 2 years.

Permeable Interlocking Pavers

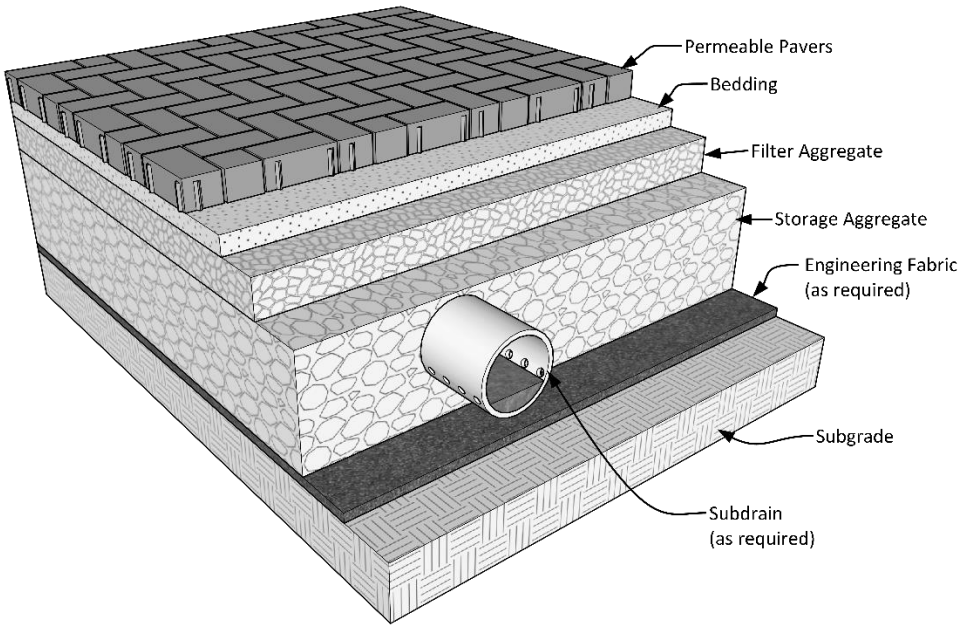
A. General

Permeable pavements are designed to infiltrate runoff, whereas runoff sheds off the surface of conventional pavements. In permeable pavements, runoff passes through the surface and is stored in the aggregate base. In pervious soils, the runoff infiltrates the soil; in less permeable soils, a subdrain system is placed to slowly discharge the runoff. Runoff volume reduction is achieved as the water is infiltrated into the underlying soils. The peak runoff rate is reduced due to the stormwater being stored in the aggregate subbase and slowly released to the downstream piping systems. Traditionally, at a minimum, the depth of the aggregate subbase is designed to meet the storage needs for the Water Quality Volume (WQv), which is 1.25 inches of rainfall in Iowa.

Permeable pavements can dramatically reduce the surface runoff from most rainfall events by disconnecting and distributing runoff through filtration and detention. The use of permeable pavements can result in stormwater runoff conditions that approximate the predevelopment site conditions for the immediate area covered by the pavers.

The design of permeable interlocking pavements (PIP) involves both structural and hydrological analyses. Figure 5L-1.01 illustrates a typical cross-section of a PIP. These two design elements are typically not interconnected and in reality are often in conflict. This is particularly the case with the subgrade treatment and volume of aggregate subbase. Structural design requires a compacted subgrade and the hydrologic design desires an uncompacted subgrade to allow as much infiltration as possible. In most instances, the hydrologic requirements for filter and storage aggregate exceed the structural needs for the unbound aggregate subbase.

Figure 5L-1.01: Permeable Interlocking Paver Cross-section



PIP are used for low speed/low volume streets, alleys, parking lots, and driveways. The design and operating speed of the facility should be below 35 mph. Permeable paver projects should only be developed in areas dominated by impermeable surfaces or surfaces that are fully vegetated so that sediment runoff is minimized and life of the pavement is maximized. PIP are capable of handling truck traffic.

The following elements should be reviewed prior to undertaking a detailed design process:

- Underlying geology and soils
- NRCS hydrologic soils groups
- History of fill, disturbance, or compaction of underlying soils
- Current drainage patterns and volume of runoff
- Local and downstream drainage facilities
- Distances to potable water supply wells
- Elevation of the static water table
- Traffic volumes, including percent trucks

Because water is stored in the subbase rock, it may be necessary to protect structures that are adjacent to the permeable paver project by sealing the foundation walls. The PIP must be a minimum of 100 feet from a municipal water supply well.

There are two types of permeable interlocking pavers. One type is concrete pavers that are 3 1/8 inches thick; the other type is clay brick pavers that are at least 2 5/8 inches thick. The concrete pavers must comply with ASTM C 936. There are two ASTM standards for brick pavers, depending on the traffic loading. ASTM C 902 is for pedestrian and light vehicular traffic locations. ASTM C 1272 is for heavier vehicular traffic and will be the type listed in the SUDAS Specifications. The clay pavers should be 2 3/4 inches thick, Type F brick for PX applications according to ASTM C 1272.

