American Association of State Highway and Transportation Officials
AASHTO
John Schroer, President
Commissioner, Tennessee Department of Transportation
Bud Wright, Executive Director
444 North Capitol Street NW, Suite 249, Washington, DC 20001
(202) 624-5800 • Fax: (202) 624-5806 • www.transportation.org

ERRATA for
AASHTO LRFD Bridge Design Specifications, 8th Edition (LRFD-8)

May 2018

Dear Customer:

Recently, we were made aware of some technical revisions that need to be applied to the *AASHTO LRFD Bridge Design Specifications,* 8th 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:

http://downloads.transportation.org/LRFD-8-Errata.pdf

Then, please replace the existing pages with the corrected pages to ensure that your edition is both accurate and current.

AASHTO staff sincerely apologizes for any inconvenience to our readers.

Sincerely,

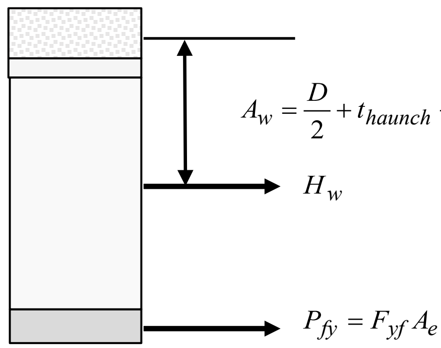
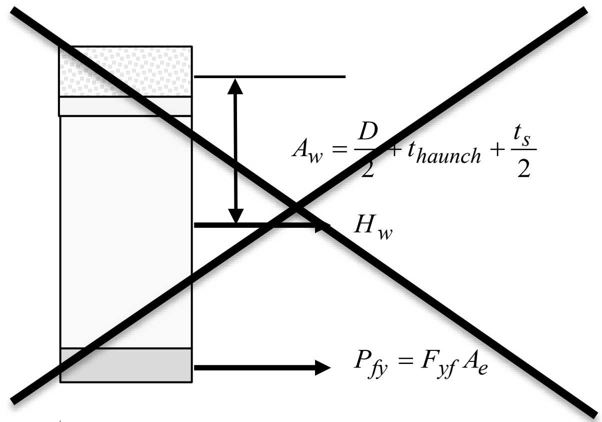
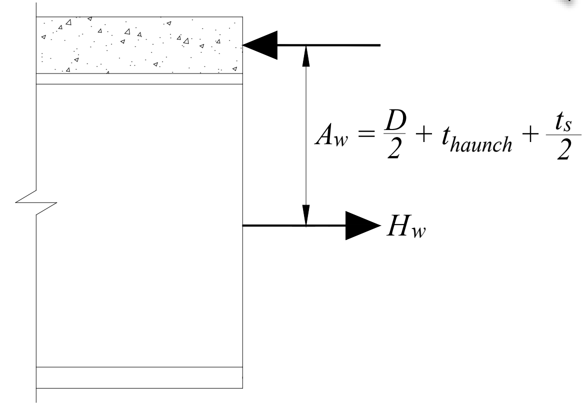
Erin Grady
Publications Director

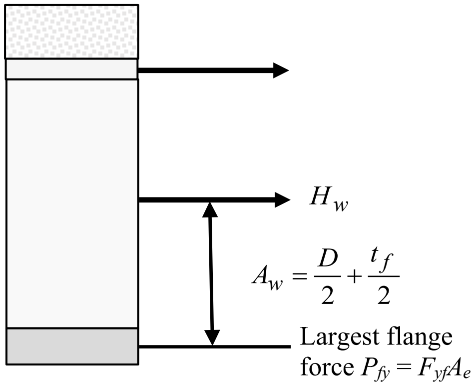
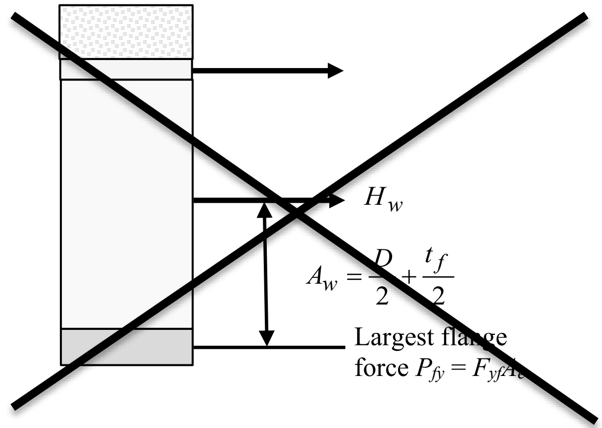
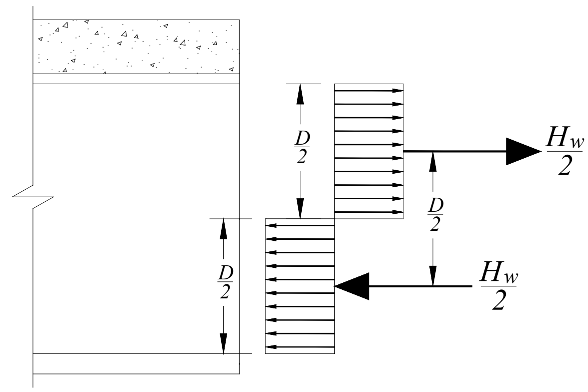
Page	Existing Text	Corrected Text
2-6 (Editor- ial)	**C2.3.3.4** …The provisions of the individual railroads and the AREMA Manual should be used to determine: • clearances, • loadings, • pier protection, • waterproofing, and blast protection.	**C2.3.3.4** …The provisions of the individual railroads and the AREMA Manual should be used to determine: • clearances, • loadings, • pier protection, • waterproofing, and • blast protection.
4-10	**4.3** η_i = load modifier relating to ductility, redundancy, and operational importance as specified in Article 1.3.2.1 (C4.2.6.7.1)	**4.3** η_i = load modifier relating to ductility, redundancy, and operational importance as specified in Article 1.3.2.1 ~~(C4.2.6.7.1)~~ (C4.6.2.7.1)
5-37	**Eq. 5.6.3.1.2-1** $f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{\ell_e}\right) \leq f_{py}$	There was an error in this equation wherein the subscripts to ℓ did not appear as intended **in the hard copy version only**. This has been restored.
5-37	**Eq. 5.6.3.1.2-2** $\ell_e = \left(\frac{2\ell_i}{2+N_s}\right)$	There was an error in this equation wherein the subscripts to ℓ did not appear as intended **in the hard copy version only**. This has been restored.
5-114	**Eq. 5.8.4.5.2-1** $f_{ca} = \frac{0.6P_u\kappa}{A_b\left(1+\ell_c\left(\frac{1}{b_{eff}}-\frac{1}{t}\right)\right)}$	There was an error in this equation wherein pieces of the denominators and subscripts to ℓ did not appear as intended **in the hard copy version only**. This has been restored.
5-149	**Eq. 5.9.5.4.4b-5** $d_{eff} = d_c + \frac{d_{duct}}{4} + \frac{\sum s_{duct}}{2}$	There was an error in this equation wherein the Σ did not appear as intended **in the hard copy version only**. This has been restored.
5-165 (Editor- ial)	Precast concrete box culverts • Top slabs used as a driving surface — 2.5 • Top slabs with less than 2.0 ft of fill not used as a driving surface — 2.0 1.0 • All other members	Precast concrete box culverts • Top slabs used as a driving surface — 2.5 • Top slabs with less than 2.0 ft of fill not used as a driving surface — 2.0 • All other members — 1.0
5-186	**Eq. 5.10.8.4.2a-1** $S_{max} = \frac{2\pi A_{sp} f_{ytr} \ell_s}{k A_\ell f_{u\ell}}$	There was an error in this equation wherein pieces of the subscripts did not appear as intended **in the hard copy version only**. These have been restored.

<table>
<tr><th>Page</th><th>Existing Text</th><th>Corrected Text</th></tr>
<tr><td>5-230</td><td>Eq. 5.12.5.3.8d-3
$$A_\ell \geq \frac{T_u p_h}{2\phi A_o f_y}$$</td><td>There was an error in this equation wherein pieces of the denominator did not appear as intended <u>in the hard copy version only</u>. These have been restored.</td></tr>
<tr><td>5-280</td><td>Eq. B5.2-5
$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5\left|V_u - V_p\right|\cot\theta - A_{ps}f_{po}\right)}{2\left(E_c A_c + E_s A_s + E_p A_{ps}\right)}$$</td><td>Eq. B5.2-5
$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5\left|V_u - V_p\right|\cot\theta - A_{ps}f_{po}\right)}{2\left(E_c A_{ct} + E_s A_s + E_p A_{ps}\right)}$$
A_c in this equation should be A_{ct}</td></tr>
<tr><td>6-80</td><td>Table 6.8.2.2-1 (<u>excerpted</u>)
<table>
<tr><th>Case</th><th>…</th><th>Shear Lag Factor, U</th></tr>
<tr><td>1</td><td>…</td><td>$U = 1.0$</td></tr>
<tr><td>2</td><td>…</td><td>$U = 1 - \frac{\bar{x}}{L}$</td></tr>
<tr><td>3</td><td>…</td><td>$U = 1.0$
and
A = area of the directly connected elements</td></tr>
<tr><td>4</td><td>…</td><td>$L \geq 2w \ldots U = 1.0$
$2w > L \geq 1.5w \ldots U = 0.87$
$1.5w > L \geq w \ldots U = 0.75$</td></tr>
<tr><td>5</td><td>…</td><td>$L \geq 1.3D \ldots U = 1.0$
$D \leq L < 1.3D \ldots U = 1 - \frac{\bar{x}}{L}$
$\bar{x} = \frac{D}{\pi}$</td></tr>
<tr><td rowspan="2">6</td><td>…</td><td>$L \geq H \ldots U = 1 - \frac{\bar{x}}{L}$
$\bar{x} = \frac{B^2 + 2BH}{4(B+H)}$</td></tr>
<tr><td>…</td><td>$L \geq H \ldots U = 1 - \frac{\bar{x}}{L}$
$\bar{x} = \frac{B^2}{4(B+H)}$</td></tr>
<tr><td rowspan="2">7</td><td>…</td><td>$b_f \geq \frac{2}{3}d \ldots U = 0.90$
$b_f < \frac{2}{3}d \ldots U = 0.85$</td></tr>
<tr><td>…</td><td>$U = 0.70$</td></tr>
<tr><td rowspan="2">8</td><td>…</td><td>$U = 0.80$</td></tr>
<tr><td>…</td><td>$U = 0.60$</td></tr>
</table></td><td>There were errors wherein pieces of the equations in the “Shear Lag Factor, U” column did not appear as intended <u>in the hard copy version only</u>. These have been restored.</td></tr>
<tr><td>6-132</td><td>Eq. 6.10.3.2.1-1
$f_{bu} + f_l \leq \phi_f R_h F_{yc}$,</td><td>There was an error in this equation wherein pieces of the variables did not appear as intended <u>in the hard copy version only</u>. These have been restored.</td></tr>
</table>

Page	Existing Text	Corrected Text
6-132	**Eq. 6.10.3.2.1-2** $f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc}$,	There was an error in this equation wherein pieces of the variables did not appear as intended **in the hard copy version only**. These have been restored.
6-140	**Eq. 6.10.4.2.2-2** $f_f + \frac{f_l}{2} \leq 0.95 R_h F_{yf}$	There was an error in this equation wherein pieces of the variables did not appear as intended **in the hard copy version only**. These have been restored.
6-140	**Eq. 6.10.4.2.2-3** $f_f + \frac{f_l}{2} \leq 0.80 R_h F_{yf}$	There was an error in this equation wherein pieces of the variables did not appear as intended **in the hard copy version only**. These have been restored.
6-153	**Eq. 6.10.7.2.1-2** $f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nt}$	There was an error in this equation wherein pieces of the variables did not appear as intended **in the hard copy version only**. These have been restored.
6-258	**6.13.6.1.3c (excerpted)** Should the moment resistance provided by the flange splices, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state at the point of splice, 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 located at the mid-depth of the web that provides the necessary moment resistance in conjunction with the flange splices. 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 flange splices divided by the appropriate moment arm to the mid-depth of the web. 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 the vertical distance from the mid-depth of the web to the mid-thickness of the top or bottom flange, whichever flange has the larger design yield resistance, P_{fy}.	**6.13.6.1.3c (excerpted)** Should the moment resistance provided by the flange splices, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state at the point of splice, 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 ~~located at the mid-depth of~~ in the web that provides the necessary moment resistance in conjunction with the flange splices. 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 flange splices divided by the appropriate moment arm ~~to the mid-depth of the web~~. 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 vertical distance from the mid-depth of the web to the mid-thickness of the top or bottom flange, whichever flange has the larger design yield resistance, P_{fy}.~~

Page	Existing Text	Corrected Text
6-258	*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 at the point of splice determined according to the provisions of Article 6.10.9 or 6.11.9, as applicable. Should the moment resistance provided by the flange splices, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state at the point of splice, 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 located at the mid-depth of the web that provides the necessary moment resistance in conjunction with the flange splices. 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 flange splices divided by the appropriate moment arm to the mid-depth of the web. 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 the vertical distance from the mid-depth of the web to the mid-thickness of the top or bottom flange, whichever flange has the larger design yield resistance, P_{fy}.	*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 at the point of splice determined according to the provisions of Article 6.10.9 or 6.11.9, as applicable. Should the moment resistance provided by the flange splices, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state at the point of splice, 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 ~~located at the mid-depth of~~ in the web that provides thet necessary moment resistance in conjunction with the flange splices. 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 flange splices divided by the appropriate moment arm ~~to the mid-depth of the web~~. 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 vertical distance from the mid-depth of the web to the mid-thickness of the top or bottom flange, whichever flange has the larger design yield resistance, P_{fy}.~~
6-258	*C6.13.6.1.3c* … 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 taken as the portion of the factored moment at the strength limit state that exceeds the moment resistance provided by the flange splices divided by the moment arm, A_w:	*C6.13.6.1.3c* … 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 at the strength limit state that exceeds the moment resistance provided by the flange splices. 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.

Page	Existing Text	Corrected Text
6-259	**Figure C6.13.6.1.3c-1** 	**Figure C6.13.6.1.3c-1**   $Web\ Moment = H_w A_w$ $H_w = \frac{Web\ Moment}{A_w}$
6-259	*C6.13.6.1.3c* … 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, taken as the portion of the factored moment at the strength limit state that exceeds the moment resistance provided by the flange splices divided by the moment arm, A_w, to the mid-thickness of the top or bottom flange, whichever flange has the larger value of P_{fy}:	*C6.13.6.1.3c* … 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 at the strength limit state that exceeds the moment resistance provided by the flange splices. 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.~~the moment arm, A_w, to the mid-thickness of the top or bottom flange, whichever flange has the larger value of P_{fy}:~~

Page	Existing Text	Corrected Text
6-259	**Figure C6.13.6.1.3c-2**  	**Figure C6.13.6.1.3c-2**   $Web\ Moment = \frac{Hw}{2}\left(\frac{D}{2}\right)$ $H_w = \frac{Web\ Moment}{D/4}$

Page	Existing Text	Corrected Text
6-259	*C6.13.6.1.3c* .. Because the resultant web force in cases where H_w is computed is divided equally to all of the bolts in this approach, the traditional vector analysis for bolt groups subject to a concentric shear and a centroidal moment is not applied.	*C6.13.6.1.3c* .. 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 in cases where H_w is computed is divided equally to all of the bolts in this approach, the traditional vector analysis for bolt groups subject to a concentric shear and a centroidal moment is not applied.
8-8 (Editor-ial)	**Table 8.4.1.1.4-1** **Douglas Fir-larch**	**Table 8.4.1.1.4-1** **Douglas Fir-Larch**
10-76	**Eq. C10.6.3.1.2e-5** $\beta_m = \frac{B}{4H}$	**Eq. C10.6.3.1.2e-5** $\beta_m = \frac{B}{4H_{s2}}$
10-77	**Eq. C10.6.3.1.2e-6** $\beta_m = \frac{B}{2H}$	**Eq. C10.6.3.1.2e-6** $\beta_m = \frac{B}{2H_{s2}}$
10-78	**Eq. 10.6.3.1.2f-1** $q_n = \left[q_2 + \left(\frac{1}{K}\right)c_1' \cot\phi_1'\right] e^{2\left[1+\left(\frac{B}{L}\right)\right] K \tan\phi_1' \left(\frac{H}{B}\right)} - \left(\frac{1}{K}\right)c_1' \cot\phi_1'$	**Eq. 10.6.3.1.2f-1** $q_n = \left[q_2 + \left(\frac{1}{K}\right)c_1' \cot\phi_1'\right] e^{2\left[1+\left(\frac{B}{L}\right)\right] K \tan\phi_1' \left(\frac{H_{s2}}{B}\right)} - \left(\frac{1}{K}\right)c_1' \cot\phi_1'$ (Note: in the new version, the $\left(\frac{H}{B}\right)$ term in the exponent has been changed to $\left(\frac{H_{s2}}{B}\right)$.)
10-78	**Eq. C10.6.3.1.2f-1** $q_n = q_2 e^{0.67\left[1+\left(\frac{B}{L}\right)\right]\frac{H}{B}}$	**Eq. C10.6.3.1.2f-1** $q_n = q_2 e^{0.67\left[1+\left(\frac{B}{L}\right)\right]\frac{H_{s2}}{B}}$ (Note: in the new version, the $\left(\frac{H}{B}\right)$ term in the exponent has been changed to $\left(\frac{H_{s2}}{B}\right)$.)

