ERRATA

Dear Customer:

Recently, we were made aware of some technical revisions that need to be applied to the *AASHTO LRFD Bridge Design Specifications,* Ninth Edition. Please scroll down to see the full erratum.

In the event that you need to download this file again, please download from AASHTO's online bookstore at:

https://downloads.transportation.org/LRFDBDS-9-Errata.pdf

Then, please replace the existing pages with the corrected pages to ensure that your edition is both accurate and current.
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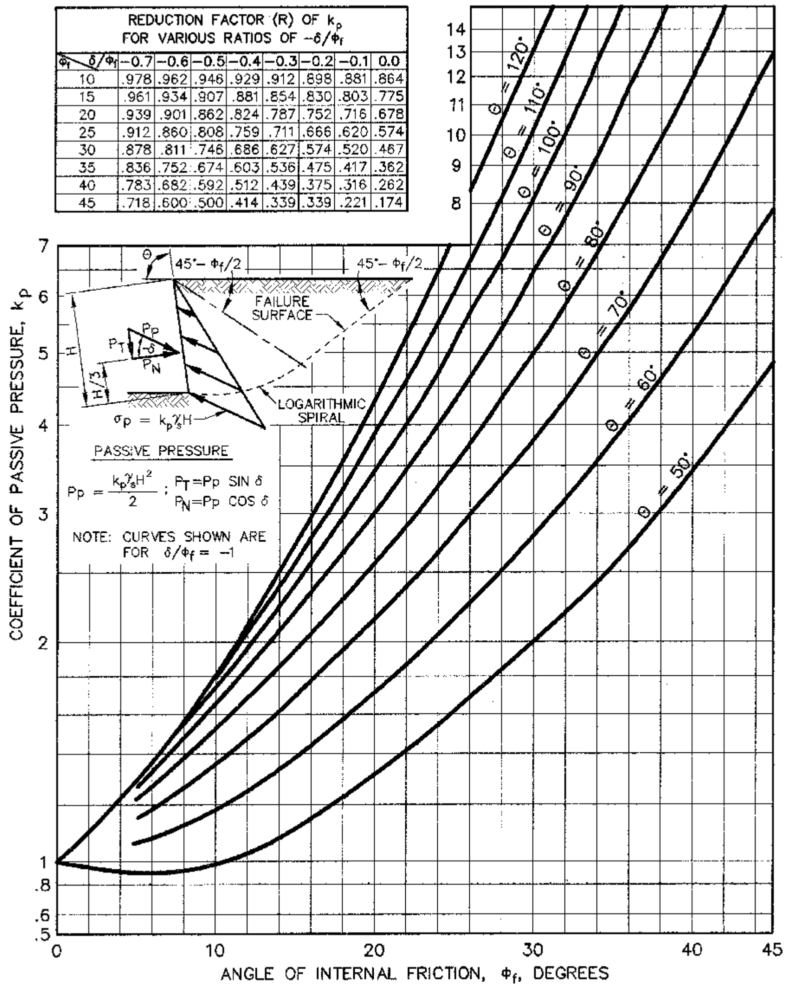
AASHTO staff sincerely apologizes for any inconvenience to our readers.

AASHTO Publications Staff
November 2021

Summary of Errata for LRFDBDS-9, November 2021

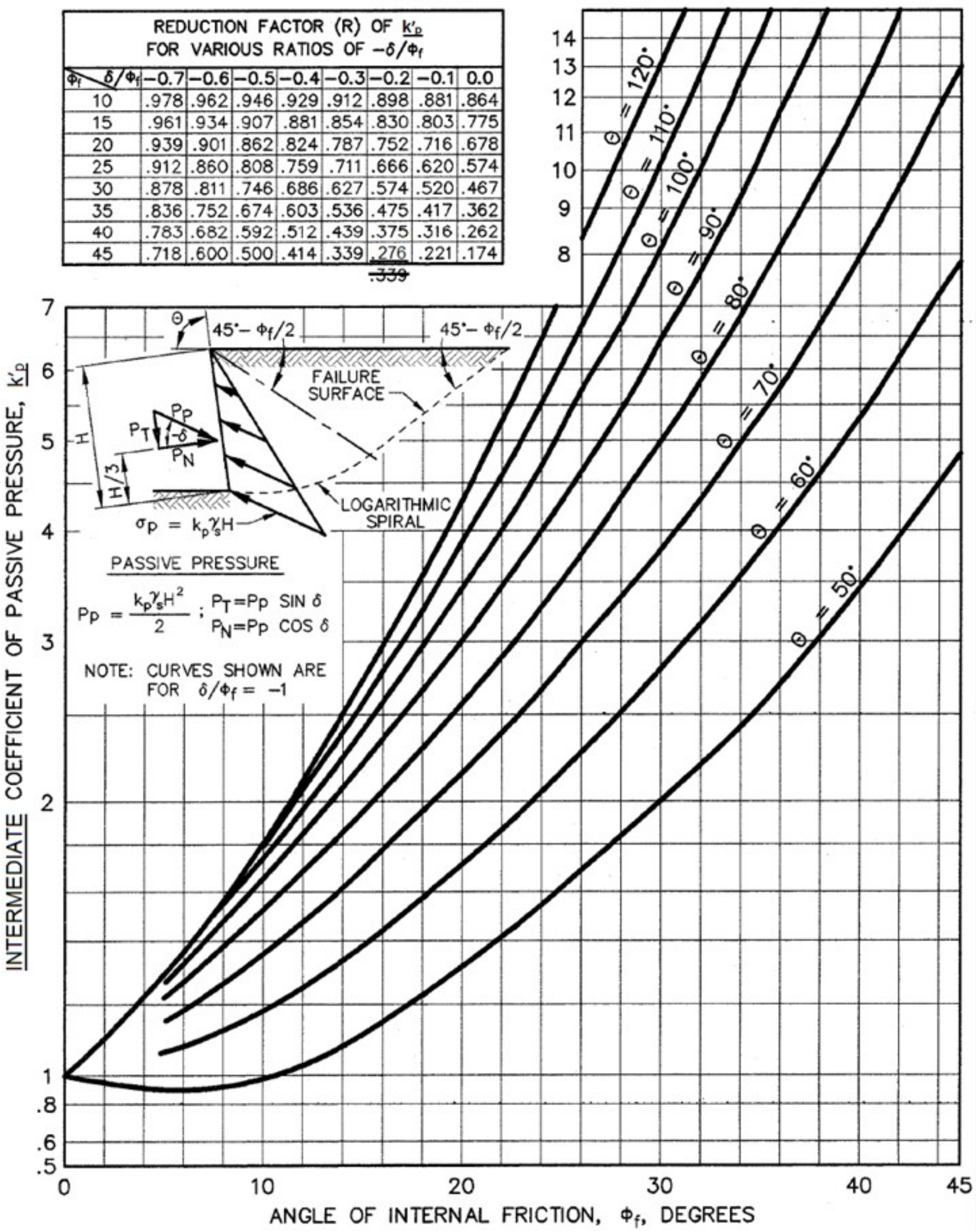
Section	Page Number(s)	Original Image
3	3-126	**Figure 3.11.5.4-1—Computational Procedures for Passive Earth Pressures for Vertical and Sloping Walls with Horizontal Backfill (U.S. Department of the Navy, 1982a)**:



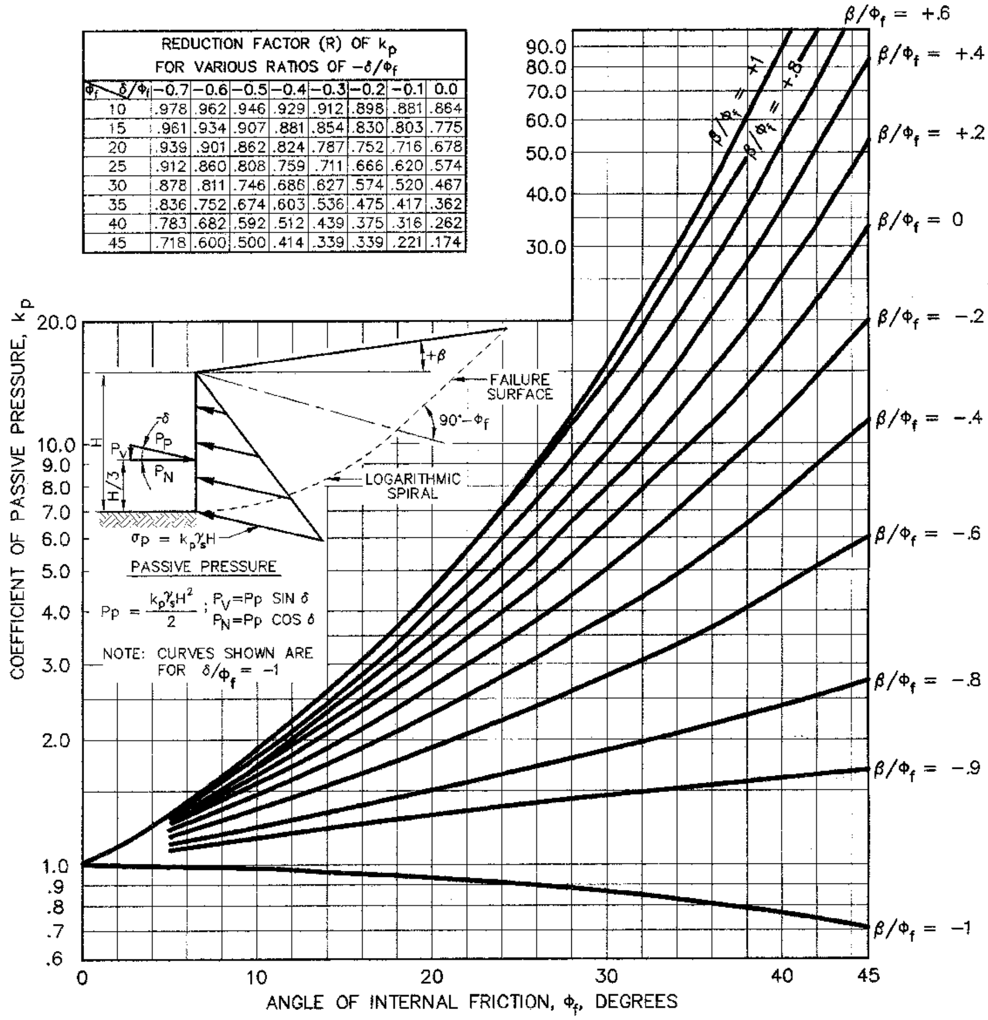
Corrected Image

REDUCTION FACTOR (R) OF $\underline{k'_p}$
FOR VARIOUS RATIOS OF $-\delta/\phi_f$

ϕ_f \ δ/ϕ_f	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276 ~~.339~~	.221	.174



Section	Page Number	Original Image
3	3-127	**Figure 3.11.5.4-2—Computational Procedures for Passive Earth Pressures for Vertical Wall with Sloping Backfill (U.S. Department of the Navy, 1982a)**:



REDUCTION FACTOR (R) OF k_p FOR VARIOUS RATIOS OF $-\delta/\phi_f$

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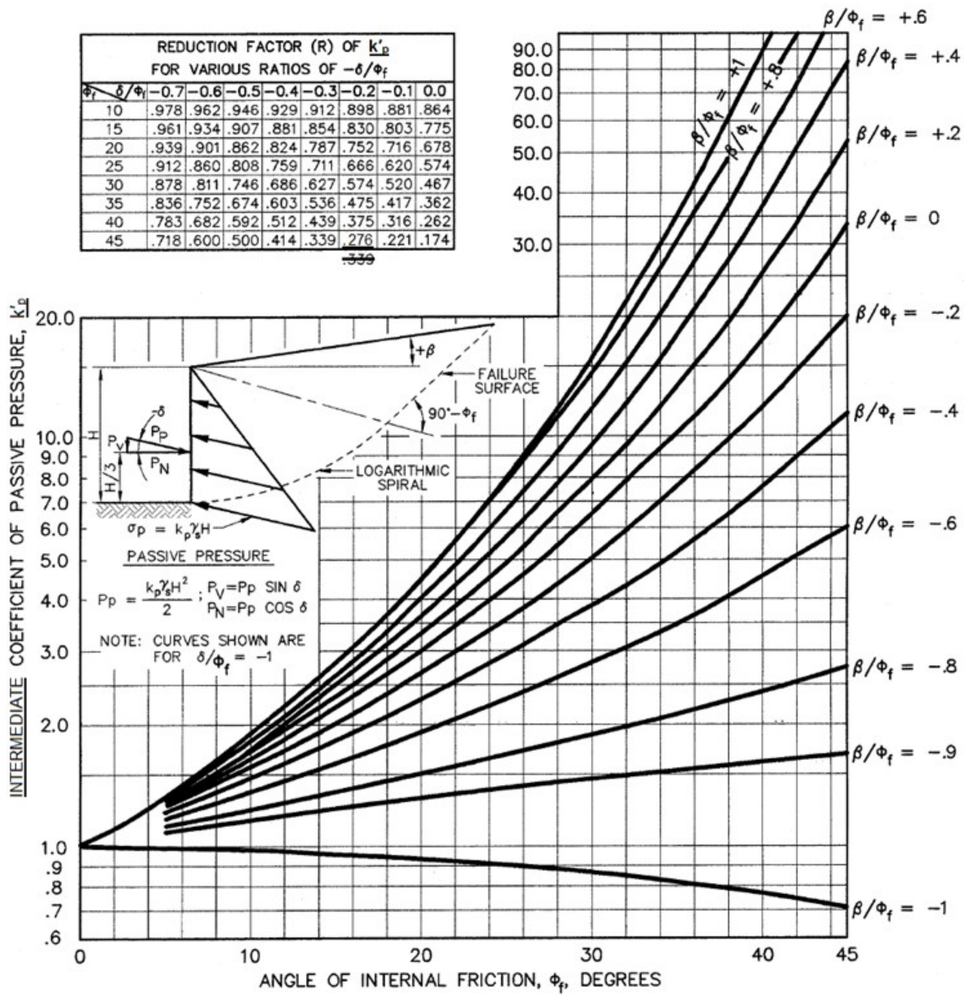
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.339	.221	.174

Corrected Image		
		See chart and reduction-factor table below



REDUCTION FACTOR (R) OF k'_p FOR VARIOUS RATIOS OF $-\delta/\Phi_f$

Φ_f \ δ/Φ_f	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	0.0
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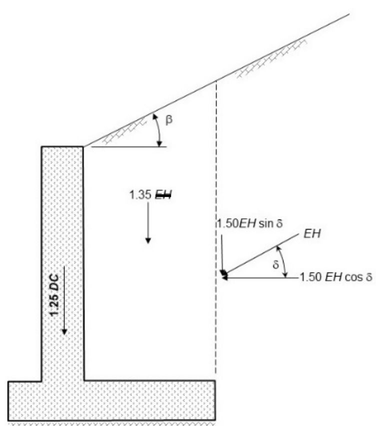
45	.718	.600	.500	.414	.339	.276 ~~.339~~	.221	.174