B. Structural Design

The design procedure for permeable interlocking pavers is the same as for flexible pavements. Research has shown that the load distribution and failure modes of PIP are similar to other flexible pavements. Because the designs are the same as for flexible pavements, the AASHTO *Guide for Design of Pavement Structures* (AASHTO, 1993) can be used. The paver used in design for concrete pavers is a 3 1/8 inch thick paver with a minimum 1 inch bedding layer. The structural coefficient is 0.44 per inch. This provides a structural number of 1.82. The clay brick paver is 2 3/4 inches thick, which has a corresponding structural number of 1.21. The remaining structural support comes from the aggregate layers and the soil subgrade.

The American Society of Civil Engineers has developed a design standard called *Structural Design of Interlocking Concrete Pavement for Municipal Streets and Roadways* (ASCE/T&DI/ICPI 58-10). The structural design for clay brick pavers is the same as for concrete pavers. The engineer will need to determine or select the following:

- Design traffic loading (ESALS)
- Design life (40 years minimum)
- Design reliability (usually 75% to 80%)
- Overall standard deviation (0.45)
- Required structural number to meet traffic loading
- Initial serviceability (flexible pavements = 4.2)
- Terminal serviceability (local streets = 2.0)
- Subgrade resilient modulus based on saturated soil characteristics, including seasonal variability

- Drainage conditions

Once these elements are determined, the design thickness of the unbound aggregate subbase can be determined. The ASCE design standard has tables showing thickness of the layers that were developed using the AASHTO 1993 Guide. Thickness is selected based on the ESALS, the soil category, and the drainage.

Three types of interlock are critical to achieve: vertical, rotational, and horizontal. Vertical interlock is achieved by the shear transfer of loads to surrounding pavers through the material in the joints. Rotational interlock is maintained by the pavers being of sufficient thickness and aspect ratio (3:1 minimum), being placed close together, and restrained by a curb from lateral forces of vehicle tires. Rotational interlock can be further enhanced if there is a slight crown to the pavement cross-section. Horizontal interlock is primarily achieved through the use of laying patterns that disperse forces from braking, turning, and accelerating vehicles. Herringbone patterns, either 45° or 90°, are the most effective patterns for maintaining interlock. A string or soldier course should be used at the interface between the pavers and the edge restraint.

A PCC edge restraint is typically used for street and alley projects. The edge restraint may be a standard curb and gutter section, a vertical curb section, or a narrow concrete slab, and should be placed on the subbase aggregates.

After placement, the pavers are compacted with a high frequency plate compactor, which forces the joint material into the joints and begins compaction of the paver into the bedding layer. The pavement is transformed from a loose collection of pavers into an interlocked system capable of spreading vertical loads horizontally through the shear forces in the joints.

One of the direct conflicts with the hydrologic design of PIP is the compaction of the subgrade soils. The structural design calls for subgrades compacted to 95% Modified Proctor Density according to AASHTO T 180. The effective compaction depth should be 12 inches minimum. This compaction requirement will prevent efficient infiltration of water through the subgrade and thus will likely necessitate a piping design to handle the stormwater that accumulates in the storage aggregate (unbound subbase).

The engineer should provide a geotextile between the subgrade and the storage aggregate (subbase) as a means of preventing mixing of the materials. The geotextile should comply with Iowa DOT Section 4196 for subsurface drainage.

C. Hydrologic Design

The design process follows traditional storm sewer procedures for pavements. The initial step in the hydrologic design is the determination of the design storm event. Some agencies may establish the storm return period and the rainfall intensity. Information on intensity-duration-frequency for various return periods can be found in Chapter 2. In addition, the contributing area must be determined. The runoff volume should be determined according to the methods described in Chapter 2 using a design rainfall depth of 1.25 inches as a minimum, unless the jurisdiction has a different policy.

The next step involves establishing the drainage area. The storm event is then applied to the drainage area and the volume of runoff is determined.

The permeability of the subgrade soil is a critical design element. If the subgrade soil permeability is less than 1/2 inch per hour, a subdrain piping network will be needed. Soil compaction to support vehicular traffic will decrease permeability. Good design practice for vehicular traffic loads is to provide a minimum CBR of 5. Thus as the soil permeability is determined it should be assessed at

the density required to realize a CBR of 5 under soaked conditions.

To maximize the effectiveness of the PIP, the pavement grade should be as flat as possible, although steeper grades can be used. The general guideline is that the longitudinal grade should be greater than 1% and less than 12%. Three design alternatives exist for the PIP. They are:

- Full infiltration: All of the stormwater runoff from the design storm is infiltrated into the subgrade soils. See Figure 5L-1.02.A.
- Partial exfiltration: Some of the design storm runoff is infiltrated and the remainder is collected in the subdrain system and slowly discharged into the downstream systems. This is accomplished by setting the subdrain pipe above the top of the subgrade. See Figure 5L-1.02.B.
- Full Exfiltration: Soil permeability is limited and thus all of the runoff volume is carried away through the subdrain piping. See Figure 5L-1.02.C.

Designers must also evaluate and provide for larger storm events. One way to provide for the larger storms but still provide for infiltration of the water quality storms is to raise the elevation of the intakes above the pavers so the small storms are infiltrated and the large storms are handled by the intakes and pipe network.