Page	Existing Text	Corrected Text
10-120	**C10.7.3.13.2** …Article C5.6.4.1 notes that compression members are usually prestressed only where they are subjected to high levels of flexure. Therefore, a method of determining nominal axial compression resistance is not given.	**C10.7.3.13.2** …~~Article C5.6.4.1 notes that~~ Compression members are usually prestressed only where they are subjected to high levels of flexure. Therefore, a method of determining nominal axial compression resistance is not given.
11-3 (Editor-ial)	**11.3.1—General** h_a distance between the base of the wall, or the mudline in front of the wall, and the resultant active seismic earth pressure force (ft) (A11.3.1)	**11.3.1—General** h_a = distance between the base of the wall, or the mudline in front of the wall, and the resultant active seismic earth pressure force (ft) (A11.3.1)
11-48	**C11.9.5.1** …A number of suitable methods for the determination of anchor loads are in common use. Sabatini et al. (1999) provides two methods which can be used: the Tributary Area Method, and the Hinge Method. These methods are illustrated in Figures C11.5.9.1-1 and C11.5.9.1-2. These figures assume that the soil below the base of the excavation has sufficient strength to resist the reaction force *R*. If the soil providing passive resistance below the base of the excavation is weak and is inadequate to carry the reaction force *R*, the lowest anchor should be designed to carry both the anchor load as shown in the figures as well as the reaction force. See Article 11.8.4.1 for evaluation of passive resistance. Alternatively, soil-structure interaction analyses, e.g., beam on elastic foundation, can be used to design continuous beams with small toe reactions, as it may be overly conservative to assume that all of the load is carried by the lowest anchor.	**C11.9.5.1** …A number of suitable methods for the determination of anchor loads are in common use. Sabatini et al. (1999) provides two methods which can be used: the Tributary Area Method, and the Hinge Method. These methods are illustrated in Figures ~~C11.5.9.1-1~~ C11.9.5.1-1 and ~~C11.5.9.1-2~~ C11.9.5.1-2. These figures assume that the soil below the base of the excavation has sufficient strength to resist the reaction force *R*. If the soil providing passive resistance below the base of the excavation is weak and is inadequate to carry the reaction force *R*, the lowest anchor should be designed to carry both the anchor load as shown in the figures as well as the reaction force. See Article 11.8.4.1 for evaluation of passive resistance. Alternatively, soil-structure interaction analyses, e.g., beam on elastic foundation, can be used to design continuous beams with small toe reactions, as it may be overly conservative to assume that all of the load is carried by the lowest anchor.

<table>
<tr><th>Page</th><th>Existing Text</th><th>Corrected Text</th></tr>
<tr>
<td>11-49</td>
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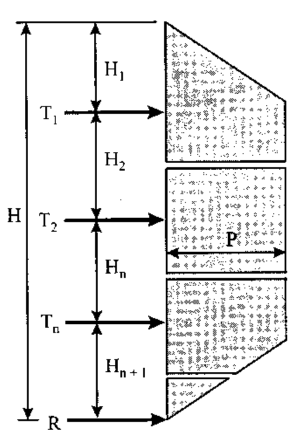
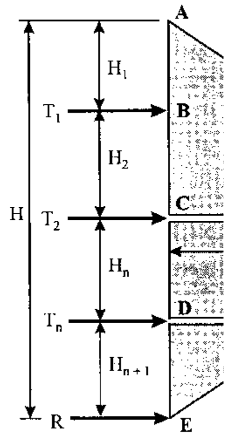
Figure C11.9.5.1-2— Calculation of Anchor Loads for Multilevel Wall after Sabatini et al. (1999)





TributaryAreaMethod **Hinge Method**

T_1 = Load over length $H_1 + H_2/2$
T_1 Calculated from $\Sigma M_C = 0$

T_2 = Load over length $H_2/2 + H_n/2$
T_{2u} = Total earth pressure ($ABCGF$) – T_1

T_n = Load over length $H_n/2 + H_{n+1}/2$
T_{2L} = Calculated from $\Sigma M_D = 0$

R = Load over length $H_{n+1}/2$
T_{nu} = Total earth pressure ($CDIH$) – T_{2L}

T_{nL} = Calculated from $\Sigma M_E = 0$

R = Total earth pressure – $T_1 - T_2 - T_n$

$T_2 = T_{2u} = T_{2L}$

$T_n = T_{nu} + T_{nL}$

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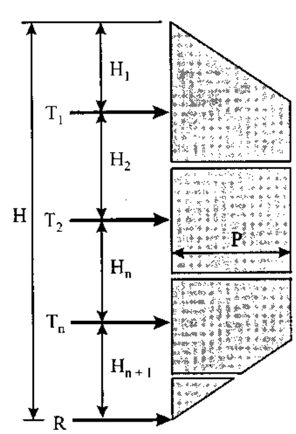
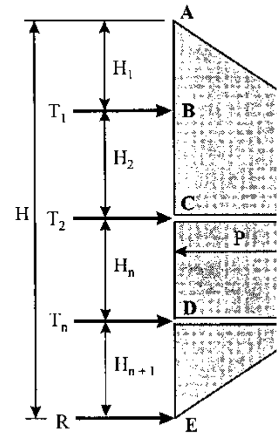
<td>

Figure C11.9.5.1-2— Calculation of Anchor Loads for Multilevel Wall after Sabatini et al. (1999)





Tributary Area Method **Hinge Method**

T_1 = Load over length $H_1 + H_2/2$
T_1 Calculated from $\Sigma M_C = 0$

T_2 = Load over length $H_2/2 + H_n/2$
T_{2u} = Total earth pressure ($ABCGF$) – T_1

T_n = Load over length $H_n/2 + H_{n+1}/2$
T_{2L} = Calculated from $\Sigma M_D = 0$

R = Load over length $H_{n+1}/2$
T_{nu} = Total earth pressure ($CDIH$) – T_{2L}

T_{nL} = Calculated from $\Sigma M_E = 0$

R = Total earth pressure – $T_1 - T_2 - T_n$

$\cancel{T_2 = T_{2u} = T_{2L}}$ $\underline{T_2 = T_{2u} + T_{2L}}$

$T_n = T_{nu} + T_{nL}$

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</table>

Page	Existing Text	Corrected Text
11-79	**11.10.6.4.2b** ... 3) *Polymer Requirements*: Polymers which are likely to have good resistance to long-term chemical degradation shall be used if a single default reduction factor is to be used, to minimize the risk of the occurrence of significant long-term degradation. The polymer material requirements provided in Table 11.10.6.4.2b-1 shall, therefore, be met if detailed product specific data as described in AASHTO R 69 and Elias, et al. (2009) is not obtained. Polymer materials not meeting the requirements in Table 11.10.6.4.2b-1 may be used if this detailed product specific data extrapolated to the design life intended for the structure are obtained.	**11.10.6.4.2b** ... ~~3)~~ *Polymer Requirements*: Polymers which are likely to have good resistance to long-term chemical degradation shall be used if a single default reduction factor is to be used, to minimize the risk of the occurrence of significant long-term degradation. The polymer material requirements provided in Table 11.10.6.4.2b-1 shall, therefore, be met if detailed product specific data as described in AASHTO R 69 and Elias, et al. (2009) is not obtained. Polymer materials not meeting the requirements in Table 11.10.6.4.2b-1 may be used if this detailed product specific data extrapolated to the design life intended for the structure are obtained.
11-123	**Eq. A11.5.2-3** $\log d = -1.51 - 0.74\log\left(\frac{k_v}{k_{h0}}\right) + 3.27\log\left(\frac{1-k_y}{k_{h0}}\right) - 0.8$	**Eq. A11.5.2-3** $\underline{\log d = -1.51 - 0.74\log\left(\frac{k_y}{k_{h0}}\right) + 3.27\log\left(1-\frac{k_y}{k_{h0}}\right) - 0.80\log(k_{h0}) + 1.59\log(PGV)}$ The k_v term in the first parenthesis should be k_y
11-123	**Eq. A11.5.2-4** $\log d = -1.31 - 0.93\log\left(\frac{k_v}{k_{h0}}\right) + 4.52\log\left(1-\frac{k_v}{k_{h0}}\right) - 0.46\log\left(k_{h0}\right) + 1.12\log\left(PGV\right)$	**Eq. A11.5.2-4** $\underline{\log d = -1.31 - 0.93\left(\log\frac{k_y}{k_{h0}}\right) + 4.52\log\left(1-\frac{k_y}{k_{h0}}\right) - 0.46\log\left(k_{h0}\right) + 1.12\log(PGV)}$ The k_v term in the first parenthesis should be k_y
12-2	**12.3—NOTATION** ... A_s = tension reinforcement area on cross-section width, b (in.^{2}/ft) (C12.10.4.2.4a) (C12.11.4) (C12.11.5)	**12.3—NOTATION** ... A_s = tension reinforcement area on cross-section width, b (in.^{2}/ft) (~~C~~12.10.4.2.4a) ~~(C12.11.4)~~ ~~(C12.11.5)~~

Page	Existing Text	Corrected Text
12-29	*12.8.3.1.1—Cross-Section* …Table A12-3 shall apply. Minimum requirements for section properties shall be taken as specified in Table 12.8.3.1.1-1. Covers that are less than that shown in Table 12.8.3.1-1 and that correspond to the minimum plate thickness for a given radius may be used if ribs are used to stiffen the plate. If ribs are used, the plate thickness may not be reduced below the minimum shown for that radius, and the moment of inertia of the rib and plate section shall not be less than that of the thicker unstiffened plate corresponding to the fill height. Use of soil cover less than the minimum values shown for a given radius shall require a special design.	*12.8.3.1.1—Cross-Section* …Table A12-3 shall apply. Minimum requirements for section properties shall be taken as specified in Table 12.8.3.1.1-1. Covers that are less than that shown in ~~Table 12.8.3.1-1~~ <u>Table 12.8.3.1.1-1</u> and that correspond to the minimum plate thickness for a given radius may be used if ribs are used to stiffen the plate. If ribs are used, the plate thickness may not be reduced below the minimum shown for that radius, and the moment of inertia of the rib and plate section shall not be less than that of the thicker unstiffened plate corresponding to the fill height. Use of soil cover less than the minimum values shown for a given radius shall require a special design.
14-24 (Editor-ial)	*C14.5.6.9.2* … The designer should consider showing the total estimated transverse and vertical movement in each direction, as well as the rotation in each direction about the three principal axes on the contract plans. Vertical movement due to vertical grade, with horizontal bearings, and vertical movement due to girder and rotation may also be considered.	*C14.5.6.9.2* … ___The designer should consider showing the total estimated transverse and vertical movement in each direction, as well as the rotation in each direction about the three principal axes on the contract plans. Vertical movement due to vertical grade, with horizontal bearings, and vertical movement due to girder and rotation may also be considered.

2.3.2.2.2—Protection of Users

Railings shall be provided along the edges of structures conforming to the requirements of Section 13.

All protective structures shall have adequate surface features and transitions to safely redirect errant traffic.

In the case of movable bridges, warning signs, lights, signal bells, gates, barriers, and other safety devices shall be provided for the protection of pedestrian, cyclists, and vehicular traffic. These shall be designed to operate before the opening of the movable span and to remain operational until the span has been completely closed. The devices shall conform to the requirements for "Traffic Control at Movable Bridges," in the *Manual on Uniform Traffic Control Devices* (MUTCD) or as shown on plans.

Where specified by the Owner, sidewalks shall be protected by barriers.

C2.3.2.2.2

Protective structures include those that provide a safe and controlled separation of traffic on multimodal facilities using the same right-of-way.

Special conditions, such as curved alignment, impeded visibility, etc., may justify barrier protection, even with low design velocities.

2.3.2.2.3—Geometric Standards

Requirements of the AASHTO publication *A Policy on Geometric Design of Highways and Streets* shall either be satisfied or exceptions thereto shall be justified and documented. Width of shoulders and geometry of traffic barriers shall meet the specifications of the Owner.

2.3.2.2.4—Road Surfaces

Road surfaces on a bridge shall be given antiskid characteristics, crown, drainage, and superelevation in accordance with *A Policy on Geometric Design of Highways and Streets* or local requirements.

2.3.2.2.5—Vessel Collisions

Bridge structures shall either be protected against vessel collision forces by fenders, dikes, or dolphins as specified in Article 3.14.15, or shall be designed to withstand collision force effects as specified in Article 3.14.14.

C2.3.2.2.5

The need for dolphin and fender systems can be eliminated at some bridges by judicious placement of bridge piers. Guidance on use of dolphin and fender systems is included in the AASHTO *Highway Drainage Guidelines*, Volume 7: *Hydraulic Analyses for the Location and Design of Bridges*.

2.3.3—Clearances

2.3.3.1—Navigational

Permits for construction of a bridge over navigable waterways shall be obtained from the U.S. Coast Guard and/or other agencies having jurisdiction. Navigational clearances, both vertical and horizontal, shall be established in cooperation with the U.S. Coast Guard.

C2.3.3.1

Where bridge permits are required, early coordination should be initiated with the U.S. Coast Guard to evaluate the needs of navigation and the corresponding location and design requirements for the bridge.

Procedures for addressing navigational requirements for bridges, including coordination with the Coast Guard, are set forth in the Code of Federal Regulations, 23 CFR, Part 650, Subpart H, "Navigational Clearances for Bridges," and 33 U.S.C. 401, 491, 511, et seq.

2.3.3.2—Highway Vertical

The vertical clearance of highway structures shall be in conformance with the AASHTO publication *A Policy on Geometric Design of Highways and Streets* for the Functional Classification of the Highway or exceptions thereto shall be justified. Possible reduction of vertical clearance, due to settlement of an overpass structure, shall be investigated. If the expected settlement exceeds 1.0 in., it shall be added to the specified clearance.

The vertical clearance to sign supports and pedestrian overpasses should be 1.0 ft. greater than the highway structure clearance, and the vertical clearance from the roadway to the overhead cross bracing of through-truss structures should not be less than 17.5 ft.

C2.3.3.2

The specified minimum clearance should include 6.0 in. for possible future overlays. If overlays are not contemplated by the Owner, this requirement may be nullified.

Sign supports, pedestrian bridges, and overhead cross bracings require the higher clearance because of their lesser resistance to impact.

2.3.3.3—Highway Horizontal

The bridge width shall not be less than that of the approach roadway section, including shoulders or curbs, gutters, and sidewalks.

Horizontal clearance under a bridge should meet the requirements of Article 2.3.2.2.1.

No object on or under a bridge, other than a barrier, should be located closer than 4.0 ft. to the edge of a designated traffic lane. The inside face of a barrier should not be closer than 2.0 ft. to either the face of the object or the edge of a designated traffic lane.

C2.3.3.3

The usable width of the shoulders should generally be taken as the paved width.

The specified minimum distances between the edge of the traffic lane and fixed object are intended to prevent collision with slightly errant vehicles and those carrying wide loads.

2.3.3.4—Railroad Overpass

Structures designed to pass over a railroad shall be in accordance with standards established and used by the affected railroad in its normal practice. These overpass structures shall comply with applicable federal, state, county, and municipal laws.

Regulations, codes, and standards should, as a minimum, meet the specifications and design standards of the American Railway Engineering and Maintenance of Way Association (AREMA), the Association of American Railroads, and AASHTO.

C2.3.3.4

Attention is particularly called to the following chapters in the *Manual for Railway Engineering* (AREMA, 2003):

- Chapter 7—Timber Structures,
- Chapter 8—Concrete Structures and Foundations,
- Chapter 9—Highway-railroad Crossings,
- Chapter 15—Steel Structures, and
- Chapter 18—Clearances.

