CHAPTER 10

BRIDGES

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10.1 Introduction

This chapter provides information for the planning and hydraulic design of highway bridges. The methodology is intended for those with an understanding of basic hydrologic and hydraulic analysis methods and some experience in the design of hydraulic structures.

Bridges are often the most expensive and complex highway structures, and considerable diligence and care are used in the hydraulic aspects of their design. These hydraulic aspects include, but are not limited to:

- changes in water surface profiles during floods due to the presence of the bridge,
- the passage of streambed material, ice, or debris,
- the potential for scour and erosion around the bridge foundation,
- clearance under the bridge deck for navigation, and
- drainage of stormwater runoff from the bridge deck.

The preceding aspects of bridge hydraulics are used in many activities, such as planning, location, design, and maintenance. In the case of moveable bridges, these aspects are also considered in bridge operation. This chapter addresses the first four of these aspects. The last aspect, bridge deck drainage, is covered in **Chapter 13**.

10.2 Policy

General ODOT policies that pertain to highway facilities such as bridges are listed in **Chapter 3**.

10.3 Bridges, Bridge Components, and Dimension Estimates

Occasionally the hydraulic designer will need to estimate the type, size, and location of a small bridge. In addition, the hydraulic designer often recommends the size or the shape of the waterway opening. To do either of these tasks, the designer should be familiar with structure types, the comparative costs of commonly used structures, their components, their advantages and limitations, and methods of estimating their dimensions.

There are many types of bridges, as shown in Figure 10-1. They range in complexity from simple single-span structures to complex multiple span girder bridges or trusses. The majority of highway bridges over water are relatively short structures such as open-bottom culverts, single or multiple span prestressed slab bridges, or single or multiple span prestressed box beam bridges. Methods of estimating the dimensions of these smaller structures are discussed in detail in this section. A detailed discussion of the more complex structures is beyond the scope of this chapter.

10.3.1 Single Versus Multiple Span Bridges

Many considerations govern the selection of a structure, such as hydraulic performance, foundation conditions, cost, aesthetics, etc. One of the most important requirements is the need to span an obstruction. From a hydraulic standpoint, this obstruction could be a waterway that has to be kept clear for navigation, the passage of floodwaters, or a number of other concerns.

In general, bridges use two means to cross obstructions. One means is to span the obstruction entirely with a single span. The other method is to use two or more shorter spans with bents between the spans. An advantage of the single-span method is the lack of interior bents and their associated costs. This cost advantage is offset to some degree by the added expense of the longer span because the cost of a span is often proportional to its length. Another advantage of a single-span is to eliminate any scour or debris related problems associated with piers. The advantage of the multi-span method can be a reduction in span costs, but this is offset by the expense of the interior bents. The hydraulic designer will often examine both of these alternatives at a site, if they are both viable.

10.3.2 Comparative Costs

Although many types of structures can be used at most crossings, only a limited number of types are cost-effective. In order to help in the selection of alternatives for hydraulic analysis, the following table has been adapted from the ODOT Bridge Section *Bridge Design and Drafting Manual*, the ODOT Bridge Standard Drawings, and trade literature. It lists common structure types in order of increased costs, and it also lists the ranges of typical span lengths. These are broad guidelines, and they are recommended for selecting alternatives for further study, only. The selection of a crossing type is usually made by the structural designer after a thorough study of hydraulic, aesthetic, geotechnical, environmental, and other considerations.

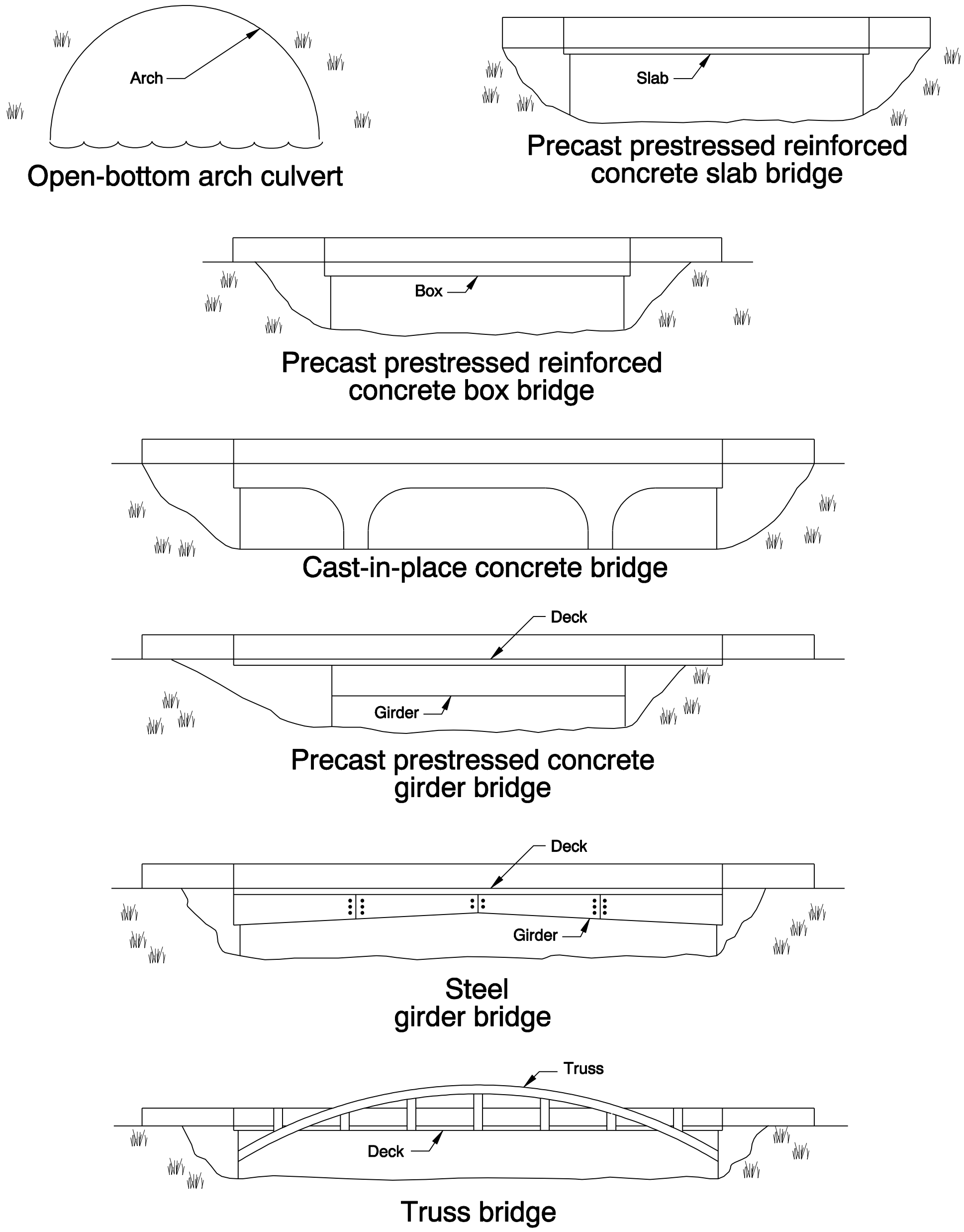


Figure 10-1 Structure Types

Table 10-1 Comparative Structure Costs
(Structures are listed in order of increasing cost.)

Structure Type	Span Lengths
Open-bottom culvert*	6 to 30 feet
3-sided reinforced concrete rigid frames	12 to 42 feet
Precast prestressed concrete slab	15 to 83 feet
Precast prestressed concrete box	54 to 116 feet
Cast-in-place concrete slab	up to 50-66-50 feet (3-span bridge)
Precast prestressed integral deck concrete girder	up to 130 feet
Precast prestressed concrete girder	71 to 164 feet
Cast-in-place box girder	**
Cast-in-place post-tensioned box girder	**
Steel girder or truss	**

* Open-bottom culverts can cost more than slab span bridges at many sites.
**Normally used for long single-span and longer multi-span bridges.

10.3.3 Open-Bottom Culverts

Open-bottom culverts are available in a wide range of span lengths from approximately 6 to 30 feet. These structures have characteristics of both bridges and culverts, and they are discussed in detail in Chapter 9. They are also discussed in this chapter because scour depth estimates, and in some cases, hydraulic performance analyses, are done using the same methods as bridges. Like bridges, they:

- have a waterway opening with a bottom composed of natural streambed materials,
- have footings that must be protected from scour, and
- in the larger sizes, have hydraulic characteristics more similar to bridges than culverts.

In a manner similar to culverts, they

- do not have a deck (the roadway crosses over the culvert on a layer of earth and/or aggregate fill), and
- in the smaller sizes, have hydraulic characteristics more similar to culverts than bridges.

The hydraulic designer should pay particular attention to the following when locating or performing a hydraulic study on an open-bottom culvert:

Foundations - Open-bottom culverts are almost always supported by spread footings, as shown in Figure 10-2. The bottoms of the footings are placed below the predicted scour elevation unless they are keyed into non-erodible rock. Sometimes it is necessary to locate the footings considerably deeper than the elevation of the bottom of an arch if the predicted scour is fairly deep. Stemwalls are used in these instances to connect the arch to the footings, as shown in Figures 10-2a and 10-2b. Stemwalls can also be used to increase the waterway area by elevating the arch above the channel bottom. Stemwalls are not used with the precast 3-sided rigid frame box culverts shown in Figure 10-2c. These culverts rest directly on the footings.

Footings, stemwalls, and the requisite excavation can often be quite expensive, and experience has shown the cost of a scour resistant foundation is usually the critical factor in determining whether an open-bottom culvert or a bridge is the most cost-effective structure. In general, the open-bottom culvert is an economical structure where the footings can be protected from scour without an extensive and costly foundation. Usually this occurs when the footings can be keyed into non-erodible rock, and the rock is at or near the ground surface. The other application is a crossing over regulated waterway such as a canal where the scour depth is limited and predictable, and the footings can be placed below the predicted scour elevations with reasonable cost. This subject is discussed in more detail in **Chapter 9**.

Fill Heights - Open-bottom culverts, unlike bridges, usually have a layer of fill between the crown of the culvert and the roadway surface, as shown in Figure 10-2. The height of this fill is critical, and it must be thick enough to distribute the live load and prevent the culvert from being distorted, or possibly failing, due to the pressures exerted by traffic. At the same time, the fill height must not be excessive, so as to prevent structural failure of the culvert due to the weight of the surrounding fill. The ranges of allowable fill heights are limited for many types of arches, in particular metal open-bottom arches. As a result, fill heights are often a critical factor in the decision about whether or not to use an arch and the selection of the shape of the arch. This subject is addressed in more detail in **Chapter 5** and **Chapter 9**.

Note: Rigid frames can be designed so traffic crosses over pavement on the top of the frame. This is not often done, and a special design is needed.

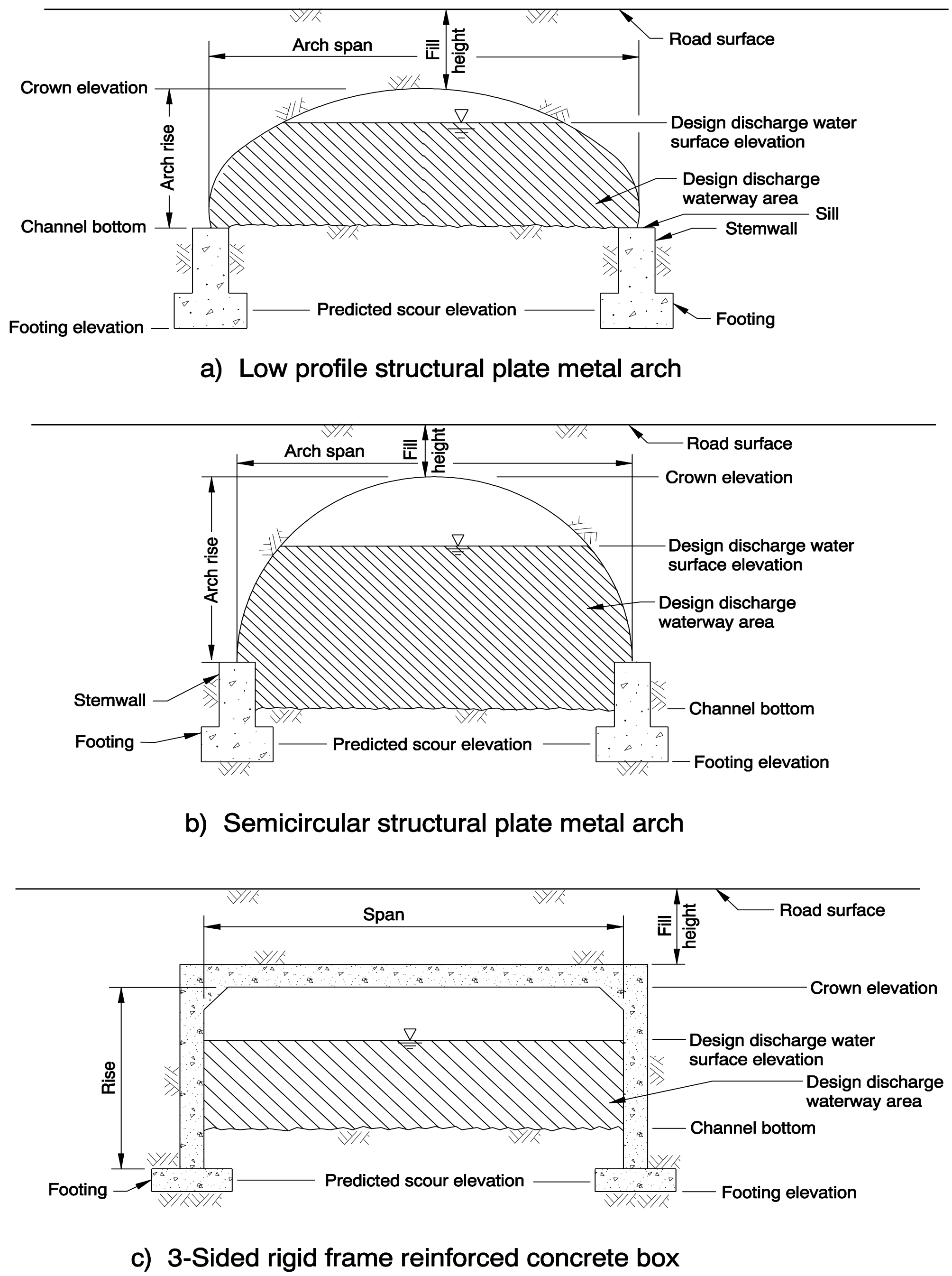


Figure 10-2 Open-Bottom Culverts

10.3.4 Precast Prestressed Slab or Box Beam Bridges

Structures with spans of precast prestressed slabs and boxes are the most common short and medium length highway bridges. These spans are self-supporting. In other words, they do not need the additional support from a superstructure such as a truss or girders. Precast prestressed slabs are used for single-span and multi-span structures with span lengths of approximately 15 to over 83 feet, and precast prestressed boxes can be used for spans with lengths from approximately 54 to 116 feet. Slabs and boxes built to ODOT Standard Plans can be used for spans with skew angles from 0 degree (an unskewed bridge) to a maximum skew angle of 45 degrees. Skew angles over 45 degrees can be made using special designs. These slabs and boxes are shown in Figure 10-3 and ODOT Standard Drawings BR400 through BR465.

A hydraulic performance advantage of these bridge spans is their shallow depth between the pavement surface and the bottom of the box or slab. These depths are considerably less than many of the other structure types, and this is an important asset when vertical clearance over the water or channel bottom is a critical factor. Another advantage of these spans is their relatively smooth under surface. This can be a benefit for structures that pass ice or large floating debris.

Waterway Opening Dimensions - The waterway opening dimensions of slab or box span bridges can be estimated using the following procedures. These estimates are usually adequate for the hydraulic modeling of the recommended alternatives. Later in the design process a recommended alternative will be selected. It will often have a waterway with different dimensions than the waterways of the alternatives described in the hydraulic study. If this occurs, and the proposed waterway is smaller than hydraulic report recommendations, it is recommended that the proposed waterway be modeled in order to verify that it has adequate hydraulic performance.

The clear span distance is the width of the structure opening, and it is the distance between the inside faces of the abutments. A single-span structure spans the entire clear span distance. Multi-span structures have one or more interior bents within the clear span distance, as shown in Figure 10-3a.

The span length of a slab or box is the distance between the centerlines of the bearing pads, as shown in Figure 10-3b. The centerline of each bearing pad is approximately 1.5 feet outside of the face of the end bent. As a result, the span length of the typical single-span structure is greater than the clear span distance. The span length for a given clear span distance can be estimated for a typical ODOT slab or box bridge by the following equation:

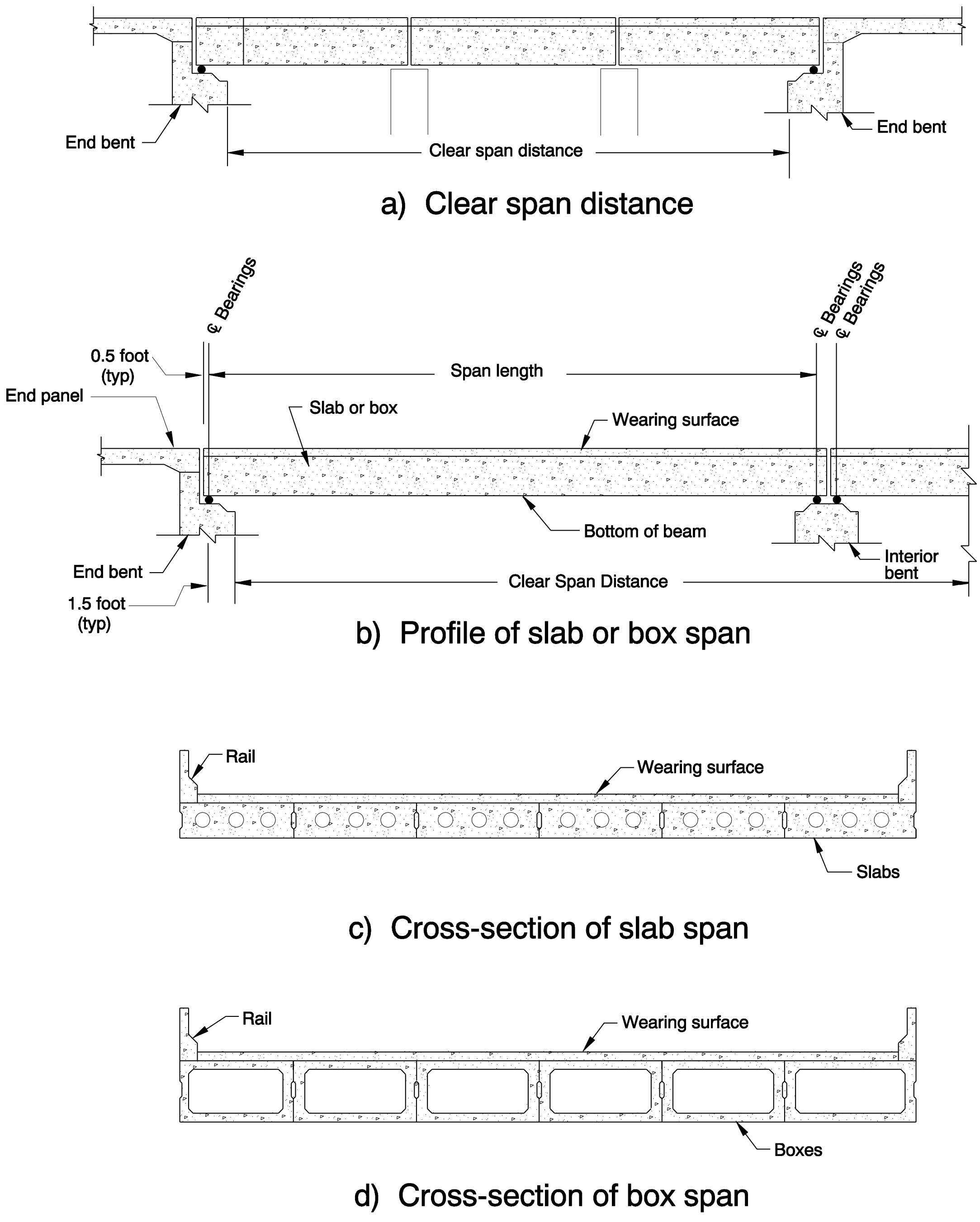


Figure 10-3 Prestressed Concrete Slab and Box Spans

$$\text{Span Length}_{\text{Single-Span}} = \text{Distance}_{\text{Clear Span}} + 3 \qquad \text{(Equation 10-1)}$$

Where:

$\text{Span Length}_{\text{Single-Span}}$	=	Estimated minimum length of a single span between bearing centers in feet
$\text{Distance}_{\text{Clear span}}$	=	Distance between the inner faces of the end bents in feet

The end of the span is 0.5 foot beyond the centerline of the bearing. It is assumed that both ends of the span abut each other at each interior bent for estimating purposes. Consequently, the minimum lengths of multiple spans that can cross a clear span distance is:

$$\text{Span Length}_{\text{Multi-Span}} = \frac{\left(\text{Distance}_{\text{Clear Span}} + 3\right) - [(1.0)(\#_{\text{interior bents}})]}{\#_{\text{Spans}}} \qquad \text{(Equation 10 - 2)}$$

Where:

$\text{Span Length}_{\text{Multi-Span}}$	=	Estimated length of each of the multiple spans between centers of bearing in feet
$\text{Distance}_{\text{Clear Span}}$	=	Distance between the inner faces of the end bents in feet
$\#_{\text{Interior Bents}}$	=	Number of interior bents
$\#_{\text{Spans}}$	=	Number of spans

The preceding equation assumes all spans have equal length. The designer will usually try to do this because it is most economical. Sometimes unequal span lengths are used to clear obstacles and for other purposes.