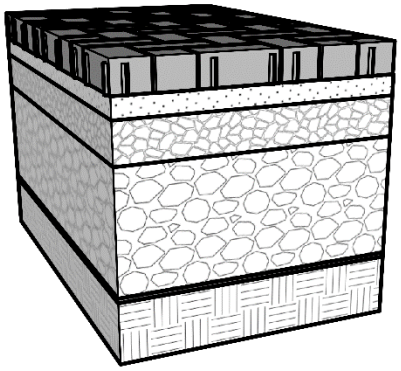
Once the volume of runoff and the soil permeability are known, the thickness of the storage aggregate layer (Iowa DOT Gradation No. 13/ASTM Gradation No. 2) can be determined. The void space (volume of voids/volume of aggregate) for Iowa DOT Gradation No. 13 is 40%. A 40% void space provides 0.4 cubic feet of stormwater storage for each cubic foot of aggregate. Thus, the volume of the storage aggregate will need to be 2.5 times the volume of water to be stored.

Due to the need to compact the subgrade soil to handle vehicles, it is very likely that subdrains will be needed to discharge at least a portion of the runoff. The elevation and sizing of the subdrains should be set to provide for full discharge of the design storm within 72 hours either through infiltration into the subgrade soil or through subdrain pipe discharge.

In order to prevent absorption of the bedding stone into the storage aggregate layer, a layer of filter aggregate (Iowa DOT Gradation No. 3/ ASTM Gradation No. 57) is needed. This layer is typically 4 inches thick. The bedding aggregate (Iowa DOT Gradation No. 29/ASTM Gradation No. 8) is then placed 2 inches thick, compacted, and leveled. Fine graded sand should not be used as the bedding and for filling of voids due to the increased clogging potential.

The pavers are placed, additional bedding stone is added to fill the voids in between the pavers, the area is swept, and finally the pavers are compacted. Sweeping prior to compaction is important to prevent stones on the surface from marring or cracking the pavers. That process may need to be repeated to entirely fill the voids. The final step is to sweep and remove any excess void filler stone.

Figure 5L-1.02: Permeable Interlocking Paver Design Alternatives



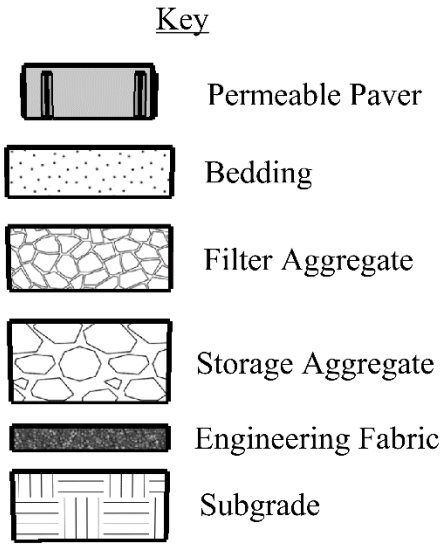


Figure 2.A
Full Infiltration
(Requires soils with infiltration rates greater than 0.5" / hour)

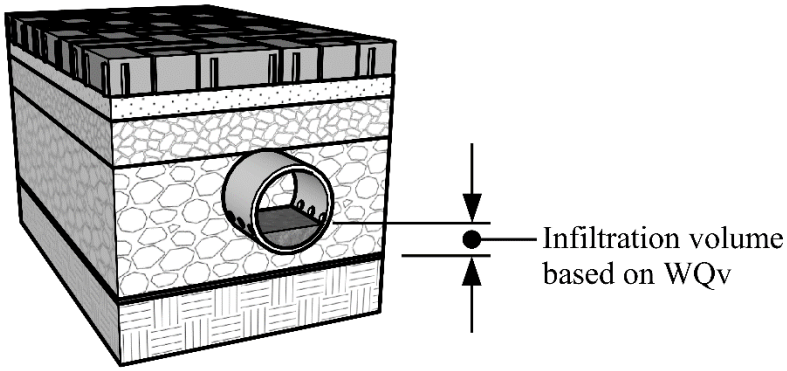


Figure 2.B
Partial Infiltration

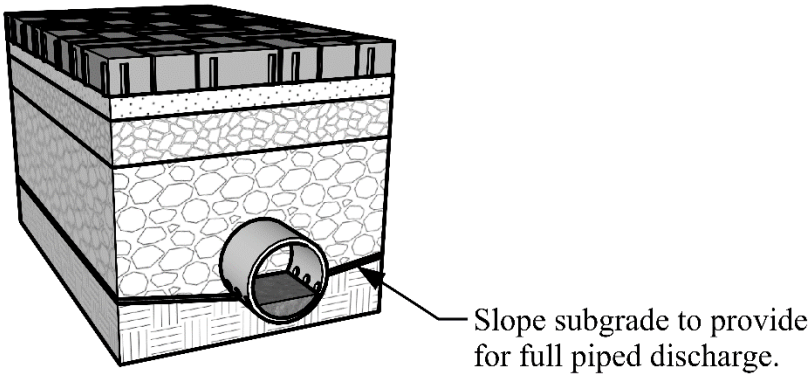


Figure 2.C
Full Exfiltration

D. Construction Elements

Monitoring and controlling the construction activities of a permeable interlocking paver project are critical to the long-term performance of the permeable pavement. Preventing and diverting sediment from entering the aggregates and pavement during construction must be of the highest priority. Aggregate stockpiles must be isolated to prevent contamination by sediment. Erosion and sediment control devices must be placed and maintained throughout the project until vegetation is fully established. All unnecessary vehicle and pedestrian traffic should be restricted once the aggregate placement has initiated. It may be necessary to wash vehicle and equipment tires to prevent tracking dirt and mud onto the aggregate layers.