The provisions of the individual railroads and the AREMA Manual should be used to determine:

- clearances,
- loadings,
- pier protection,
- waterproofing, and
- blast protection.

n_{cf} = minimum number of intermediate cross-frames or diaphragms within the individual spans of the bridge or bridge unit at the stage of construction being evaluated (4.6.3.3.2)
P = axle load (kip) (4.6.2.1.3)
P_D = design horizontal wind pressure (ksf) (C4.6.2.7.1)
P_e = Euler buckling load (kip) (4.5.3.2.2b)
P_u = factored axial load (kip) (4.5.3.2.2b) (4.7.4.5)
P_w = lateral wind force applied to the brace point (kips) (C4.6.2.7.1)
p = tire pressure (ksi) (4.6.2.1.8)
p_e = equivalent uniform static seismic loading per unit length of bridge that is applied to represent the primary mode of vibration (kip/ft) (C4.7.4.3.2c)
$p_e(x)$ = the intensity of the equivalent static seismic loading that is applied to represent the primary mode of vibration (kip/ft) (C4.7.4.3.2b)
p_o = a uniform load arbitrarily set equal to 1.0 (kip/ft) (C4.7.4.3.2b)
R = girder radius (ft); load distribution to exterior beam in terms of lanes; minimum radius of curvature at the centerline of the bridge cross-section throughout the length of the bridge or bridge unit at the construction stage and/or loading condition being evaluated (ft); radius of curvature; R-Factor for calculation of seismic design forces due to inelastic action (C4.6.1.2.4b) (C4.6.2.2.2d) (4.6.3.3.2) (C4.6.6) (4.7.4.5)
R_d = R_d-factor for calculation of seismic displacements due to inelastic action (4.7.4.5)
r = reduction factor for longitudinal force effect in skewed bridges (4.6.2.3)
S = spacing of supporting components (ft); spacing of beams or webs (ft); clear span (ft); skew of support measured from line normal to span (degrees) (4.6.2.1.3) (4.6.2.2.1) (4.6.2.10.2) (4.7.4.4)
S_b = spacing of grid bars (in.) (4.6.2.1.3)
SM = single-mode elastic method (4.7.4.3.1)
s = length of a side element (in.) (C4.6.2.2.1)
T = period of fundamental mode of vibration (sec.) (4.7.4.5)
T_G = temperature gradient (Δ°F) (C4.6.6)
TH = time history method (4.7.4.3.1)
T_m = period of mth mode of vibration (sec.) (C4.7.4.3.2b)
T_S = reference period used to define shape of seismic response spectrum (sec.) (4.7.4.5)
T_u = uniform specified temperature (°F) (C4.6.6)
T_{UG} = temperature averaged across the cross-section (°F) (C4.6.6)
t = thickness of plate-like element (in.); thickness of flange plate in orthotropic steel deck (in.) (C4.6.2.2.1) (4.6.2.6.4)
t_g = depth of steel grid or corrugated steel plank including integral concrete overlay or structural concrete component, less a provision for grinding, grooving, or wear (in.) (4.6.2.2.1)
t_o = depth of structural overlay (in.) (4.6.2.2.1)
t_s = depth of concrete slab (in.) (4.6.2.2.1)
V_{LD} = maximum vertical shear at $3d$ or $L/4$ due to wheel loads distributed laterally as specified herein (kips) (4.6.2.2.2a)
V_{LL} = distributed live load vertical shear (kips) (4.6.2.2.2a)
V_{LU} = maximum vertical shear at $3d$ or $L/4$ due to undistributed wheel loads (kips) (4.6.2.2.2a)
$v_s(x)$ = deformation corresponding to p_o (ft) (C4.7.4.3.2b)
$v_{s,MAX}$ = maximum value of $v_s(x)$ (ft) (C4.7.4.3.2c)
W = edge-to-edge width of bridge (ft); factored wind force per unit length (kip/ft); total weight of cable (kip); total weight of bridge (kip) (4.6.2.2.1) (C4.6.2.7.1) (4.6.3.7) (C4.7.4.3.2c)
W_e = half the web spacing, plus the total overhang (ft) (4.6.2.2.1)
W_1 = modified edge-to-edge width of bridge taken to be equal to the lesser of the actual width or 60.0 for multilane loading, or 30.0 for single-lane loading (ft) (4.6.2.3)
w = width of clear roadway (ft); width of element in cross-section (in.) (4.6.2.2.2b) (C4.6.6)
$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure (kip/ft) (C4.7.4.3.2) (4.7.4.3.2c)
w_p = plank width (in.) (4.6.2.1.3)
w_g = maximum width between the girders on the outside of the bridge cross-section at the completion of the construction or at an intermediate stage of the steel erection (ft) (4.6.3.3.2)
X = distance from load to point of support (ft) (4.6.2.1.3)
X_{ext} = horizontal distance from the center of gravity of the pattern of girders to the exterior girder (ft) (C4.6.2.2.2d)
x = horizontal distance from the center of gravity of the pattern of girders to each girder (ft) (C4.6.2.2.2d)

Z	=	a factor taken as 1.20 where the lever rule was not utilized, and 1.0 where the lever rule was used for a single lane live load distribution factor (4.6.2.2.4)
z	=	vertical distance from center of gravity of cross-section (in.) (C4.6.6)
α	=	angle between cable and horizontal (degrees); coefficient of thermal expansion (in./in./°F); generalized flexibility (4.6.3.7) (C4.6.6) (C4.7.4.3.2b)
β	=	generalized participation (C4.7.4.3.2b)
γ	=	load factor; generalized mass (C4.6.2.7.1) (C4.7.4.3.2b)
Δ	=	displacement of point of contraflexure in column or pier relative to point of fixity for the foundation (in.) (4.7.4.5)
Δ_e	=	displacement calculated from elastic seismic analysis (in.) (4.7.4.5)
Δw	=	overhang width extension (in.) (C4.6.2.6.1)
δ_b	=	moment or stress magnifier for braced mode deflection (4.5.3.2.2b)
δ_s	=	moment or stress magnifier for unbraced mode deflection (4.5.3.2.2b)
ε_u	=	uniform axial strain due to axial thermal expansion (in./in.) (C4.6.6)
η_i	=	load modifier relating to ductility, redundancy, and operational importance as specified in Article 1.3.2.1 ~~(C4.2.6.7.1)~~ (C4.6.2.7.1)
θ	=	skew angle (degrees); maximum skew angle of the bearing lines at the end of a given span, measured from a line taken perpendicular to the span centerline (degrees) (4.6.2.2.1) (4.6.3.3.2)
μ	=	Poisson's ratio (4.6.2.2.1)
σ_E	=	internal stress due to thermal effects (ksi) (C4.6.6)
ϕ	=	rotation per unit length; flexural resistance factor (C4.6.6) (4.7.4.5)
ϕ_K	=	stiffness reduction factor = 0.75 for concrete members and 1.0 for steel and aluminum members (4.5.3.2.2b)

4.4—ACCEPTABLE METHODS OF STRUCTURAL ANALYSIS

Any method of analysis that satisfies the requirements of equilibrium and compatibility and utilizes stress-strain relationships for the proposed materials may be used, including, but not limited to:

- classical force and displacement methods,
- finite difference method,
- finite element method,
- folded plate method,
- finite strip method,
- grid analogy method,
- series or other harmonic methods,
- methods based on the formation of plastic hinges, and
- yield line method.

The Designer shall be responsible for the implementation of computer programs used to facilitate structural analysis and for the interpretation and use of results.

The name, version, and release date of software used should be indicated in the contract documents.

C4.4

Many computer programs are available for bridge analysis. Various methods of analysis, ranging from simple formulae to detailed finite element procedures, are implemented in such programs. Many computer programs have specific engineering assumptions embedded in their code, which may or may not be applicable to each specific case.

When using a computer program, the Designer should clearly understand the basic assumptions of the program and the methodology that is implemented.

A computer program is only a tool, and the user is responsible for the generated results. Accordingly, all output should be verified to the extent possible.

Computer programs should be verified against the results of:

- universally accepted closed-form solutions,
- other previously verified computer programs, or
- physical testing.

The purpose of identifying software is to establish code compliance and to provide a means of locating bridges designed with software that may later be found deficient.

$$c = \frac{A_{ps} f_{pu} + A_s f_s - A'_s f'_s - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}} \quad (5.6.3.1.1\text{-}3)$$

for rectangular section behavior:

$$c = \frac{A_{ps} f_{pu} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b + k A_{ps} \dfrac{f_{pu}}{d_p}} \quad (5.6.3.1.1\text{-}4)$$

where:

A_{ps} = area of prestressing steel (in.^{2})
f_{pu} = specified tensile strength of prestressing steel (ksi)
f_{py} = yield strength of prestressing steel (ksi)
A_s = area of nonprestressed tension reinforcement (in.^{2})
A'_s = area of compression reinforcement (in.^{2})
f_s = stress in the nonprestressed tension reinforcement at nominal flexural resistance (ksi), as specified in Article 5.6.2.1
f'_s = stress in the nonprestressed compression reinforcement at nominal flexural resistance (ksi), as specified in Article 5.6.2.1
b = width of the compression face of the member; for a flange section in compression, the effective width of the flange as specified in Article 4.6.2.6 (in.)
b_w = web width or diameter of a circular section (in.)
h_f = compression flange depth (in.)
d_p = distance from extreme compression fiber to the centroid of the prestressing force (in.)
c = distance from the extreme compression fiber to the neutral axis (in.)
α_1 = stress block factor specified in Article 5.6.2.2
β_1 = stress block factor specified in Article 5.6.2.2

Table C5.6.3.1.1-1—Values of *k*

Type of Tendon	f_{py}/f_{pu}	Value of k
Low relaxation strand	0.90	0.28
Type 1 high-strength bar	0.85	0.38
Type 2 high-strength bar	0.80	0.48

5.6.3.1.2—Components with Unbonded Tendons

For rectangular or flanged sections subjected to flexure about one axis and for biaxial flexure with axial load as specified in Article 5.6.4.5, where the approximate stress distribution specified in Article 5.6.2.2 is used, the average stress in unbonded prestressing steel may be taken as:

$$f_{ps} = f_{pe} + 900\left(\frac{d_p - c}{\ell_e}\right) \leq f_{py} \quad (5.6.3.1.2\text{-}1)$$

in which:

$$\ell_e = \left(\frac{2\,\ell_i}{2 + N_s}\right) \quad (5.6.3.1.2\text{-}2)$$

C5.6.3.1.2

A first estimate of the average stress in unbonded prestressing steel may be made as:

$$f_{ps} = f_{pe} + 15.0 \text{ (ksi)} \quad (C5.6.3.1.2\text{-}1)$$

In order to solve for the value of f_{ps} in Eq. 5.6.3.1.2-1, the equation of force equilibrium at ultimate is needed. Thus, two equations with two unknowns (f_{ps} and c) need to be solved simultaneously to achieve a closed-form solution.

for T-section behavior:

$$c = \frac{A_{ps} f_{ps} + A_s f_s - A'_s f'_s - \alpha_1 f'_c (b - b_w) h_f}{\alpha_1 f'_c \beta_1 b_w} \tag{5.6.3.1.2-3}$$

for rectangular section behavior:

$$c = \frac{A_{ps} f_{ps} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b} \tag{5.6.3.1.2-4}$$

where:

c = distance from extreme compression fiber to the neutral axis assuming the tendon prestressing steel has yielded, given by Eqs. 5.6.3.1.2-3 and 5.6.3.1.2-4 for T-section behavior and rectangular section behavior, respectively (in.)
ℓ_e = effective tendon length (in.)
ℓ_i = length of tendon between anchorages (in.)
N_s = number of plastic hinges at supports in an assumed failure mechanism crossed by the tendon between anchorages or discretely bonded points assumed as:

- For simple spans……………………….0
- End spans of continuous units…………1
- Interior spans of continuous units……....2

f_{py} = yield strength of prestressing steel (ksi)
f_{pe} = effective stress in prestressing steel after losses (ksi)

5.6.3.1.3—Components with Both Bonded and Unbonded Tendons

5.6.3.1.3a—Detailed Analysis

Except as specified in Article 5.6.3.1.3b, for components with both bonded and unbonded tendons, the stress in the prestressing steel shall be computed by detailed analysis. This analysis shall take into account the strain compatibility between the section and the bonded prestressing steel. The stress in the unbonded prestressing steel shall take into account the global displacement compatibility between bonded sections of tendons located within the span. Bonded sections of unbonded tendons may be anchorage points and any bonded section, such as deviators. Consideration of the possible slip at deviators shall be taken into consideration. The nominal flexural strength should be computed directly from the stresses resulting from this analysis.

5.8.4.5—General Zone of Post-Tensioning Anchorages

5.8.4.5.1—Limitations of Application

Concrete compressive stresses ahead of the anchorage device, location and magnitude of the bursting force, and edge tension forces may be estimated using Eqs. 5.8.4.5.2-1 through 5.8.4.5.3-2, provided that:

- the member has a rectangular cross-section and its longitudinal extent is not less than the larger transverse dimension of the cross-section;
- the member has no discontinuities within or ahead of the anchorage zone;
- the minimum edge distance of the anchorage in the main plane of the member is not less than 1.5 times the corresponding lateral dimension, a, of the anchorage device;
- only one anchorage device or one group of closely spaced anchorage devices is located in the anchorage zone; and
- the angle of inclination of the tendon, as specified in Eqs. 5.8.4.5.3-1 and 5.8.4.5.3-2, is between −5.0 degrees and +20.0 degrees.

C5.8.4.5.1

The equations specified herein are based on the analysis of members with rectangular cross-sections and on an anchorage zone at least as long as the largest dimension of that cross-section. For cross-sections that deviate significantly from a rectangular shape, for example I-girders with wide flanges, the approximate equations should not be used.

Discontinuities, such as web openings, disturb the flow of forces and may cause higher compressive stresses, bursting forces, or edge tension forces in the anchorage zone. Figure C5.8.4.5.1-1 compares the bursting forces for a member with a continuous rectangular cross-section and for a member with a noncontinuous rectangular cross-section. The approximate equations may be applied to standard I-girders with end blocks if the longitudinal extension of the end block is at least one girder height and if the transition from the end block to the I-section is gradual.

Anchorage devices may be treated as closely spaced if their center-to-center spacing does not exceed 1.5 times the width of the anchorage devices in the direction considered.

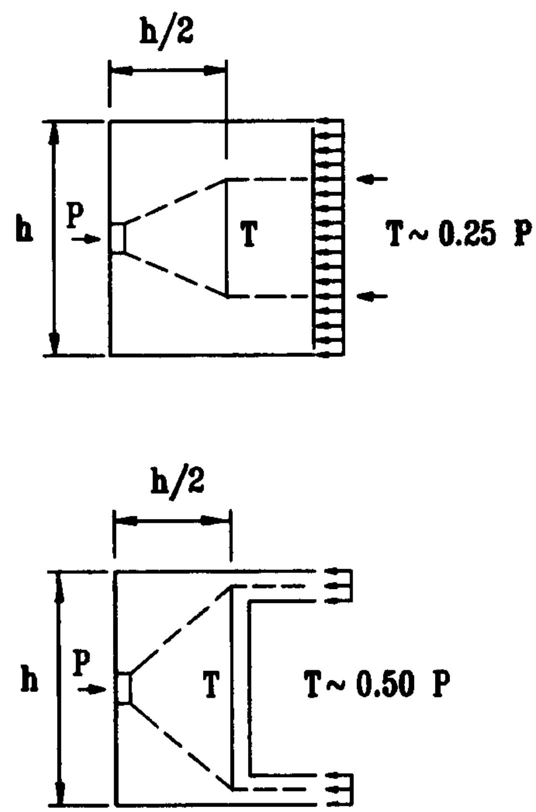


Figure C5.8.4.5.1-1—Effect of Discontinuity in Anchorage Zone

The approximate equations for concrete compressive stresses are based on the assumption that the anchor force spreads in all directions. The minimum edge distance requirement satisfies this assumption and is illustrated in Figure C5.8.4.5.1-2. The approximate equations for bursting forces are based on finite element analyses for a single anchor acting on a rectangular cross-section. Eq. 5.8.4.5.3-1 gives conservative results for the bursting reinforcement, even if the anchors are not closely spaced, but the resultant of the bursting force is located closer to the anchor than indicated by Eq. 5.8.4.5.3-2.

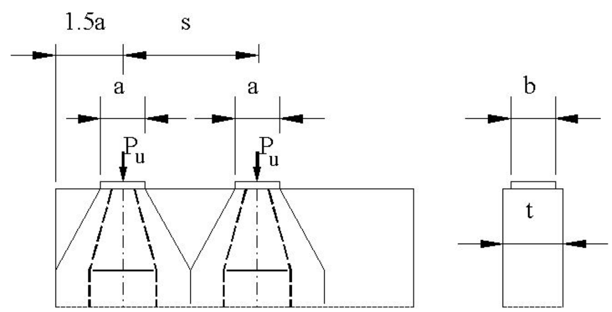


Figure C5.8.4.5.1-2—Edge Distances and Notation

5.8.4.5.2—Compressive Stresses

The concrete compressive stress ahead of the anchorage devices, f_{ca}, calculated using Eq. 5.8.4.5.2-1, shall not exceed the limit specified in Article 5.9.5.6.5a:

$$f_{ca} = \frac{0.6 P_u \kappa}{A_b \left(1 + \ell_c \left(\frac{1}{b_{eff}} - \frac{1}{t}\right)\right)} \qquad (5.8.4.5.2\text{-}1)$$

in which:

if $a_{eff} \leq s < 2a_{eff}$, then :

$$\kappa = 1 + \left(2 - \frac{s}{a_{eff}}\right)\left(0.3 + \frac{n}{15}\right) \qquad (5.8.4.5.2\text{-}2)$$

if $s \geq 2a_{eff}$, then :

$$\kappa = 1 \qquad (5.8.4.5.2\text{-}3)$$

C5.8.4.5.2

This check of concrete compressive stresses is not required for basic anchorage devices satisfying Article 5.8.4.4.2.