The span width must be sufficient to accommodate the roadway, bridge rails, and sidewalks, if used. Information on the roadway width, rail type, and sidewalk requirements can be obtained from the roadway or bridge designer. Typically the span width is a multiple of 4 feet because the individual slabs and boxes are this wide. The slabs and boxes do not fit together exactly, so it is customary to add an extra 0.042 foot (1/2 inch) of deck width for each slab or box used, as follows:

$$\text{Width}_{\text{Span}} = (4.042)\ (\#_{\text{Slabs or Boxes}}) \qquad \text{(Equation 10-3)}$$

Where:

$\text{Width}_{\text{Span}}$	=	Estimated total width of the span in feet
$\#_{\text{Slabs or Boxes}}$	=	Number of slabs or boxes in each span

The total depth of the span can be estimated by adding the thickness of the surfacing to the depth of the slab or box. The surfacing thickness can be provided by the bridge designer. If this information is not available and the bridge is on a road with a constant gradient, a thickness of 2 inches can be assumed at midspan and a thickness of 3 inches at the span ends. The surfacing is thicker at the ends of the span because the slab or box has a slight upward arch shape due to camber. If the bridge is on a vertical curve, this assumption cannot be made and the bridge designer should be contacted for an elevation estimate.

The depth of the slab and box can be estimated using the tables in Standard Drawings BR450 and BR460, respectively. The name of each table gives the span or box depth. For example, "SLAB 12" represents a slab with a depth of 12 inches in Drawing BR450, and "Box 33" represents a box with a depth of 33 inches in Drawing BR460. The following equation can be used:

$$Depth_{Total} = Depth_{Slab\ or\ Box} + Thickness_{Surfacing} \quad \text{(Equation 10-4)}$$

Where:

$Depth_{Total}$ = Estimated total depth of span in feet
$Depth_{Slab\ or\ Box}$ = Depth of slab or box in feet
$Thickness_{Surfacing}$ = Thickness of surfacing in feet

The elevations of the bottom of the slab or box can be estimated with either the centerline profile and cross-slope, or 3-line profile. This information can be provided by the bridge or roadway designer. In most cases, the roadway cross-slope will also be the cross-slopes of the upper and lower surfaces of the bridge deck. An exception occurs when the roadway has gutters. Bridge decks generally do not have gutters, and the cross-slope of the road and bridge may differ. The bridge designer should be contacted for assistance. The following formula can be used in most applications to determine the bottom of beam elevations:

$$EL_{BOB} = EL_{CL} + [(Distance)\ (Cross\text{-}slope)] - Depth_{Total} \quad \text{(Equation 10-5)}$$

Where:

EL_{BOB} = Estimated bottom-of-beam elevation in feet
EL_{CL} = Elevation of road centerline in feet
Distance = Distance from centerline to deck edge in feet
Cross-slope = Cross-slope of roadway or deck surface in feet per foot (This is a positive value if the edge of the deck is higher than the highway centerline, and it is a negative value if the edge of the deck is lower than the centerline.)
$Depth_{Total}$ = Depth of span including surfacing in feet (Use Equation 10-4)

10.3.4.1 Example - Estimating Bridge Span Dimensions

A slab or box bridge will be included in a hydraulic model. The clear span distance and roadway elevations are known. Estimates are needed of the deck width and the bottom of slab or box elevations.

The bridge is on a road with a constant 0.5 percent grade. It is also on a horizontal curve where the roadway is superelevated and it has a constant 0.025 foot per foot cross-slope across its entire width. The clear span distance must be at least 95 feet and the waterway opening is not skewed in relation to the road centerline. An interior bent can be located within that clear span distance, if needed. The road centerline elevations at the ends of the clear span distance are 95.26 and 95.73 feet. The total roadway width is 45 feet. The deck must accommodate this roadway and provide sufficient width for the bridge rails. A width of 1.33 feet is assumed for the rail. This is the width of the commonly used "Type F" bridge rail.

A single span would have to be longer than the 95-foot clear distance, as shown in Figure 10-4. Using Equation 10-1, the span length from centerline of bearing to centerline of bearing would be:

$$\text{Span Length}_{\text{Single-Span}} = 95.0 + 3.0 = 98.0 \text{ feet}$$

This distance can be spanned by a single reinforced concrete box. A two-span bridge would be needed if slabs are used. Using equation 10-2, the span lengths between the bearing centerlines of a two-span bridge would be:

$$\text{Span Length}_{\text{Multi-Span}} = \frac{(95.0 + 3.0) - [(1.0)(1)]}{2} = 48.5 \text{ feet}$$

The number of slabs or boxes can be calculated by dividing needed roadway and rail width by 4.042 as follows:

$$\frac{45 + 1.33 + 1.33}{4.042} = 11.8 \text{ slabs or boxes}$$

It could be assumed that 12 slabs or boxes would be adequate if the bridge was on a tangent. This bridge is located on a horizontal curve, and some added width will be needed to accommodate the curved roadway, as shown in Figure 10-4. The bridge and end panels were sketched to see how many extra slabs or boxes would be needed. One extra slab or box is sufficient, for a total of 13 slabs or boxes. The deck width is estimated by Equation 10-3 to be:

$$\text{Width}_{\text{Span}} = (4.042)\ (13) = 52.5 \text{ feet}$$

The depth of the box for the single-span bridge is estimated from Standard Drawing BR460 to be 4 feet (48 inches). The thickness of the surfacing is assumed to be 0.25 feet (3 inches) at the ends of the bridge, as discussed in the previous section. Using Equation 10-4, the total depth at the ends of a single span is:

$Depth_{Total}$ = 4.0 + 0.25 = 4.25 feet (for a single-span bridge)

Note: A 42-inch deep box can also be used. It is able to handle span lengths up to 102 feet. This box, however, is near its design length limit with a 95-foot span. The 48-inch deep box can be used for spans up to 116 feet. This depth is selected for estimating purposes because it is not near its design length limit and it has reserve capacity for more weight, such as a pavement overlay. The practice of selecting a box or slab depth providing extra capacity is recommended for waterway dimension estimating purposes.

The slab depth of the two-span bridge is estimated from Drawing BR450 to be 1.75 feet (21 inches). This depth is adequate for spans up to 55 feet long. Using Equation 10-4, the total depth of a span for a two-span bridge is:

$Depth_{Total}$ = 1.75 + 0.25 = 2.00 feet (for two spans)

The bottom-of-box or bottom-of-slab elevations are estimated for all four corners of each of the two bridges using Equation 10-5. The distances from the road centerline to the edges of the deck are scaled from the drawing in Figure 10-4. The elevation of the bottom of the box at the upstream left corner of the single-span bridge is determined by a typical calculation, as follows:

$EL_{BOB} = 95.26 + [(52.5/2)\ (0.025)] – 4.25 = 91.67$ feet

In general, the dimensions used in hydraulic modeling of bridge decks abutments and bents are:

- elevations rounded to the nearest 0.01 foot, and
- horizontal measurements to the nearest 0.1 foot.

The bridge opening dimensions, rounded according to the preceding guidelines, are:

- clear span distance = 95.0 feet, and
- span width = 52.5 feet

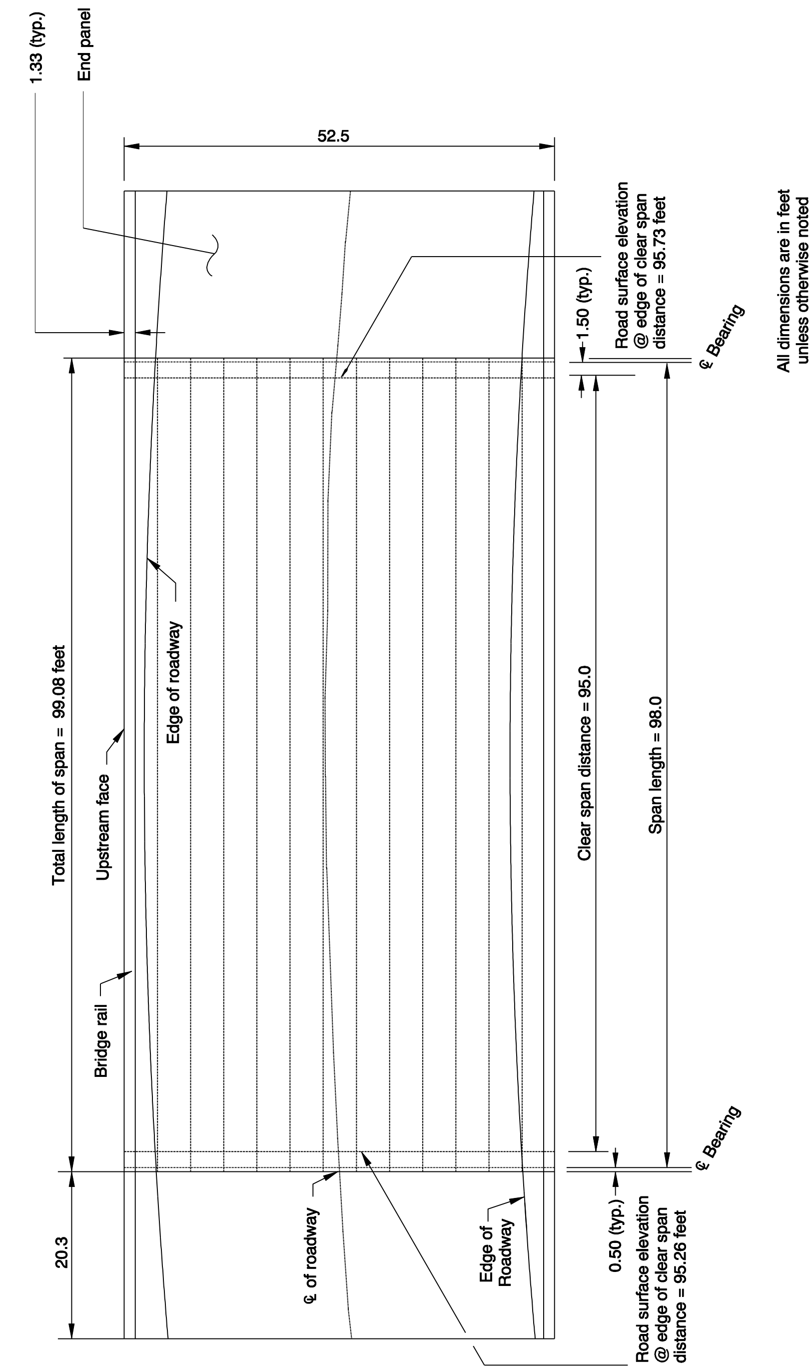


Figure 10-4 Plan View of Span in Example

The elevations of the bottom of the boxes or slabs at the corners of the end bents are:

Bridge	Centerline Elevation (feet)	Distance to Deck Edge (feet)	Total Cross-slope (foot per foot)	Span Depth (feet)	Corner Location	Corner Elevation (feet)
Single-span	95.26	26.3	+ 0.025	4.25	Upst. Left	91.67
	95.73	26.3	+ 0.025	4.25	Upst. Right	92.14
	95.26	26.3	- 0.025	4.25	Dnst. Left	90.35
	95.73	26.3	- 0.025	4.25	Dnst. Right	90.82
Two-span	95.26	26.3	+ 0.025	2.00	Upst. Left	93.92
	95.73	26.3	+ 0.025	2.00	Upst. Right	94.39
	95.26	26.3	- 0.025	2.00	Dnst. Left	92.60
	95.73	26.3	- 0.025	2.00	Dnst. Right	93.07

The opening dimensions of the upstream face of a single-span bridge are shown in Figure 10-5a, and the dimensions of the downstream face of a two-span bridge are shown in Figure 10-5b. The elevations and dimensions should be verified by the bridge designer.

The area obstructed by the Type “F” bridge rail is shown in Figure 10-5a. It is assumed the rail will be 2.67 feet tall and it will extend the entire length of the bridge, including end panels. The area obstructed by the rail must be included in the hydraulic model if roadway overtopping occurs. Typical rails are shown in the ODOT bridge and roadway standard drawings. An interior bent with a 3 foot width is shown in Figure 10-5b. This width is a commonly used estimate of the interior bent width for a slab or box span bridge.

10.3.5 Prestressed Beam, Cast-in-Place, Girder, Truss, and Other Bridges

Precast beam bridges, cast-in-place bridges, girder bridges, truss bridges and other types of relatively long or complex structures are used in many applications. These bridges are often designed for a specific site, and their dimensions cannot be estimated by the relatively simple procedures used for slab and box bridges. The structural designer can provide the necessary dimensions for the hydraulic modeling of these bridges. Additional information about these bridges is in publications such as the ODOT Bridge Design and Drafting Manual, the publications of the American Association of State Highway and Transportation Officials (AASHTO), and the many textbooks on the subject. Methods for estimating the dimensions of these structures are not included in this chapter.

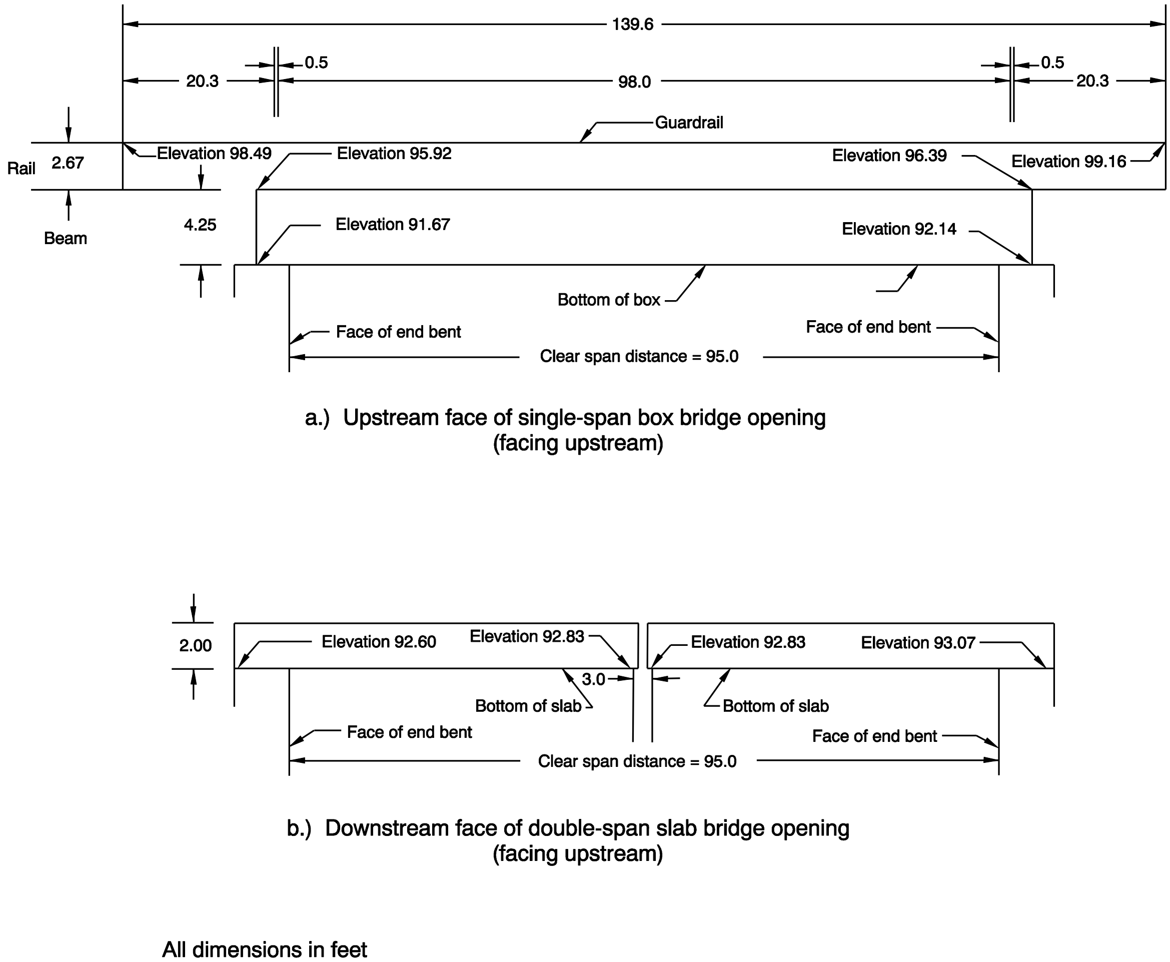


Figure 10-5 Estimated Waterway Opening Dimensions for Example

10.3.6 Interior Bents (Piers)

An interior bent is an intermediate support for a multi-span structure. Interior bents in the water are often called piers, and both terms are used interchangeably in this chapter. The hydraulic designer should be familiar with the different types of bents and their hydraulic characteristics. The types, sizes, and locations of the interior bents are usually determined by the structural designer after consultation with the geotechnical and hydraulic designers. Many factors are considered, such as structural needs, aesthetics, environmental concerns, seismic safety, costs, hydraulic characteristics, the potential for debris accumulation, and foundation requirements.

Many foundation types are used to support interior bents. Piling is often an economical foundation in softer soils. Drilled shafts are often used in rock or in soils that are too rocky to accommodate piling. Spread footings can be used on rock or on erodible materials in locations where the footings are protected from scour or founded below scour depth. The foundation type is usually recommended by the geotechnical designer after consultation with the structural and hydraulic designers.

There are many types of interior bents, and the more common types and their foundations are discussed in this chapter and shown in Figure 10-6. The dimensions of interior bents are difficult to estimate without experience in structural design, and it is recommended that a structural designer be consulted for the needed dimensions.

10.3.6.1 Single Column Bents

These bents are comprised of a single column, and they are often supported by piling with caps, drilled shafts, or spread footings, as shown in Figure 10-6a. These bents are sometimes called cantilever or hammerhead bents because of their shape.

Single column bents have hydraulic advantages. They are relatively easy to keep clear of debris. There is only one face that can catch and retain debris, and usually this face is accessible for cleaning. In addition, single column bents can sometimes be made with round or nearly round shapes, and scour depth is the same for all flow attack angles. In other words, the scour depths are not strongly influenced by the direction of the flow. This characteristic can make single column bents desirable where unskewed structures are used at skewed road/stream crossings, where the flow can attack the bent from varying angles, or where the attack angle is likely to change during the life of the structure.

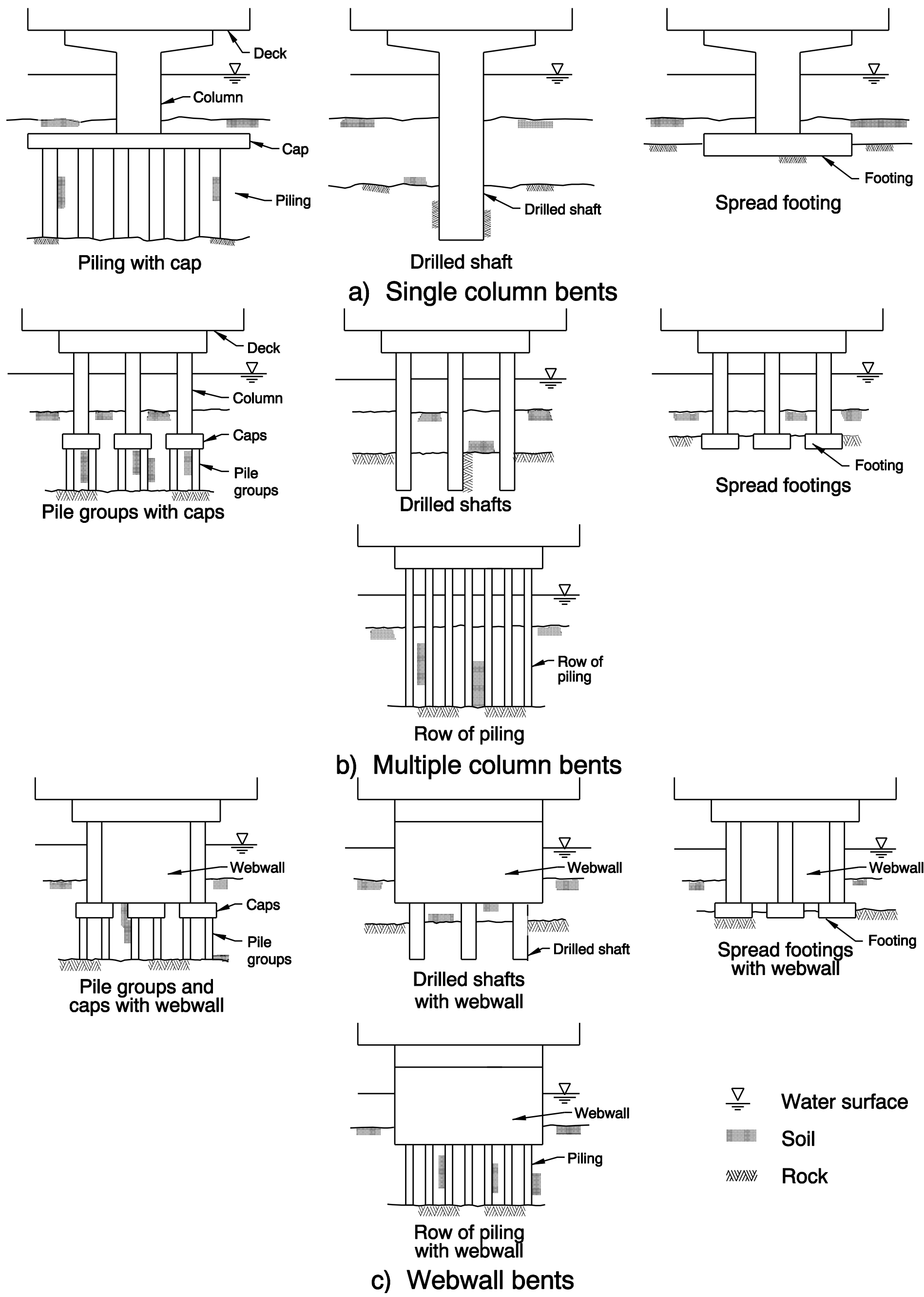


Figure 10-6 Interior Bents

A hydraulic disadvantage of single column bents can be their relatively deep scour depths. These depths are due to the relatively large width of the column, and they can be deeper than the scour depths associated with the thinner columns of multiple column bents.