A test section (approximately 5 feet by 5 feet) should be constructed to provide a basis for construction monitoring. The test section should be placed on the prepared subgrade to illustrate the processes used to place the pavers and illustrate the paver pattern and the edge details.

Restrict all equipment and workers from the paver placement area once the bedding stone has been placed, leveled, and compacted. Pavers may be placed by hand or mechanically. Placement should proceed from one end or side and continue work from the completed placement areas. An important consideration with mechanically placed pavers for large projects is to ensure the wear on the paver molds does not change the size of the pavers and thus impact the ability to correctly place the pavers.

E. Maintenance

As with any pavement, particularly permeable pavements, specific maintenance activities are necessary to achieve the design life of the pavement. PIP can become clogged with sediment that affects its infiltration rate. The rate of sedimentation can depend on the number and type of vehicles using the pavement, as well as the control of erosive soils adjacent to the pavement. The most important element of maintenance is keeping the sediment out of the pavement by vacuum sweeping. Regular vacuum street sweeping will maintain a high infiltration rate and keep out vegetation. Calibration of the vacuum force may be necessary to remove the sediment but minimize removal of the filler material from the joints. Over time, it may be necessary to add additional joint filler material to prevent intrusion by sediment.

Winter maintenance involves plowing snow and applications of de-icing chemicals. Although not required, snowplows can be equipped with rubber edged blades to minimize chipping of the pavers. Use of de-icing chemicals is often not necessary because the PIP remains warmer throughout the winter. Sand should not be used as an abrasive for traction. The sand will clog the filler material in the pavement joints.

Complete Streets

A. Background

Design professionals face an increasingly complex set of competing demands in development and delivery of street projects involving public rights-of-way. Designing a safe facility, completing construction, and installing various traffic control measures are only a part of a much larger picture. Street projects today also need to meet the objectives of regulatory, policy, and community requirements aimed at integrating the roadway into the existing natural and built environments. Among the many factors influencing the planning, design, and operation of today's streets are concerns about minimizing transportation costs; improving public health, creating and maintaining vibrant neighborhoods; accommodating the needs of the young, the physically challenged, as well as an aging population; and adopting greener and more sustainable lifestyles.

In the past, street design was focused on the need to move motor vehicles. The number and width of lanes was determined based on future projected traffic volumes or a set of standards based on the functional classification of the street. The functional classification and the adjacent land use also determined the general operating speed that was to be used for the design. Integration of facilities for pedestrians and bicyclists was not always a high priority. Some observers claim if you do not design for all modes of travel, then you preclude them.

Citizens within some cities are asking agencies to change the way they look at streets and the street function within each community. These agencies are looking to make their streets more "complete." Complete streets are designed and operated to enable safe access to all motorists, pedestrians, bicyclists, and transit users, regardless of age and ability. According to the National Complete Streets Coalition, there are in excess of 600 agencies that have adopted some form of a complete streets policy. Nineteen Iowa agencies, both small communities and larger cities, have adopted complete streets policies. Many other Iowa communities are looking into the concepts of complete streets. Complete streets also complement the principles of context sensitive design by ensuring that streets are sensitive to the needs of all users for the land use within the area. Proponents of complete streets note that by rethinking the design to include all users, the "balance of power" is altered by indicating that streets have many purposes and are not exclusively for motor vehicle traffic. The objectives of the complete streets philosophy are met by slowing vehicles down and providing better facilities for transit, pedestrians, and bicyclists. It is important to understand that safe and convenient walking and bicycling facilities may look different depending on the context. Appropriate facilities in a rural area will be different from facilities in a dense urban area.

There is no one size fits all design for complete streets. While the ultimate design goal for a complete street is a street that is safe and convenient for all users, every design should take into account a number of factors, some of which may be in conflict with each other. The factors include such elements as:

- Number and types of users - vehicles, trucks, transit buses, pedestrians, bicyclists
- Available right-of-way
- Existing improvements
- Land use
- Available budget
- Parking needs
- Community desires

In larger communities where the traffic volumes are heavy and land use density is greater, all of the above elements may be factors to consider. However, in smaller communities with lower traffic volumes and less dense developments, only a few may be important. The application of complete streets principles is most effective when neighborhoods are compact, complete, and connected to encourage walking and biking comfortable distances to everyday destinations such as work, schools, and retail shops. Past land use practices of large tracts for single use development are less effective in encouraging short walking or biking trips.

Complete streets are designed to respect the context of their location. For example, downtown locations may involve greater emphasis on pedestrians, bicyclists, and transit users than single family neighborhoods. Additionally context includes social and demographic factors that influences who is likely to use the street. For example, low income families and those without their own vehicle have the need for an interconnected pedestrian, bicycle, and transit network serving important destinations in the community.