Eqs. 5.8.4.5.2-1 and 5.8.4.5.2-2 are based on a strut-and-tie model for a single anchor with the concrete stresses determined as indicated in Figure C5.8.4.5.2-1 (Burdet, 1990), with the anchor plate width, b, and member thickness, t, being equal. Eq. 5.8.4.5.2-1 was modified to include cases with values of $b < t$.

5.9.5.4.4b—Shear Resistance to Pull-Out

The shear resistance per unit length of the concrete cover against pull-out by deviation forces, V_r, shall be taken as:

$$V_r = \phi V_n \tag{5.9.5.4.4b-1}$$

in which:

$$V_n = 0.15 d_{eff} \lambda \sqrt{f'_{ci}} \tag{5.9.5.4.4b-2}$$

where:

V_n = nominal shear resistance of two shear planes per unit length (kips/in.)
ϕ = resistance factor for shear, 0.75
d_{eff} = one-half the effective length of the failure plane in shear and tension for a curved element (in.)
f'_{ci} = design concrete compressive strength at time of application of tendon force (ksi)
λ = concrete density modification factor as specified in Article 5.4.2.8

For single duct stack or for $s_{duct} < d_{duct}$, d_{eff}, shown in Detail (a) in Figure 5.9.5.4.4b-1, shall be taken as:

$$d_{eff} = d_c + \frac{d_{duct}}{4} \tag{5.9.5.4.4b-3}$$

For $s_{duct} \geq d_{duct}$, d_{eff} shall be taken as the lesser of the following based on Paths 1 and 2 shown in Detail (b) in Figure 5.9.5.4.4b-1:

$$d_{eff} = t_w - \frac{d_{duct}}{2} \tag{5.9.5.4.4b-4}$$

$$d_{eff} = d_c + \frac{d_{duct}}{4} + \frac{\sum s_{duct}}{2} \tag{5.9.5.4.4b-5}$$

where:

s_{duct} = clear distance between tendon ducts in vertical direction (in.)
d_{duct} = outside diameter of post-tensioning duct (in.)
d_c = minimum concrete cover over the tendon duct (in.)
t_w = web thickness (in.)

C5.9.5.4.4b

The two shear planes for which Eq. 5.9.5.4.4b-2 gives V_n are as indicated in Figure 5.9.5.4.4b-1 for single and multiple tendons.

Where a staggered or side-by-side group of ducts is located side by side in a single web, all possible shear and tension failure planes should be considered in determining d_{eff}.

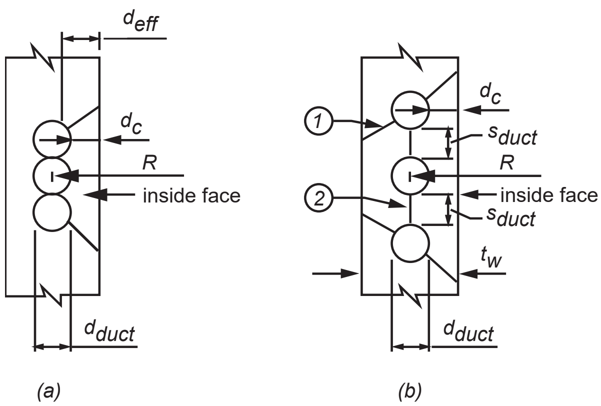


Figure 5.9.5.4.4b-1—Definition of d_{eff}

If the factored in-plane deviation force exceeds the factored shear resistance of the concrete cover, as specified in Eq. 5.9.5.4.4b-1, fully anchored stirrups and duct ties to resist the in-plane deviation forces shall be provided in the form of either nonprestressed or prestressed reinforcement. The duct ties shall be anchored beyond the ducts either by in-plane 90 degree hooks or by hooking around the vertical bar.

Additional information on deviation forces can be found in Nutt, et al. (2008) and Van Landuyt (1991).

Common practice is to limit the stress in the duct ties to 36.0 ksi at the maximum unfactored tensile force.

A generic stirrup and duct tie detail is shown in Figure C5.9.5.4.4b-1. Small diameter reinforcing bars should be used for better anchorage of these bars. There have been no reported web failures where this detail has been used.

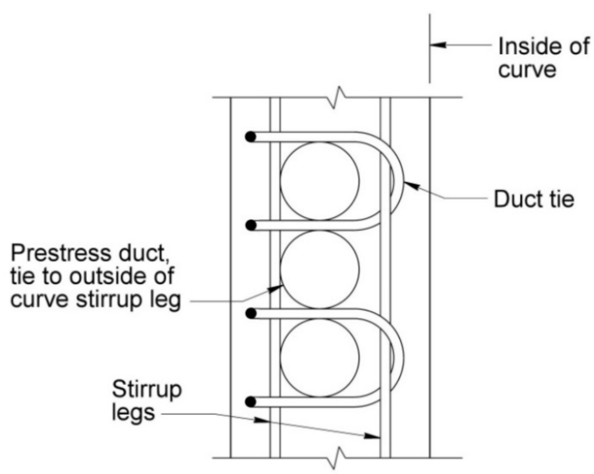


Figure C5.9.5.4.4b-1—Generic Duct Tie Detail

5.9.5.4.4c—Cracking of Cover Concrete

When the clear distance between ducts oriented in a vertical column is less than 1.5 in., the ducts shall be considered stacked. Resistance to cracking shall be investigated at the ends and at midheight of the unreinforced cover concrete.

The applied local moment per unit length at the ends of the cover shall be taken as:

$$M_{end} = \frac{\left(\Sigma F_{u-in} / h_{ds}\right) h_{ds}^2}{12} \qquad (5.9.5.4.4c\text{-}1)$$

C5.9.5.4.4c

Figure C5.9.5.4.4c-1 illustrates the concept of an unreinforced cover concrete beam to be investigated for cracking. Experience has shown that a vertical stack of more than three ducts can result in cracking of the cover concrete. When more than three ducts are required, it is recommended that at least 1.5 in. spacing be provided between the upper and lower ducts of the two stacks.

The resistance factor is based on successful performance of curved post-tensioned box girder bridges in California.

exposed to noncorrosive soil, where the minimum cover shall be 3.0 in.

Cover to epoxy-coated steel may be as shown for interior exposure in Table 5.10.1-1.

Table 5.10.1-1—Cover for Unprotected Main Reinforcing Steel (in.)

Situation	Cover (in.)
Direct exposure to salt water	4.0
Cast against earth	3.0
Coastal	3.0
Exposure to deicing salts	2.5
Deck surfaces subject to tire stud or chain wear	2.5
Exterior other than above	2.0
Interior other than above • Up to No. 11 bar • No. 14 and No. 18 bars	 1.5 2.0
Bottom of cast-in-place slabs • Up to No. 11 bar • No. 14 and No. 18 bars	 1.0 2.0
Precast soffit form panels	0.8
Precast reinforced piles • Noncorrosive environments • Corrosive environments	 2.0 3.0
Precast prestressed piles	2.0
Cast-in-place piles • Noncorrosive environments • Corrosive environments ○ General ○ Protected • Shells • Auger-cast, tremie concrete, or slurry construction	 2.0 3.0 3.0 2.0 3.0
Precast concrete box culverts • Top slabs used as a driving surface • Top slabs with less than 2.0 ft of fill not used as a driving surface • All other members	 2.5 2.0 1.0

"Corrosive" water or soil contains greater than or equal to 500 parts per million (ppm) of chlorides. Sites that are considered corrosive due solely to sulfate content greater than or equal to 2,000 ppm, a pH of less than or equal to 5.5, or both shall be considered noncorrosive in determining minimum cover.

5.10.2—Hooks and Bends

5.10.2.1—Standard Hooks

For the purpose of these Specifications, the term “standard hook” shall mean one of the following:

- For longitudinal reinforcement:

  (a) 180-degree bend, plus a $4.0d_b$ extension, but not less than 2.5 in. at the free end of the bar, or

  (b) 90-degree bend, plus a $12.0d_b$ extension at the free end of the bar.

C5.10.2.1

These requirements are similar to the requirements of ACI 318-14 and CRSI's *Manual of Standard Practice*.

Tests by Shahrooz et al. (2011) showed that standard hooks are adequate for reinforcement with specified minimum yield strengths between 75.0 and 100 ksi if transverse, confining reinforcement as specified in Article 5.10.8.2.4 is provided.

- For transverse reinforcement:
  - (a) No. 5 bar and smaller—90-degree bend, plus a $6.0d_b$ extension at the free end of the bar;
  - (b) No. 6, No. 7 and No. 8 bars—90-degree bend, plus a $12.0d_b$ extension at the free end of the bar; and
  - (c) No. 8 bar and smaller—135-degree bend, plus a $6.0\ d_b$ extension at the free end of the bar.

where:

d_b = nominal diameter of reinforcing bar (in.)

Standard hooks may be used with reinforcing steel having a specified minimum yield strength between 75.0 and 100 ksi for elements and connections specified in Article 5.4.3.3 only if ties specified in Article 5.10.8.2.4 are provided.

5.10.2.2—Seismic Hooks

Seismic hooks meeting the requirements of Article 5.11.4.1.4 shall be used for transverse reinforcement in regions of expected plastic hinges and elsewhere as indicated in the contract documents.

5.10.2.3—Minimum Bend Diameters

The diameter of a bar bend, measured on the inside of the bar, shall not be less than that specified in Table 5.10.2.3-1.

Table 5.10.2.3-1—Minimum Diameters of Bend

Bar Size and Use	Minimum Diameter
No. 3 through No. 5—General	$6.0d_b$
No. 3 through No. 5—Stirrups and Ties	$4.0d_b$
No. 6 through No. 8—General	$6.0d_b$
No. 9, No. 10, and No. 11	$8.0d_b$
No. 14 and No. 18	$10.0d_b$

The inside diameter of bend for stirrups and ties in plain or deformed welded wire reinforcement shall not be less than $4.0d_b$ for deformed wire larger than D6 and $2.0d_b$ for all other wire sizes. Bends with inside diameters of less than $8.0d_b$ shall not be located less than $4.0d_b$ from the nearest welded intersection.

For each end of a single-leg stirrup of welded plain or deformed wire reinforcement, two longitudinal wires at a minimum spacing of 2.0 in. and with the inner wire at not less than $d/4$ or 2.0 in. from middepth of member shall be provided. The outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.

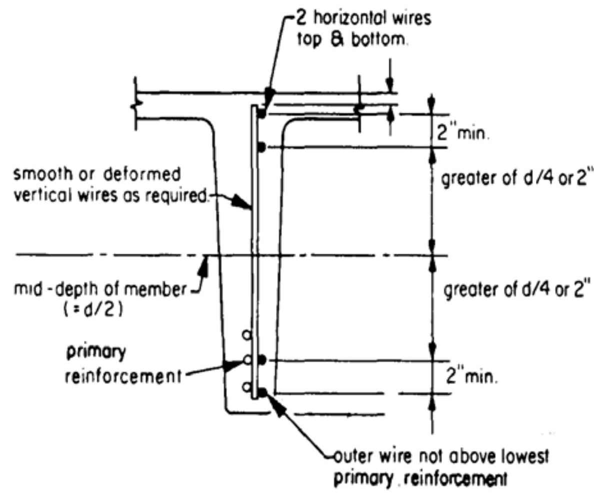


Figure C5.10.8.2.6c-1—Anchorage of Single-Leg Welded Wire Reinforcement Shear Reinforcement, ACI 318-14

5.10.8.2.6d—Closed Stirrups

Pairs of U-stirrups or ties that are placed to form a closed unit shall be considered properly anchored and spliced where length of laps are not less than 1.3 ℓ_d, where ℓ_d in this case is the development length for bars in tension.

In members not less than 18.0 in. deep, closed stirrup splices in stirrup legs extending the full available depth of the member, and with the tension force resulting from factored loads, $A_b f_y$, not exceeding 9.0 kips per leg, may be considered adequate.

Transverse torsion reinforcement shall be made fully continuous and shall be anchored by 135-degree standard hooks around longitudinal reinforcement.

5.10.8.3—Development by Mechanical Anchorages

Any mechanical device capable of developing the strength of reinforcement without damage to concrete may be used as an anchorage. Performance of mechanical anchorages shall be verified by laboratory tests.

Development of reinforcement may consist of a combination of mechanical anchorage and the additional embedment length of reinforcement between the point of maximum bar stress and the mechanical anchorage.

If mechanical anchorages are to be used, complete details shall be shown in the contract documents.

C5.10.8.3

Standard details for such devices have not been developed.

5.10.8.4—Splices of Bar Reinforcement

Reinforcement with specified minimum yield strengths up to 100 ksi may be used in elements and connections specified in Article 5.4.3.3. For spliced bars having a specified minimum yield strength greater than 75.0 ksi, transverse reinforcement satisfying the

C5.10.8.4

Confining reinforcement is not required in slabs or decks.

Research by Shahrooz et al. (2011) verified the use of these provisions for tensile splices for reinforcement with specified minimum yield strengths up to 100 ksi in requirements of Article 5.7.2.5 for beams and Article 5.10.4.3 for columns shall be provided over the required splice length.

applications in Seismic Zone 1. See Article C5.4.3.3 for further information.

5.10.8.4.1—Detailing

Permissible locations, types, and dimensions of splices, including staggers, for reinforcing bars shall be shown in the contract documents.

5.10.8.4.2—General Requirements

5.10.8.4.2a—Lap Splices

This provision of this Article shall apply only to the grades of reinforcement noted.

The lengths of lap for lap splices of individual bars shall be as specified in Articles 5.10.8.4.3a and 5.10.8.4.5a.

Lap splices within bundles shall be as specified in Article 5.10.8.2.3. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced.

For reinforcement in tension, lap splices shall not be used for bars larger than No. 11.

Bars spliced by noncontact lap splices in flexural members shall not be spaced farther apart transversely than the lesser of the following:

- one-fifth the required lap splice length; or
- 6.0 in.w

For columns with longitudinal reinforcement that anchors into oversized shafts, where bars are spliced by noncontact lap splices, and longitudinal column and shaft reinforcement are spaced farther apart transversely than the greater of the following:

- one-fifth the required lap splice length; or
- 6.0 in.,

the spacing of the shaft transverse reinforcement in the splice zone shall meet the requirements of the following equation:

$$S_{\max} = \frac{2\pi A_{sp} f_{ytr} \ell_s}{k A_\ell f_{u\ell}} \qquad (5.10.8.4.2a\text{-}1)$$

where:

S_{max} = spacing of transverse shaft reinforcement (in.)
A_{sp} = area of shaft spiral or transverse reinforcement (in.^{2})
f_{ytr} = specified minimum yield strength of shaft transverse reinforcement (ksi)
ℓ_s = required tension lap splice length of the column longitudinal reinforcement (in.)
A_ℓ = area of longitudinal column reinforcement (in.^{2})

C5.10.8.4.2a

This ratio, k, could be determined from the column moment-curvature analysis using appropriate computer programs. For simplification, k = 0.5 could safely be used in most applications.

The development length of column longitudinal reinforcement in drilled shafts is from WSDOT-TR$_{AC}$ Report WA-RD 417.1 titled *Noncontact Lap Splices in Bridge Column-Shaft Connections*. Eq. 5.10.8.4.2a-1 is based upon a strut-and-tie analogy of the noncontact splice with an assumed strut angle of 45 degrees.

applied shear, introduces compression into the support region of the member and no concentrated load occurs within a distance, h, from the face of the support.