Section	Page Number	Original Text	Corrected Text
6	6-308–309	First paragraph of **6.13.6.1.3c—Web Splices**: “As a minimum, web splice plates and their connections shall be designed at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web splice plates, $V_r = \phi_v V_n$, on either side of the splice determined according to the provisions of Article 6.10.9 or 6.11.9, as applicable.” Second paragraph of **6.13.6.1.3c—Web Splices:** “Should the moment resistance provided by the flanges at the point of splice, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state, the web splice plates and their connections shall instead be designed for a design web force taken equal to the vector sum of the smaller factored shear resistance and a horizontal force in the web that provides the necessary moment resistance in conjunction with the flanges.”	First paragraph of **6.13.6.1.3c—Web Splices**: “As a minimum, web splice plates and their connections shall be designed at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web, $V_r = \phi_v V_n$, on either side of the splice determined according to the provisions of Article 6.10.9 or 6.11.9, as applicable.” Second paragraph of **6.13.6.1.3c—Web Splices:** “Should the moment resistance provided by the flanges at the point of splice, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state, the web splice connections shall instead be designed for a design web force taken equal to the vector sum of the smaller factored shear resistance and a horizontal force in the web that provides the necessary moment resistance in conjunction with the flanges.” **Note: This change applies only to the PDF edition, and brings it in line with the print edition.**

Section	Page Number	Original Image
11	11-13	**Figure C11.5.6-1—Typical Application of Load Factors for Bearing Resistance** 

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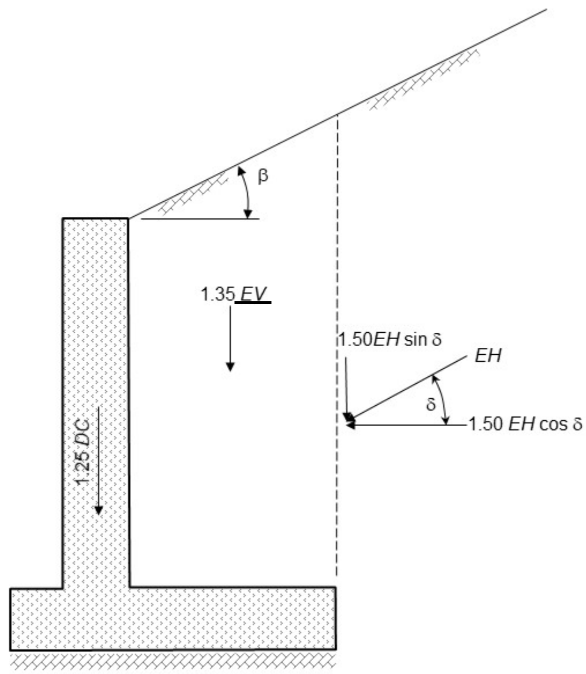
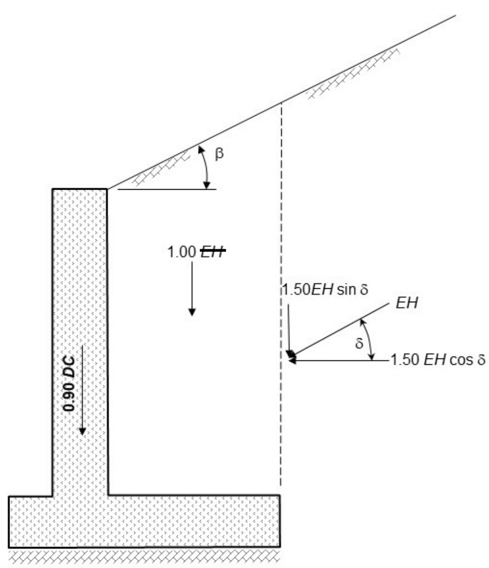
		
Section	**Page Number**	**Original Image**
11	11-13	**Figure C11.5.6-2—Typical Application of Load Factors for Sliding and Eccentricity** 

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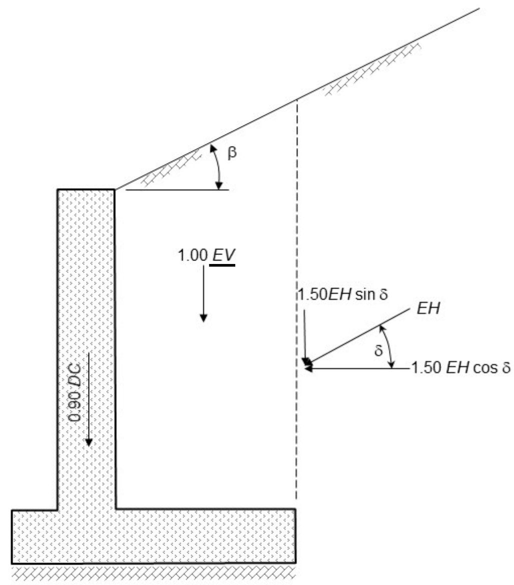
		

Section	Page Number	Original Text	Corrected Text
12	12-22		**Table 12.6.6.3-1—Minimum Cover:** The note in the table has been moved to eliminate potential confusion regarding concrete pipe measurements.
12	12-95	**12.14.5.7—Crack Control**: “The provisions of Article 5.6.4 for buried structures shall apply.”	**12.14.5.7—Crack Control**: “The provisions of Article 5.6.7 for buried structures shall apply.”

$$p_p = k_p \gamma_s z + 2c\sqrt{k_p} \quad (3.11.5.4\text{-}1)$$

where:

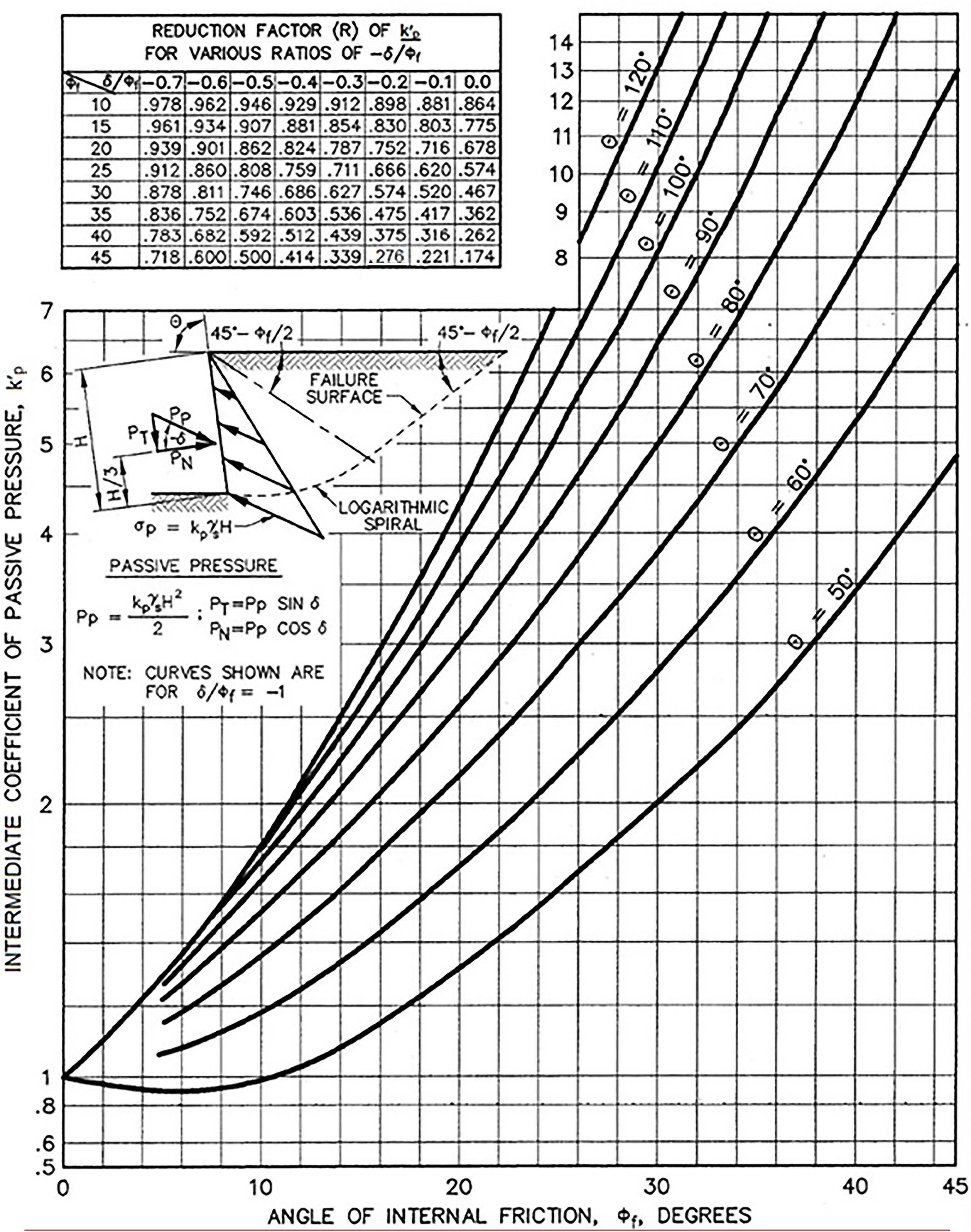
p_p = passive lateral earth pressure (ksf)
γ_s = unit weight of soil (kcf)
z = depth below surface of soil (ft)
c = soil cohesion (ksf)
k_p = coefficient of passive lateral earth pressure specified in Figures 3.11.5.4-1 and 3.11.5.4-2, as appropriate