The U.S. DOT adopted a policy statement regarding bicycle and pedestrian accommodations in March of 2010. It states:

> "The U.S. DOT policy is to incorporate safe and convenient walking and bicycling facilities into transportation projects. Every transportation agency has the responsibility to improve conditions and opportunities for walking and bicycling and to integrate walking and biking into their transportation systems. Because of the numerous individual and community benefits that walking and bicycling provide – including health, safety, environmental, transportation, and quality of life – transportation agencies are encouraged to go beyond minimum standards to provide safe and convenient facilities for these modes."

In addition to the U.S. DOT policy, members from the U.S. House of Representatives and the U.S. Senate have introduced a bill entitled “Safe Streets Act of 2014” that calls for all state DOTs and TMAs/MPOs to adopt a complete streets policy for all federally funded projects.

B. Design Guidance

There are a myriad of ways to address the development of complete streets in terms of a planning function, but there are not specific complete streets design elements identified for engineers to use to develop construction or reconstruction projects. The concept of complete streets goes beyond safety, tying in issues of health, livability, economic development, sustainability, and aesthetics.

Applying flexibility in street design to address the complete streets philosophy requires an understanding of each street’s functional basis. It also requires understanding how adding, altering, or eliminating any design element will impact different users. For instance, large radii may make it easier for trucks to navigate the street, but they create wider streets for pedestrians to cross. Designers of complete streets should understand the relationship between each criterion and its impact on the safety and mobility of all users.

Various manuals are available to provide design guidance including:

- AASHTO’s A Policy on Geometric Design of Highways and Streets (the Green Book)
- The Manual on Uniform Traffic Control Devices (MUTCD)
- The Highway Capacity Manual (HCM)
- AASHTO Guide for the Development of Bicycle Facilities
- ITE Traffic Engineering Manual
- NFPA Fire Code
- Local design ordinances
- The Access Board’s PROWAG

Some elements within these manuals are specific standards and some are guidelines with ranges of acceptable values. The MUTCD has been adopted as law; therefore the standards within it need to be met. In addition, there may be different standards for facilities that are under the Iowa DOT's jurisdiction than those for local control. If federal or state funding is being used to assist in a project's financing, the standards may be different yet. Local jurisdictions utilize the above manuals for design as a means of protection from lawsuits. Thus from a liability standpoint, it is very important that the design guidance meet the standards or fall within the range of acceptable guidelines provided by the above manuals.

As always, functional classification, traffic volumes, and level of service are factors to consider in any street design, and may be the highest priority for certain facilities. Through stakeholder input, it is important to identify the core issues, develop a spectrum of alternatives, and reach a design decision considering the needs of all of the users. The project development process may determine vehicular level of service is not the critical element and improved service for the other travel modes for pedestrians, bicyclists, and transit users is equal or more important.

C. Design Elements

If a complete streets design is contemplated, many elements must be determined during the design process. Traditionally designers have focused on those related to motor vehicles. With a complete streets design, other elements are also addressed. Each of those elements will be discussed and design guidance presented.

1. **Land Use:** The type of adjacent land use provides insight into several factors. For instance, in industrial areas, the expectation is that truck volumes will be higher. Also in commercial/retail areas, there is an expectation that pedestrians, transit, and bicyclists will have a greater impact. In residential land use areas, the street and right-of-way should accommodate pedestrians of all ages and abilities, and shared use of the street by motorists and bicyclists should be expected.

   Land use will influence speed, curb radii, lane width, on-street parking, transit stops, sidewalks, and bicycle facilities.

2. **Functional Classification:** Most jurisdictions classify their streets as a means of identifying how they serve traffic. Streets are generally classified as arterial, collector, or local facilities. Complete streets projects must take into consideration each street classification because it helps determine how the street and network needs to be treated to handle traffic volumes and other conflicts that may arise if design changes are made.

   Street classifications and the functions of each type are explained in detail in Section 5B-1. It is important to note that all jurisdictions, regardless of size have at least one street in each category. That means that in a larger community an arterial street may carry 20,000 vehicles per day, but in a smaller city the volume on their arterial street might be 2,000 vehicles per day. Similar differences exist in the collector classifications. Generally arterial streets are designated because their primary purpose is to move traffic. Collectors serve the traffic mobility function, but also provide access to adjacent property. Local streets are primarily there to serve adjacent property and should not have through traffic. Designs appropriate for low density residential areas are not likely to fit in the downtown commercial areas due to the likelihood of more pedestrians, bicyclists, trucks, and buses.

3. **Speed:** Because of the differences from community to community in functional classifications, a better criteria to use for design is speed. There are two types of speed to consider in design. The first is operating speed and the other is design speed. Operating speed is typically the posted speed limit and the design speed is often set at 5 miles per hour greater as a factor of safety. It is

also permissible to set the design speed and the posted speed the same. The design speed determines various geometric requirements for safe operations at that speed. These include stopping sight distance, passing sight distance, intersection sight distance, and horizontal and vertical curve elements. These standards are from the AASHTO Green Book and are outlined in Tables 5C-1.01 and 5C-1.02 and for liability reasons should be met at all times, especially for new streets. If it is not possible for any design element to meet the geometric standards on existing streets, warning signs and other safety treatments must be used.