The nominal shear resistance, V_n, shall be determined as the lesser of the following:

$$V_n = V_c + V_s \quad (5.12.5.3.8c\text{-}1)$$

$$V_n = 0.379\lambda\sqrt{f'_c}\, b_v d \quad (5.12.5.3.8c\text{-}2)$$

in which:

$$V_c = 0.0632K\lambda\sqrt{f'_c}\; b_v d \quad (5.12.5.3.8c\text{-}3)$$

$$V_s = \frac{A_v f_y d}{s} \quad (5.12.5.3.8c\text{-}4)$$

$$K = \sqrt{1 + \frac{f_{pc}}{0.0632\lambda\sqrt{f'_c}}} \leq 2.0 \quad (5.12.5.3.8c\text{-}5)$$

Where the effects of torsion are required to be considered by Article 5.7.2.1, the cross-sectional dimensions shall be such that:

$$\left(\frac{V_u}{b_v\, d}\right) + \left(\frac{T_u}{2\, A_o\, b_e}\right) \leq 0.474\lambda\sqrt{f'_c} \quad (5.12.5.3.8c\text{-}6)$$

where:

b_v = effective web width taken as the total minimum width of all webs within the depth d adjusted for the effect of openings or ducts as specified in Article 5.7.2.8.

d = $0.8h$ or the distance from the extreme compression fiber to the centroid of the prestressing reinforcement, whichever is greater (in.)

f'_c = compressive strength of concrete for use in design (ksi)

f_{pc} = unfactored compressive stress in concrete after prestress losses have occurred either at the centroid of the cross-section resisting transient loads or at the junction of the web and flange where the centroid lies in the flange (ksi)

s = spacing of stirrups (in.)

A_v = total area of transverse reinforcing in all webs in the cross-section within a distance s (in.^{2})

V_u = factored design shear including any normal component from the primary prestressing force (kip)

T_u = applied factored torsional moment (kip-in.)

A_o = area enclosed by shear flow path, including any area of holes therein (in.^{2})

Eq. 5.12.5.3.8c-4 is based on an assumed 45-degree truss model.

Eqs. 5.12.5.3.8c-3 and 5.12.5.3.8c-6 are only used to establish appropriate concrete section dimensions.

b_e = the effective thickness of the shear flow path of the elements making up the space truss model resisting torsion calculated in accordance with Article 5.7.2.1 (in.)
ϕ = resistance factor for shear specified in Article 5.5.4.2
λ = concrete density modification factor as specified in Article 5.4.2.8

5.12.5.3.8d—Torsional Reinforcement

Where consideration of torsional effects is required by Article 5.7.2.1 torsion reinforcement shall be provided, as specified herein. This reinforcement shall be in addition to the reinforcement required to resist the factored shear, as specified in Article 5.12.5.3.8c, flexure and axial forces that may act concurrently with the torsion.

The longitudinal and transverse reinforcement required for torsion shall satisfy:

$$T_u \leq \phi T_n \quad (5.12.5.3.8d\text{-}1)$$

The nominal torsional resistance from transverse reinforcement shall be based on a truss model with 45-degree diagonals and shall be computed as:

$$T_n = \frac{2A_o A_t f_y}{s} \quad (5.12.5.3.8d\text{-}2)$$

The minimum additional longitudinal reinforcement for torsion, A_ℓ, shall satisfy:

$$A_\ell \geq \frac{T_u\, p_h}{2\phi A_o f_y} \quad (5.12.5.3.8d\text{-}3)$$

where:

A_t = total area of transverse torsion reinforcing in the exterior web and flange (in.^{2})
A_ℓ = total area of longitudinal torsion reinforcement in a box girder (in.^{2})
T_u = applied factored torsional moment (kip-in.)
p_h = perimeter of the polygon defined by the centroids of the longitudinal chords of the space truss resisting torsion. p_h may be taken as the perimeter of the centerline of the outermost closed stirrups (in.)
A_o = area enclosed by shear flow path, including any area of holes therein (in.^{2})
f_y = yield strength of additional longitudinal reinforcement (ksi)
ϕ = resistance factor for shear specified in Article 5.5.4.2

A_λ shall be distributed around the outer-most webs and top and bottom slabs of the box girder in accordance

C5.12.5.3.8d

Use of reinforcement with f_y greater than 75.0 ksi has not been verified by tests.

In determining the required amount of longitudinal reinforcement, the beneficial effect of longitudinal prestressing is taken into account by considering the longitudinal prestressing force in excess of that required for concurrent flexure and shear as an equivalent area of reinforcement.

The total area of transverse reinforcing, A_t, must be placed in each exterior web and flange that forms the closed section.

Unlike solid sections, when designing the webs of segmental bridges the shear and torsion reinforcing should be directly added together. Reinforcing for transverse bending in the webs and other box girder elements should be accounted for in the total reinforcing demand.

APPENDIX B5—GENERAL PROCEDURE FOR SHEAR DESIGN WITH TABLES

B5.1—BACKGROUND

The general procedure herein is an acceptable alternative to the procedure specified in Article 5.7.3.4.2. The procedure in this Appendix utilizes tabularized values of β and θ instead of Eqs. 5.7.3.4.2-1, 5.7.3.4.2-2, and 5.7.3.4.2-3. Appendix B5 is a complete presentation of the general procedures in LRFD Design (AASHTO 2007) without any interim changes.

B5.2—SECTIONAL DESIGN MODEL—GENERAL PROCEDURE

For sections containing at least the minimum amount of transverse reinforcement specified in Article 5.7.2.5, the values of β and θ shall be as specified in Table B5.2-1. In using this table, ε_x shall be taken as the calculated longitudinal strain at the middepth of the member when the section is subjected to M_u, N_u, and V_u as shown in Figure B5.2-1.

For sections containing less transverse reinforcement than specified in Article 5.7.2.5, the values of β and θ shall be as specified in Table B5.2-2. In using this table, ε_x shall be taken as the largest calculated longitudinal strain which occurs within the web of the member when the section is subjected to N_u, M_u, and V_u as shown in Figure B5.2-2.

Where consideration of torsion is required by the provisions of Article 5.7.2, V_u in Eqs. B5.2-3 through B5.2-5 shall be replaced by V_{eff}.

For solid sections:

$$V_{eff} = \sqrt{V_u^2 + \left(\frac{0.9 p_h T_u}{2A_o}\right)^2} \qquad \text{(B5.2-1)}$$

For hollow sections:

$$V_{eff} = V_u + \frac{T_u d_s}{2A_o} \qquad \text{(B5.2-2)}$$

Unless more accurate calculations are made, ε_x shall be determined as:

- If the section contains at least the minimum transverse reinforcement as specified in Article 5.7.2.5:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5|V_u - V_p|\cot\theta - A_{ps}f_{po}\right)}{2(E_sA_s + E_pA_{ps})} \qquad \text{(B5.2-3)}$$

CB5.2

The shear resistance of a member may be determined by performing a detailed sectional analysis that satisfies the requirements of Article 5.7.3.1. Such an analysis (see Figure CB5.2-1) would show that the shear stresses are not uniform over the depth of the web and that the direction of the principal compressive stresses changes over the depth of the beam. The more direct procedure given herein assumes that the concrete shear stresses are uniformly distributed over an area b_v wide and d_v deep, that the direction of principal compressive stresses (defined by angle θ) remains constant over d_v, and that the shear strength of the section can be determined by considering the biaxial stress conditions at just one location in the web. See Figure CB5.2-2.

For solid cross-section shapes, such as a rectangle or an "I," there is the possibility of considerable redistribution of shear stresses. To make some allowance for this favorable redistribution it is safe to use a root-mean-square approach in calculating the nominal shear stress for these cross-sections, as indicated in Eq. B5.2-1. The 0.9 p_h comes from 90 percent of the perimeter of the spalled concrete section. This is similar to multiplying 0.9 times the lever arm in flexural calculations.

For a hollow girder, the shear flow due to torsion is added to the shear flow due to flexure in one exterior web, and subtracted from the opposite exterior web. In the controlling web, the second term in Eq. B5.2-2 comes from integrating the distance from the centroid of the section, to the center of the shear flow path around the circumference of the section. The stress is converted to a force by multiplying by the web height measured between the shear flow paths in the top and bottom slabs, which has a value approximately equal that of d_s. If the exterior web is sloped, this distance should be divided by the sine of the web angle from horizontal.

Members containing at least the minimum amount of transverse reinforcement have a considerable capacity to redistribute shear stresses from the most highly strained portion of the cross-section to the less highly strained portions. Because

The initial value of ε_x should not be taken greater than 0.001.

- If the section contains less than the minimum transverse reinforcement as specified in Article 5.7.2.5:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5\left|V_u - V_p\right|\cot\theta - A_{ps}f_{po}\right)}{E_s A_s + E_p A_{ps}} \tag{B5.2-4}$$

The initial value of ε_x should not be taken greater than 0.002.

- If the value of ε_x from Eqs. B5.2-3 or B5.2-4 is negative, the strain shall be taken as:

$$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + 0.5\left|V_u - V_p\right|\cot\theta - A_{ps}f_{po}\right)}{2\left(E_c \underline{A_{ct}} + E_s A_s + E_p A_{ps}\right)} \tag{B5.2-5}$$

where:

A_{ct} = area of concrete on the flexural tension side of the member as shown in Figure B5.2-1 (in.^{2})
A_{ps} = area of prestressing steel on the flexural tension side of the member, as shown in Figure B5.2-1 (in.^{2})
A_o = area enclosed by the shear flow path, including any area of holes therein (in.^{2})
A_s = area of nonprestressed steel on the flexural tension side of the member at the section under consideration, as shown in Figure B5.2-1. In calculating A_s for use in this equation, bars which are terminated at a distance less than their development length from the section under consideration shall be ignored (in.^{2})
d_s = distance from extreme compression fiber to the centroid of the nonprestressed tensile reinforcement (in.)
f_{po} = a parameter taken as modulus of elasticity of prestressing steel multiplied by the locked-in difference in strain between the prestressing steel and the surrounding concrete. For the usual levels of prestressing, a value of $0.7f_{pu}$ will be appropriate for both pretensioned and post-tensioned members (ksi)

of this capacity to redistribute, it is appropriate to use the middepth of the member as the location at which the biaxial stress conditions are determined. Members that contain no transverse reinforcement, or contain less than the minimum amount of transverse reinforcement, have less capacity for shear stress redistribution. Hence, for such members, it is appropriate to perform the biaxial stress calculations at the location in the web subject to the highest longitudinal tensile strain; see Figure B5.2-2.

The longitudinal strain at the middepth of the member, ε_x, can be determined by the procedure illustrated in Figure CB5.2-3. The actual section is represented by an idealized section consisting of a flexural tension flange, a flexural compression flange, and a web. The area of the compression flange is taken as the area on the flexure compression side of the member, i.e., the total area minus the area of the tension flange as defined by A_{ct}. After diagonal cracks have formed in the web, the shear force applied to the web concrete, $V_u - V_p$, will primarily be carried by diagonal compressive stresses in the web concrete. These diagonal compressive stresses will result in a longitudinal compressive force in the web concrete of $(V_u - V_p)\cot\theta$. Equilibrium requires that this longitudinal compressive force in the web needs to be balanced by tensile forces in the two flanges, with half the force, that is $0.5(V_u - V_p)\cot\theta$, being taken by each flange. To avoid a trial and error iteration process, it is a convenient simplification to take this flange force due to shear as $V_u - V_p$. This amounts to taking $0.5\cot\theta = 1.0$ in the numerator of Eqs. B5.2-3, B5.2-4, and B5.2-5. This simplification is not expected to cause a significant loss of accuracy. After the required axial forces in the two flanges are calculated, the resulting axial strains, ε_t and ε_c, can be calculated based on the axial force-axial strain relationship shown in Figure CB5.2-4.

For members containing at least the minimum amount of transverse reinforcement, ε_x can be taken as:

$$\varepsilon_x = \frac{\varepsilon_t + \varepsilon_c}{2} \tag{CB5.2-1}$$

where ε_t and ε_c are positive for tensile strains and negative for compressive strains. If, for a member subject to flexure, the strain ε_c is assumed to be negligibly small, then ε_x becomes one half of ε_t. This is the basis for the expression for ε_x given in Eq. B5.2-3. For members containing less than the minimum amount of transverse reinforcement, Eq. B5.2-4 makes the conservative simplification that ε_x is equal to ε_t.

In some situations, it will be more appropriate to determine ε_x using the more accurate procedure of Eq. CB5.2-1 rather than the simpler Eqs. B5.2-3 through B5.2-5. For example, the shear capacity of sections near the ends of precast, pretensioned simple beams made continuous for live load will be estimated in a very conservative manner by

6.8.2—Tensile Resistance

6.8.2.1—General

The factored tensile resistance, P_r, shall be taken as the lesser of the values given by Eqs. 6.8.2.1-1 and 6.8.2.1-2.

$$P_r = \phi_y P_{ny} = \phi_y F_y A_g \quad (6.8.2.1\text{-}1)$$

$$P_r = \phi_u P_{nu} = \phi_u F_u A_n R_p U \quad (6.8.2.1\text{-}2)$$

where:

P_{ny} = nominal tensile resistance for yielding in gross section (kip)
F_y = specified minimum yield strength (ksi)
A_g = gross cross-sectional area of the member (in.^{2})
F_u = tensile strength (ksi)
A_n = net area of the member as specified in Article 6.8.3 (in.^{2})
R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size
U = reduction factor to account for shear lag; 1.0 for components in which force effects are transmitted to all elements, and as specified in Article 6.8.2.2 for other cases
ϕ_y = resistance factor for yielding of tension members as specified in Article 6.5.4.2
ϕ_u = resistance factor for fracture of tension members as specified in Article 6.5.4.2

C6.8.2.1

The reduction factor, U, does not apply when checking yielding on the gross section because yielding tends to equalize the nonuniform tensile stresses caused over the cross-section by shear lag. The reduction factor, R_p, conservatively accounts for the reduced fracture resistance in the vicinity of bolt holes that are punched full size (Brown et al., 2007). No reduction in the net section fracture resistance is required for holes that are drilled full size or subpunched and reamed to size. The reduction in the factored resistance for punched holes was previously accounted for by increasing the hole size for design by 0.625 in., which penalized drilled and subpunched and reamed holes and did not provide a uniform reduction for punched holes since the reduction varied with the hole size.

Due to strain hardening, a ductile steel loaded in axial tension can resist a force greater than the product of its gross area and its yield strength prior to fracture. However, excessive elongation due to uncontrolled yielding of gross area not only marks the limit of usefulness but it can precipitate failure of the structural system of which it is a part. Depending on the ratio of net area to gross area and the mechanical properties of the steel, the component can fracture by failure of the net area at a load smaller than that required to yield the gross area. General yielding of the gross area and fracture of the net area both constitute measures of component strength. The relative values of the resistance factors for yielding and fracture reflect the different reliability indices deemed proper for the two modes.

The part of the component occupied by the net area at fastener holes generally has a negligible length relative to the total length of the member. As a result, the strain hardening is quickly reached and, therefore, yielding of the net area at fastener holes does not constitute a strength limit of practical significance, except perhaps for some builtup members of unusual proportions.

For welded connections, A_n is the gross section less any access holes in the connection region.

6.8.2.2—Reduction Factor, *U*

The shear lag reduction factor, *U,* shall be used when investigating the tension fracture check specified in Article 6.8.1 at the strength limit state.

In the absence of more refined analysis or tests, the reduction factors specified herein may be used to account for shear lag in connections.

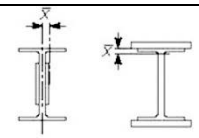
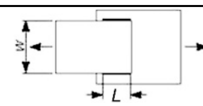
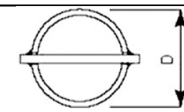
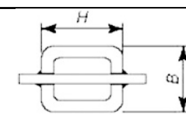
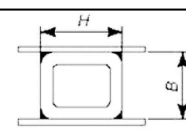
The shear lag reduction factor, *U,* may be calculated as specified in Table 6.8.2.2-1. For members composed of more than one element, the calculated value of *U* should not be taken to be less than the ratio of the gross area of the connected element or elements to the member gross area.

C6.8.2.2

The provisions of Article 6.8.2.2 are adapted from the 2005 AISC Specification Section D3.3, Effective Net Area for design of tension members. The 2005 AISC provisions are adapted such that they are consistent with updated draft 2010 AISC provisions. These updated provisions specify that, for members composed of more than one element, the calculated value of *U* should not be taken to be less than the ratio of the gross area of the connected element or elements to the member gross area.

Examples of the distances $\bar{x}$ and L used in the calculation of the reduction factor *U* for all types of tension members, except plates and Hollow Structural Section (HSS) members, are illustrated in Figure C6.8.2.2-1.