10.3.6.2 Multiple Column Bents

These bents are comprised of a row of columns, and they can be supported by pile groups with caps, drilled shafts, spread footings, or rows of piles, as shown in Figure 10-6b.

Multiple column bents have several hydraulic advantages. These bents often have relatively shallow scour depths. Often the scour depths are proportional to the widths of the columns, and the smaller columns of multiple column bents often have shallower depths than the large columns of single column bents. Also, like single column bents, multiple column bents with widely spaced round or square columns often have similar scour depths for varying flow attack angles.

A hydraulic disadvantage of multiple column bents is the possibility that they can catch and retain debris between the columns. This debris can be relatively inaccessible and difficult to remove - especially during flood events.

10.3.6.3 Multiple Column Bents with Webwalls

These bents are comprised of columns with a connecting webwall between the columns. These bents are sometimes called "wall bents," and they are supported by the same foundations as multiple column bents without webwalls, as shown in Figure 10-6c.

The addition of a webwall to a multiple column bent has a considerable effect on its hydraulic characteristics. A bent with a webwall can have relatively shallow scour depths if the direction of the flow is parallel to the bent centerline. Scour depths can increase considerably if the flow attacks the bent at an angle. As a result, these bents perform best if they are aligned with the flow and the flow always comes from the same direction. Another advantage of these bents is their ability to pass debris. The webwalls prevent debris from lodging between the columns.

10.3.7 Abutments (End Bents)

Almost all bridges require support at the ends to retain the approach embankments and carry the vertical and horizontal loads from the superstructure. These supports are often called "abutments" or "end bents." "Abutment" is the most commonly used term in the field of hydraulic engineering, and "end bent" is often preferred by structural designers. Both terms are used in this chapter with the following distinctions. The term "abutment" applies to the entire supporting structure, including earth fill, the structural members, the foundation, and the revetment needed to provide scour and erosion protection. The term "end bent" is used to describe the load bearing structural members that support the span, such as the pilings, footings, drilled shafts, pilecaps, wingwalls, abutment walls, etc.

There are many different types of abutments, and the hydraulic designer should be familiar with their components, their advantages and disadvantages, and methods of estimating their dimensions. The choice of the type, size, and location of the abutment is made by the structural designer after consultation with the foundation and hydraulic designers. The more common abutments are discussed in this chapter.

10.3.7.1 Spillthrough Abutments

This abutment has a fill slope on the channel side of the end bent, as shown in Figure 10-7. This abutment is often used where the bent is supported by piles or drilled shafts, and in almost all cases, the face of the abutment is protected from scour or erosion by a layer of riprap. As a general rule, a spillthrough abutment is more cost-effective than a vertical abutment where the height of the vertical abutment is greater than 10 feet. There are three types of spillthrough abutments defined in the ODOT Bridge Design and Drafting Manual, Options A, B, and C. Cross-sections of these abutment options are shown in Figure 10-8.

Option A - This spillthrough abutment option does not have wingwalls to retain the fill around the end bent, as shown in Figure 10-8a. This is the abutment with the lowest cost because there are no wingwalls. This cost advantage is often offset by the added expense of the bridge span, because a longer structure is needed to provide an adequate waterway opening. Of the three options, this type has the smallest waterway opening area for any given span length.

Option B - This option uses short wingwalls to retain the embankment fill, as shown in Figure 10-8b. This option is often cost-effective because the wingwalls allow the use of a shorter span. This option is used more frequently than the other two. For this reason, it is recommended that it be used for the initial modeling of a spillthrough abutment bridge.

Option C - This option uses taller wingwalls and an extended pile cap to retain the embankment fill, as shown in Figure 10-8c. It is often a cost-effective abutment in some applications because it can allow the use of a shorter span. A disadvantage of this option is the greater longitudinal forces from lateral soil loads that must be resisted by the end bent. The added structural strength needed to resist these forces can add to the cost of the abutment. This option is often used if the additional cost of the abutment can be offset by a reduction in the cost of the span.

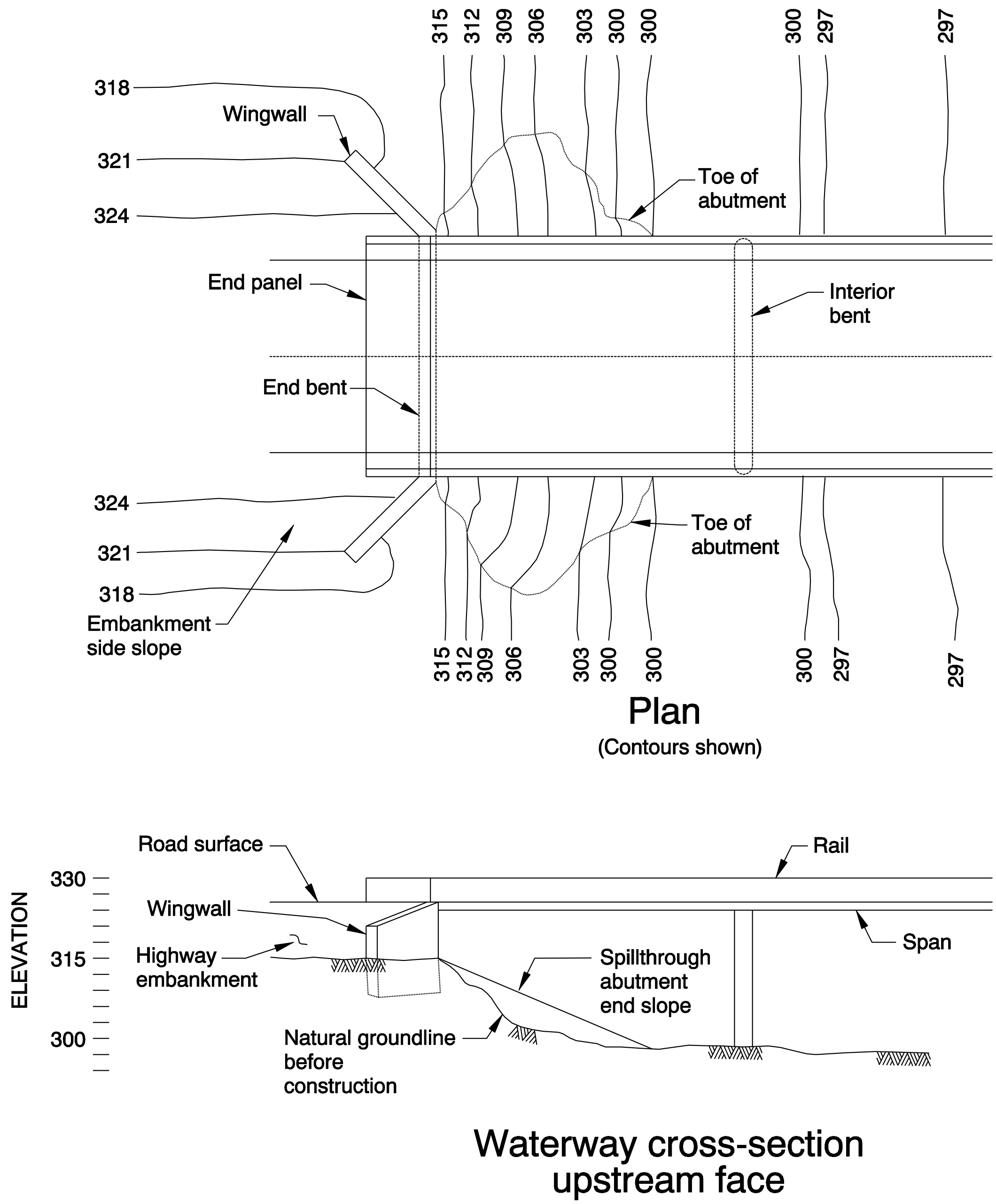


Figure 10-7 Spillthrough Abutment with Wingwalls

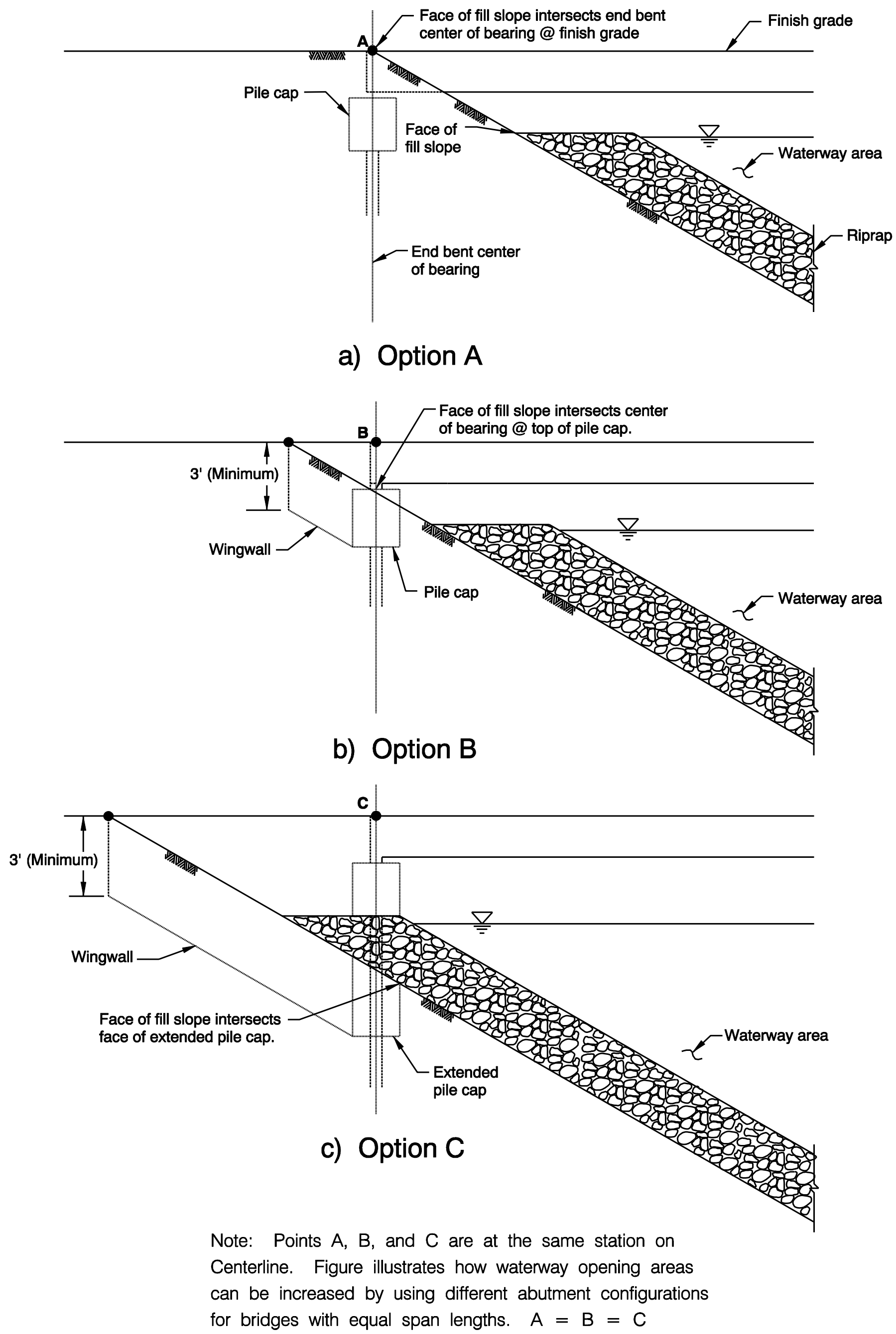


Figure 10-8 Spillthrough Abutment Options

Abutment Dimensions - The hydraulic modeling of a spillthrough abutment bridge waterway opening requires an estimate of its dimensions. The revetment protection on the abutment faces must be included in the hydraulic model if it obstructs a portion of the opening. Methods to estimate spillthrough abutment dimensions are included in this section. Methods to calculate revetment blanket thickness are in **Chapter 15**. Methods to estimate the clear span distance and the span dimensions are described previously in this subsection.

Abutment end slopes and embankment side slopes are shown in Figure 10-7. The end slope can be assumed to be 2 units of vertical distance to 1 unit of horizontal distance (1V:2H) for scoping estimates and the initial waterway opening hydraulic modeling. In almost all cases, the fill and revetment will be stable at this slope. Steeper end slopes are often desired to reduce structure costs and for other reasons. The end slope should not be steeper than 1V: 1-1/2H if it will be protected by loose riprap. The embankment side slope can be estimated for scoping and initial estimates from the guidelines in the ODOT Highway Design Manual. Regardless of the abutment or embankment slopes chosen, the stability of the fill and revetment should be verified in the geotechnical and hydraulic designs, respectively.

The location of the face of the abutment fill slope is needed for the hydraulic modeling of the structure. The face of the abutment fill slope in Option A can be approximated by extending a line downward from the end bent centerline of bearing on the top of the span, as shown in the Figure 10-8a. The face of the fill slope for Option B can be approximated by extending a line downward from the end bent centerline of bearing on the top of the pile cap, as shown in the Figure 10-8b. The fill slope for Option C can be approximated by extending a line downward from a point on the face of the pile cap, as shown in the Figure 10-8c.

10.3.7.2 Vertical Abutments

This abutment type differs from a spillthrough abutment because it does not have a fill slope on the channel side. The embankment fill is retained behind an abutment wall and wingwalls, as shown in Figure 10-9. These abutments are often supported by spread footings protected by loose riprap.

Abutment Dimensions - The dimensions of a vertical abutment waterway opening are relatively easy to estimate. The clear span distance can be estimated using Equation 10-1, and the faces of the abutment walls can be assumed to project vertically downward from the ends of the clear span. The wingwalls usually project out from the abutment wall at an angle. This angle is generally half the angle between the centerlines of the road and the stream, and the walls extend outward to the toes of the embankment fill, as shown in Figure 10-9. An exception is the “U” abutment where the wingwalls are parallel to the road centerline.

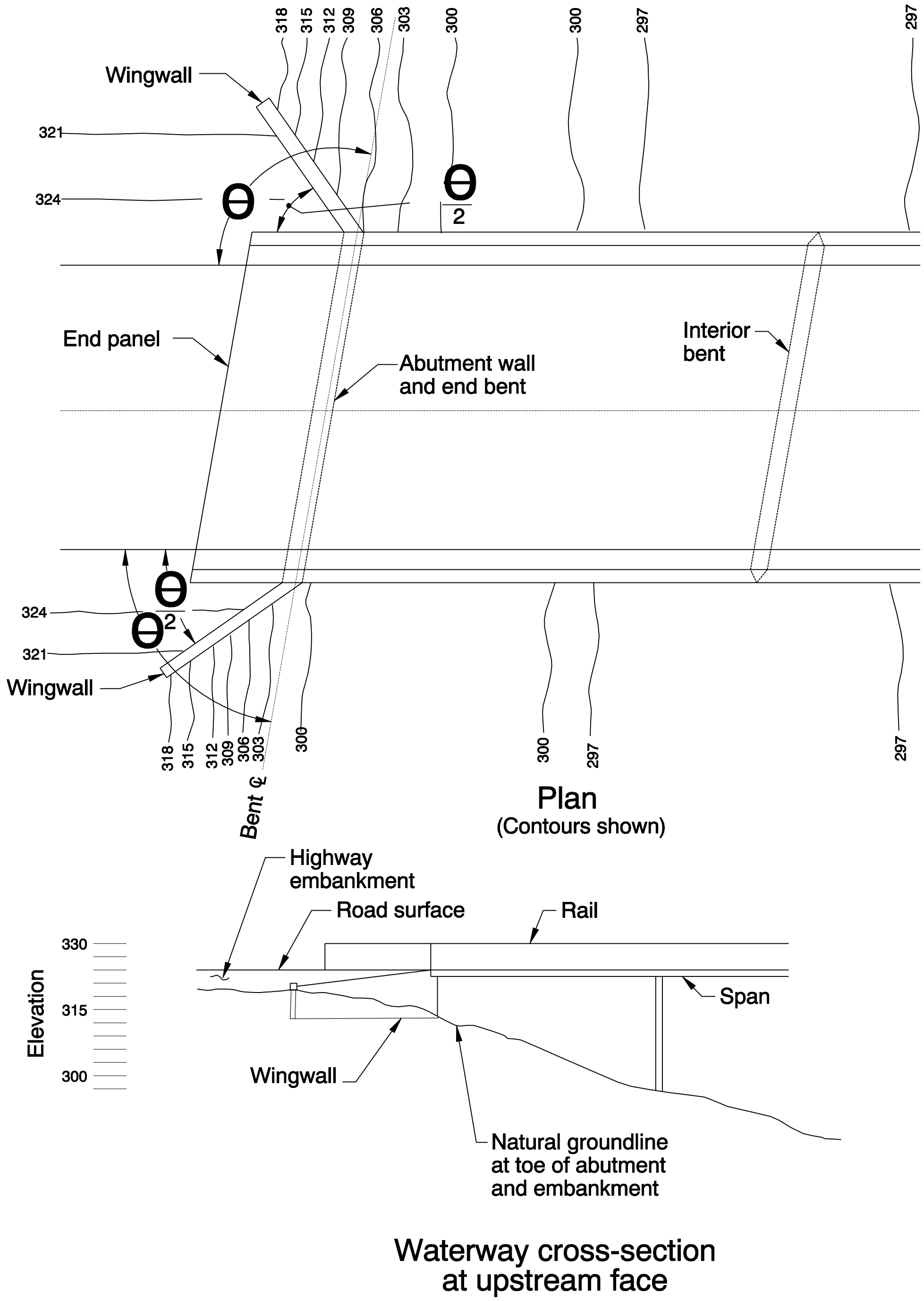


Figure 10-9 Vertical Abutment

In most cases, the revetment protecting the abutment wall toes does not reduce the cross-sectional area of the bridge opening, and it does not have to be included in the hydraulic model. The revetment protecting the abutment is usually placed in a trench adjacent to the wall face above the wall toe.

10.3.7.3 Mechanically Stabilized Earth (MSE) Wall Abutments

The mechanically stabilized earth (MSE) wall abutment is a variation of the vertical abutment. This abutment is often a cost-effective alternative to a tall conventional vertical abutment, or a conventional abutment founded on a material with a low bearing capacity. Unlike the vertical abutment described in the previous subsection, the MSE abutment does not convey substantial concentrated loads to the underlaying earth or rock. The embankment fill is retained by walls on the outside of the abutment. The structural loads from the bridge are transferred to the fill within the abutment by a shallow footing under the end bent, and the structural loads are transmitted to the underlaying material by the abutment fill. A MSE wall abutment is in Figure 10-10.

MSE wall abutments are different than the typical conventional vertical abutments. Unlike conventional abutments which can be skewed up to 45 degrees or more, MSE wall abutments are usually unskewed, and if they are skewed, the maximum skew angle is 20^{o}. In order to avoid a skewed configuration, they are often placed a distance back from the main channel at skewed highway/waterway crossings. This results in a longer bridge. In addition, unlike the angled wingwalls used on most conventional vertical abutments, wingwalls parallel to the road centerline are often preferred for MSE wall abutments.

The MSE wall abutment has been used in Oregon in a limited number of recent applications. Almost all of the abutments have been constructed away from the water on the streambank with little or no in-water work. In addition, most of the bridge spans and supporting footings have been located above the elevation of the check flood. The designer should use caution when considering a MSE wall abutment for a site where the abutment will need to be built in the water or where floodwaters are expected to contact the footings or the bottom of the span.

Abutment Dimensions - The dimensions of MSE wall abutments and the spans supported by these abutments should be estimated by a structural designer. These structures are often used on large and complex girder bridges, and the dimension estimating guidelines in this chapter may not apply.