$$k_p = R\,k'_p \quad (3.11.5.4\text{-}2)$$

where:

k_p = coefficient of passive lateral earth pressure
R = reduction factor for coefficient of passive lateral earth pressure for various ratios of $-\delta/\phi_f$
k'_p = intermediate coefficient of passive lateral earth pressure determined from Figures 3.11.5.4-1 and 3.11.5.4-2

Wedge solutions are inaccurate and unconservative for larger values of wall friction angle.



REDUCTION FACTOR (R) OF k'_p FOR VARIOUS RATIOS OF $-\delta/\phi_f$

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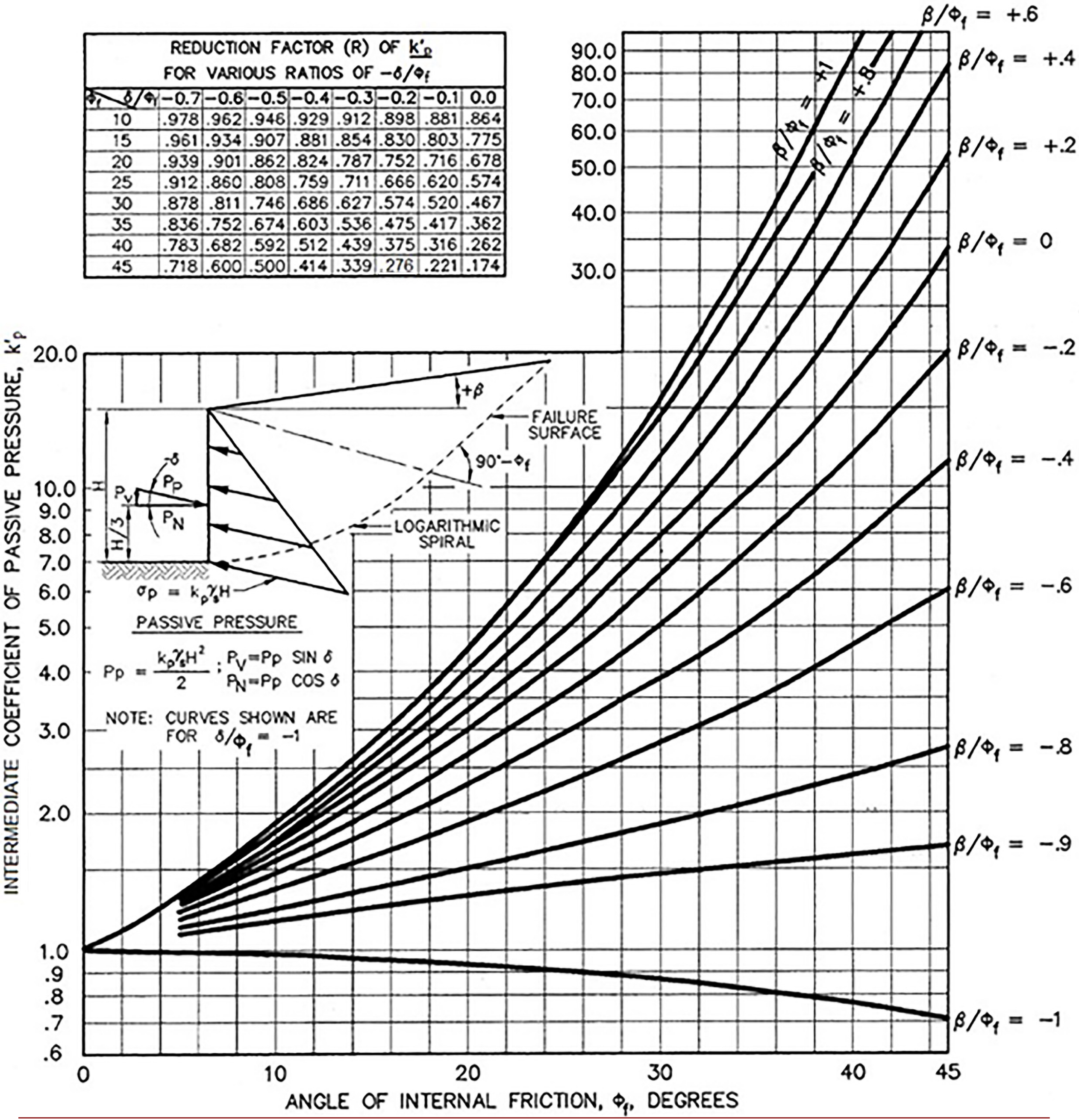
Figure 3.11.5.4-1—Computational Procedures for Passive Earth Pressures for Vertical and Sloping Walls with Horizontal Backfill (U.S. Department of the Navy, 1982a)



REDUCTION FACTOR (R) OF k'_p FOR VARIOUS RATIOS OF $-\delta/\phi_f$								
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45	.718	.600	.500	.414	.339	.276	.221	.174

Figure 3.11.5.4-2—Computational Procedures for Passive Earth Pressures for Vertical Wall with Sloping Backfill (U.S. Department of the Navy, 1982a)

3.11.5.5—Equivalent-Fluid Method of Estimating Rankine Lateral Earth Pressures

The equivalent-fluid method may be used where Rankine earth pressure theory is applicable.

The equivalent-fluid method shall only be used where the backfill is free-draining. If this criterion cannot be satisfied, the provisions of Articles 3.11.3, 3.11.5.1, and 3.11.5.3 shall be used to determine horizontal earth pressure.

C3.11.5.5

Applicability of Rankine theory is discussed in Article C3.11.5.3.

Values of the unit weights of equivalent fluids are given for walls that can tolerate very little or no movement as well as for walls that can move as much as 1.0 in. in 20.0 ft. The concepts of equivalent fluid unit weights have taken into account the effect of soil creep on walls.