Table 6.8.2.2-1—Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of cross-sectional elements by fasteners or welds (except as in Cases 3, 4, 5, and 6).		$U = 1.0$	—
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds. (Alternatively, for W, M, S, and HP, Case 7 may be used.)		$U = 1 - \frac{\bar{x}}{L}$	
3	All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A = area of the directly connected elements	—
4	Plates where the tension load is transmitted by longitudinal welds only.		$L \geq 2w \ldots U = 1.0$ $2w > L \geq 1.5w \ldots U = 0.87$ $1.5w > L \geq w \ldots U = 0.75$	
5	Round HSS with a single concentric gusset plate.		$L \geq 1.3D \ldots U = 1.0$ $D \leq L < 1.3D \ldots U = 1 - \frac{\bar{x}}{L}$ $\bar{x} = \frac{D}{\pi}$	
6	Rectangular HSS	with a single concentric gusset plate	$L \geq H \ldots U = 1 - \frac{\bar{x}}{L}$ $\bar{x} = \frac{B^2 + 2BH}{4(B+H)}$	
		with 2 side gusset plates	$L \geq H \ldots U = 1 - \frac{\bar{x}}{L}$ $\bar{x} = \frac{B^2}{4(B+H)}$	
7	W, M, S, or HP Shapes or Tees cut from these shapes (If U is calculated per Case 2, the larger value is permitted to be used.)	with flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq \frac{2}{3}d \ldots U = 0.90$ $b_f < \frac{2}{3}d \ldots U = 0.85$	—
		with web connected with 4 or more fasteners in direction of loading	$U = 0.70$	—
8	Single angles (If U is calculated per Case 2, the larger value is permitted to be used.)	with 4 or more fasteners per line in direction of loading	$U = 0.80$	—
		with 2 or 3 fasteners per line in direction of loading	$U = 0.60$	—

where:

L = length of connection (in.)
w = plate width (in.)
$\bar{x}$ = connection eccentricity (in.)
B = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection (in.)
H = overall height of rectangular HSS member, measured in the plane of the connection (in.)
d = full nominal depth of section (in.)
b_f = flange width (in.)

I_{yc} = moment of inertia of the compression flange of the steel section about the vertical axis in the plane of the web (in.^{4})

I_{yt} = moment of inertia of the tension flange of the steel section about the vertical axis in the plane of the web (in.^{4})

that the boundary conditions assumed at the web-flange juncture in the web bend-buckling and compression-flange local buckling formulations within these Specifications are sufficiently accurate. The ratio of the web area to the compression flange area is always less than or equal to 5.45 for members that satisfy Eqs. 6.10.2.2-2 and 6.10.2.2-3. Therefore, the AISC (2016) limit of 10 on this ratio is not required.

An I-section with a ratio of I_{yc}/I_{yt} outside the limits specified in Eq. 6.10.2.2-4 is more like a tee-section with the shear center located at the intersection of the larger flange and the web. The limits of Eq. 6.10.2.2-4 are similar to the limits specified in previous Specifications, but are easier to apply
since they are based on the ratio of I_{yc} to I_{yt} rather than to I_y of the entire steel section. Eq. 6.10.2.2-4 ensures more efficient flange proportions and prevents the use of sections that may be particularly difficult to handle during construction. Also, Eq. 6.10.2.2-4 ensures the validity of the equations for $C_b > 1$ in cases involving moment gradients. Furthermore, these limits tend to prevent the use of extremely monosymmetric sections for which the larger of the yield moments, M_{yc} or M_{yt}, may be greater than the plastic moment, M_p. If the flanges are composed of plates of equal thickness, these limits are equivalent to $b_{fc} \geq 0.46 b_{ft}$ and $b_{fc} \leq 2.15\ b_{ft}$.

The advent of composite design has led to a significant reduction in the size of compression flanges in regions of positive flexure. In addition to satisfying the proportion limits given in this Article, the minimum compression-flange width in these regions for preliminary design should also be established based on the L/b_{fc} guideline suggested in Eq. C6.10.3.4.1-1.

6.10.3—Constructibility

6.10.3.1—General

The provisions of Article 2.5.3 shall apply. In addition to providing adequate strength, nominal yielding or reliance on post-buckling resistance shall not be permitted for main load-carrying members during critical stages of construction, except for yielding of the web in hybrid sections. This shall be accomplished by satisfying the requirements of Articles 6.10.3.2 and 6.10.3.3 at each critical construction stage. For sections in positive flexure that are composite in the final condition, but are noncomposite during construction, the provisions of Article 6.10.3.4 shall apply. For investigating the constructibility of flexural members, all loads shall be factored as specified in Article 3.4.2. For the calculation of deflections, the load factors shall be taken as 1.0.

Potential uplift at bearings shall be investigated at each critical construction stage.

Webs without bearing stiffeners at locations subjected to concentrated loads not transmitted through a deck or deck system shall satisfy the provisions of Article D6.5.

C6.10.3.1

If uplift is indicated at any critical stage of construction, temporary load may be placed to prevent lift-off. The magnitude and position of any required temporary load should be provided in the contract documents.

Factored forces at high-strength bolted joints of load carrying members are limited to the slip resistance of the connection during each critical construction state to ensure that the correct geometry of the structure is maintained.

If there are holes in the tension flange at the section under consideration, the tension flange shall also satisfy the requirement specified in Article 6.10.1.8.

Load-resisting bolted connections either in or to flexural members shall be proportioned to prevent slip under the factored loads at each critical construction stage. The provisions of Article 6.13.2.8 shall apply for investigation of connection slip.

6.10.3.2—Flexure

6.10.3.2.1—Discretely Braced Flanges in Compression

For critical stages of construction, each of the following requirements shall be satisfied. For sections with slender webs, Eq. 6.10.3.2.1-1 shall not be checked when f_ℓ is equal to zero. For sections with compact or noncompact webs, Eq. 6.10.3.2.1-3 shall not be checked.

$$f_{bu} + f_l \leq \phi_f R_h F_{yc}, \qquad (6.10.3.2.1\text{-}1)$$

$$f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nc}, \qquad (6.10.3.2.1\text{-}2)$$

and

$$f_{bu} \leq \phi_f F_{crw} \qquad (6.10.3.2.1\text{-}3)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2.

f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)

f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)

F_{crw} = nominal bend-buckling resistance for webs specified in Article 6.10.1.9 (ksi)

F_{nc} = nominal flexural resistance of the flange (ksi). F_{nc} shall be determined as specified in Article 6.10.8.2. For sections in straight I-girder bridges with compact or noncompact webs, the lateral torsional buckling resistance may be taken as M_{nc} determined as specified in Article A6.3.3 divided by S_{xc}. In computing F_{nc} for constructibility, the web load-shedding factor, R_b, shall be taken as 1.0.

M_{yc} = yield moment with respect to the compression flange determined as specified in Article D6.2 (kip-in.)

R_h = hybrid factor specified in Article 6.10.1.10.1. For hybrid sections in which f_{bu} does not exceed the specified minimum yield strength of the web, the hybrid factor shall be taken equal to 1.0.

C6.10.3.2.1

A distinction is made between discretely and continuously braced compression and tension flanges because for a continuously braced flange, flange lateral bending need not be considered.

This Article gives constructibility requirements for discretely braced compression flanges, expressed by Eqs. 6.10.3.2.1-1, 6.10.3.2.1-2, and 6.10.3.2.1-3 in terms of the combined factored vertical and flange lateral bending stresses during construction. In making these checks, the stresses f_{bu} and f_ℓ must be determined according to the procedures specified in Article 6.10.1.6.

Eq. 6.10.3.2.1-1 ensures that the maximum combined stress in the compression flange will not exceed the specified minimum yield strength of the flange times the hybrid factor; that is, it is a yielding limit state check.

Eq. 6.10.3.2.1-2 ensures that the member has sufficient strength with respect to lateral torsional and flange local buckling based limit states, including the consideration of flange lateral bending where these effects are judged to be significant. For horizontally-curved bridges, flange lateral bending effects due to curvature must always be considered in discretely braced flanges during construction.

Eq. 6.10.3.2.1-3 ensures that theoretical web bend-buckling will not occur during construction.

Eq. 6.10.3.2.1-2 addresses the resistance of the compression

flange by considering this element as an equivalent beam-column. This equation is effectively a beam-column interaction equation, expressed in terms of the flange stresses computed from elastic analysis (White and Grubb, 2005). The f_{bu} term is analogous to the axial load and the f_ℓ term is analogous to the bending moment within the equivalent beam-column member. The factor of 1/3 in front of the f_ℓ term in Eq. 6.10.3.2.1-2 gives an accurate linear approximation of the equivalent beam-column resistance within the limits on f_ℓ specified in Article 6.10.1.6 (White and Grubb, 2005).

Eq. 6.10.3.2.1-1 often controls relative to Eq. 6.10.3.2.1-2, particularly for girders with large f_ℓ and for members with compact or noncompact webs. However, for members with noncompact flanges or large unsupported lengths during construction combined with small or zero

I_y = noncomposite moment of inertia about the vertical centroidal axis of a single girder within the span under consideration (in.^{4})

L = length of the span under consideration (in.)

t = distance from the centroid of the noncomposite steel section under consideration to the centroid of the tension flange (in.). The distance shall be taken as positive.

w_g = girder spacing for a two-girder system or the distance between the two exterior girders of the unit for a three-girder system (in.)

Should the sum of the largest total factored girder moments across the width of the unit within the span under consideration exceed 70 percent of M_{gs}, the following alternatives may be considered:

- The addition of flange level lateral bracing adjacent to the supports of the span may be considered as discussed in Article 6.7.5.2;
- The unit may be revised to increase the system stiffness; or
- The amplified girder second-order displacements of the span during the deck placement may be evaluated to verify that they are within tolerances permitted by the Owner.

provided in Yura et. al. (2008) be used, as Eq. 6.10.3.4.2-1 becomes more conservative in this case. Yura et al. (2008) further indicates the adjustments that need to be made to the more general buckling equation for singly symmetric girders and/or for three-girder systems.

Large global torsional rotations signified by large differential vertical deflections between the girders and also large lateral deflections, as determined from a first-order analysis, are indicative of the potential for significant second-order global amplification. Situations exhibiting potentially significant global second-order amplification include phased construction involving narrow unsupported units with only two or three girders and possibly unevenly applied deck weight. One suggested method of increasing the global buckling resistance in such cases is to consider the addition of flange level lateral bracing to the system. Yura et al. (2008) suggest adjustments to be made when estimating the elastic global lateral-torsional buckling resistance of the system where a partial top-flange lateral bracing system is present at the ends of the span, along with some associated bracing design recommendations.

The elastic global buckling resistance should only be used as a general indicator of the susceptibility of horizontally-curved I-girder systems to second-order amplification under noncomposite loading conditions. Narrow horizontally-curved I-girder bridge units that meet both of the conditions stated in this article in their noncomposite condition during the deck placement may be subject to significant second-order amplification and should instead be analyzed using a global second-order load-deflection analysis to evaluate the behavior. As an alternative, the addition of flange level lateral bracing adjacent to the supports of the span may be considered as discussed in Article 6.7.5.2, or the unit can be braced to other structural units or by external bracing within the span.

6.10.3.5—Dead Load Deflections

The provisions of Article 6.7.2 shall apply, as applicable.

C6.10.3.5

If staged construction is specified, the sequence of load application should be recognized in determining the camber and stresses.

6.10.4—Service Limit State

6.10.4.1—Elastic Deformations

The provisions of Article 2.5.2.6 shall apply, as applicable.

C6.10.4.1

The provisions of Article 2.5.2.6 contain optional live load deflection criteria and criteria for span-to-depth ratios. In the absence of depth restrictions, the span-to-depth ratios should be used to establish a reasonable minimum web depth for the design.

6.10.4.2—Permanent Deformations

6.10.4.2.1—General

For the purposes of this Article, the Service II load combination specified in Table 3.4.1-1 shall apply.

The following methods may be used to calculate stresses in structural steel at the Service II limit state:

- For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 6.10.1.7, flexural stresses in the structural steel caused by Service II loads applied to the composite section may be computed using the short-term or long-term composite section, as appropriate. The concrete deck may be assumed to be effective for both positive and negative flexure, provided that the maximum longitudinal tensile stresses in the concrete deck at the section under consideration caused by the Service II loads are smaller than $2f_r$, where f_r is the modulus of rupture of the concrete specified in Article 6.10.1.7.
- For sections that are composite for negative flexure with maximum longitudinal tensile stresses in the concrete deck greater than or equal to $2f_r$, the flexural stresses in the structural steel caused by Service II loads shall be computed using the section consisting of the steel section and the longitudinal reinforcement within the effective width of the concrete deck.
- For sections that are noncomposite for negative flexure, the properties of the steel section alone shall be used for calculation of the flexural stresses in the structural steel.

The longitudinal stresses in the concrete deck shall be determined as specified in Article 6.10.1.1.1d.

C6.10.4.2.1

These provisions are intended to apply to the design live load specified in Article 3.6.1.1. If this criterion were to be applied to a design permit load, a reduction in the load factor for live load should be considered.

Article 6.10.1.7 requires that one percent longitudinal deck reinforcement be placed wherever the tensile stress in the concrete deck due to either factored construction loads or due to Load Combination Service II exceeds the factored modulus of rupture of the concrete. By controlling the crack size in regions where adequate shear connection is also provided, the concrete deck may be considered effective in tension for computing flexural stresses on the composite section due to Load Combination Service II.

The cracking behavior and the partial participation of the physically cracked slab in transferring forces in tension is very complex. Article 6.10.4.2.1 provides specific guidance that the concrete slab may be assumed to be uncracked when the maximum longitudinal concrete tensile stress is smaller than $2f_r$. This limit between the use of an uncracked or cracked section for calculation of flexural stresses in the structural steel is similar to a limit suggested in CEN (2004) beyond which the effects of concrete cracking should be considered.

6.10.4.2.2—Flexure

Flanges shall satisfy the following requirements:

- For the top steel flange of composite sections:

$$f_f \leq 0.95 R_h F_{yf} \quad (6.10.4.2.2\text{-}1)$$

- For the bottom steel flange of composite sections:

$$f_f + \frac{f_l}{2} \leq 0.95 R_h F_{yf} \quad (6.10.4.2.2\text{-}2)$$

- For both steel flanges of noncomposite sections:

$$f_f + \frac{f_l}{2} \leq 0.80 R_h F_{yf} \quad (6.10.4.2.2\text{-}3)$$

where:

C6.10.4.2.2

Eqs. 6.10.4.2.2-1 through 6.10.4.2.2-3 are intended to prevent objectionable permanent deflections due to expected severe traffic loadings that would impair rideability. For homogeneous sections with zero flange lateral bending, they correspond to the overload check in the 2002 AASHTO Standard Specifications and are based on successful past practice. Their development is described in Vincent (1969). A resistance factor is not applied in these equations because the specified limits are serviceability criteria for which the resistance factor is 1.0.

Eqs. 6.10.4.2.2-1 through 6.10.4.2.2-3 address the increase in flange stresses caused by early web yielding in hybrid sections by including the hybrid factor R_h.

For continuous-span members in which noncomposite sections are utilized in negative flexure regions only, it is recommended that Eqs. 6.10.4.2.2-1 and 6.10.4.2.2-2, as applicable, be applied in those regions.

by Eq. 6.10.4.2.2-2 will often govern the design of the bottom flange of compact composite sections in positive flexure wherever the nominal flexural resistance at the strength limit state is based on either Eq. 6.10.7.1.2-1, 6.10.7.1.2-2, or 6.10.7.1.2-3. Thus, it is prudent and expedient to initially design these types of sections to satisfy this permanent deflection service limit state criterion and then to subsequently check the nominal flexural resistance at the strength limit state according to the applicable Eq. 6.10.7.1.2-1, 6.10.7.1.2-2, or 6.10.7.1.2-3.

6.10.7.2—Noncompact Sections

6.10.7.2.1—General

At the strength limit state, the compression flange shall satisfy:

$$f_{bu} \leq \phi_f F_{nc} \quad (6.10.7.2.1\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)

F_{nc} = nominal flexural resistance of the compression flange determined as specified in Article 6.10.7.2.2 (ksi)

The tension flange shall satisfy:

$$f_{bu} + \frac{1}{3} f_l \leq \phi_f F_{nt} \quad (6.10.7.2.1\text{-}2)$$

where:

f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)

F_{nt} = nominal flexural resistance of the tension flange determined as specified in Article 6.10.7.2.2 (ksi)

The maximum longitudinal compressive stress in the concrete deck at the strength limit state, determined as specified in Article 6.10.1.1.1d, shall not exceed $0.6f'_c$.

C6.10.7.2.1

For noncompact sections, the compression flange must satisfy Eq. 6.10.7.2.1-1 and the tension flange must satisfy Eq. 6.10.7.2.1-2 at the strength limit state. The basis for Eq. 6.10.7.2.1-2 is explained in Article C6.10.8.1.2. For composite sections in positive flexure, lateral bending does not need to be considered in the compression flange at the strength limit state because the flange is continuously supported by the concrete deck.

For noncompact sections, the longitudinal stress in the concrete deck is limited to $0.6f'_c$ to ensure linear behavior of the concrete, which is assumed in the calculation of the steel flange stresses. This condition is unlikely to govern except in cases involving: (1) shored construction, or unshored construction where the noncomposite steel dead load stresses are low, combined with (2) geometries causing the neutral axis of the short-term and long-term composite section to be significantly below the bottom of the concrete deck.