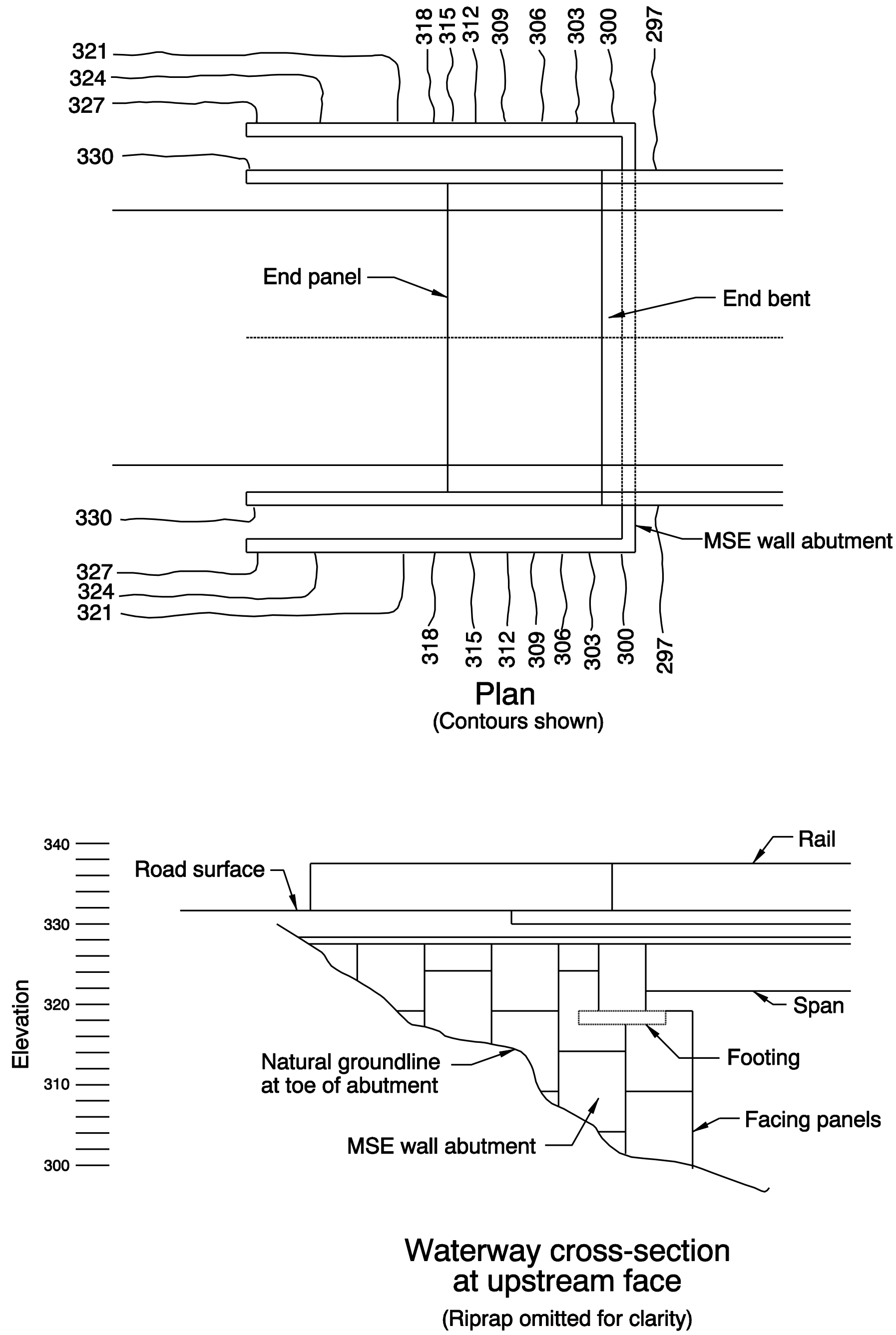


Figure 10-10 Mechanically Stabilized Earth (MSE) Wall Abutment

10.3.7.4 Set Back Abutments

The set back abutment is a spillthrough, vertical, or mechanically stabilized earth abutment located at a distance away from the waterway. Spillthrough abutment set back distance is the horizontal distance between the toe of the abutment fill and the edge of the stream or other water body during ordinary high water, as shown in Figure 10-11. Vertical abutment or mechanically stabilized earth abutment set back is the horizontal distance between the inside face of the abutment and the edge of the waterway during the ordinary high water, as shown in Figure 10-12.

It is important to define the location on the abutment where the set back is measured. Abutment set back can seldom be defined by a single distance. A single distance could be used if the bridge crossed a prismatic waterway at a right angle. This rarely occurs. As a result, set back distances often vary at different locations on the abutment fill toe or the end bent.

Set back abutments are used for many purposes, as follows.

- Many streams and rivers have significant overbank flow beyond the limits of the ordinary high water during large floods. Set back abutments can enlarge the waterway opening to provide adequate hydraulic capacity and to reduce contraction scour depths.
- Abutments can be set back to provide pedestrian and animal passage on the stream banks under the bridge.
- Set back abutments can be used to reduce or eliminate the need for revetment or other scour protection.

Note: The use of set back distance as scour protection must only be done after an engineering analysis proves it is viable. This is discussed in the scour protection section of this chapter.

10.3.8 End Panels

An end panel is a reinforced concrete slab across the approach fill at the end of the bridge that absorbs the impact from traffic entering or leaving the bridge deck. It prevents the fill from settling or the pavement from cracking near the end bent. This panel is often called an "impact panel," and it is used on state highway bridges and many bridges built for local agencies. End panels are shown in Figures 10-7, 10-9, and 10-10.

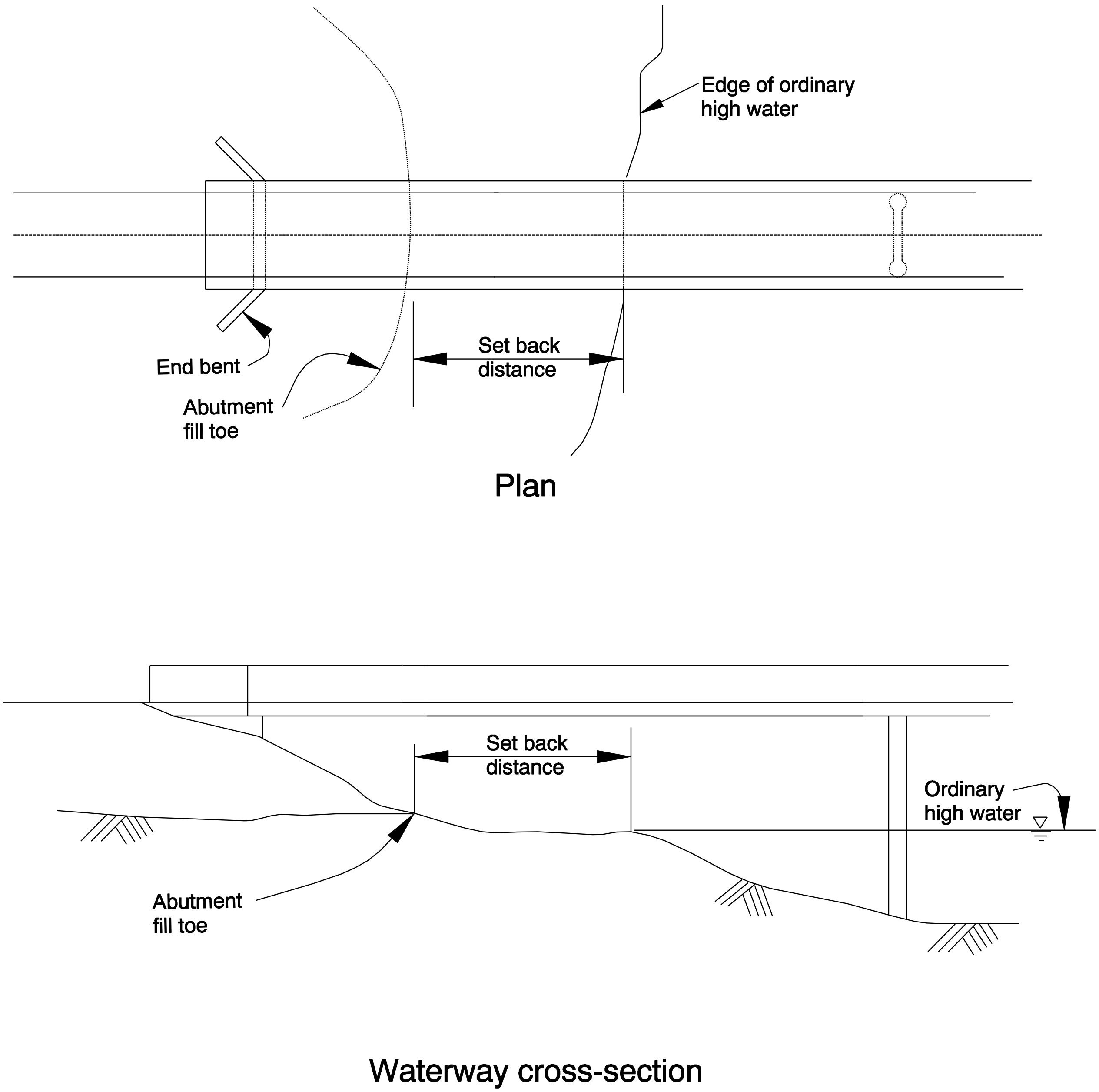


Figure 10-11 Set Back Distance for Spillthrough Abutment Bridge

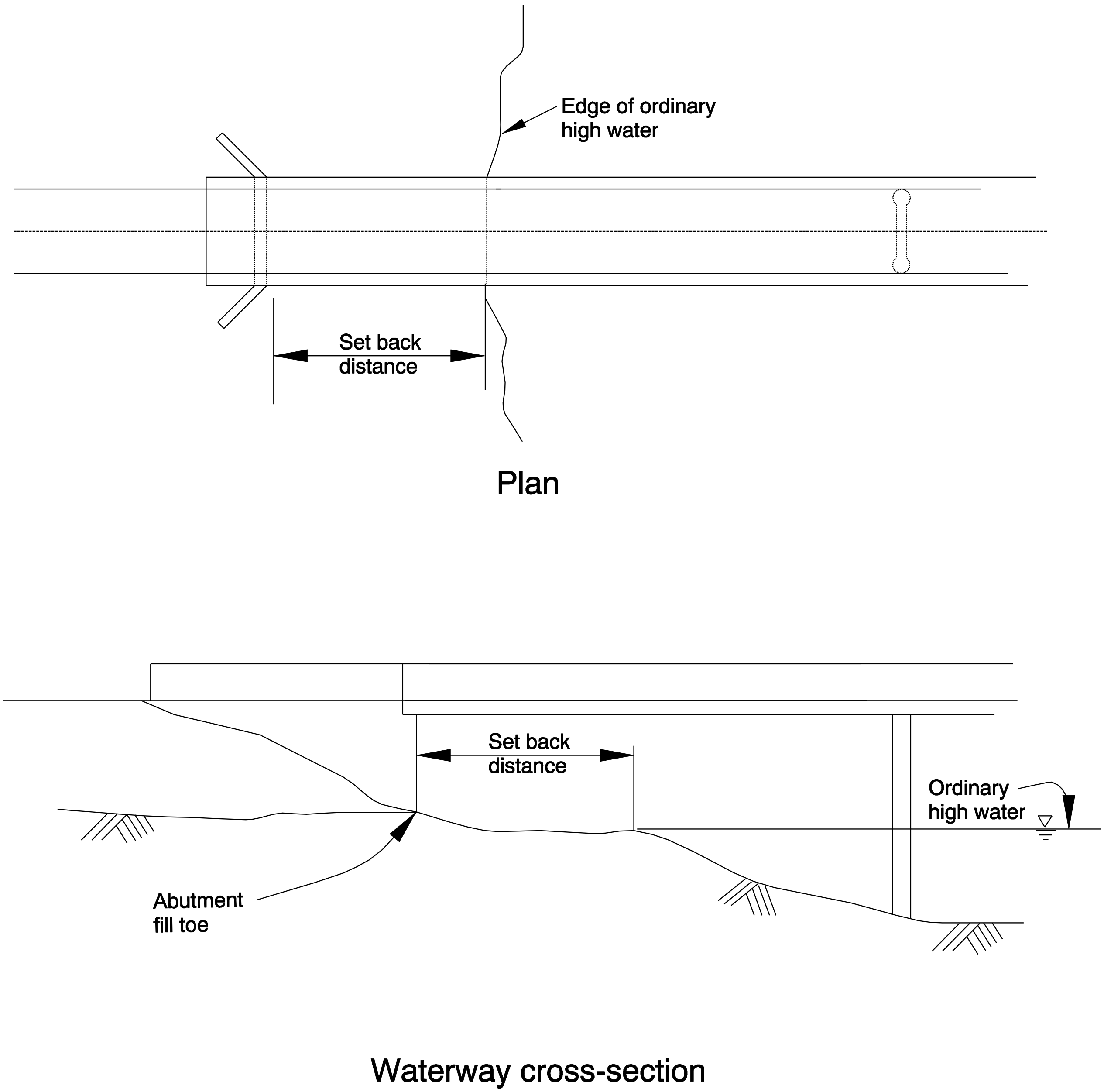


Figure 10-12 Set Back Distance for Vertical and Mechanically Stabilized Earth Abutments

Occasionally it is necessary to estimate the length of the end panel in order to design or specify erosion protection for the abutment supporting the panel. As general guidance, the end panel is 30 feet long for a bridge on a major highway, an abutment fill where excessive settlement is anticipated, a bridge with a deep abutment, or a bridge with severely skewed end bents. The end panel is 20 feet long if the foregoing situations do not occur.

10.3.9 Bridge Railing

In most instances the bridge railing is above the flood elevations, and its type, size, and location are of no concern to the hydraulics designer. In some instances, however, the bridge railing is within the path of flowing water and its hydraulic characteristics are of concern. Methods to model the hydraulic characteristics of rails that obstruct or retard flow are discussed in the user's manuals for the bridge analysis computer programs.

In general, most bridge rail types can be classified into two groups for hydraulic purposes, solid rail and flow-through rail. Solid rail includes solid concrete parapets and the often used ODOT Type F bridge rail shown in ODOT Standard Drawing BR200. These rails completely obstruct the flow unless the water goes over or around them. Flow through rails are also modeled as solid rails at sites where they may clog with floating debris.

Flow-through rail includes sheet metal guardrail mounted on posts and the tubular rails. Although these flow-through rails do not completely obstruct flow, they differ in their ability to retard flow. One of the most restrictive rails is the thrie beam rail in Standard Drawing BR233 and one of the least restrictive rails is the 2-tube side mount rail in Drawing BR226.

10.4 Design Criteria

This section includes guidance for the hydraulic aspects of bridge waterway opening sizing and location. Design and check flood discharges are listed in **Chapter 3**, guidance for hydraulic report writing is in **Chapter 4**, guidelines for riprap sizing are in **Chapter 15**.

10.4.1 Discharges and Tailwater Elevations

Several discharges are used for different purposes in bridge hydraulic studies. The criteria for discharge selection are in Section 10.8. Nearby downstream water bodies may considerably influence the hydraulic characteristics at the site, and these influences are often called "tailwater effects." Tailwater effects are addressed in Section 10.9.

10.4.2 Backwater

Backwater is the increase in water surface elevation due to the constriction caused by the bridge or approach roadway. It is the increase above the water surface elevation if the bridge or roadway were not in place. This elevation change is measured at the approach section. This section is located one waterway opening width upstream from the upstream face of the bridge constriction, as shown in Figure 10-13. Backwater is determined by a comparison of these two water surface profiles:

- the crossing without the subject structure and the approaches to the structure that occupy the floodplain, often called the "natural channel" conditions (this condition may occasionally include backwater from other existing downstream structures), and
- the crossing with the existing or proposed structure, approaches, and channel modifications, often called the "with bridge" conditions.

Note: Changes in riparian vegetation at the crossing site should be considered in the backwater analysis. The comparison should reflect the vegetation in its mature state. As an example, groups of willows planted as a riparian habitat enhancement can significantly increase backwater depths. The vegetation effects are accounted for in the selection of Manning's roughness coefficient "n."

Backwater depths are determined for both the existing and proposed crossings if the crossing is a replacement of an existing structure. The backwater depths should not be excessive. General criteria follow.

Road Overtopping - The backwater from the bridge or approaches should not cause water to overtop the road more frequently than the design flood recurrence interval. Excessively frequent overtopping cannot always be prevented by enlarging the bridge opening. The roadway may need to be raised at some locations to satisfy overtopping criteria. Minimum overtopping recurrence intervals are listed in **Chapter 3**.

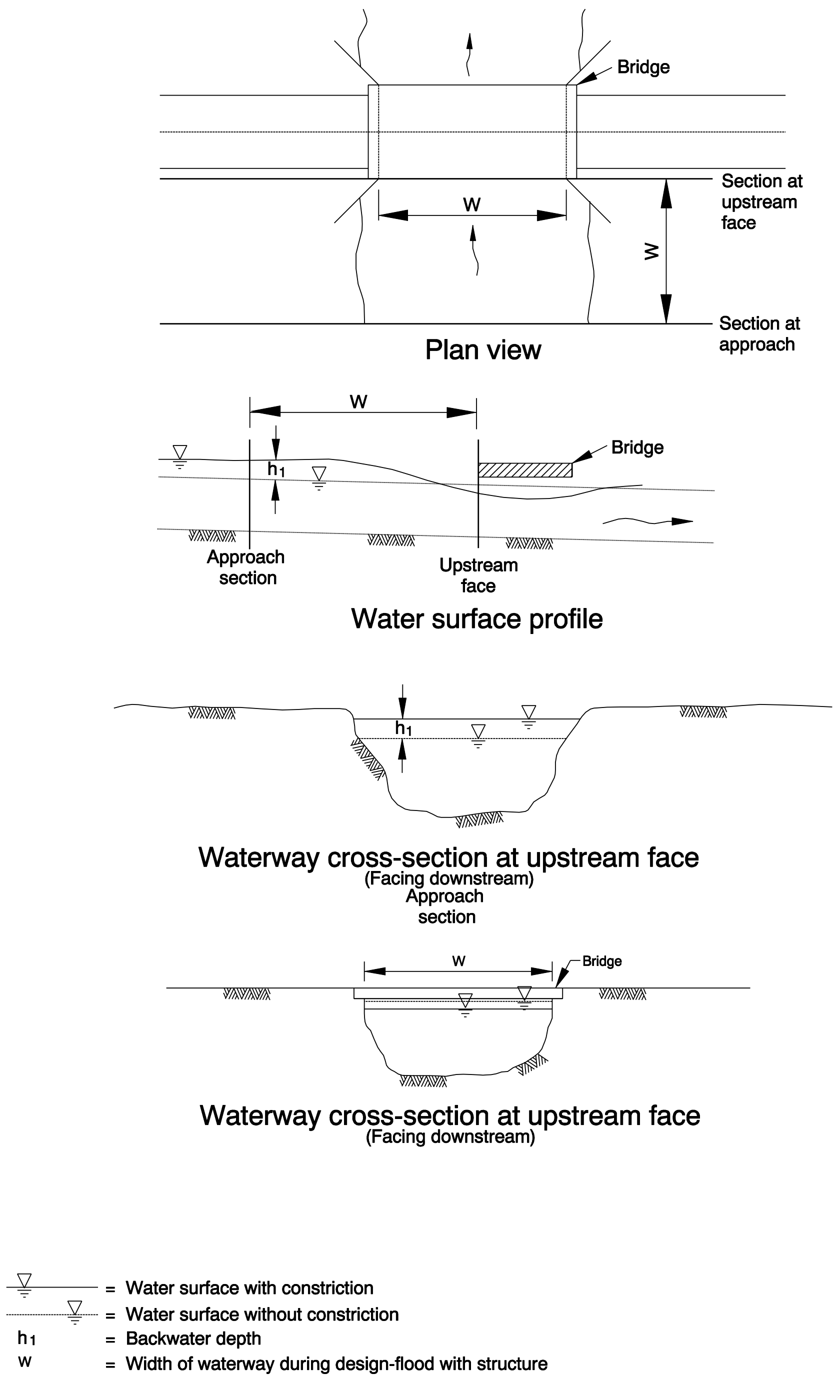


Figure 10-13 Backwater

In some circumstances, the bridge required to pass the design flood would be excessively long or high. If this occurs, a combination of flow through the bridge waterway opening and overtopping at low points in the adjacent roadway may be an alternative to accommodating the entire design flood under the bridge. This may be acceptable and allowable in certain circumstances, such as the following.

- The overflow does not damage adjacent property.
- The overflow does not travel across adjacent properties in paths it did not use before, or in significantly greater or lesser quantities than it did before, unless a drainage easement is acquired or the property is purchased.
- The approach roadway is also overtopped in other locations and traffic cannot travel to the bridge.
- Alternate routes are available to traverse or to evacuate the area.
- The road is useable when overtopped (shallow overtopping).

Crossings at Locations of Existing Structures - In general, backwater elevations should not be increased when replacing existing structures. A reduction in backwater should be considered if there is a flooding problem upstream from the existing crossing, or there is a scour problem at the existing bridge that can be alleviated by reducing the backwater.

Crossings at New Locations - In general, backwater elevations should not be higher than the site can tolerate. This is often governed by the road overtopping criteria. Backwater should not be increased on nearby upstream properties unless the needed drainage easements or right-of-way have been acquired. See **Chapter 3**.