Where the equivalent-fluid method is used, the basic earth pressure, p (ksf), may be taken as:

$$p = \gamma_{eq}\ z \qquad (3.11.5.5\text{-}1)$$

where:

γ_{eq} = equivalent fluid unit weight of soil, not less than 0.030 (kcf)
z = depth below surface of soil (ft)

The resultant lateral earth load due to the weight of the backfill shall be assumed to act at a height of H/3 above the base of the wall, where H is the total wall height, measured from the surface of the ground to the bottom of the footing.

Typical values for equivalent fluid unit weights for design of a wall of height not exceeding 20.0 ft may be taken from Table 3.11.5.5-1, where:

Δ = movement of top of wall required to reach minimum active or maximum passive pressure by tilting or lateral translation (ft)
H = height of wall (ft)
β = angle of fill to the horizontal (degrees)

The magnitude of the vertical component of the earth pressure resultant for the case of sloping backfill surface may be determined as:

$$P_v = P_h \tan\beta \qquad (3.11.5.5\text{-}2)$$

where:

$$P_h = 0.5\gamma_{eq}H^2 \qquad (3.11.5.5\text{-}3)$$

If the backfill qualifies as free-draining (i.e., granular material with less than 5 percent passing a No. 200 sieve), water is prevented from creating hydrostatic pressure.

For discussion on the location of the resultant of the lateral earth force see Article C3.11.5.1.

The values of equivalent fluid unit weight presented in Table 3.11.5.5-1 for Δ/H = 1/240 represent the horizontal component of active earth pressure based on Rankine earth pressure theory. This horizontal earth pressure is applicable for cantilever retaining walls for which the wall stem does not interfere with the sliding surface defining the Rankine failure wedge within the wall backfill (Figure C3.11.5.3-1). The horizontal pressure is applied to a vertical plane extending up from the heel of the wall base, and the weight of soil to the left of the vertical plane is included as part of the wall weight.

For the case of a sloping backfill surface in Table 3.11.5.5-1, a vertical component of earth pressure also acts on the vertical plane extending up from the heel of the wall.

Table 3.11.5.5-1—Typical Values for Equivalent Fluid Unit Weights of Soils

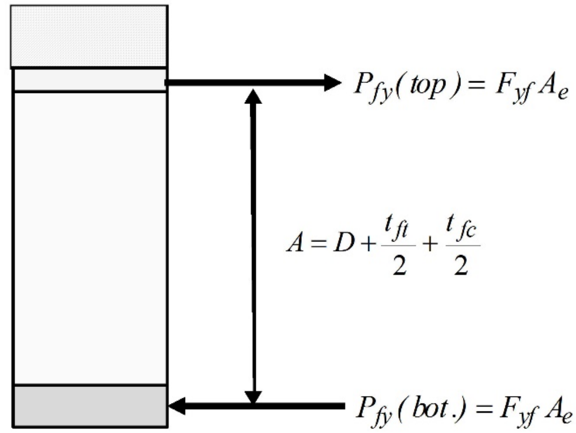
	Level Backfill		Backfill with β = 25 degrees	
Type of Soil	At-Rest γ_{eq} (kcf)	Active Δ/H = 1/240 γ_{eq} (kcf)	At-Rest γ_{eq} (kcf)	Active Δ/H = 1/240 γ_{eq} (kcf)
Loose sand or gravel	0.055	0.040	0.065	0.050
Medium dense sand or gravel	0.050	0.035	0.060	0.045
Dense sand or gravel	0.045	0.030	0.055	0.040

exceed the moment resistance provided by the nominal slip resistance of the flange splice bolts, the additional moment shall be resisted by the web as specified in Article 6.13.6.1.3c. The factored moments for checking slip shall be taken as the moment at the point of splice under Load Combination Service II, as specified in Table 3.4.1-1, and also the factored moment at the point of splice due to the deck casting sequence as specified in Article 3.4.2.1.

For the following box sections:

- single box sections in straight bridges;
- multiple box sections in straight bridges not satisfying the requirements of Article 6.11.2.3;
- single or multiple box sections in horizontally curved bridges; or
- single or multiple box sections with box flanges that are not fully effective according to the provisions of Article 6.11.1.1,

the vector sum of the St. Venant torsional shear in the bottom flange and P_{fy} shall be considered in the design of the bottom flange splice at the strength limit state. For checking slip, the St. Venant torsional shear shall be subtracted from the nominal slip resistance of the bottom flange splice bolts prior to computing the moment resistance.



Moment resistance is equal to $P_{fy(top)}$ or $P_{fy(bot.)}$, whichever is smaller, times the moment arm, A.

Figure C6.13.6.1.3b-2—Calculation of the Moment Resistance Provided by the Flanges for Composite Sections Subject to Negative Flexure and Noncomposite Sections

The moment resistance provided by the flanges can potentially be increased by staggering the flange bolts.

When checking for slip, the moment resistance provided by the nominal slip resistance of the flange splice bolts is calculated as shown in Figures C6.13.6.1.3b-1 and C6.13.6.1.3b-2, with the appropriate nominal slip resistance of the flange splice bolts substituted for P_{fy}. For checking slip due to the factored deck casting moment, the moment resistance of the noncomposite section is used.

Flange splice plates subjected to tension are to be checked for yielding on the gross section, fracture on the net section, and block shear rupture at the strength limit state according to the provisions of Article 6.13.5.2. Block shear rupture will usually not govern the design of splice plates of typical proportion. Flange splice plates subjected to compression at the strength limit state are to be checked only for yielding on the gross section of the plates. The factored yield resistance of splice plates in compression is the same as the factored yield resistance of splice plates in tension, and therefore, need not be checked. Buckling of splice plates in compression is not a concern since the unsupported length of the plates is limited by the maximum bolt spacing and end distance requirements.

For a flange splice with inner and outer splice plates, P_{fy} at the strength limit state may be assumed divided equally to the inner and outer plates and their connections when the areas of the inner and outer plates do not differ by more than ten percent. For this case, the connections are proportioned assuming double shear. Should the areas of the inner and outer plates differ by more than ten percent, the design force in each splice plate and its connection at the strength limit state should instead be determined by multiplying P_{fy} by the ratio of the area of the splice plate under consideration to the total area of the inner and outer splice plates. For this case, the connections are proportioned for the maximum calculated splice-plate force acting on a single shear plane. When checking for slip of the connection for a flange splice with inner and outer splice plates, the flange slip force is assumed divided equally to the two slip planes regardless of the ratio of the splice plate areas. Slip of the connection cannot occur unless slip occurs on both planes.

For the box sections cited in this Article, the vector sum of the St. Venant torsional shear in the bottom flange and P_{fy} is to be considered in the design of the bottom flange splice at the strength limit state. When checking for slip, the St. Venant torsional shear is conservatively subtracted from the nominal slip resistance of the bottom flange splice bolts prior to computing the moment resistance, rather than using the vector sum. St. Venant torsional shears and longitudinal warping stresses due to cross-section distortion are typically neglected in top flanges of tub-girder sections once the flanges are continuously braced. Longitudinal warping stresses due to cross-section distortion do not need to be considered in the design of the bottom flange splices at the strength limit state since the flange splices are designed to develop the full design yield resistance of the flanges. These stresses are typically relatively small in the bottom flange at the service limit state and for constructibility and may be neglected when checking the bottom flange splices for slip.

For flanges with one web in straight girders and in horizontally curved girders, the effects of flange lateral bending need not be considered in the design of the bolted flange splices since the combined areas of the flange splice plates will typically equal or exceed the area of the smaller flange to which they are attached. The flange is designed so that the yield stress of the flange is not exceeded at the flange tips under combined major-axis and lateral bending for constructibility and at the strength limit state. Flange lateral bending is also less critical at locations in-between the cross-frames or diaphragms where bolted splices are located. The rows of bolts provided in the flange splice on each side of the web provide the necessary couple to resist the lateral bending. Flange lateral bending will increase the flange slip force on one side of the splice and decrease the slip force on the other side of the splice; slip cannot occur unless it occurs on both sides of the splice.