6.10.7.2.2—Nominal Flexural Resistance

The nominal flexural resistance of the compression flange shall be taken as:

$$F_{nc} = R_b R_h F_{yc} \quad (6.10.7.2.2\text{-}1)$$

where:

R_b = web load-shedding factor determined as specified in Article 6.10.1.10.2

C6.10.7.2.2

The nominal flexural resistance of noncompact composite sections in positive flexure is limited to the moment at first yield. Thus, the nominal flexural resistance is expressed simply in terms of the flange stress. For noncompact sections, the elastically computed stress in each flange due to the factored loads, determined in accordance with Article 6.10.1.1.1a, is compared with the yield stress of the flange times the appropriate flange-strength reduction factors.

R_h = hybrid factor determined as specified in Article 6.10.1.10.1

The nominal flexural resistance of the tension flange shall be taken as:

$$F_{nt} = R_h F_{yt} \qquad (6.10.7.2.2\text{-}2)$$

6.10.7.3—Ductility Requirement

Compact and noncompact sections shall satisfy:

$$D_p \leq 0.42\, D_t \qquad (6.10.7.3\text{-}1)$$

where:

D_p = distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment (in.)

D_t = total depth of the composite section (in.)

C6.10.7.3

The ductility requirement specified in this Article is intended to protect the concrete deck from premature crushing. The limit of $D_p < 5D'$ in AASHTO (1998) corresponds to $D_p/D_t < 0.5$ for $\beta = 0.75$. The D_p/D_t ratio is lowered to 0.42 in Eq. 6.10.7.3-1 to ensure significant yielding of the bottom flange when the crushing strain is reached at the top of concrete deck for all potential cases. In checking this requirement, D_t should be computed using a lower bound estimate of the actual thickness of the concrete haunch, or may be determined conservatively by neglecting the thickness of the haunch.

6.10.8—Flexural Resistance—Composite Sections in Negative Flexure and Noncomposite Sections

6.10.8.1—General

6.10.8.1.1—Discretely Braced Flanges in Compression

At the strength limit state, the following requirement shall be satisfied:

$$f_{bu} + \frac{1}{3} f_\ell \leq \phi_f F_{nc} \qquad (6.10.8.1.1\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)

f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)

F_{nc} = nominal flexural resistance of the flange determined as specified in Article 6.10.8.2 (ksi)

C6.10.8.1.1

Eq. 6.10.8.1.1-1 addresses the resistance of the compression flange by considering this element as an equivalent beam-column. This equation is effectively a beam-column interaction equation, expressed in terms of the flange stresses computed from elastic analysis (White and Grubb, 2004). The f_{bu} term is analogous to the axial load and the f_ℓ term is analogous to the bending moment within the equivalent beam-column member. The factor of one-third in front of the f_ℓ term in Eq. 6.10.8.1.1-1 gives an accurate linear approximation of the equivalent beam-column resistance within the limits on f_ℓ specified in Article 6.10.1.6 (White and Grubb, 2005).

Eqs. 6.10.8.1.1-1, 6.10.8.1.2-1, and 6.10.8.1.3-1 are developed specifically for checking of slender-web noncomposite sections and slender-web composite sections in negative flexure. These equations may be used as a simple conservative resistance check for other types of composite sections in negative flexure and noncomposite sections. The provisions specified in Appendix A6 may be used for composite sections in negative flexure and for noncomposite sections with compact or noncompact webs in straight bridges for which the specified minimum yield strengths of the flanges and web do not exceed 70 ksi and for which the flanges satisfy Eq. 6.10.6.2.3-2. The Engineer should give consideration to utilizing the provisions of Appendix A6 for such sections in straight moment arm taken as the vertical distance between the mid-thickness of the top and bottom flanges.

At the strength limit state, the design force in splice plates subjected to tension shall not exceed the factored resistance in tension specified in Article 6.13.5.2. The design force in splice plates subjected to compression shall not exceed the factored resistance, R_r, in compression taken as:

$$R_r = \phi_c F_y A_s \qquad (6.13.6.1.3b\text{-}3)$$

where:

ϕ_c = resistance factor for compression as specified in Article 6.5.4.2
F_y = specified minimum yield strength of the splice plate (ksi)
A_s = gross area of the splice plate (in.^{2})

Bolted connections for flange splices shall be checked for slip under a flange slip force determined as the factored moment at the point of splice divided by the appropriate moment arm defined as specified herein. The factored moment 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, or the moment at the point of splice due to the deck casting sequence, whichever governs.

The computed flange 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 flange splice bolts required on one side of the splice to resist slip. For all single box sections, and for multiple box sections in bridges not satisfying the requirements of Article 6.11.2.3, including horizontally-curved bridges, or with box flanges that are not fully effective according to the provisions of Article 6.11.1.1, longitudinal warping stresses due to cross-section distortion shall be considered when checking bolted flange splices for slip and for fatigue. Longitudinal warping stresses may be ignored at the strength limit state. The vector sum of the St. Venant torsional shear and the flange slip force or design yield resistance shall be considered in the design of box-flange bolted splices for these sections at the corresponding applicable limit state.

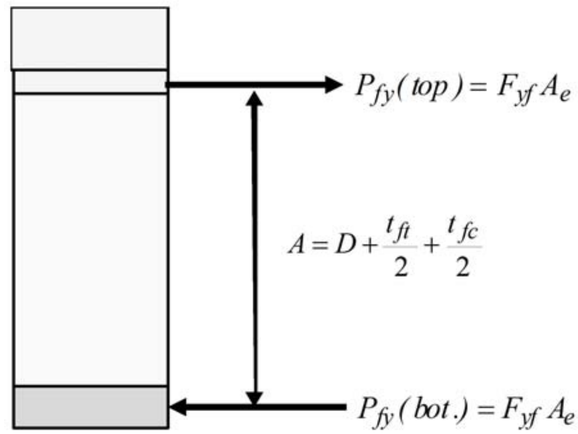


Figure C6.13.6.1.3b-2—Calculation of the Moment Resistance Provided by the Flange Splices for Composite Sections Subject to Negative Flexure and Noncomposite Sections

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 according to Eq. 6.13.6.1.3b-3. Eq. 6.13.6.1.3b-3 assumes an unbraced length of zero for the splice plates.

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 slip resistance should always be determined by dividing the flange slip force 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, including sections in horizontally-curved bridges, longitudinal warping stresses due to cross-section distortion can be more significant under construction and service conditions and must therefore be considered when checking the connections of bolted flange splices for slip and for fatigue. The warping stresses in these cases can typically be ignored in checking the top-flange splices once the flange is continuously braced. The warping stresses can also be ignored when checking splices in both the top and bottom flanges at the strength limit state. For these sections, St. Venant torsional shear must also be considered in the design of box-flange bolted splices at all limit states. St. Venant torsional shears are typically neglected in top flanges of tub sections once the flanges are continuously braced.

For straight girders where flange lateral bending is deemed significant, and for horizontally-curved girders, the effects of the lateral bending need not be considered in the design of the bolted splices for discretely braced top flanges of tub sections or discretely braced flanges of I-sections at all limit states. At the strength limit state, flange splices are to be designed to develop the full yield resistance of the flange, which cannot be exceeded in such cases under combined major-axis and lateral bending at the strength limit state. 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 at the point of splice determined according to the provisions of Article 6.10.9 or 6.11.9, as applicable.

Should the moment resistance provided by the flange splices, determined as specified in Article 6.13.6.1.3b, not be sufficient to resist the factored moment at the strength limit state at the point of splice, 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 ~~located at the mid-depth of~~ in the web that provides the necessary moment resistance in conjunction with the flange splices.

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 flange splices divided by the appropriate moment arm ~~to the mid-depth of the web~~. 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 vertical distance from the mid-depth of the web to the mid-thickness of the top or bottom flange, whichever flange has the larger design yield resistance,~~ ~~P~~$_{fy}$~~.~~

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.

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, the effect of the small moment introduced by the eccentricity of the web connection may be ignored at all limit states. Also, for all single box sections, and for multiple box sections in bridges not satisfying the requirements of Article 6.11.2.3, including horizontally-curved bridges, or with box flanges that are not fully effective according to the provisions of Article 6.11.1.1, the effect of the additional St. Venant torsional shear in the web may be ignored at the strength limit state.

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 at the strength limit state that exceeds the moment resistance provided by the flange splices. 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.

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.

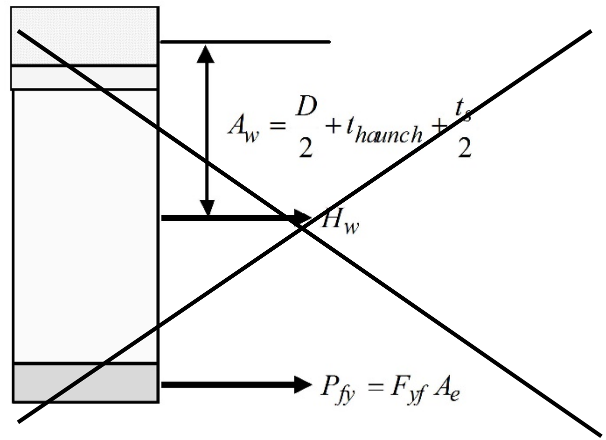
The design web force at the strength limit state shall not exceed the lesser of the factored shear resistances of the web splice plates determined as specified in Articles 6.13.4 and 6.13.5.3.

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 nominal slip resistance provided by the flange bolts not be sufficient to resist the flange slip force due to the factored moment at the point of splice, determined as specified in Article 6.13.6.1.3b, 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 the portion of the flange slip force that exceeds the nominal slip resistance of the flange bolts. 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 shear in the web at the point of splice due to the deck casting sequence, whichever governs.

For all single box sections, and for multiple box sections in bridges not satisfying the requirements of Article 6.11.2.3, including horizontally-curved bridges, or with box flanges that are not fully effective according to the provisions of Article 6.11.1.1, 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 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. For bolted web splices with thickness differences of 0.0625 in. or less, filler plates should not be provided.



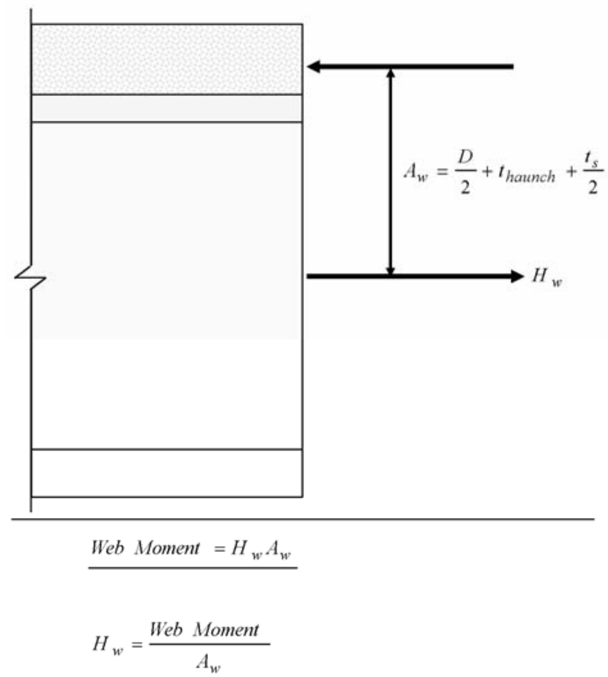
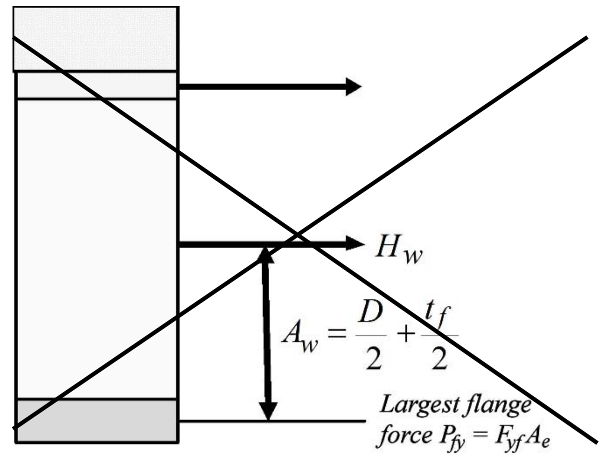
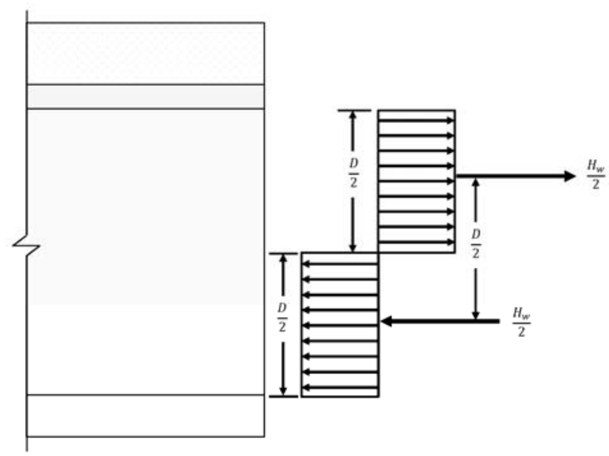


Figure C6.13.6.1.3c-1—Calculation of the Horizontal Force in the Web, H_w, for Composite Sections Subject to Positive Flexure

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 at the strength limit state that exceeds the moment resistance provided by the flange splices. 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.~~the moment arm, A_w, to the mid-thickness of the top or bottom flange, whichever flange has the larger value of P_{fy}:~~





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

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

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 in cases where H_w is computed is divided equally to all of the bolts in this approach, the traditional vector analysis for bolt groups subject to a concentric shear and a centroidal moment 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 described above when checking for slip.

When checking the bearing resistance of the web at bolt holes for an inclined resultant design web force, the resistance of an outermost hole, calculated using the clear edge distance, can conservatively be checked against the resultant force assumed to be acting on the extreme bolt in the connection as shown on the left of Figure C6.13.6.1.3c-3. This check is conservative since the resultant force acts in the direction of an inclined distance that is larger than the clear edge distance. 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. In cases where the bearing resistance of the web splice plates controls, the smaller of the clear edge or end distance on the splice plates can be used to compute the bearing resistance of the outermost hole as shown on the right of Figure C6.13.6.1.3c-3.

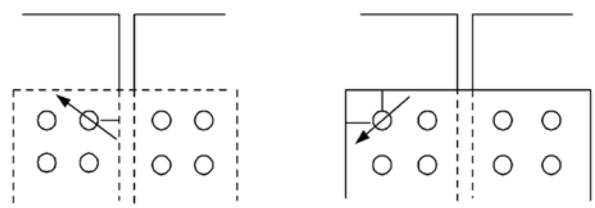


Figure C6.13.6.1.3c-3—Critical Locations for Checking Bearing Resistance of Outermost Web Splice Bolt Holes for an Inclined Resultant Design Web Force

Required bolt assembly clearances are given in AISC (2011).

6.13.6.1.4—Fillers

When bolts carrying loads pass through fillers 0.25 in. or more in thickness in axially loaded connections, including girder flange splices, either:

- The fillers shall be extended beyond the gusset or splice material, and the filler extension shall be secured by enough additional bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler or
- As an alternative, the fillers need not be extended and developed provided that the factored resistance of the bolts in shear at the strength limit state, specified in Article 6.13.2.2, is reduced by the following factor:

C6.13.6.1.4

Fillers are to be secured by means of additional fasteners so that the fillers are, in effect, an integral part of a shear-connected component at the strength limit state. The integral connection results in well-defined shear planes and no reduction in the factored shear resistance of the bolts.

In lieu of extending and developing the fillers, the reduction factor given by Eq. 6.13.6.1.4-1 may instead be applied to the factored resistance of the bolts in shear. This factor compensates for the reduction in the nominal shear resistance of a bolt caused by bending in the bolt and will typically result in the need to provide additional bolts in the connection. The reduction factor is only to be applied on the side of the connection with the fillers. The factor in Eq. 6.13.6.1.4-1 was developed mathematically (Sheikh-Ibrahim, 2002), and verified by comparison to the results from an experimental program on axially

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)} \right] \quad (6.13.6.1.4\text{-}1)$$

where:

γ = A_f/A_p

A_f = sum of the area of the fillers on both sides of the connected plate (in.^{2})

A_p = smaller of either the connected plate area or the sum of the splice plate areas on both sides of the connected plate (in.^{2}); for truss gusset plate chord splices, when considering the gusset plate(s), only the portion of the gusset plate(s) that overlaps the connected plate shall be considered in the calculation of the splice plate areas

For slip-critical connections, the nominal slip resistance of a bolt, specified in Article 6.13.2.8, shall not be adjusted for the effect of the fillers.

Fillers 0.25 in. or more in thickness shall consist of not more than two plates, unless approved by the Engineer.