It has been past practice to set the design speed at the highest level that will meet the safety and mobility needs of motor vehicles using the street. One of the principles of complete streets provides for slowing vehicles down to improve safety for all users, especially pedestrians and bicyclists. In general, the maximum speed chosen for design should reflect the network needs and the adjacent land use. The speed limit should not be artificially set low to accomplish complete streets objectives if the roadway environment does not create the driver expectation that they should slow down.

The maximum speed for arterial streets should be 45 miles per hour (mph), but only in rural sections or situations where access control is established and free flowing traffic is the normal situation. A maximum of 35 mph is more typical for most arterial streets in urban developed areas.

Collector streets serve both a mobility and property access function and thus the maximum speed is generally 30 mph. In some cases, 35 mph could be used but only when property access is very limited.

Local streets should be designed at 25 mph since their primary function is for property access.

4. **Design Vehicle:** The selection of the design vehicle is an important element in complete streets design. Lane width and curb radii are directly influenced by the design vehicle. It is not always practical to select the largest vehicle that may occasionally use a street as the design vehicle. In contrast, selection of a smaller vehicle if a street is regularly used by larger vehicles can invite serious operational and safety problems for all types of users.

   When selecting a design vehicle, the designer should consider the largest vehicle that will frequently use the street and must be accommodated without encroaching into opposing traffic lanes during turns. It is generally acceptable to have encroachment during turns into multiple same-direction lanes on the receiving street but not opposing lanes. The choice of a design vehicle is particularly important in intersection design where pedestrians, bicyclists, and vehicles routinely share the same space.

   All street designs must meet the minimum standards for fire departments and other emergency vehicle access and must consider the needs of garbage trucks and street cleaning equipment.

5. **Lane Width:** The AASHTO Green Book provides for lane widths from 9 to 12 feet wide. Narrower lanes force drivers to operate their vehicles closer to each other than they would normally desire. The drivers then slow down and potentially stagger themselves so they are not as close. The actual lane widths for any given street are subject to professional engineering judgment as well as applicable design standards and design criteria. The width of traffic lanes sends a specific message about the type of vehicles expected on the street, as well as indicating how fast drivers should travel. With painted lane lines being 4 to 6 inches wide, the actual "feel" to the driver will be about 1 foot narrower than the design lane width. Wider lanes are generally expected on arterial and collector streets due to truck traffic and higher operating speeds. Snow plowing and removal practices must also be considered as lane width decisions are being made,

especially for the curb lane. Narrower curb lane widths may necessitate different handling of snow because no space is available to plow the snow and it may require loading and removing on a more frequent basis.

It is preferred that arterial streets with 3 to 5% trucks or buses or operating speeds of 35 mph or greater have lanes that are 12 feet wide. That is especially important on the outside lane of multi-lane facilities. It is acceptable to have 11 foot wide lanes on arterial streets when speeds are 30 mph or less, but the entire street context, such as the presence of on-street parking, bike lanes, buffer areas, turn lanes, and volume of trucks and buses, needs to be considered before lane widths are chosen.

Collector streets can have 11 foot wide lanes if the number of trucks and buses is low. Collector street speeds should not exceed 35 mph.

Local commercial and industrial streets should be no narrower than 11 feet due to the larger volume of trucks expected with that land use. Local streets can have lane widths down to 10 foot wide in residential areas. For low volume local residential streets, two free flowing lanes are generally not required. This creates a yield situation when two vehicles meet.

The designer should recognize that there is an impact to the capacity of a street as the lanes are narrowed. According to the Highway Capacity Manual, capacity is lowered by 3% if lane widths are narrowed from 12 feet to 11 feet and 7% if lanes are narrowed to 10 feet.

6. **Curb Radii:** The curb radius of intersection corners impacts turning vehicles and pedestrian crossing distances. Larger radii allow larger vehicles, such as trucks and buses, to make turns without encroaching on opposing travel lanes or the sidewalk, but increase the crossing distance for pedestrians and allows smaller vehicles to turn at faster speeds. Shorter curb radii slow turning traffic and create shorter crossing distances, but make it difficult for larger vehicles to safely navigate the intersection. The curb radii that is chosen by the designer should reflect the number of pedestrians, the number of right turns by larger vehicles, length of the pedestrian crossing, and the width of intersecting streets.

   The curb radii must meet the AASHTO Green Book turning templates for the design vehicle selected. The curb radii may be modified if parking lanes and or bike lanes are present. It is acceptable to have encroachment into same-direction lanes on the receiving street. It is not acceptable to design a curb radius that calls for turning vehicles to encroach upon the opposing traffic lanes. The minimum curb radii in all cases should be 15 feet.

7. **Curb Extensions or Bump-outs:** Curb extensions or bump-outs are expansion of the curb line into the adjacent street. They are traditionally found at intersections where on-street parking exists, but may be located mid-block. Bump-outs narrow the street both physically and visually, slow turning vehicles, shorten pedestrian crossing distances, make pedestrians more visible to drivers, and provide space for street furniture. Use of curb extensions does not preclude the necessity to meet the turning radii needs of the selected design vehicle.