6.13.6.1.3c—Web Splices

As a minimum, web splice plates and their connections shall be designed at the strength limit state for a design web force taken equal to the smaller factored shear resistance of the web ~~splice plates~~, $V_r = \phi_v V_n$, on either side of the splice determined according to the provisions of Article 6.10.9 or 6.11.9, as applicable. The factored shear resistance of the web, V_r, at the

C6.13.6.1.3c

The factored shear resistance of the bolts should be based on threads included in the shear planes, unless the web splice-plate thickness exceeds 0.5 in. As a minimum, two vertical rows of bolts spaced at the maximum spacing for sealing bolts specified in Article 6.13.2.6.2 should be provided, with a closer spacing and/or additional rows provided only as needed.

strength limit state shall not exceed the lesser of the factored shear resistances of the web splice plates determined as specified in Articles 6.13.4 and 6.13.5.3.

Should the moment resistance provided by the flanges at the point of splice, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state, the web splice ~~plates and their~~ connections shall instead be designed for a design web force taken equal to the vector sum of the smaller factored shear resistance and a horizontal force in the web that provides the necessary moment resistance in conjunction with the flanges.

The horizontal force in the web shall be computed as the portion of the factored moment at the strength limit state at the point of splice that exceeds the moment resistance provided by the flanges divided by the appropriate moment arm. For composite sections subject to positive flexure, the moment arm shall be taken as the vertical distance from the mid-depth of the web to the mid-thickness of the concrete deck including the concrete haunch. For composite sections subject to negative flexure and noncomposite sections subject to positive or negative flexure, the moment arm shall be taken as one-quarter of the web depth.

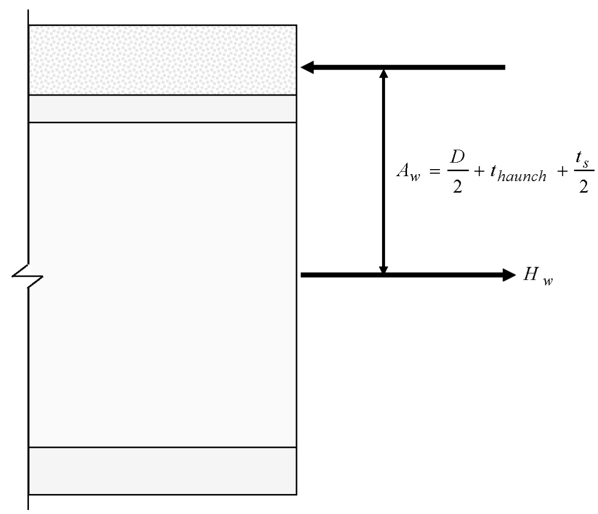
The computed design web force shall be divided by the factored shear resistance of the bolts, determined as specified in Article 6.13.2.2, to determine the total number of web splice bolts required on one side of the splice at the strength limit state. The bearing resistance of the web at bolt holes shall also be checked at the strength limit state as specified in Article 6.13.2.9.

As a minimum, bolted connections for web splices shall be checked for slip under a web slip force taken equal to the factored shear in the web at the point of splice. Should the moment resistance provided by the nominal slip resistance of the flange splice bolts, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment for checking slip at the point of splice, the web splice bolts shall instead be checked for slip under a web slip force taken equal to the vector sum of the factored shear and a horizontal force in the web that provides the necessary slip resistance in conjunction with the flange splices. The horizontal force in the web shall be computed as the portion of the factored moment for checking slip at the point of splice that exceeds the moment resistance provided by the nominal slip resistance of the flange splice bolts divided by the appropriate moment arm determined as specified herein. The factored shear for checking slip shall be taken as the shear in the web at the point of splice under Load Combination Service II, as specified in Table 3.4.1-1, or the factored shear in the web at the point of splice due to the deck casting sequence as specified in Article 3.4.2.1, whichever governs.

For the box sections specified in Article 6.13.6.1.3b, the shear for checking slip shall be taken as the sum of the factored flexural and St. Venant torsional shears in the web subjected to additive shears. For boxes with inclined

Since the web splice is being designed to develop the full factored shear resistance of the web as a minimum at the strength limit state and the eccentricity of the shear is small relative to the depth of the connection, the effect of the small moment introduced by the eccentricity of the web connection may be ignored at all limit states. Also, for the box sections specified in Article 6.13.6.1.3b, the effect of the additional St. Venant torsional shear in the web may be ignored at the strength limit state since the web splice is being designed as a minimum for the full factored shear resistance of the web.

Figure C6.13.6.1.3c-1 illustrates the computation of the horizontal force in the web, H_w, where necessary for composite sections subject to positive flexure. The web moment is taken as the portion of the factored moment that exceeds the moment resistance provided by the flanges. H_w is then taken as the web moment divided by the moment arm, A_w, taken from the mid-depth of the web to the mid-thickness of the concrete deck including the concrete haunch.



$$Web\ Moment = H_w A_w$$

$$H_w = \frac{Web\ Moment}{A_w}$$

A_w is measured from the mid-depth of the web to the mid-thickness of the deck.

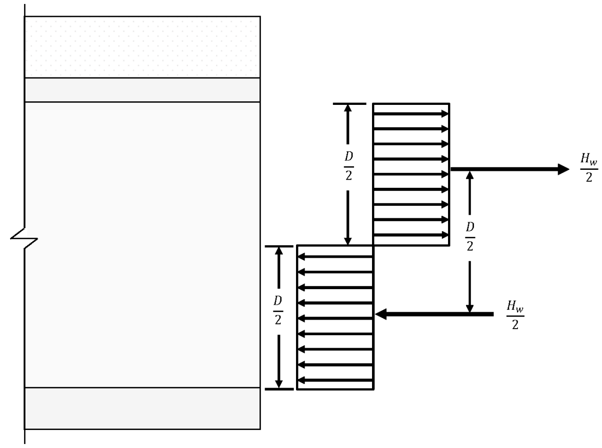
Figure C6.13.6.1.3c-1—Calculation of the Horizontal Force in the Web, H_w, for Composite Sections Subject to Positive Flexure

Figure C6.13.6.1.3c-2 illustrates the computation of the horizontal force in the web, H_w, where necessary for composite sections subject to negative flexure and noncomposite sections. The web moment is again taken as the portion of the factored moment that exceeds the moment resistance provided by the flanges. In this case, however, H_w is taken as the web moment divided by D/4, as shown in Figure C6.13.6.1.3c-2.

webs, the factored shear shall be taken as the component of the factored vertical shear in the plane of the web.

The computed web slip force shall be divided by the nominal slip resistance of the bolts, determined as specified in Article 6.13.2.8, to determine the total number of web splice bolts required on one side of the splice to resist slip.

Webs shall be spliced symmetrically by plates on each side. The splice plates shall extend as near as practical for the full depth between flanges without impinging on bolt assembly clearances or fillet areas on rolled beams. For bolted web splices with thickness differences of 0.0625 in. or less, filler plates should not be provided.



$$Web\ Moment = \frac{H_w}{2}\left(\frac{D}{2}\right)$$

$$H_w = \frac{Web\ Moment}{D/4}$$

Figure C6.13.6.1.3c-2—Calculation of the Horizontal Force in the Web, H_w, for Composite Sections Subject to Negative Flexure and Noncomposite Sections

The required moment resistance in the web for the case shown in Figure C6.13.6.1.3c-1 is provided by a horizontal tensile force, H_w, assumed acting at the mid-depth of the web that is equilibrated by an equal and opposite horizontal compressive force in the concrete deck. The required moment resistance in the web for the case shown in Figure C6.13.6.1.3c-2 is provided by two equal and opposite horizontal tensile and compressive forces, $H_w/2$, assumed acting at a distance $D/4$ above and below the mid-height of the web. In each case, there is no net horizontal force acting on the section.