Crossings in Regulatory Floodplains or Over Regulatory Floodways - Regulatory floodplains or floodways often have restrictions on the depth of the backwater that can be produced by a structure or its approaches. See **Chapter 2**. All proposed changes to water crossings should be evaluated to determine if they are in a regulatory floodplain or floodway, and it should be verified that they meet all applicable regulations. The ODOT Region Technical center hydraulic staff should be consulted about crossings in or over floodways subject to the federal Flood Insurance Program administered by the Federal Emergency Management Agency (FEMA). The ODOT Geo-Environmental Section’s Engineering and Asset Management Unit or the FEMA website has floodway maps that can be used to determine if a crossing is in a floodway.

10.4.3 Clearance

The vertical distance between the design flood water surface and the bottom of the bridge slab, box or beam is often called "clearance." The bottom of the bridge span is considered to be the elevation of the lowest structural member. This is the bottom of the lowest slab, box, girder, or beam.

The normal minimum clearance is 1.0 foot above the design flood elevation or 3 feet if drift or debris is a concern. The appropriate design flood recurrence intervals are listed in **Chapter 3**. If practical, 1 foot of clearance above the 100-year flood is provided.

Exceptions to the clearance requirement can be obtained with ODOT approval. An exception is for city or county bridges whose approaches are overtopped more frequently than once every ten years. The minimum bottom-of-beam elevation for these situations is 1.0 foot above the 10-year flood elevation. Under rare circumstances such as a park setting or where other controls on grade lines make it necessary, high water above the beam bottoms or over the deck may be allowed.

Many of the larger rivers, as well as bays, lakes, and estuaries in Oregon are considered to be navigable. The appropriate ODOT permit liaison officer should be contacted to determine if a crossing is on a navigable waterway and the needed navigational requirements. In addition, canals and irrigation ditches frequently have clearance requirements and the appropriate operators or irrigation districts should be contacted.

Clearance distances should be maintained or increased on crossings over streams that carry debris or ice during floods. Maintenance personnel should be contacted, and maintenance records reviewed, to see if debris or ice passage has been a problem. Three to four feet of clearance may be needed at some sites to accommodate debris or ice passage.

10.4.4 Waterway Alignment

At a minimum, the waterway opening should span the channel. The channel is the portion of the waterway that:

- conveys the moving bed material during large floods,
- typically is scoured of vegetation during the flood season, and
- often has steep and well defined banks.

The channel on many waterways can be defined as the area that is subject to flow every year. This is the waterway under the Ordinary High Water (OHW) elevation. The OHW is defined and discussed in **Chapter 6**.

The waterway opening should discharge flow with a direction and velocity that minimizes or eliminates damage to downstream facilities or property. This will often require a skewed structure at a location where the highway crosses the stream at an angle. The abutments should be skewed to match the direction of flow in the channel if the stream and roadway centerlines are skewed and the abutment faces are near the edges of the channel. An unskewed structure (i.e., bents normal to the roadway centerline) can often be used at a skewed crossing if the abutments are set well behind the channel in areas with low flow velocities. In instances where low flow and flood flow pass through the crossing in different directions, the waterway opening should be aligned to pass the flood flow.

The waterway opening should provide adequate performance throughout its design life with anticipated changes in streambed profile and alignment. As an example, additional streambed to bottom-of-span clearance may be needed over a waterway with a history of streambed aggradation, or a longer bridge might be needed to accommodate meander migration.

The waterway opening size and shape should consider construction and maintenance practices. As an example, additional deck to streambed clearance may be needed for maintenance equipment at a crossing where debris may need to be removed from under the bridge.

10.4.5 Environmental Concerns

Bridges and bridge approaches are often located in environmentally sensitive waterways and riparian areas. The need to minimize disturbance to these areas can often govern the location of the abutments and the interior bents, and consequently, the waterway opening size and alignment. The Region Environmental Coordinator or project team environmental representative should be contacted early in the design process for a description of the environmental concerns and requirements.

10.4.6 Scour Elevations and Countermeasures

The proposed structure and approaches need to resist scour damage. In order to do this, the flow depths and velocities through the structure must not be in excess of those that the scour countermeasures can withstand. The need to keep these depths and velocities at acceptable levels can often govern the size and alignment of the waterway opening and the elevation of the bottom of the bridge span.

10.5 Design Procedures

The hydraulic design of a stream crossing is a complex procedure that involves many tasks. The design procedures for state highway system bridges are in this chapter. Design tasks for local agency bridges are in the current ODOT Local Agency Project Manual. The typical bridge hydraulic design includes many, if not all, of the following tasks;

- visit the site to provide scoping assistance, investigate vertical and lateral channel stability, and request specialized hydraulic survey data,
- determine or assist in determining the Ordinary High Water (OHW) elevations,
- compile data,
- determine the hydrology for facility design and temporary water management (TWM),
- determine if variable tailwater elevations occur,
- create a hydraulic model of the existing crossing, if one is present,

- calibrate the hydraulic model of the existing crossing so the modeled performance reflects the historical performance,
- determine the "existing bridge" hydraulic performance,
- create a hydraulic model of the crossing with no bridge or approaches in place to determine the "natural channel" hydraulic performance,
- create a hydraulic model of the probable alternatives and analyze the alternatives,
- design the scour protection countermeasures,
- reanalyze the proposed alternatives with the scour countermeasures in place, if needed,
- analyze the OHW flood through the proposed alternatives,
- analyze the detour or temporary crossing, if used,
- compile and submit the Hydraulics Report,
- provide hydraulic assistance for the deck drainage design,
- review hydraulic aspects of the design as shown on plans during various stages of the design,
- develop TWM plans and specifications,
- provide data and support to permit specialist in order to obtain permits required for the project, and
- provide hydraulic design assistance to the project team, as needed.

Not all tasks need to be performed for every situation, as shown in the following list. These are generalities, and engineering judgment should be used when planning the hydraulic study procedure for a specific location.

1. A backwater analysis may not be possible or needed for bridges over pooled or slowly flowing waters (less than 3 feet per second during the check flood), such as crossings over sloughs, wetlands, lakes, or tidal areas such as mudflats. At these crossings environmental concerns rather than hydraulics often determine the required bridge length.

   The hydraulic study:

   - should provide the water surface elevation in the waterway opening,
   - should include revetment designed to resist wave action, if applicable, and
   - it does not need to provide approach section data, backwater depths, velocities, or scour elevations.

2. An existing channel water surface profile, only, (i.e. bridge does not need to be included in the hydraulic model) may need to be calculated for bridges over flowing water where:

   - the abutments or piers do not contact the water during the 500-year flood,
   - the waterway is in a fixed location where it cannot move and contact the structure, and
   - revetment is not needed as a protection from flowing water.

3. A scour analysis is not required if the structure is solidly founded on non-erodible rock and no revetment is needed.

4. Calculation of hydraulic data may not be needed if a previous hydraulic analysis provides the needed information. This previously analyzed information should be used with care. The data that is used should reflect current conditions and have sufficient accuracy. The Hydraulics Report should mention which data is from previous studies, and it should also list the sources.

5. The proposed work on the structure does not involve the waterway. Bridge rail replacement and deck rehabilitation are examples. Bridge widening may not need a hydraulic study if it does not affect the hydraulic performance of the structure, and scour elevations or revetment sizes are not needed for the design.

The design steps are described in detail in the following sections. Guidance on the information to be included in the Hydraulics Report is in **Chapter 4**.

10.6 Scoping and Preliminary Structure Estimate

A preliminary design task is to assist the project team with the preliminary structure type, size, and location estimate. This task is called "scoping", and it usually requires a site inspection. The site visit is a convenient time to determine the specialized survey data for the hydraulic study. The survey data is requested during this visit or shortly thereafter. The site visit is also a good time to investigate channel stability because this factor will affect the preliminary design recommendations. Hydraulic study data collection is discussed in detail in **Chapter 6**.
The preliminary structure type, size, and location estimate is made during or shortly after the site visit, and the hydraulic designer provides input from the hydraulic perspective. The following general guidelines can be used.

1. The structure should span the active stream channel, at a minimum. Regardless of the bridge type, the abutments should not intrude into the channel. This requirement often results in a spillthrough structure alternative that is longer than a vertical abutment structure. Abutment face slopes of 1V: 2H can be used for preliminary estimates.

2. The structure may need to span both the main channel and floodplain overbank areas at sites where significant contraction scour would occur if the bridge spans the channel, only.

3. The structure should be skewed, as needed, to match the flood flow direction. In some cases an unskewed structure can be used. The decision to use an unskewed structure at a skewed crossing is usually made later in the design process.

4. The most likely structure type is used for the preliminary estimate and it is determined by the bridge designer. In absence of this information, vertical abutment bridges are often used where there is non-erodible rock at or near the ground surface at the abutment locations, and the abutment heights are less than 10 feet. Spillthrough abutment bridges are often used at other locations.

5. Environmental setbacks and the nature of allowable in-stream work should be addressed at this time. These requirements can often greatly influence structure type, size, and location.

6. Regulatory floodway requirements should be addressed at this time if the structure, roadway embankments, riparian enhancement, or riparian mitigation are in a floodway. These requirements can control the structure type and location, the roadway alignment, the landscaping and riparian habitat modifications, the choice of sidewalks, guardrail, etc. A hydraulics designer should be consulted if any part of the project is within a floodway. The floodway is defined in **Chapter 2**.

10.7 Compiling Project Design Data

Data compilation for a hydraulic study includes collecting the preliminary data, as discussed in the previous section and Chapter 6. It also includes compiling project design information such as the survey and utility location, and datum verification.

An accurate site survey and resulting design information is especially critical for structures such as bridges and large culverts. The typical survey is done using computerized electronic equipment and must be processed prior to use in design. This data reduction is typically done in the survey office. The survey office verifies the quality of the survey and the information submitted to the designers. Experience has shown it is important for the hydraulic designer to also carefully review the design information and to verify it realistically represents the hydraulic structures. The following are some of the more common hydraulic survey errors.

- The terrain survey extends down to the water surface, only. The terrain model produced by the survey erroneously shows a flat channel bottom at the water surface elevation during the survey.
- The terrain model extends down to the edge of the water, and the only underwater data collected are the channel thalweg locations and elevations. The survey terrain model will represent the underwater channel as “V” shaped, although this might not be the case.
- The culvert invert elevations are measured at the surface of the sediment layer covering the invert, rather than on the pipe itself. This will represent the culvert being at a higher than actual elevation.

There are several methods to verify survey accuracy. Many, such as statistical analysis of confidence points, are complex and done by people experienced with surveying. One of the more simple methods is also the most effective. It is to “cut’ cross-sections from the terrain model at critical locations, to take these sections to the site, and to visually compare their shape and elevation to actual features. Errors in the survey or data reduction are often clearly visible.

The utility location information should also be verified. Visible utilities such as poles should be shown on the terrain model and be seen in the field. The site should be inspected for signs or other markers indicating underground utilities are present. These utilities should also be shown on the terrain model.

The project survey elevation datum should be noted and compared to the elevation datums used for the reference studies, reports, and publications. This is especially critical where there are regulatory floodplains and floodways. Several different vertical control reference datums have been used throughout the state since the beginning of the highway system, and many of these datums are in use today. The elevation differences between these datums are often several feet. It is essential to convert all elevations from other sources to the project datum before they are used in design.

10.8 Hydrology

One of the most important steps in bridge hydraulic design is to determine the study discharges, or "hydrology." Hydrologic methods are provided in **Chapter 7** and the data needed in the hydrology section of a hydraulics report are listed in **Chapter 4**. The discharges used in bridge design are:

- the design discharge (the 25-year, 50-year, or roadway overtopping discharge),
- the base flood (100-year) discharge,
- the check discharge (the road overtopping or 500-year discharge),
- scour calculation discharges,
- discharges needed for environmental and permit application data, such as the average annual (2-year) discharge or the ordinary high water (1.5-year discharge), and
- monthly peak or daily discharges for temporary water management during construction.

Changes in sea level during the bridge design life are considered in coastal structure design.

10.8.1 Design Discharge

Design discharges for bridges in various applications are listed in **Chapter 3**. The design discharge is used to compare the hydraulic characteristics of the existing and proposed crossings. It is also used to determine the minimum recommended waterway area and the minimum bottom-of-span elevation, and to design the revetment. The 25 or 50-year flows are the most common design discharges. A 100-year flood design discharge is used in regulatory floodways and floodplains. The road overtopping flood is the design discharge if overtopping occurs more frequently than the design flood recurrence interval listed in **Chapter 3**. The hydraulic performance during the design discharge is reported for both the existing and proposed structures.

10.8.2 Base Flood

This is a discharge with a 100-year recurrence interval. It is used to compare the hydraulic performance of the existing and proposed crossings, to design the revetment, and to calculate scour elevations. The base flood is often used to assess the effects of inundation caused by the proposed structure and to compare the effects of inundation caused by the existing and proposed structures. The base flood is the design flood in regulatory floodplains and floodways. The hydraulic performance during the base flood discharge is reported for both the existing and proposed structures.

10.8.3 Check Discharge

The check discharge is used to compare the hydraulic performance of the existing and proposed crossings, to check the stability of the structure and revetment, and to estimate the scour elevations. The check discharge is either the 500-year flood or the roadway overtopping flood. The check discharges are reported for both the existing and proposed crossings. It is common to have different check discharges for the existing and proposed structure if the road overtopping floods are the check discharges. Hydraulic performance during the check flood is reported for the existing and proposed structures.

500-Year Flood - This flood is expected to occur, on the average, once every 500 years. This is the check discharge if road overtopping occurs less frequently than once in 500 years.

Road Overtopping Discharge - The road overtopping discharge is the flood when either the road overtops, the bridge overtops, or another area near the upstream side of the bridge overtops and the water enters an adjacent drainage. In general, this occurs when the energy grade line elevation at the approach section is at the same elevation as the point of overtopping. The energy grade line represents the surface elevation of water pooled on the upstream side of the crossing. The road overtopping flood is not analyzed if it occurs less frequently than once every 500-years.

The hydraulic stresses caused by an overtopping flood with a recurrence interval less than 500-years will often create greater hydraulic stresses than the 500-year flood. This is not always the case. Occasionally a 500-year flood with overtopping will create greater stresses than a more frequent overtopping flood. It is good practice to always analyze scour and waterway opening velocities caused by the 500-year flood. The flood causing the deepest scour and highest waterway opening velocities should be reported.

10.8.4 Scour Calculation Discharges

Scour is calculated for the 100-year flood, the 500-year flood, and the roadway overtopping flood. The scour depths are compared to determine the design and check discharges for scour estimates. The scour design and check discharges depend on the site hydraulic characteristics, as follows.

Case 1: The incipient roadway overtopping flood occurs more frequently than the 100-year flood and creates deeper scour than the 100-year flood. The overtopping flood is the design flood and the check flood is either the 100-year flood or the 500-year flood, whichever creates the deepest scour.

Case 2: The incipient roadway overtopping flood occurs more frequently than the 100-year flood and creates shallower scour than the 100-year flood. The 100-year flood is the design flood and the check flood is either the overtopping flood or the 500-year flood, whichever creates the deepest scour.

Case 3: The incipient roadway overtopping flood occurs less frequently than the 100-year flood but more frequently than the 500-year flood. The 100-year flood is the design flood and the check flood is either the overtopping flood or the 500-year flood, whichever creates the deepest scour.

Case 4: The incipient roadway overtopping flood occurs less frequently than the 500-year flood. The 100-year flood is the design flood and the 500-year flood is the check flood.

10.8.5 Average Annual Discharge and Ordinary High Water Discharge

The average annual discharge has a 2.33-year recurrence interval. It is the flood that occurs at least once every two years on the average. This flood was often used to determine the riparian area for regulatory purposes. It is still used by some agencies.

Experience has shown that the 2-year flood can often overestimate the riparian area. Many regulatory agencies have adopted the ordinary high water (OHW) discharge as an indicator of the riparian area. This is the flood expected to occur every year, on the average. The OHW elevation is often determined by field marks, as discussed in **Chapter 6**. OHW elevation estimates based on field marks are often uncertain and difficult, especially after floods. It is good practice to verify the OHW elevations using a hydraulic model. The OHW flood is approximately the 1.5-year event.

10.8.6 Temporary Water Management Discharges

Temporary Water Management (TWM) is used to keep water from being contaminated by construction activities, to keep water from interfering with construction operations, and in the case of streams, to ensure there is an uninterrupted flow of water through the work site.

Several discharges are used in TWM. Mean daily exceedance discharges are often used to estimate pump sizes, bypass pipe diameters, and other TWM features. Maximum predicted discharges are often used to design critical temporary structures such as cofferdams. Methods to calculate TWM discharges are in **Chapter 7**.

10.8.7 Sea Level Change

Sea level change should be considered in coastal structure design. This change is primarily a combination of global sea level rise combined with local vertical land movement. Trends in sea level change have been calculated for three tide level gaging stations on the Oregon Coast and a nearby gage in California by the National Ocean Service (NOS) in cooperation with the National Oceanic and Atmospheric Administration (NOAA). A positive trend means the sea level appears to be rising in relation to the land. A negative trend means the opposite. The following summaries are based on information from NOAA.

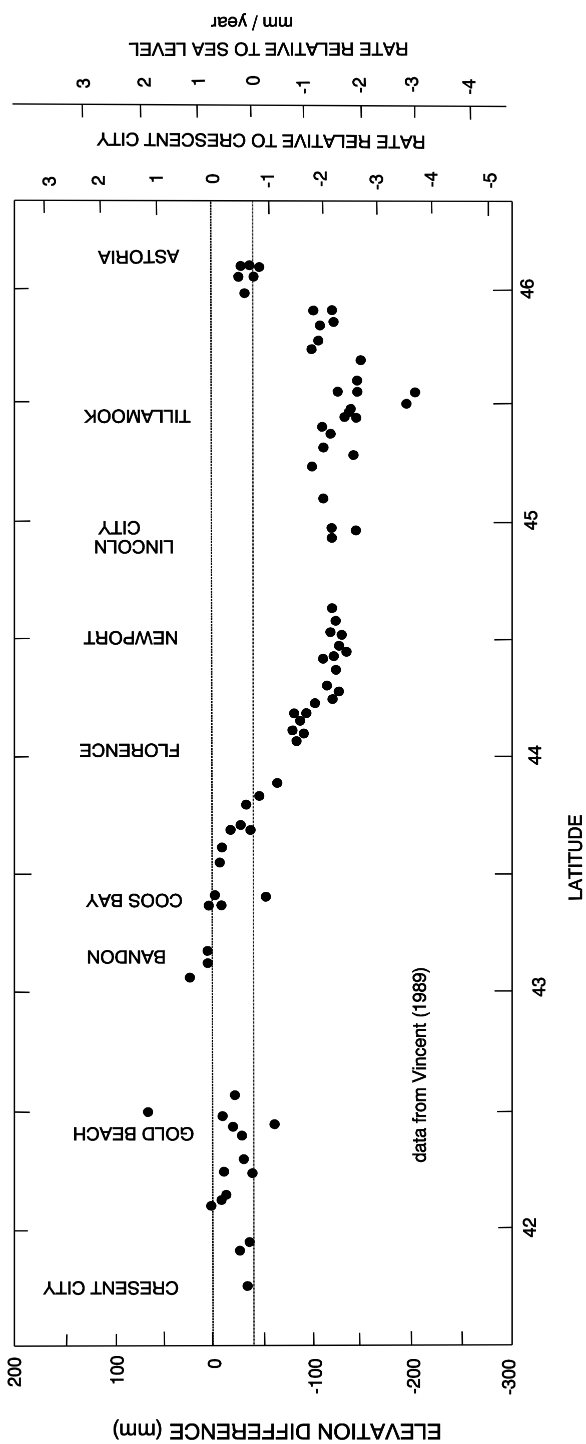
Gage 9419750 Crescent City, California (near Brookings) – "The mean sea level trend is - 0.16 feet per century (- 0.48 millimeters per year) with a standard error of 0.009 inches (0.23 millimeters) per year based on monthly mean sea level data from 1933 to 1999.

Gage 9432780 Charleston, Oregon (near Coos Bay) – "The mean sea level trend is +0.57 feet per century (+1.74 millimeters per year) with a standard error of 0.034 inches (0.87 millimeters) per year based on monthly mean sea level data from 1970 to 1999.

Gage 9435380 South Beach, Oregon (near Newport) – "The mean sea level trend is +1.15 feet per century (+3.51 millimeters per year) with a standard error of 0.029 inches (0.73 millimeters) per year based on monthly mean sea level data from 1967 to 1999.

Gage 9439040 Astoria, Oregon – "The mean sea level trend is (-0.05 feet per century (–0.16 millimeters per year) with a standard error of 0.009 inches (0.24 millimeters) per year based on monthly mean sea level data from 1925 to 1999.

Sea level change can also be estimated using this chart developed by P. Vincent for a 1989 Masters of Science Dissertation at the University of Oregon, Eugene, Oregon.



Data is metric

Figure 10-14 Sea Level Change on the Oregon Coast

10.9 Tailwater Effects

Many bridges cross streams and rivers upstream from their confluence with other streams and rivers, lakes, estuaries, bays, or the ocean. The water surface elevation in the downstream water body should be considered during the hydraulic design when it can influence the flow velocity and water surface elevation at the bridge. Examples of different situations follow.

Situation 1- The bridge is on a stream or river, the downstream water body is also a stream or river, and neither watercourse has significant regulation or flow diversion upstream from their confluence. In addition, both the tributary and downstream water body watersheds should respond to the same hydrological events. In this situation, it may be possible that floods with different recurrence intervals can occur simultaneously on either watercourse. The difference in flood recurrence interval increases with the ratio of watershed size, and the relationship is shown in the "Frequencies for Coincidental Occurrence" table in **Chapter 13**.