8. **Bicycle Facilities:** Bicycle facilities provide opportunities for a range of users and are a fundamental element of complete streets design. In Iowa, bicycles are legally considered a vehicle and thus have legal rights to use any street facility unless specifically prohibited. They also have legal responsibilities to obey all traffic regulations as a vehicle. Bicycle facilities generally are one of the following three types:
   a. **Shared Use Paths:** Separate travel ways for non-motorized uses. Bicycles, pedestrians, skaters, and others use these paths for commuting and recreation. Generally used by less experienced bicyclists.

b. **Shared Lanes:** These are lanes shared by vehicles and bikes without sufficient width or demand for separate bike lanes. They made be marked or unmarked. Low speed, low volume residential streets generally will not have pavement markings. For higher speed facilities, sharrow pavement markings and signage are used to remind drivers of the presence of bicyclists in the travel lane. Placing the sharrow markings between vehicle wheel tracks increases the life of the marking. These types of shares lanes are used more for commuting than recreation.
c. **Bike Lanes:** Dedicated lanes used on higher speed, higher volume streets separated from vehicle lanes or on-street parking spaces by pavement markings. No specific standards for when to use bike lanes exist, but conflicts between bikes and vehicles in shared lanes generally become problematic when vehicular volumes exceed 3,000 to 5,000 ADT and operating speeds are 30 mph or greater. Bicycle lanes should be a minimum of 5 feet wide on curbed pavements and 4 feet wide on rural cross-sections. If possible, a buffer zone of 3 feet should be provided between the bike lane and the on-street parking area to minimize conflicts with bikes and opening vehicle doors. These lanes are generally used by experienced bicyclists for commuting.

Snow and ice control activities impact vehicular lanes and bike lanes differently. Generally plows will leave some snow on the pavement. Vehicles are able to travel through this material but bicyclists may have more difficulty. In addition, the material may refreeze and make bike use more treacherous.

Design information for each bicycle facility type is detailed in Sections 12B -1 through 12B -3. Bicycle parking facilities at destination points will assist in encouraging bike usage.

9. **On-Street Parking:** On-street parking can be an important element for complete street design by calming traffic, providing a buffer for pedestrians if the sidewalk is at the back of curb, in addition to benefiting adjacent retail or residential properties. The width of parallel parking stalls can vary from 7 to 10 feet. Streets with higher traffic volumes and higher speeds should have wider parking spaces or a combination of parking space and buffer zone. Narrower parking spaces can be used if a 3 feet buffer zone is painted between the parking stall and a bike or traffic lane. The buffer zone will minimize exposure of doors opening into bicyclists, as well as facilitate faster access into and out of the parking space. Placement of parking stalls near intersections or mid-block crossings is critical so as to not impede sight lines of pedestrians entering crosswalks. Snow plowing could impact the availability of on-street parking intermittently. Requirements for ADA accessible on-street parking numbers and stall design must be adhered to. Information on those requirements can be found in Section 12A-2.

10. **Sidewalks:** Sidewalks are the one element of a complete street that is likely to provide a facility for all ages and abilities. Often sidewalks are the only way for young and older people alike to move throughout the community. Sidewalk connectivity is critical to encourage users. Sidewalks should be provided on both sides of all streets unless specific alternatives exist or safety is of concern. All sidewalks are required to meet ADA guidelines or be a part of a transition plan to be upgraded. Sections 12A-1 and 12A-2 identify the specific ADA requirements for sidewalks.
Sidewalks that are set back from the curb are safer than if the sidewalk is located at the back of curb. Street furniture and landscaping can add character and improve safety for sidewalks that are located at the back of curb. Providing seating areas within the sidewalk area can further enhance the urban environment and encourage pedestrian activity.

11. **Turn Lanes:** Turn lanes located at intersections provide opportunities for vehicles to exit the through lanes and improve capacity of the street. Two Way Left Turn Lanes (TWLTL) provide the opportunity to access midblock driveways without causing backups in the through lanes.

Turn lanes also allow faster speeds in the through lanes so a trade-off with safety exists especially at intersections.

Width of turn lanes should reflect the character of the traffic. Dedicated left and right turn lane widths should match the width of the lanes on the street. Local streets should not provide separate turn lanes. TWLTL should be a minimum of 12 feet wide because of the presence of through traffic on each side.

12. **Medians:** Medians provide for access management, pedestrian refuge, and additional space for landscaping, lighting, and utilities. Use of medians and the functions provided are dependent upon the width of available right-of-way and the other types of facilities that are included. The minimum width for pedestrian refuge is 6 feet. The minimum width of a median for access control and adjacent to left turn lanes is 4 feet. The minimum width for landscaped medians is 10 feet. Greater widths provide more opportunities for more extensive landscaping.

13. **Transit:** Bus service within the state is limited to the larger metropolitan areas. Currently there are a number of fixed route systems in the state. Smaller communities do not have fixed route service due to lack of demand. Children, elderly, and low-income people are the primary users of a fixed route transit system. In addition to system reliability, use of transit systems as a viable commuting option is directly dependent on the frequency of service and the destinations within the fixed route. To have a successful transit system, stops must be within walking or biking distance of residential areas to attract riders and it must have major retail, employment, and civic centers along its route system.