Because the resultant web force is assumed divided equally to all of the bolts, the traditional vector analysis is not applied.

Since slip is a serviceability requirement, the effect of the additional St. Venant torsional shear in the web is to be considered for the box sections specified in Article 6.13.6.1.3b when checking for slip.

When a moment contribution from the web is required, the resultant forces causing bearing on the web bolt holes are inclined. The bearing resistance of each bolt hole in the web can conservatively be calculated in this case using the clear edge distance, as shown on the left of Figure C6.13.6.1.3c-3. This calculation is conservative since the resultant forces act in the direction of inclined distances that are larger than the clear edge distance. This calculation is also likely to be a conservative calculation for the bolt holes in the adjacent rows. Should the bearing resistance be exceeded, it is recommended that the edge distance be increased slightly in lieu of increasing the number of bolts or thickening the web. Other options would be to calculate the bearing resistance based on the inclined distance or to resolve the resultant force in the direction parallel to the edge distance, or to refine the

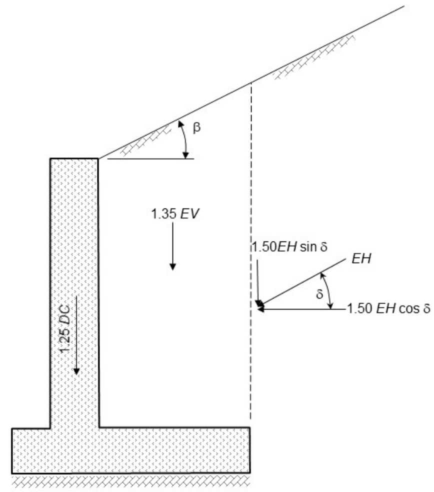


Figure C11.5.6-1—Typical Application of Load Factors for Bearing Resistance

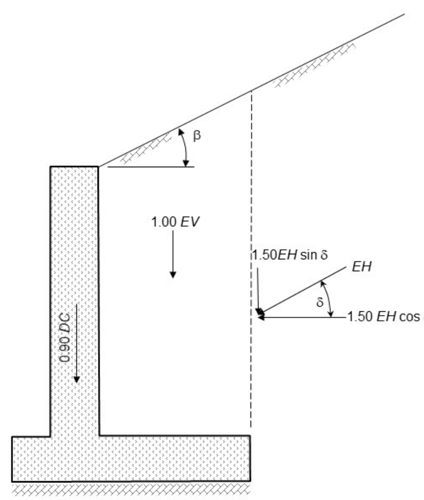


Figure C11.5.6-2—Typical Application of Load Factors for Sliding and Eccentricity

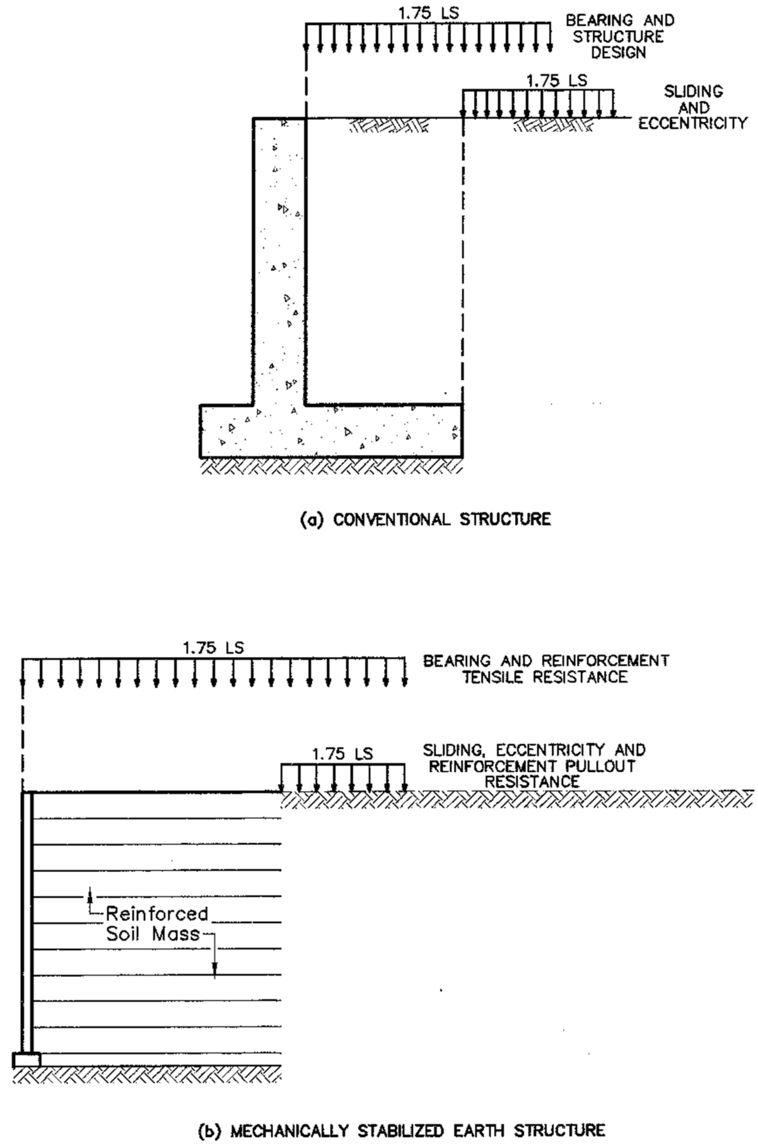


Figure C11.5.6-3—Typical Application of Live Load Surcharge

12.6.6.2—Embankment Installations

The minimum width of the soil envelope shall be sufficient to ensure lateral restraint for the buried structure. The combined width of the soil envelope and embankment beyond shall be adequate to support all the loads on the culvert and to comply with the movement requirements specified in Article 12.6.2.

C12.6.6.2

As a guide, the minimum width of the soil envelope on each side of the buried structure should not be less than the widths specified in Table C12.6.6.2-1:

Table C12.6.6.2-1—Minimum Width of Soil Envelope

Diameter, *S* (in.)	Minimum Envelope Width (ft)
<24	*S*/12
24–144	2.0
>144	5.0

12.6.6.3—Minimum Cover

The minimum cover, including a well-compacted granular subbase and base course, shall not be less than that specified in Table 12.6.6.3-1, where:

S = diameter of pipe (in.)
B_c = outside diameter or width of the structure (ft)
B'_c = out-to-out vertical rise of pipe (ft)
ID = inside diameter (in.)

C12.6.6.3

McGrath et al. (2005) has shown that the significant thermal expansion in thermoplastic pipe can affect pavement performance under shallow fills. Depending on the pipe material and the pavement type above it, the minimum cover may include the pavement thickness and base course, along with the sub-base.

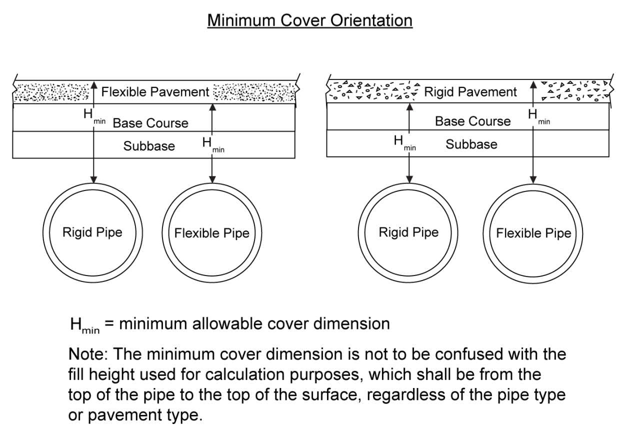


Figure 12.6.6.3-1—Minimum Cover Orientation

If the minimum cover provided in Table 12.6.6.3-1 is not sufficient to avoid placement of the pipe within the pavement layer, then the minimum cover should be increased to a minimum of the pavement thickness, unless an analysis is performed to determine the effect on both the pipe and the pavement.