The crossing should be analyzed with the given flood passing through the tributary and a coincident flood in the downstream water body. The crossing should also be analyzed with the given flood in the downstream water body and a coincident flood in the tributary. The hydraulic characteristics should be reported for the combination(s) that result in the highest water surface elevation at the bridge and the highest velocity through the waterway opening. These velocities and elevations will be used in the hydraulic and structural designs.

Situation 2- The bridge is at a site where the coincidental occurrence relationship in Situation 1 does not apply. This could occur when the crossing is over a regulated stream, the downstream water body is a regulated stream or reservoir, or the tributary crossed by the structure and the downstream water body does not respond to the same hydrologic events. An example where Situation 2 applies is a flooding stream that discharges into a downstream reservoir that has been previously drained to provide flood storage. Unlike the coincident floods that would occur during Situation 1, in this instance there would be no tailwater effect from the downstream water body.

In Situation 2, the crossing should be analyzed when the given flood passes through the bridge and the flow is at normal depth in the downstream channel (there is no backwater from the neighboring water body). This combination will usually result in the highest velocity through the bridge opening. The crossing should also be analyzed when the given flood passes through the opening and there is a flood of identical recurrence interval in the downstream water body. This combination will usually result in the highest water surface elevation. The hydraulic data for both combinations of discharge and backwater should be reported.

Situation 3 - The bridge is on a stream or river and the elevation of the downstream water body is influenced by the tides. This occurs in estuaries and bays, near the ocean, and on large tidally influenced rivers such as the lower reaches of the Columbia River. The highest velocity is expected when the given flood passes through the bridge opening, and there is mean lower low tide in the downstream water body. The highest water surface elevation at the bridge is expected when the given flood passes through the bridge opening and there is mean higher high tide in the downstream water body.

Situation 4 - The bridge is at a site where the tailwater elevations are influenced by another structure, such as a weir, a dam, or another bridge. The structure should be analyzed with and without the downstream structure in place if there is any chance the structure will be removed or wash out during the design life of the proposed structure. (In the case of a nearby bridge, it is always assumed that the structure will be removed.) The highest water surface elevations typically occur when the downstream structure is in-place, and the greatest flow velocities usually happen after it is removed.

10.10 Hydraulic Modeling

Hydraulic modeling is the process to determine the water surface profiles through the bridge site. It is also used to estimate flow velocities and to calculate scour depths.

Water flowing through a bridge is a complex process. In most instances, the predominate flow direction is in the longitudinal (upstream-downstream) direction, and there is limited flow in the vertical (up-down) direction, or in the transverse (side-to-side) direction. The hydraulic characteristics of this flow are modeled by one-dimensional analysis, which recognizes flow in the longitudinal direction and ignores flow in other directions. These one-dimensional analyses are often made using the following computer programs:

- U.S. Corps of Engineers Hydraulic Engineering Center - River Analysis System (HEC-RAS), or
- Federal Highway Administration (FHWA) Water Surface Profile (WSPRO). This program is a module in the FHWA HYDRAIN software package.

The HEC-RAS program is preferred by ODOT.

Sometimes, transverse flow across the waterway is predominate and the one-dimensional method may not be appropriate. In these instances, more accurate answers may be provided using a two-dimensional modeling method. Two dimensional (2-D) models simulate flow in the longitudinal and transverse directions at a series of user defined node points. Flow in the vertical direction is assumed to be negligible. These models can account for transverse flow due to lateral velocities and water surface gradients that cannot be accounted for with one-dimensional models. A 2-D model should be considered for major projects with complex flow patterns that one-dimensional models cannot adequately analyze, such as the following.

- Wide floodplains with multiple openings, particularly on skewed embankments.
- Floodplains with significant variations in roughness or complex geometry, such as ineffective flow areas, flow around islands, or multiple channels.
- Sites where more accurate flow patterns and velocities are needed to design better and more cost-effective countermeasures such as riprap along embankments and/or abutments
- Tidally affected river crossings of tidal inlets, bays, and estuaries.
- High risk or sensitive locations where potential losses or liability costs are high.

Two commonly used 2-D computer programs are the U.S. Corps of Engineers (USCOE) RMA-2 and the Federal Highway Administration (FHWA) Finite Element Surface-Water Modeling System: Two-Dimensional Flow in a Horizontal Plane (FESWMS-2DH). Both RMA-2 and FESWMS model steady and unsteady flow. FESWMS is recommended for highway crossings of rivers and floodplains because it supports both super and subcritical flow analysis, and it can analyze weirs (roadway overtopping), culverts, and bridges.

The Surface Water Modeling System (SMS) developed by Brigham Young University in a cooperative project with the USCOE and the FHWA can be used to develop the finite element mesh and associated boundary conditions necessary for RMA2 and FESWMS. The solution files for both programs, which contain surface elevation, velocity, or other functional data at each mesh node, can be read into SMS to generate vector plots, color-shaded contour plots, time variant curve plots, and dynamic animation sequences.

Note: The RMA2 and FESWMS-2DH methods require considerable input data and user expertise, and as a consequence, it can be an expensive analysis. It is recommended that the ODOT Region Technical Center hydraulics staff be contacted before this method is used on ODOT projects.

10.10.1 Existing Structure

The first step in the hydraulic modeling is to analyze the existing structure, if one is present. The terrain data is provided by the field survey, and the structural data is based on the survey data and the plans for the existing structure.

The predicted existing crossing hydraulic performance should be compared to historical flood records and observed highwater data, if it is available. The hydraulic model may need to be adjusted if the predicted flood frequency relationship is significantly different than the flooding history. This adjustment is called "calibration" and it is usually made by adjusting the waterway or structure Manning's roughness coefficients. It is also made by designating ineffective flow areas in the cross-section.

Backwater depths for the existing structure are based on a comparison between the existing structure and natural channel water surface profiles.

10.10.2 Natural Channel

The water surface profile computed with the natural channel hydraulic model provides the baseline of comparison for the hydraulic performance of both the existing (if present) and proposed structures. The natural channel model is the same for both structures, and it represents the hydraulic conditions (i.e. highwater elevations, flow velocities, flow distributions, etc) at the site without the roadway, roadway embankment, or structure.

10.10.3 Proposed Structures

The proposed structure or alternative structure geometries are included in these hydraulic models. The structural designer can often list and describe the possible alternatives for these analyses. In absence of this information, it is customary to report data on spillthrough abutment and vertical abutment bridge alternatives. Throughout the modeling process it is important to verify that the modeled structure is a viable option. As an example, a hydraulic analysis of a single-span bridge is of little use if a multi-span structure is needed.

Special care should be used when modeling crossings where pressure flow or roadway overtopping occurs. The hydraulic performance of these sites are often controlled by features such as the bottom of beam elevations, the bridge deck profile, the roadway profile, and the flow resistance of the guardrails, curbs, and road surface. Guidance on the hydraulic modeling of these sites is included in Section 10.3 and the user's manual for the analysis program.

The proposed structure(s) should be modeled for both the post-construction conditions and future conditions that may occur during the structure design life. An example is a pavement overlay that blocks the roadway overtopping flow. In this case the structure should be analyzed with and without the overlay in place. The highest upstream water surface elevations and velocities through the bridge opening may occur after the road grade is overlaid. Another example is the case where the roadway grade may be raised in the future to eliminate overtopping. It may be necessary to construct a longer bridge to accommodate a future grade raise.

Backwater depths for the proposed structure(s) are based on a comparison between the proposed structure(s) and natural channel water surface profiles.

10.11 Scour and Erosion Protection

Bridges in riverine and marine environments are often exposed to the scouring action of flowing water and the erosive action of waves on standing water. As a result, scour and erosion countermeasures are used to protect the structure from these destructive forces. In this section the term "scour protection" applies to countermeasures against both erosion and scour damage.

This section discusses countermeasures to protect bridges and abutments from scour caused by river and stream discharges and flows in estuaries due to tidal fluctuations. These countermeasures can also be used on inland waterways where flow velocities are low and wave action is the dominant cause of erosion. This could occur on deep bodies of water such as tidal estuaries, bays, lakes, and large rivers. In these cases, it is recommended that the revetment be designed to resist wave action and checked to verify that it will withstand the forces caused by flowing water. Methods of sizing riprap to resist wave damage are presented in **Chapter 15**.

Countermeasures to protect structures from ocean wave action are beyond the scope of this manual. Coastal engineering design methods should be used. Specialized publications on coastal engineering are available from several sources. The United States Corps of Engineers “Coastal Engineering Manual” is a widely used reference. The design method for coastal protection on an ODOT structure should be discussed with the Region Technical Center hydraulics staff before it is used in design. ODOT must verify that it is an acceptable procedure for the application.

Erosion damage to spillthrough abutments can be caused by roadway or bridge deck runoff flowing down the fill slope. This damage should be prevented, and methods to do this are presented in **Chapter 13**.

10.11.1 Extent of Scour Protection

The scour protection should prevent damage to the structure or supporting embankments from streamflow and wave action during floods of lesser or equal magnitude to the design event. The protection should survive the check flood with minimal damage to the structure or supporting embankment. Design and check floods are discussed in Section 10.8.

The protection should last throughout the design life of the structure without major maintenance. Structure design life is typically between 75 and 120 years. The structural designer can provide this information.

Scour protection with a reduced design life can be used in some circumstances. This should be clearly stated in the Hydraulic Report if it is the case. The frequency, type, and extent of anticipated repair or replacement should be described. Access requirements to repair or replace the scour protection should be described. This includes physical access, easements, and possibly added right-of-way. Responsible parties should be aware of, and agree to, the anticipated repairs or replacement.

10.11.2 Scour or Erosion Protection Countermeasure Preference and Selection

Almost all bridges are protected by one or more countermeasures. Countermeasures for each abutment and pier should be analyzed and preferred methods used if they will provide the required protection. As an example, riprap revetment may be the only practical protection for an abutment located on an eroding bank at the outside of a river bend. Set back without revetment may be the practical and preferred scour protection for the other abutment located behind the inside edge of the bend. The following list includes common scour protection methods in the general order of preference.

Avoiding areas subject to scour. The structure and supporting abutments are located away from scour prone areas, in the horizontal and/or vertical directions. Set back abutments, deep foundations, or retaining walls are often used to avoid scour prone areas.

Keying critical foundation members into scour and erosion resistant materials. The most commonly used methods are keying spread footings, drilling shafts, or driving pilings into erosion resistant materials. These methods cannot be used for spillthrough abutment embankments.

Biotechnical protection. These methods include planted embankments, vegetated biodegradable geogrids, live cribwalls, willow fascines, etc. These methods rely solely on vegetation for erosion protection. Many of these methods are discussed in **Chapter 15**. Biotechnical protection may be a suitable alternative for less scour critical areas with suitable growing conditions where it can be periodically maintained and replaced.

Biologically enhanced revetment. This includes riprap, concrete cribs, and concrete jacks planted with vegetation; articulated concrete block mats with vegetal cover; riprap with rootwads; and boulder toes. A common characteristic of enhanced revetment is vegetation backed or supported by an erosion resistant and durable layer. It does not depend solely on the vegetation to provide adequate scour protection.

Revetment. The most commonly used revetment material is loose rock riprap. Other revetment types, such as concrete cribs or jacks are also used. Revetment is the most common protection in scour critical areas where other more preferred methods are not practical.

Relocating the waterway. This includes relocating the channel bank or banks, or use of spur fields, barbs, or rootwads in riprap to move the waterway away from the structure. This is often the most practical method for protecting existing structures from scour damage.

These factors should be considered when selecting a scour protection method. The most preferred method should be used unless it is impractical due to one or more factors. These factors cannot be rated in importance. Any or all may be critical to select a scour protection type.

The factors are as follows.

Public safety. The consequences of scour protection failure should be considered. This includes the danger to the public during a failure, both to the traveling public and to people in the vicinity of the failure.

Expense. Structure repair or replacement can be very costly. There is the direct cost to the agency of the repair or replacement, and the cost to the public due to the inconvenience of interrupted commerce and travel. There is also the cost of liability from damage claims.

Ease of repair or replacement if the scour protection is damaged or fails. Many forms of scour protection are extremely difficult to repair or replace after the bridge deck is placed or after construction access roads are removed.

Environmental impacts of failure. Failure of scour protection can cause considerable environmental damage. Typical damage is accelerated embankment erosion and streambed contamination from embankment soils. This usually occurs during the winter when sensitive fish species are in the stream. Bridges often carry utilities across waterways. A critical concern can be a spill from a petroleum pipeline, gas pipeline, sewage pipeline, or other conduit that could rupture if the structure collapses.

Road closure. Scour protection failure often results in a road closure. This could be a prolonged closure if a complicated structural repair or replacement is needed. This is especially critical in Oregon. The largest floods in the state are almost always regional events that affect most of the bridges in the vicinity. Adequate scour protection of structures on critical highways is especially important during these events.

Certainty of design. Hydraulic conditions are often difficult to predict with certainty. Scour protection methods with extra strength and durability are often desired in applications where future conditions are uncertain.

10.11.3 Definition of Structure

The definition of a structure, for scour protection design is:

- the structure itself, including end bents, interior bents, end panels, and
- the abutment fills as needed to prevent structural damage.

In some cases, additional facilities are located away from the bridge to prevent damage to the structure from scour or erosion. These items should also be designed to the same standards as the structural protection. Examples include:

- weirs or check dams arresting headcuts that would undermine the bridge foundation if they were allowed to progress through the waterway opening (headcuts are described in **Chapter 9**), or
- spurs or embankment protection that direct the water through the bridge opening if structural damage could occur if the items were not in place and functioning as designed (spurs and embankment protection are discussed in **Chapter 15**.)

10.11.4 Variable Tailwater Elevations

Scour protection at sites with variable tailwater elevations should be checked to verify it is adequate for the range of expected conditions. Typically the greatest design and check flood waterway opening velocities occur when the tailwater elevation is at its lowest. This usually determines the hydraulic shear forces the scour protection must resist. This would be the rock size, using riprap for an example. Conversely, the highest design and check flood elevations usually occur when the tailwater elevation is at its highest. This combination of discharge and tailwater often governs the elevation of the top of the revetment protection. This is discussed in more detail in Section 10.9.

10.12 Scour Protection Design

A scour protection design is made for each application. Standard materials and designs are used where they provide adequate protection. As an example, riprap revetment would be of a standard ODOT classification unless circumstances require a special size or gradation. Several design methods for scour protection are discussed in the following subsections.

10.12.1 Scour Depths for Scour Protection Design

The primary means of scour protection is to have sufficient foundation burial into the supporting materials to be stable if scour occurs. The total scour depths discussed in Section 10.13 are used in the scour protection design. Shallower foundations can be used if they are protected by revetment adequate to resist damage from the check flood, or by sufficient set back distance.

Tops of pier footings are to be lower than the base flood scour elevations. Bottoms of pier footings are to be lower than the check flood scour elevations, or 6 feet below the thalweg, whichever is lower. Shallower footing burial is acceptable where the footings are keyed into solid erosion resistant rock.

Abutments that project into shallow and calm water do not need to be protected unless wave action is a concern or a new unprotected abutment fill has just been constructed. Shallow and calm water is assumed to be less than 3 feet deep with velocities less than 3 feet per second during the check flood.

Scour depths are to be calculated without considering nearby structures as scour protection, unless they will be intact and maintained throughout the structure design life. As an example, a new structure will be built immediately upstream from an existing bridge. The new structure should be designed so it is not damaged by scour if the existing structure is removed.

Additional information about foundations is in the ODOT Technical Services Foundations Manual and the Bridge Design and Drafting Manual.

10.12.2 Set Back Abutment Design

Set back abutments are placed in a location on the streambank or shore where they will not be undermined by scour or erosion, as shown in Section 10.3. The design is to assure the abutment is far enough from the waterway to not be undermined during the structure design life. In some cases, revetment, bioprotection, or river training structures are needed in addition to set back to prevent undermining.

10.12.2.1 Set Back Abutment Preliminary Estimates

It is often necessary to estimate if set back abutments are feasible early in project development when design data is not available. Set back abutments without additional protection may be feasible if all of these conditions occur during events of lesser or equal magnitude to the scour protection check flood.

- Pressure flow does not occur due to water contacting the bridge superstructure, ice jammed against the superstructure, or debris lodged against the superstructure.
- Piers and abutments are located out of areas with flowing water, i.e. water depths less than 3 feet and flow velocities less than 3 feet per second.
- Erosive wave action does not occur.
- The waterway banks or shore are not expected to recede toward the abutment.
- It is unlikely future development within the floodplain during the structure design life will direct the river or stream toward the abutment.

Additional scour protection should be planned for abutments where all of the foregoing conditions do not occur. Set back abutments should not be selected based on preliminary estimates alone. An abutment design must be made to assure the preliminary estimates are valid.

10.12.2.2 Set back Abutment Design Information

The following information is required for set back abutment design.

- The structure design life.
- The site hydraulics, including stages and velocities for floods up to and including the revetment check flood.
- The ordinary high water elevation.
- The proposed foundation type, size, and location.
- The superstructure bottom-of-beam elevation.
- The toe location, soil angle of repose, end erodibility for earthen embankments.

- The geological characteristics of the soils and rocks supporting the foundation or embankment in scour prone areas.
- The streambank or shore recession rate if lateral erosion or scour is a concern. The rate of streambed degradation if vertical scour or erosion is expected.

Often detailed information is not available and assumed conditions are used in the design. All assumed conditions should be biased towards the structure safety. For example, if rock or soil conditions are not known, it should be assumed the supporting material is erodible.

10.12.2.3 Set Back Abutment Design Sequence

The following procedure is used to design the typical set back abutment. This procedure can also be used to evaluate and design urgent and emergency repairs to existing structures. Cross-sections of the structure and streambank are the primary tool for this analysis.

Step 1 - Locate the proposed foundations and end panels in relation to the scour prone areas on the cross-sections. The foundation is considered to be structural items such as footings, pilings, and retaining walls. Locate placed embankments supporting the foundation and end panels.

Step 2 - Locate the ordinary high water on the cross-section. Set back is the distance between the foundation and the ordinary high water, as shown in figures 10-11 and 10-12. Almost always the set back distances will be different at various locations on the structure.

Step 3 - Locate the active scour areas. These are locations where scour has occurred, is occurring, or has the potential to occur during the structure design life, such as:

- scour described in historical data such as maintenance files, bridge inspection reports, flood photos, and other records,
- scour observed during a site inspection,
- scour caused by the presence of the structure in the waterway (contraction, abutment, pier, and pressure flow scour), and
- potential scour due to changes in future conditions.

Changes in future conditions can be:

- removal of objects that restrain waterway movement such as foundations of obsolete structures,
- the potential for floodplain development that would direct the stream or river toward the structure,
- debris or ice collection on a proposed pier or superstructure,
- the change in sea level associated with climate change.

Step 4 - Determine the rate of streambank recession if it will occur. This rate is often based on observed scour. Stream movement is discussed in detail in the Federal Highway Administration Hydraulic Design Series Number 6, “River Engineering for Highway Encroachments.” Procedures to predict movement in meandering streams are in the National Cooperative Highway Research Program Report 533 “Handbook for Predicting Stream Meander Migration.”

Step 5 - The urgent repair process is triggered when observed scour is threatening the foundation or end panel, and the repair can wait until the in-water work period. Often it will take a year or two to design and permit an urgent repair. Draw the threshold for an urgent repair on the cross-sections. This is shown in Figure 10-15a.

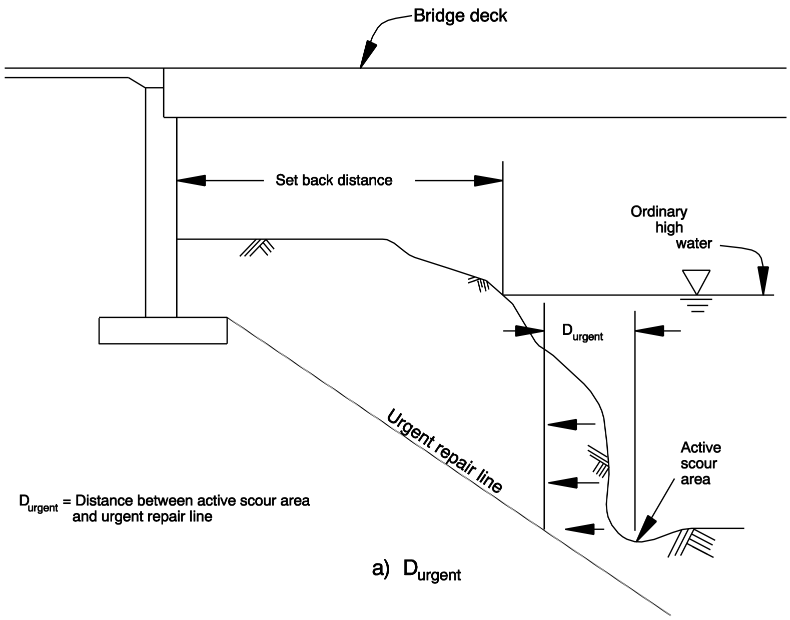
Step 6 - The emergency repair process is triggered when scour is endangering or damaging the foundation or end panel, and the repair must be done immediately to save the structure. Draw the threshold for an emergency repair on the cross-sections. This is shown in Figure 10-15b.