    Transit stops should be located on the far side of intersections to help reduce delays, minimize conflicts between buses and right turning vehicles, and encourage pedestrians to cross behind the bus where they are more visible to traffic. Far side stops also allow buses to take advantage of gaps in vehicular traffic.

    Bus turn out lanes are also best located on the far side of intersections. These turn outs free up the through lanes adjacent to the bus stop. Transit bulb outs are more pedestrian friendly than turnouts because they provide better visibility of the transit riders, as well as potentially providing space for bus shelters without creating congestion along the sidewalk. With buses stopping in the through lane, bulb-outs also provide traffic calming for the curb lane.

14. **Traffic Signals:** Traffic signals are not usually considered an element of complete streets, but they have many components with direct implications for complete streets. The timing, phasing, and coordination of traffic signals impacts all modes. Well-planned signal cycles reduce delay and unnecessary stops at intersections, thus improving traffic flow without street widening. Traffic signal timing can be designed to control vehicle operating speed along the street and to provide differing levels of protection for crossing pedestrians.

    The flashing don't walk pedestrian phase should be set using a 3.5 feet per second walking speed and the full pedestrian crossing time (walk/flashing don't walk) set using 3.0 feet per second. Some agencies representing the elderly are indicating that the overall walking speed should be 2.7 feet per second to cover a larger portion of the elderly population. ADA accessible pedestrian signal elements, such as audible signal indications, should be included in all new pedestrian signal installations and any installations being upgraded. See Section 13D-1, F for more information on accessible pedestrian signals.

15. **Summary:** The table below summarizes some of the critical design elements that should be examined if a complete streets project is implemented. Other geometric elements can be found in Table 5C-1.02. Some of the lane width values shown in the table below differ from the

acceptable values from Section 5C-1 because the expectation is that the complete street environment includes the potential for on-street parking and/or bike lanes. Adjustments in the values may be necessary to accommodate large volumes of trucks or buses. Contact the Jurisdictional Engineer if design exceptions are being considered.

Table 5M-1.01: Preferred Design Elements for Complete Streets

Classification	**Local**				**Collector**						**Arterial**					
Posted Speed (mph)	25		30		25		30		35 and Up		25		30		35 and Up	
Land use[1]	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*	*Res.*	*C/I*
Travel lane width (ft)[2]	10[3]	11	10	11	11	11	11	11	11	12	11	11	11	12	12	12
Turn lane width (ft)	--	--	--	--	11	11	11	11	11	12	11	11	11	12	12	12
Two-way left-turn lanes width (ft)	--	--	--	--	12	12	12	12	12	12	12	12	12	12	12	12
Curb Offset (ft)[4]	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	2	2	2	2	2	2
Parallel parking width (no buffer) (ft)[5]	8	8	8	8	8	9	8	9	9	9	10	10	10	10	10	10
Curb radii (ft)[6]	15	15	15	15	15	25	15	25	25	30	15	25	15	25	25	30
Bike lane width (ft)[7]	--	--	--	--	5	5	5	5	5	5	5	5	5	5	5	5

[1] Res. = Residential, C/I = Commercial/Industrial
[2] Minimum sharrow lane width is 13 feet.
[3] For low volume residential streets, two free flowing lanes are not required. They can operate as yield streets if parking is allowed on both sides and vehicles are parked across from each other.
[4] Curb offset, less the width of the curb, may be used in the parallel parking lane width.
[5] For arterial or high speed collectors, the parallel parking stall width may be reduced if a minimum 3 feet wide buffer strip is included.
[6] Curb radii may be adjusted based on design vehicle, presence of bike lanes or parking lanes, and the number of receiving lanes. Encroachment of turning vehicles into opposing lanes is not allowed.
[7] If paving is integral without a longitudinal gutter joint, the curb offset, less the width of the curb, may be used as part of a bike lane.

D. Traffic Calming

Traffic calming is different from but related to complete streets philosophies. Through design measures, traffic calming aims to slow traffic down to a desired speed. By slowing vehicular traffic, biking and pedestrian activities are made safer.

It is absolutely critical that traffic calming measures recognize the need to maintain access for emergency vehicles. Unless the situation is unusual, realizing slower speeds involves a series of traffic calming measures. However, too many measures along a street is likely to divert vehicles to adjacent streets and just move the problem or frustrate drivers to the point of complaining to the level necessary for removal of the traffic calming measures. Because of the anticipation that traffic will be just displaced to adjacent streets, it is very important to study a larger area than a single street when evaluating traffic calming measures.

Many design elements will accomplish traffic calming. These include the following.

- Reduction in lane widths:
  - Short medians
  - Bulb outs
  - Lane striping
- Lateral shifts
  - Chicanes
- Raised/tabled intersections
- Raised/tabled cross walks
- Speed humps or speed cushions
- Traffic circles
- Radar speed signs

Choosing the design elements to use for a particular area will depend on the neighborhood context and the specific concern to be addressed. Prior to evaluating alternative measures, stakeholders must be educated so they can have meaningful involvement. The evaluation needs to involve all stakeholders in the definition of the problem. If possible, all stakeholders, including drivers, pedestrians, bicyclists, and area property owners, would achieve some level of agreement on the traffic calming plan prior to implementation.

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