Table 12.6.6.3-1—Minimum Cover

Type	Condition	Minimum Cover*
Corrugated Metal Pipe	—	$S/8 \geq 12.0$ in.
Spiral Rib Metal Pipe	Steel Conduit	$S/4 \geq 12.0$ in.
	Aluminum Conduit where $S \leq 48.0$ in.	$S/2 \geq 12.0$ in.
	Aluminum Conduit where $S > 48.0$ in.	$S/2.75 \geq 24.0$ in.
Structural Plate Pipe Structures	—	$S/8 \geq 12.0$ in.
Long-Span Structural Plate Pipe Structures	—	Refer to Table 12.8.3.1.1-1
Structural Plate Box Structures	—	1.4 ft. as specified in Article 12.9.1
Deep Corrugated Structural Plate Structures	—	See Article 12.8.9.4
Fiberglass Pipe	—	12.0 in.
Thermoplastic Pipe	Under unpaved areas	$ID/8 \geq 12.0$ in.
	Under paved roads	$ID/2 \geq 24.0$ in.
Steel-Reinforced Thermoplastic Culverts	—	$S/5 \geq 12.0$ in.
* Minimum cover taken from top of rigid pavement or bottom of flexible pavement		
Reinforced Concrete Pipe	Under unpaved areas or top of flexible pavement	$B_c/8$ or $B'_c/8$, whichever is greater, $\geq$ 12.0 in.
Reinforced Concrete Pipe	Under bottom of rigid pavement	9.0 in.
~~* Minimum cover taken from top of rigid pavement or bottom of flexible pavement~~		

If soil cover is not provided, the top of precast or cast-in-place reinforced concrete box structures shall be designed for direct application of vehicular loads.

Additional cover requirements during construction shall be taken as specified in Article 30.5.5 of the *AASHTO LRFD Bridge Construction Specifications.*

12.6.7—Minimum Spacing between Multiple Lines of Pipe

The spacing between multiple lines of pipe shall be sufficient to permit the proper placement and compaction of backfill below the haunch and between the structures.

Contract documents should require that backfilling be coordinated to minimize unbalanced loading between multiple, closely spaced structures. Backfill should be kept level over the series of structures when possible. The effects of significant roadway grades across a series of structures shall be investigated for the stability of flexible structures subjected to unbalanced loading.

C12.6.7

As a guide, the minimum spacing between pipes should not be less than that shown in Table C12.6.7-1.

Table C12.6.7-1—Minimum Pipe Spacing

Type of Structure	Minimum Distance Between Pipes (ft)
Round Pipes Diameter, D (ft)	
<2.0	1.0
2.0–6.0	$D/2$
>6.0	3.0
Pipe Arches Span, S (ft)	
<3.0	1.0
3.0–9.0	$S/3$
9.0–16.0	3.0
Arches Span, S (ft)	
All Spans	2.0

12.14.5.6—Resistance Factors

The provisions of Articles 12.5.5 and 1.3.1 shall apply as appropriate.

12.14.5.7—Crack Control

The provisions of Article 5.6.~~4~~7 for buried structures shall apply.

12.14.5.8—Minimum Reinforcement

The provisions of Article 5.9.5.5 shall not be taken to apply to precast three-sided structures.

The primary flexural reinforcement in the direction of the span shall provide a ratio of reinforcement area to gross concrete area at least equal to 0.002. Such minimum reinforcement shall be provided at all cross-sections subject to flexural tension, at the inside face of walls, and in each direction at the top of slabs of three-sided sections with less than 2.0 ft of fill.

12.14.5.9—Deflection Control at the Service Limit State

The deflection limits for concrete structures specified in Article 2.5.2.6.2 shall be taken as mandatory and pedestrian usage as limited to urban areas.

12.14.5.10—Footing Design

Design shall include consideration of differential horizontal and vertical movements and footing rotations. Footing design shall conform to the applicable Articles in Sections 5 and 10.

12.14.5.11—Structural Backfill

Specification of backfill requirements shall be consistent with the design assumptions used. The contract documents should require that a minimum backfill compaction of 90 percent Standard Proctor Density be achieved to prevent roadway settlement adjacent to the structure. A higher backfill compaction density may be required on structures utilizing a soil–structure interaction system.

12.14.5.12—Scour Protection and Waterway Considerations

The provisions of Article 2.6 shall apply as appropriate.

12.15—FIBERGLASS PIPE

12.15.1—General

The provisions of this Article shall apply to the structural design of solid wall buried fiberglass pipe.

C12.15.1

The provisions of this Article are based on Chapter 5 of AWWA's *Manual of Water Supply Practices—M45*, "Fiberglass Pipe Design." The internal pressure design requirements of M45 have been omitted here as culverts are typically designed for gravity flow.

Specific design requirements rely on the provisions for thermoplastic pipe as modified in this Article. Not all thermoplastic pipe design requirements are applicable to fiberglass pipe design.

12.15.2—Section Properties

Section properties for service and strength limit state calculations for solid wall fiberglass pipe shall be determined from the manufacturer's published nominal wall thickness, and from the nominal diameter requirements of ASTM D3262, Tables 2 and 3. Fiber-reinforced wall thickness shall be determined in accordance with ASTM D3567.

12.15.3—Mechanical Requirements

Solid wall fiberglass pipe mechanical properties shall be determined as specified in this Article.

C12.15.3

Because of the composite nature of fiberglass pipe, variety of manufacturing processes, and variables such as the amount, type, and orientation of fiber reinforcement, mechanical properties used in design must be actual properties from test results and measurements of the pipe in accordance with ASTM D3262 and ASTM D3567, as required in this Article.

12.15.3.1—Circumferential Flexural Modulus

The design value for circumferential flexural modulus of the pipe, E_{cf}, shall be calculated from pipe stiffness test results, conducted in accordance with ASTM D3262, using the pipe's fiber-reinforced wall thickness determined in accordance with Article 12.15.2.

C12.15.3.1

Appendix 2 of ASTM D2412 includes discussion on determining the circumferential flexural modulus from pipe stiffness test results.

12.15.3.2—Long-Term Ring-Bending Strain

The design value for long-term ring-bending strain, S_b, shall be determined from tests in accordance with ASTM D5365 using a water test solution with a pH between 5 and 9. Test data shall be statistically extrapolated to 75 years. Alternatively, the results from tests in accordance with ASTM D3681 may be used when tested in a solution of 1N H_2SO_4 with the results extrapolated to 75 years.

C12.15.3.2

As fiberglass pipe was originally designed for sanitary sewer applications, testing in acid is common and results are generally available from manufacturers. Testing in acid gives conservative results over water, and, if the results are already available, eliminates the need for the manufacturer to perform additional testing in water.

12.15.4—Total Allowable Deflection

The total allowable deflection, Δ_A (in.), shall be 5.0 percent of the inside diameter of the pipe or a lower deflection as specified by the Engineer. The total allowable deflection and the upper deflection limit where remediation or replacement is required shall be prominently displayed in the construction documents.

C12.15.4

Higher stiffness fiberglass pipe is easily produced for specific installations by increasing the quantity of glass fiber reinforcement or the wall thickness, or both. The allowable deflection for higher stiffness pipes may need to be reduced.

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