Step 7 - Measure the distances between the scour prone areas and the threshold lines for urgent and emergency repairs. These distances are D_{urgent} and $D_{emergency}$ shown in Figure 10-15.

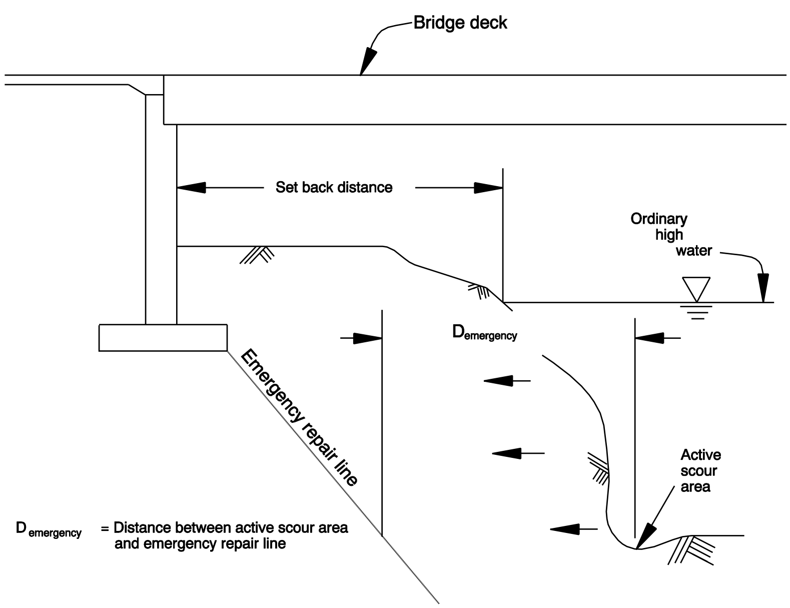
Step 8 - Divide the distances calculated in the previous step by the rates of recession. This will estimate time intervals until an urgent or emergency repair is needed.

Structures should not be designed with urgent or emergency repairs anticipated to occur within their design lives. Any exceptions to this practice should be approved by the people responsible for the structure.

Increased set back distance or scour protection should be used if the setback distance is insufficient to protect an unshielded foundation or end panel, as shown in Figure 10-16. Scour protection methods discussed in this chapter can be used.



a) D_{urgent}



b) $D_{emergency}$

Figure 10-15 D_{urgent} and $D_{emergency}$

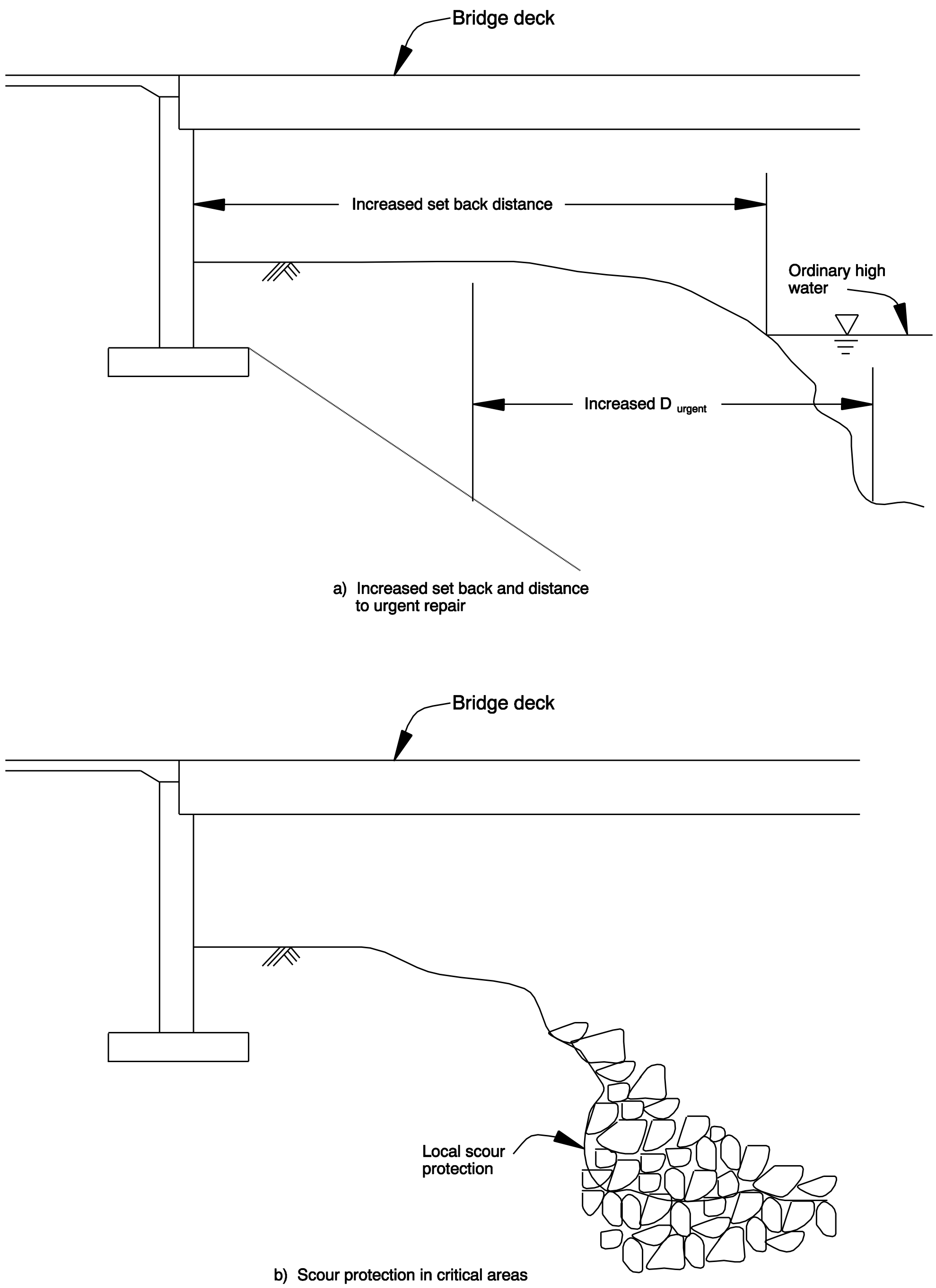


Figure 10-16 Increased Setback and Scour Protection

10.12.3 Biotechnical Protection and Habitat Enhancement

Crossing structures are often located in environmentally sensitive riparian areas. In most of these locations it is desired that the structure have minimal impact on the riparian corridor. This is often accomplished by locating the abutments on the streambanks away from the channel, minimizing the use of revetment in the waterway through use of biotechnical stabilization, and planting native vegetation to enhance the riparian environment. This subsection provides some broad guidelines on biotechnical issues. **Chapter 15** and personnel that specialize in the biological disciplines can provide more specific guidance.

10.12.3.1 Growing Conditions

The root and stem structure provided by healthy plants is essential to biotechnical protection. In general, plants grow best in areas where they are protected from destructive forces, have adequate room to grow, enough sunlight and water, and sufficient soil of the right type. These requirements vary greatly among the plant species. The hydraulic forces caused by extreme winter flows, for example, could destroy a western red cedar and be harmless to a sandbar willow. Often a careful examination of the existing vegetation at the site and their locations can provide insight on the appropriate plants for biological enhancement. Qualified environmental personnel and landscape designers can often provide information on this subject.

These habitat requirements influence the choice of plants at a bridge site. Small bushes and similar plants are usually used within a few yards of the structure. Large plants such as trees should be used with caution. The root systems of many large trees can damage the structure foundation. The damage can be caused by the expansion of the roots as the tree grows, or the forces exerted on the foundation by the tree as it sways in the wind. In other instances, the root system of the plant cannot fully develop in the presence of the nearby structure, and the plant will have inadequate support to resist toppling in the wind. Locating large trees near the bridge deck is also avoided because of clearance problems, and plants of all types have limited success growing under the bridge deck because it blocks both light and precipitation.

10.12.3.2 Site Use and Maintenance

In addition to environmental requirements of plant growth, the vegetation should also be compatible with site use and planned maintenance activities. The mature plants should not cause damage to the structure, utilities, or roadway. In addition, they should not obstruct the driver's vision or prevent maintenance access to the site. Landscape maintenance in most rural and many undeveloped urban locations is limited. Watering, for example, is only practical in landscaped areas with irrigation systems. Another example is pruning. The size and shape of the mature plant should be considered so that periodic trimming or pruning is minimal or not needed.

Local roadway, landscape, and bridge maintenance personnel should be contacted for input before the plants and planting locations are selected, and areas where plants are preferred and discouraged should be discussed. In these discussions, it is important to address that maintenance will be provided after construction.

10.12.3.3 Environmental Concerns

Environmental concerns and agreements can often influence plant choice. Himalayan blackberry and scotch broom are examples of plants that are not specified for riparian enhancements because of environmental concerns about exotic (non-native) species. Environmental personnel should be contacted for input before plants and their locations are selected.

10.12.3.4 Hydraulic Concerns

The presence of vegetation in the floodplain can greatly influence the hydraulic characteristics of the waterway, and these effects are usually the greatest for smaller structures. As a result, the hydraulic aspects of vegetation should be considered before the plants and their locations are selected. These effects should also be considered when reviewing the planting plans developed by others. In both cases, the roughness effects of the vegetation should be included in the hydraulic model of the site, and it should be verified that the hydraulic performance will be satisfactory with the vegetation mature and in place.

In general, plants increase the waterway roughness ("n" value). This can have two affects on hydraulic characteristics, it increases the depth of flow, and it can change the distribution of flow velocity in the stream cross-section. In both cases, the step-backwater software programs commonly used to analyze bridge crossings can also model the hydraulic effects of the plants. A typical procedure to analyze the hydraulic effects of vegetation in the waterway follows. It assumes a step-backwater analysis was made previously to model the bridge without plantings.

Step 1 - Add cross-sections, as necessary, to model the planted area. It is best to plant vegetation in ineffective flow areas.

Step 2 - Subdivide the waterway cross-sections at the planting locations, based on varying Manning's roughness coefficients. A good reference for the roughness coefficients is the Federal Highway Administration Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains, Report No. FHWA-TS-84-204 (FHWA: Washington D.C., 1984).

Step 3 - Execute the program.

Step 4 - Examine the water surface profiles to determine any changes in backwater elevation.

Step 5 - Verify that the backwater with the proposed vegetation is within the acceptable limit. If not, modify the planting plan as needed.

Step 6 - Examine the cross-section velocity distributions to determine any changes in the velocity or the distribution of velocity in the channel.

Step 7 - Verify that any changes in the velocities or velocity distribution do not affect scour depths. If scour depths are affected, include the effects in the scour depth calculations.

Step 8 - Verify that the changes in velocities or the velocity distribution do not require changes in the revetment design. If the revetment design is affected, revise the design or modify the planting plan.

Step 9 - Verify that changes in the velocity distribution do not increase velocities in erosion prone areas such as channel banks. If increased erosion is predicted, determine if the erosion is tolerable. If it is not, protect the erodible area or modify the planting plan.

10.12.3.5 Choice of Biological Enhancement Methods

Plants should not be solely relied upon to provide bridge scour protection. It has been difficult to accurately predict the degree of protection plants provide. This does not mean plants cannot be used. Plants can provide biological enhancement of the bridge site, and they can also be an element in the scour and erosion protection. The enhanced protection should provide the same degree of protection required in Section 10.11. The enhanced protection should not depend on the plants in order to be effective. A backup system should be incorporated to provide adequate protection if the plants do not grow or if they die. Recent experience with biotechnical bank protection indicates that unforeseen damage from animals, insects, or diseases can decimate the plantings.

The most common method of biological enhancement in current use at bridge sites is to plant desirable trees and shrubs in suitable locations, and not to rely on them for scour and erosion protection. Often these plantings include willows in the riprap revetment. In most cases, bush size willows are used in areas where their roots will receive adequate moisture. Often other bushy plants are used higher on the embankments, such as red-osier dogwoods and snowberries. Bush alders of different varieties are also used in riparian habitats.

Willows and dogwoods are often planted as cuttings inserted into soil placed over and within the riprap. These cuttings are most successful when:

- they are cut and inserted during their dormant season (usually December and January),
- the lower end of the cutting is in an area with moist or saturated soil,
- 75 to 95 percent of the cutting length is buried, and
- the correct polarity is used. In other words, the top end of the cutting on the host plant is the top end of the installed cutting.

10.12.4 Standard Revetment Designs

Revetment protection is designed for each individual structure at each site, and the typical procedure is to modify a standard revetment design to fit the application. This section includes standard revetment designs for spillthrough abutments, vertical abutments, mechanically stabilized earth wall (MSE wall) abutments, and interior bents. Additional information about revetment, including gradation and properties, is discussed in **Chapter 15**. Biological enhancement of revetment is discussed in the previous subsection.

Revetment or equivalent protection should be used if there is flowing water against the abutment during events up to and including the revetment check flood. Flowing water is considered to be water more than 3 feet deep or having flow velocities in excess of 3 feet per second.

Estimated scour depths and elevations are an essential tool for revetment design. The scour types used in revetment design are discussed in Section 10.13.

10.12.4.1 Spillthrough Abutment Revetment

The objective of this revetment is to protect the abutment fill from scour damage during events up to and including the revetment design flood. Typical cross-sections of spillthrough abutment revetment are shown in Figures 10-7 and 10-8, and details are shown in Figure 10-17. The rock size is determined by methods in **Chapter 15** using the highest velocity though the bridge opening. This velocity usually occurs at the downstream face. The maximum elevation of the revetment protection is determined by the highest water surface elevation in the bridge opening plus 1 foot, and this is often the energy grade line elevation at the upstream faces of the abutments. An exception occurs if water is ponded at the upstream faces of the abutments during the revetment design flood. In this case, the elevation of the energy grade line at the approach section should be used. Wave action on the ponded water should also be considered, if present, and guidelines are included in **Chapter 15** for sizing rock at inland locations. The rock should extend upward on the abutment face as needed to prevent wave damage.

The revetment should wrap around the sides of the bridge abutments and protect the fresh embankment slopes below the revetment design flood elevation. A toe trench may be needed on the upstream side of the bridge abutments if there is significant flow contraction. The revetment should extend along the embankments a distance sufficient to protect the bridge end panels. This distance can be approximated by the guidelines in Subsection 10.3.8. An example of an end panel undermined by wave action from a flooding river is shown in Figure 10-18. This end panel was supported by stone embankment material with little scour resistance. A layer of revetment would have prevented this damage.

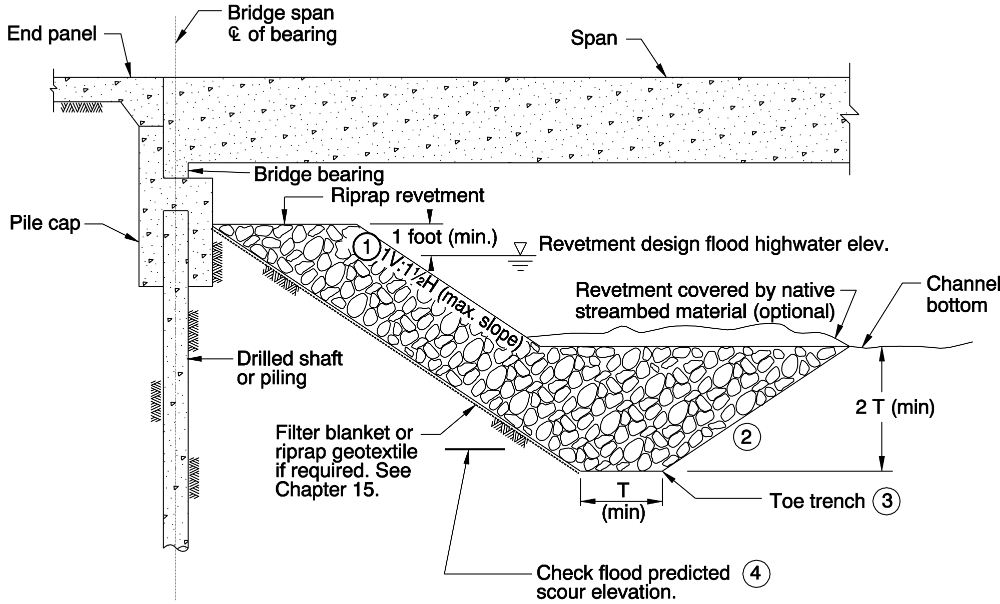


TABLE		
Riprap Class	T (feet)	Filter Blanket or Riprap Geotextile
50	1.0	No
100	1.5	No
200	2.0	Yes
700	3.0	Yes
2000	4.0	Yes

Notes:

1. The maximum slope of an abutment with riprap protection is 1 unit vertical to 1½ units horizontal. Steeper abutment faces require special scour protection such as paved end slopes or rock filled gabion baskets.

2. The maximum slope of the inside of the toe trench is the angle of repose of the streambed material. A flatter slope may be needed in con-cohesive materials such as saturated sands or gravels. A 1 unit horizontal to 1 unit vertical slope can be used for quantity estimates if site specific information is not available.

3. The bottom of the riprap filled toe trench must be below the lower of:
   - the elevation of predicted scour during the check flood,
   - the elevation of the channel thalweg minus the toe trench depth.

4. Predicted total scour based on Subsection 10.13.7.

Figure 10-17 Spillthrough Abutment Revetment Details

Figure 10-18 Undermined End Panel

10.12.4.2 Pile or Shaft Supported Vertical Abutment

Vertical abutments can be supported by piling or drilled shafts. An essential part of this design is to extend the bottom of the pile cap down to an elevation lower than the bottom of the adjacent riprap filled toe trench and the check flood scour elevation, as shown in Figure 10-19. This extended cap often requires structural modifications to withstand the lateral earth pressure behind the abutment face.

The embankment wrapping around the edges of the wingwalls should be protected from erosion if significant wave action occurs. Guidelines for sizing riprap to resist wave action are in **Chapter 15**. The riprap should extend upward to a high enough elevation to prevent wave damage.

10.12.4.3 Footing Supported Vertical Abutment and MSE Wall Abutment Revetment

The primary objective of the revetment on these abutments is to protect the footing or bottom edge of the facing panel from scour damage during events up to and including the revetment check flood. Typical revetment cross-sections for footing supported vertical and MSE wall abutments are shown in Figures 10-20 and 10-21, respectively. Like spillthrough abutments, the rock size is determined by methods in **Chapter 15** using the highest velocity though the bridge opening, which is usually at the downstream face. The maximum elevation of the revetment protection is determined by the highest water surface elevation in the bridge opening, and this is often the energy grade line elevation at the upstream face of the abutment. An exception occurs if water is ponded at the upstream faces of the abutments during the revetment design flood. In this case, the elevation of the energy grade line at the approach section should be used.

The embankment wrapping around the edges of the wingwalls should be protected from erosion if significant flow velocity or wave action occurs. Guidelines for sizing riprap to resist wave action are in **Chapter 15**. The riprap should extend upward to a high enough elevation to prevent wave damage.

Note: The MSE wall abutment is a unique design with limited use in Oregon. Almost all of the abutments have been constructed away from the water on the streambank with little or no in-water work. In addition, most of the bridge spans and supporting footings have been located above the elevation of the check flood. Caution should be used when considering MSE wall abutments for sites where the abutment will need to be built in the water or where floodwaters are expected to contact the footings or the bottom of the span.

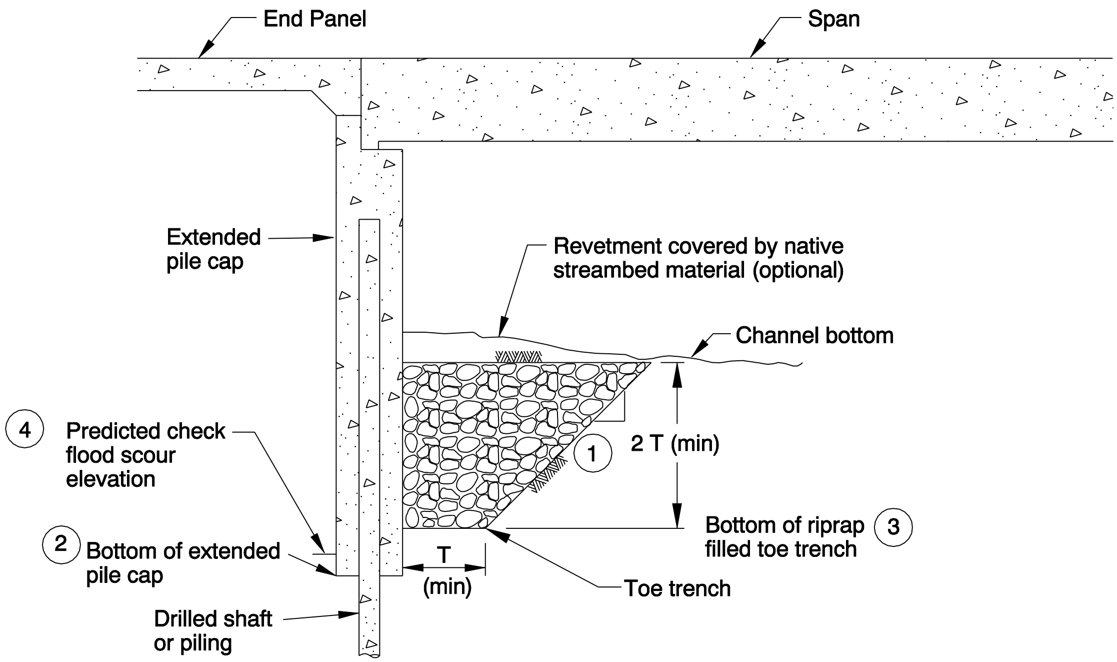


TABLE	
Riprap Class	T (feet)
50	1.0
100	1.5
200	2.0
700	3.0
2000	4.0

Notes:

1. The maximum slope of the inside of the toe trench is the angle of the repose of the streambed material. A flatter slope may be needed in non-cohesive materials such as saturated sands or gravels. A 1 unit horizontal to 1 unit vertical slope can be used for quantity estimates if site specific information is not available.

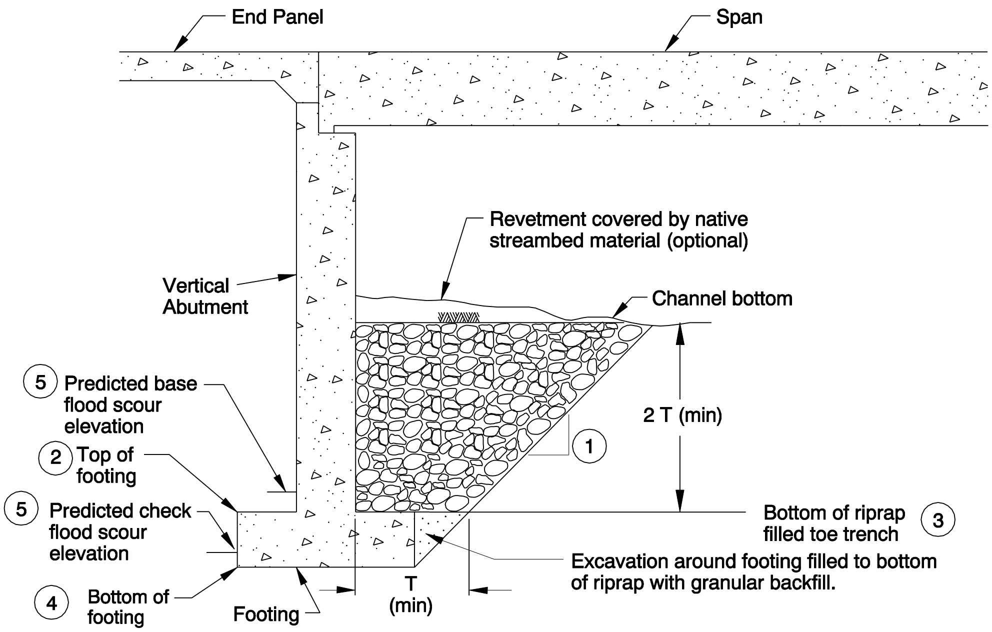
2. The bottom of the extended pile cap must be below the bottom of the riprap filled toe trench.

3. The bottom of the riprap filled toe trench must be below the lower of:
   - the elevation of predicted scour during the check flood, or
   - the elevation of the channel thalweg minus the toe trench depth.

   Higher elevations may be permitted or the revetment eliminated if the bottom of the extended pile cap is keyed into solid rock.

4. Predicted total scour based on Subsection 10.13.7.

Figure 10-19 Pile or Shaft Supported Vertical Abutment Revetment Details



TABLE

Riprap Class	T (feet)
50	1.0
100	1.5
200	2.0
700	3.0
2000	4.0

Notes:

1. The maximum slope of the inside of the toe trench is the angle of repose of the streambed material. A flatter slope may be needed in non-cohesive materials such as saturated sands or gravels. A 1 unit horizontal to 1 unit vertical slope can be used for quantity estimates if site specific information is not available.

2. The top of the footing must be below the lower of, the bottom of the riprap filled toe trench, or the base flood scour elevation.

3. The bottom of the riprap filled toe trench must be below the lower of:
   - the elevation of predicted scour during the base (100-year) flood, or
   - the elevation of the channel thalweg minus the toe trench depth.

   Higher elevations may be permitted or the revetment eliminated, if the footings are keyed into non-erodible rock.

4. The bottom of the footing must be below the elevation of predicted scour during the check flood or 6 feet below the elevation of the thalweg, whichever is lower.

5. Predicted total scour based on Subsection 10.13.7.

Figure 10-20 Footing Supported Vertical Abutment Revetment Details

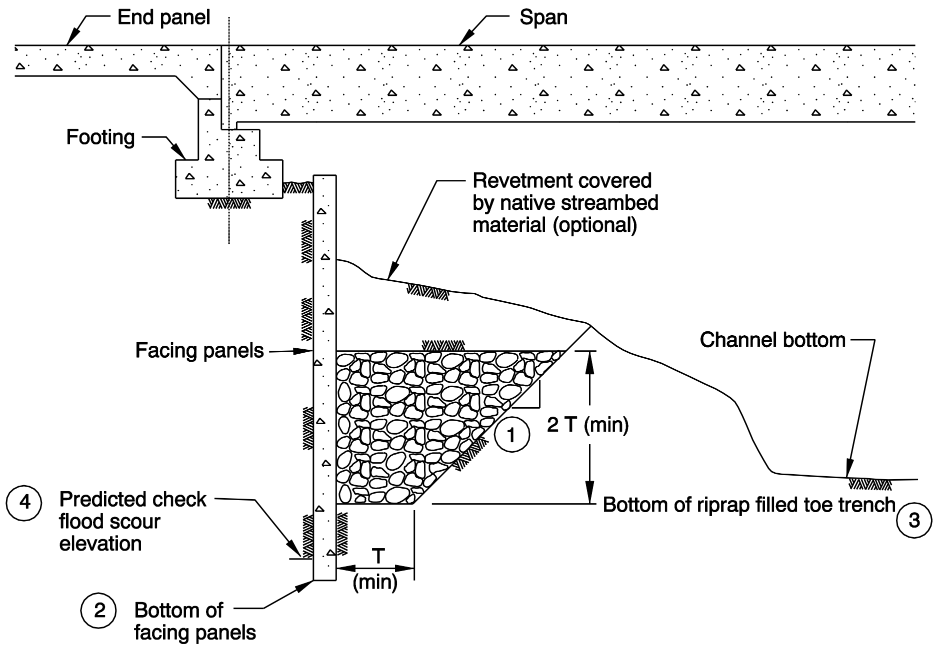


TABLE	
Riprap Class	T (feet)
50	1.0
100	1.5
200	2.0
700	3.0
2000	4.0

1. The maximum slope of the inside of the toe trench is the angle of repose of the streambed material. A flatter slope may be needed in non-cohesive materials such as saturated sands or gravels. A 1 unit horizontal to 1 unit vertical slope can be used for quantity estimates if site specific information is not available.

2. The bottoms of the facing panels must be below the elevation of the bottom of the riprap fill toe trench.

3. The bottom of the riprap filled toe trench must be below the lower of:
   - the elevation of predicted scour during the check flood, or
   - the elevation of the channel thalweg minus the toe trench depth.

   Higher elevations may be permitted, or the revetment eliminated, if the bottoms of the facing panels are keyed into non-erodible rock.

4. Predicted total scour based on Subsection 10.13.7.

Figure 10-21 MSE Wall Abutment Revetment Details

10.12.4.4 Interior Bent (Pier) Revetment

The interior bent of a bridge, often called a pier, is supported by the underlaying soil or rock, and scour of these underlaying materials will reduce the strength of the bent. The desired practice is to embed the foundation sufficiently deep to have adequate strength after estimated scour occurs. In some cases, this may not be practical, and it may be necessary to protect the foundation with riprap, articulated block mats, or other means. This is most often done when protecting existing structures. This should not be done for new structures unless approved by the ODOT Region Technical Center hydraulics staff. Riprap, in most cases, is considered to be temporary scour protection for piers.

Footing bases on new bridges are designed to be below the estimated check flood scour elevation, as described in Subsection 10.12.1. Debris, if it is anticipated to be present, should be considered in the scour calculations. Many designers also include revetment around the pier as an additional countermeasure against unanticipated scour. A typical detail for pier revetment is shown in Figure 10-22. Pier revetment size is based on the approach flow velocity upstream from the pier. Methods to calculate pier riprap size are in **Chapter 15**.

10.13 Scour

Scour at bridges is a complex phenomenon, and it often has multiple and interrelated causes. In general, there are two categories of scour. One category is general scour, and this is a change in elevation over most or all of the stream bottom. The other category is local scour, and it occurs at specific locations in the waterway opening. The major causes of scour are:

- long-term changes in channel profile or location, typically called "aggradation" (raising), "degradation" (lowering), or lateral shifting "plan form changes" of the channel bottom,
- general scour due to the contraction of flow as the discharge passes through the bridge opening, called "contraction scour,"
- local scour adjacent to the faces of the abutments, called "abutment scour,"
- local scour around the interior bents, called "pier scour," and
- other types of pier scour, such as the scour caused by a buildup of debris on a nearby part of the structure.

The primary purpose for scour calculations is bridge foundation and scour protection design. The scour types to be calculated are discussed in section 10.12. Each of the listed scour types is described in more detail in the remainder of this section. Bridge scour and calculation methods are described in detail in the Federal Highway Administration (FHWA) Hydraulic Engineering Circular No. 18 Evaluating Scour at Bridges.

Notes:

1. The maximum slope of the outside of the revetment trench is the angle of repose of the streambed material. A flatter slope may be needed in non-cohesive materials such as saturated sands or gravels. A 1 unit horizontal to 1 unit vertical slope can be used for quantity estimates if site specific information is not available.

2. The elevation of the top of the footing should not be higher than bottom of the riprap filled trench.

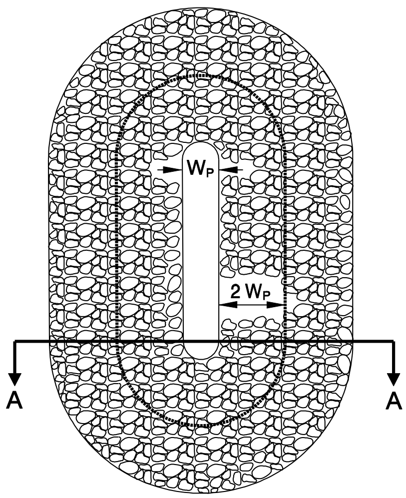
3. The bottom of the riprap filled toe trench should be lower than:
   - the predicted base (100-year) scour elevation, or
   - the channel thalweg elevation minus the toe trench depth.

   Higher elevations may be permitted or the revetment eliminated if the footing is keyed into solid rock.

4. The top of the footing should be below the lower of, the base (100-year) flood scour elevation, or the bottom of the riprap filled toe trench.

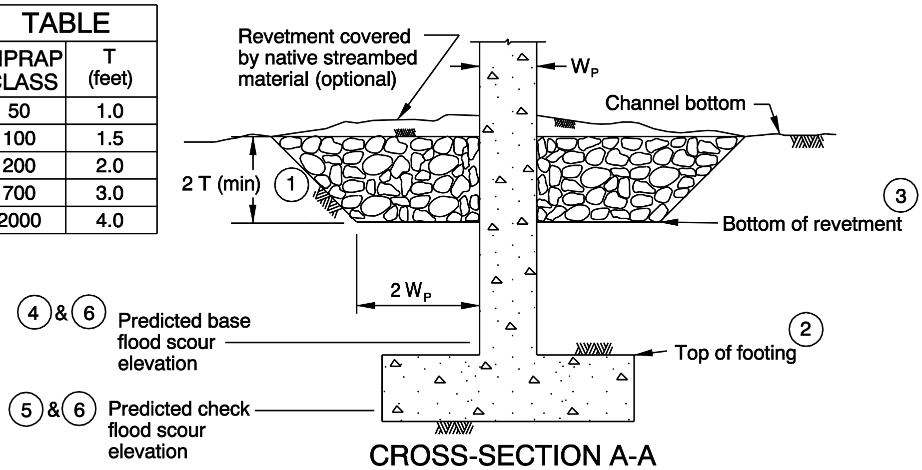
5. The bottom of footing elevation should be below the predicted scour elevation during the check flood scour elevation or 6 feet below the elevation of the thalweg, whichever is lowest.

6. Predicted total scour based on Subsection 10.13.7.



PLAN

TABLE	
RIPRAP CLASS	T (feet)
50	1.0
100	1.5
200	2.0
700	3.0
2000	4.0



CROSS-SECTION A-A

W_P = Pier Width

Revetment around footing supported interior bent shown.
Revetment around other bent types are similar.

Figure 10-22 Interior Bent Revetment Details

10.13.1 Discharges and Tailwater Effects Used in Scour Analyses

The scour elevations are reported for the scour design and check discharges. The scour elevations are calculated for the combinations of discharges and tailwater elevations that create the deepest scour depths in the bridge opening. If changes to the hydrology, structure, waterway, roadway, or other items are expected within the design life of the structure that may cause deeper scour depths, scour elevations based on these deeper depths should be reported. Guidance on selecting discharges and tailwater effects are included in Sections 10.8 and 10.9, respectively.

10.13.2 Aggradation and Degradation and Plan Form Changes

Aggradation and degradation are long-term channel profile changes caused by the buildup or removal, respectively, of bed material on the channel bottom. These changes are usually considered to be permanent, and they are often caused by changes in the stream discharge, the amount or size of the moving bed load, or a change in the energy grade line profile. These channel profile changes are especially critical in the design of fish passage culverts, and they are discussed in **Chapter 9**. A useful reference on this subject is the FHWA Hydraulic Engineering Circular No. 20 Stream Stability at Highway Structures.

Plan form changes are changes in location of the channel banks in the horizontal plane. Plan form changes of concern to hydraulic designers are usually one or more of two types, meander migration or channel widening.

Meander migration is a shifting in the lateral and downstream direction the lowest point of the channel cross-section, often called the channel "thalweg." This lateral instability can occur on streams that have meandering paths. It can also occur on streams with straight channels, such as previously meandering streams that have been straightened by channel changes.

Channel widening occurs when the banks scour away and the channel widens. This widening can be caused by many sources, such as flow from a large flood, an increase in the sediment discharge, livestock or land use practices eliminating bank vegetation, or many other sources.
Plan form changes are difficult to predict on a long-term basis. As a result, in the hydraulic design it should be assumed that the channel thalweg or banks can shift laterally unless they are physically constrained.

Considerable survey data is available for estimating changes in waterway cross-section and profile at ODOT bridges and local agency bridges that are included in the ODOT bridge inspection program. This information is helpful when estimating aggradation or degradation. Regardless of the source of the survey data, all elevations should be converted to a common datum before they are compared. Survey data is available from the following sources:

- waterway profiles and cross-sections are almost always made before a bridge installation or replacement, and this data is in the bridge construction files (ODOT Bridge Section) or the hydraulic study files (ODOT Geo-Environmental Section's Engineering and Asset Management Unit),

- waterway profiles and cross-sections are often made before a detailed scour study, and this data is in the scour reports (ODOT Bridge Section), and
- waterway cross-sections are made periodically during bridge inspections, and this data is in the bridge inspection reports (ODOT Bridge Section or ODOT Region Bridge Inspectors).

Aggradation should be estimated at bridge sites where it can be predicted, and it should not be included in the predicted total scour. The estimated channel elevation after aggradation should be mentioned separately in the report with a cautionary note to the structural designer. The bridge waterway opening may need to be enlarged to accommodate the anticipated aggradation.

Degradation, if it can be predicted, should be included as a component of the total predicted scour.

10.13.3 Contraction Scour

Contraction scour is general scour caused by increased flow velocities within the bridge opening in comparison to the slower velocities in the upstream and downstream waterway. Contraction scour can occur in the bridge opening due to the constriction caused by the bridge abutments and/or internal bents. This type of scour can also be caused by contraction due to constrictions in the natural channel. This is not uncommon because bridges are often placed across natural constrictions.

Equations in HEC-18 are used to calculate contraction scour in most applications, and detailed instructions on their use is included in the publication. One equation, the clearwater scour equation, is used where there is no live-bed movement, such as:

- the transport of bed material from upstream of the contraction is small in quantity or composed of fine material that washes through the contraction in suspension, or
- coarse sediments are present that may armor the channel bottom and limit the depth of live-bed contraction scour.

The other equation, the live-bed scour equation, is used where substantial amounts of bed material are washed into and out of the contracted area during floods, and the clearwater equation is not applicable.

Both equations should be used to determine the potential contraction scour and the lowest elevation reported at sites where the scour type cannot be predicted with certainty, or locations where both types of scour may occur.

ODOT practice is to assume the channel thalweg can change location and the predicted contraction scour depth can occur anywhere in the bridge opening. Exceptions to this general rule occur at sites where the waterway has a fixed alignment and movement is not possible. Examples are canals, channels incised in solid rock, channels restrained by spur dikes or levees, etc.

10.13.4 Abutment Scour

Abutment scour is local scour that occurs at the faces of abutments that project into the waterway or floodplain. The obstruction causes flow vortexes to form at the toe of the abutment, and this turbulent flow scours away the underlaying bed material. At present, equations to predict abutment scour are mainly based on laboratory data and they tend to predict conservative scour depths. In other words, it is likely the actual abutment scour will be less than the predicted value, and unlikely the abutment scour will be greater than the prediction.

ODOT recommended practice is to protect the toe of the abutment with revetment in lieu of including abutment scour in the predicted scour elevation. An exception occurs when revetment protection is omitted from the face of the abutment and the toe of the abutment is not solidly keyed into non-erodible rock. In this case, abutment scour is calculated and included in the predicted total scour elevation.

10.13.5 Pier Scour

Pier scour is a form of local scour that occurs around interior bents that are exposed to flow. Methods in HEC-18 are recommended to calculate pier scour. The calculation of pier scour should consider the effects of varying angles of flow attack, possible movement of the channel thalweg, and if present, the effects of pressure flow and debris accumulation.

ODOT practice is to assume the channel thalweg can change location within the bridge opening, and the varying thalweg locations can change the directions from which the flow approaches the pier. The flow direction can influence the predicted scour depth, and the maximum pier scour that results from the various possible flow directions should be reported. In addition, it is assumed that due to a moving thalweg, the deepest predicted pier scour depth can occur at any pier in the bridge opening. Exceptions to this general rule occur at sites where the waterway has a fixed alignment and movement is not possible. Examples are canals, channels incised in solid rock, channels restrained by spur dikes or levees, etc.

The pier scour depth can also be influenced by occurrence of pressure flow during a flood event. Pressure flow scour occurs when the water surface elevation at the upstream face of the bridge is greater than or equal to the elevation of the low chord of the bridge superstructure. Pressure flow causes additional pier scour when the water that collects at the upstream face of the bridge and plunges downward into the bridge opening. The additional pier scour due to pressure flow can be estimated by the procedures in HEC-18.

10.13.6 Pier Scour Due to Other Causes

Increased scour can occur at piers due to the collection of debris on the piers or superstructure, the presence of ice, or numerous other causes. Procedures in HEC-18 can be used to estimate pier scour due to debris. It is necessary to estimate the size of the debris buildup in order to use this procedure. Maintenance records, bridge inspection reports, photographs of floods, and recollections of witnesses can all be used to estimate the debris buildup during large floods.

10.13.7 Total Scour

Total scour elevation at a structure where the abutment toes are protected from scour, and the scour protection is designed to withstand the check flood:

- for piers, the thalweg elevation – pier scour – contraction scour – channel degradation, and
- for abutments, the thalweg elevation – contraction scour – channel degradation.

Total scour elevation at a structure where the abutment toes are not protected from scour, or the scour protection is inadequate to withstand the check flood:

- for piers, the thalweg elevation – pier scour – contraction scour – channel degradation,
- for piers within the abutment scour zone, or abutments having piers within the abutment scour zone, the thalweg elevation – abutment scour – pier scour – contraction scour – channel degradation,
- for abutments, the thalweg elevation – abutment scour - contraction scour – channel degradation, and
- for abutments having piers within the abutment scour zone, the thalweg elevation – abutment scour – pier scour – contraction scour – channel degradation.

The abutment scour zone is an anticipated scour cavity adjacent to the abutment toe. It extends downward to the total scour elevation. The top width of the cavity is twice the total scour depth. Pressure flow and debris components should be included in the scour calculations if they occur.

Not all forms of scour may be present at all crossings. Scour depths may be limited by the presence of non-erodible rock. The erodibility of the rock can be determined by the foundation or geotechnical designer.

10.14 Temporary Crossings

Temporary crossings such as construction bridges and detour bridges are often used during the installation, replacement, and rehabilitation of structures. These bridges are often designed by the contractors or subcontractors who do the construction of the permanent crossing. The locations and lengths of these structures are often governed by available right-of-way or construction easements, environmental considerations, navigational requirements, as well as hydraulic needs.

The typical design information provided for a temporary structure by a hydraulic designer includes the predicted low flow discharge during the construction season, the predicted 5-year flood discharge for structures in place through the flood season, and flood elevations. Instructions for calculating the discharges are included in **Chapter 7** and flood elevations are often determined from the hydraulic model for the permanent structure.

Temporary structures across floodways subject to Federal Emergency Management Agency requirements need special consideration. These temporary structures must meet additional hydraulic requirements if they are in place across the floodway between November 1 and May 31. The Region Technical Center hydraulics staff should be contacted for assistance as soon as possible during the design process if a temporary structure will be needed across a floodway during the aforementioned period.