The specified minimum yield strength of fillers 0.25 in. or greater in thickness should not be less than the larger of 70 percent of the specified minimum yield strength of the connected plate and 36.0 ksi.

loaded bolted splice connections with undeveloped fillers (Yura, et al., 1982). The factor is more general than a similar factor given in AISC (2016) in that it takes into account the areas of the main connected plate, splice plates and fillers and can be applied to fillers of any thickness. Unlike the empirical AISC factor, the factor given by Eq. 6.13.6.1.4-1 will typically be less than 1.0 for connections utilizing 0.25-in. thick fillers in order to ensure both adequate shear resistance and limited deformation of the connection.

For slip-critical connections, the nominal slip resistance of a bolt need not be adjusted for the effect of the fillers. The resistance to slip between filler and either connected part is comparable to that which would exist between the connected parts if fillers were not present.

For fillers 0.25 in. or greater in thickness in axially loaded bolted connections, the specified minimum yield strength of the fillers should theoretically be greater than or equal to the specified minimum yield strength of the connected plate times the factor $[1/(1+\gamma)]$ in order to provide fully developed fillers that act integrally with the connected plate. However, such a requirement may not be practical or convenient due to material availability issues. As a result, premature yielding of the fillers, bolt bending and increased deformation of the connection may occur in some cases at the strength limit state. To control excessive deformation of the connection, a lower limit on the specified minimum yield strength of the filler plate material is recommended for fillers 0.25 in. or greater in thickness. Connections where the fillers are appropriately extended and developed or where additional bolts are provided according to Eq. 6.13.6.1.4-1 in lieu of extending the fillers, but that do not satisfy the recommended yield strength limit, will still have adequate reserve shear resistance in the connection bolts. However, such connections will have an increased probability of larger deformations at the strength limit state. For fillers less than 0.25 in. in thickness, the effects of yielding of the fillers and deformation of the connection are considered inconsequential. For applications involving the use of weathering steels, a weathering grade product should be specified for the filler plate material.

6.13.6.2—Welded Splices

Welded splice design and details shall conform to the requirements of the latest edition of AASHTO/AWS D1.5M/D1.5 *Bridge Welding Code* and the following provisions specified herein.

Welded splices for tension and compression members shall be designed to resist the design axial force specified in Article 6.13.1. Tension and compression members may be spliced by means of full penetration butt welds. Flexural members shall be spliced by means of full penetration butt welds. The use of splice plates should be avoided.

Welded field splices should be arranged to minimize overhead welding.

C6.13.6.2

Flange width transition details typically show the transition starting at the butt splice. Figure 6.13.6.2-1 shows a preferred detail where the splice is located a minimum of 3.0 in. from the transition for ease in fitting runoff tabs. Where possible, constant width flanges are preferred in a shipping piece.

Material of different widths spliced by butt welds shall have symmetric transitions conforming to Figure 6.13.6.2-1. The type of transition selected shall be consistent with the detail categories of Table 6.6.1.2.3-1 for the groove-welded splice connection used in the design of the member. The contract documents shall specify that butt weld splices joining material of different thicknesses be ground to a uniform slope between the offset surfaces, including the weld, of not more than one in 2.5.

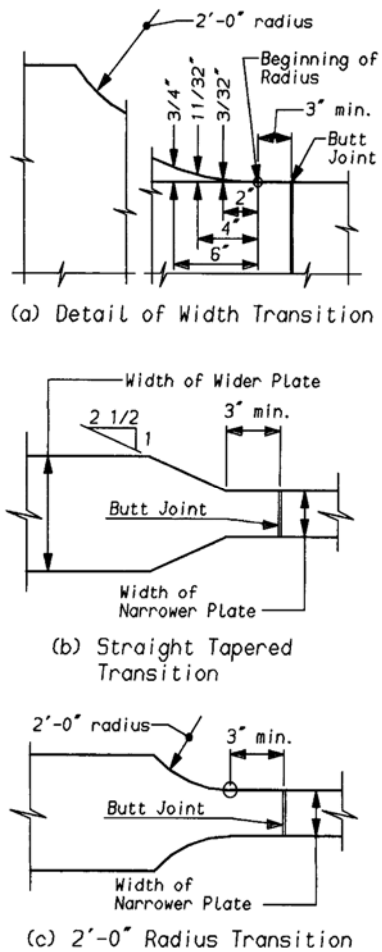


Figure 6.13.6.2-1—Splice Details

6.13.7—Rigid Frame Connections

6.13.7.1—General

All rigid frame connections shall be designed to resist the moments, shear, and axial forces due to the factored loading at the strength limit state.

C6.13.7.1

The provisions for rigid frame connections are well documented in Chapter 8 of ASCE (1971).

The rigidity is essential to the continuity assumed as the basis for design.

6.13.7.2—Webs

The thickness of an unstiffened beam web shall satisfy:

C6.13.7.2

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For machine evaluated lumber (MEL) commercial grades M-17, M-20 and M-27, F_{co}, requires qualification and quality control shall be required.

Reference design values specified in Table 8.4.1.1.4-2 shall be taken as applicable to lumber that will be used under dry conditions. For 2.0-in. to 4.0-in. thick lumber, the dry dressed sizes shall be used regardless of the moisture content at the time of manufacture or use.

than offsets the design effect of size reductions due to shrinkage.

For any given bending design value, F_{bo}, the modulus of elasticity, E_o, and tension parallel to grain, F_{to}, design value may vary depending upon species, timber source or other variables. The E_o and F_{to} values included in the F_{bo}-E_o grade designations in Table 8.4.1.1.4-2 are those usually associated with each F_{bo} level. Grade stamps may show higher or lower values if machine rating indicates the assignment is appropriate.

Higher G values may be claimed when (a) specifically assigned by the rules writing agency or (b) when qualified by test, quality controlled for G and provided for on the grade stamp. When a different G value is provided on the grade stamp, higher F_{vo} and F_{cpo} design values may be calculated in accordance with the grading rule requirements.

Table 8.4.1.1.4-1—Reference Design Values for Visually Graded Sawn Lumber

Species and Commercial Grade	Size Classification	Design Values (ksi) Bending F_{bo}	Tension parallel to grain F_{to}	Shear parallel to grain F_{vo}	Compression perpendicular to grain F_{cpo}	Compression parallel to grain F_{co}	Modulus of Elasticity E_o	Grading Rules Agency
Douglas Fir-Larch								
Select Structural	Dimension ≥2 in. Wide	1.500	1.000	0.180	0.625	1.700	1,900	WCLIB WWPA
No. 1 & Btr		1.200	0.800	0.180	0.625	1.550	1,800	
No. 1		1.000	0.675	0.180	0.625	1.500	1,700	
No. 2		0.900	0.575	0.180	0.625	1.350	1,600	
Dense Select Structural	Beams and Stringers	1.900	1.100	0.170	0.730	1.300	1,700	WCLIB
Select Structural		1.600	0.950	0.170	0.625	1.100	1,600	
Dense No. 1		1.550	0.775	0.170	0.730	1.100	1,700	
No. 1		1.350	0.675	0.170	0.625	0.925	1,600	
No. 2		0.875	0.425	0.170	0.625	0.600	1,300	
Dense Select Structural	Posts and Timbers	1.750	1.150	0.170	0.730	1.350	1,700	
Select Structural		1.500	1.000	0.170	0.625	1.150	1,600	
Dense No. 1		1.400	0.950	0.170	0.730	1.200	1,700	
No. 1		1.200	0.825	0.170	0.625	1.000	1,600	
No. 2		0.750	0.475	0.170	0.625	0.700	1,300	
Dense Select Structural	Beams and Stringers	1.900	1.100	0.170	0.730	1.300	1,700	WWPA
Select Structural		1.600	0.950	0.170	0.625	1.100	1,600	
Dense No. 1		1.550	0.775	0.170	0.730	1.100	1,700	
No. 1		1.350	0.675	0.170	0.625	0.925	1,600	
No. 2 Dense		1.000	0.500	0.170	0.730	0.700	1,400	
No. 2		0.875	0.425	0.170	0.625	0.600	1,300	
Dense Select Structural	Posts and Timbers	1.750	1.150	0.170	0.730	1.350	1,700	
Select Structural		1.500	1.000	0.170	0.625	1.150	1,600	
Dense No. 1		1.400	0.950	0.170	0.730	1.200	1,700	
No. 1		1.200	0.825	0.170	0.625	1.000	1,600	
No. 2 Dense		0.850	0.550	0.170	0.730	0.825	1,400	
No. 2		0.750	0.475	0.170	0.625	0.700	1,300	
Eastern Softwoods								
Select Structural	Dimension ≥2 in. Wide	1.250	0.575	0.140	0.335	1.200	1,200	NELMA NSLB
No. 1		0.775	0.350	0.140	0.335	1.000	1,100	
No. 2		0.575	0.275	0.140	0.335	0.825	1,100	
Hem-Fir								
Select Structural	Dimension ≥2 in. Wide	1.400	0.925	0.150	0.405	1.500	1,600	WCLIB WWPA
No. 1 & Btr		1.100	0.725	0.150	0.405	1.350	1,500	
No. 1		0.975	0.625	0.150	0.405	1.350	1,500	
No. 2		0.850	0.525	0.150	0.405	1.300	1,300	
Select Structural	Beams and Stringers	1.300	0.750	0.140	0.405	0.925	1,300	
No.1		1.050	0.525	0.140	0.405	0.750	1,300	
No.2		0.675	0.350	0.140	0.405	0.500	1,100	
Select Structural	Posts and Timbers	1.200	0.800	0.140	0.405	0.975	1,300	
No.1		0.975	0.650	0.140	0.405	0.850	1,300	
No.2		0.575	0.375	0.140	0.405	0.575	1,100	
Mixed Southern Pine								
Select Structural	Dimension 2 in.–4 in. Wide	2.050	1.200	0.175	0.565	1.800	1,600	SPIB
No.1		1.450	0.875	0.175	0.565	1.650	1,500	
No.2		1.100	0.675	0.175	0.565	1.450	1,400	
Select Structural	Dimension 5 in.–6 in. Wide	1.850	1.100	0.175	0.565	1.700	1,600	
No.1		1.300	0.750	0.175	0.565	1.550	1,500	
No.2		1.000	0.600	0.175	0.565	1.400	1,400	
Select Structural	Dimension 8 in. Wide	1.750	1.000	0.175	0.565	1.600	1,600	
No.1		1.200	0.700	0.175	0.565	1.450	1,500	
No.2		0.925	0.550	0.175	0.565	1.350	1,400	

Table 10.6.3.1.2c-2 (cont.)

			β=10°				β=20°				β=30°				β=40°			
			Ns				*Ns*				*Ns*				*Ns*			
ϕ (°)	***B/H***	***b/B***	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**	**0**	**2**	**4**	***c'*=0**
30	**0.2**	0	0.93	0.92	0.91	0.76	0.65	0.64	0.63	0.39	0.51	0.50	0.48	0.11	0.40	0.37	0.36	0.00
		0.5	0.74	0.81	0.80	0.75	0.70	0.66	0.65	0.50	0.57	0.52	0.49	0.21	0.47	0.42	0.39	0.00
		1.25	0.78	0.85	0.86	0.86	0.74	0.73	0.72	0.72	0.63	0.60	0.59	0.38	0.54	0.50	0.47	0.00
		2.5	0.84	0.92	0.93	0.99	0.81	0.82	0.83	0.94	0.72	0.73	0.74	0.74	0.64	0.62	0.61	0.00
		5	0.95	1.00	1.00	1.00	0.93	0.98	1.00	1.00	0.88	0.95	1.00	0.97	0.80	0.85	0.87	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	0.5	0	0.79	0.79	0.78	0.70	0.63	0.59	0.55	0.36	0.50	0.43	0.39	0.13	0.39	0.32	0.27	0.00
		0.5	0.76	0.87	0.87	0.74	0.72	0.71	0.70	0.51	0.58	0.56	0.54	0.24	0.49	0.46	0.43	0.00
		1.25	0.79	0.85	0.92	0.87	0.75	0.73	0.76	0.72	0.63	0.62	0.61	0.45	0.54	0.52	0.50	0.00
		2.5	0.87	0.91	1.00	0.99	0.84	0.85	0.90	0.98	0.74	0.78	0.80	0.80	0.67	0.70	0.71	0.00
		5	0.97	1.00	1.00	1.00	0.95	1.00	1.00	1.00	0.90	1.00	1.00	1.00	0.85	0.94	0.98	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	1	0	0.79	0.75	0.73	0.67	0.63	0.53	0.49	0.41	0.55	0.41	0.35	0.24	0.48	0.33	0.26	0.00
		0.5	0.78	0.87	0.89	0.74	0.75	0.74	0.74	0.51	0.64	0.62	0.60	0.35	0.59	0.56	0.54	0.00
		1.25	0.81	0.90	0.91	0.88	0.78	0.78	0.78	0.72	0.68	0.67	0.66	0.58	0.64	0.62	0.61	0.00
		2.5	0.88	0.99	1.00	0.96	0.85	0.90	0.92	0.95	0.78	0.81	0.84	0.88	0.75	0.78	0.80	0.00
		5	0.97	1.00	1.00	1.00	0.96	1.00	1.00	1.00	0.92	1.00	1.00	1.00	0.89	0.98	1.00	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
	2	0	0.88	0.88	0.87	0.65	0.87	0.85	0.83	0.48	0.85	0.82	0.80	0.38	0.83	0.80	0.76	0.00
		0.5	0.89	0.91	0.91	0.75	0.89	0.89	0.87	0.58	0.88	0.86	0.84	0.51	0.87	0.85	0.82	0.00
		1.25	0.90	0.92	0.93	0.88	0.90	0.90	0.90	0.75	0.89	0.87	0.87	0.70	0.89	0.87	0.86	0.00
		2.5	0.97	1.00	1.00	1.00	0.96	0.97	0.98	0.98	0.92	0.94	0.96	0.95	0.91	0.92	0.94	0.00
		5	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
40	**0.2**	0	0.69	0.69	0.69	0.78	0.51	0.48	0.47	0.37	0.37	0.33	0.30	0.16	0.27	0.23	0.20	0.05
		0.5	0.65	0.73	0.71	0.74	0.60	0.55	0.53	0.38	0.64	0.38	0.35	0.25	0.34	0.29	0.25	0.13
		1.25	0.68	0.77	0.75	0.86	0.63	0.60	0.58	0.55	0.74	0.44	0.42	0.39	0.39	0.34	0.31	0.25
		2.5	0.72	0.83	0.84	1.00	0.68	0.68	0.68	0.76	0.87	0.53	0.53	0.62	0.45	0.43	0.41	0.48
		5	0.80	0.93	0.95	1.00	0.76	0.82	0.85	1.00	1.00	0.72	0.76	1.00	0.57	0.61	0.63	0.94
		10	0.94	1.00	1.00	1.00	0.91	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.76	0.93	1.00	1.00
	0.5	0	0.67	0.69	0.67	0.69	0.50	0.45	0.43	0.35	0.36	0.30	0.26	0.17	0.27	0.20	0.17	0.07
		0.5	0.68	0.81	0.81	0.73	0.63	0.62	0.61	0.46	0.47	0.44	0.41	0.25	0.39	0.35	0.32	0.09
		1.25	0.70	0.82	0.84	0.85	0.65	0.65	0.66	0.60	0.51	0.49	0.47	0.40	0.43	0.41	0.39	0.18
		2.5	0.76	0.92	0.96	1.00	0.72	0.77	0.80	0.81	0.59	0.62	0.63	0.60	0.54	0.56	0.56	0.37
		5	0.84	1.00	1.00	1.00	0.81	0.91	0.94	1.00	0.71	0.82	0.88	1.00	0.67	0.77	0.83	0.84
		10	0.96	1.00	1.00	1.00	0.94	1.00	1.00	1.00	0.89	1.00	1.00	1.00	0.86	1.00	1.00	1.00
	1	0	0.69	0.64	0.62	0.70	0.63	0.48	0.43	0.45	0.58	0.39	0.33	0.32	0.54	0.33	0.27	0.24
		0.5	0.77	0.81	0.82	0.74	0.75	0.73	0.72	0.49	0.71	0.66	0.62	0.38	0.68	0.62	0.57	0.30
		1.25	0.78	0.84	0.85	0.84	0.77	0.76	0.75	0.64	0.73	0.69	0.66	0.55	0.71	0.66	0.63	0.48
		2.5	0.83	0.92	0.95	1.00	0.81	0.85	0.87	0.85	0.76	0.78	0.79	0.76	0.75	0.76	0.77	0.72
		5	0.89	1.00	1.00	1.00	0.87	0.95	0.98	1.00	0.80	0.90	0.95	1.00	0.80	0.89	0.94	1.00
		10	0.98	1.00	1.00	1.00	0.97	1.00	1.00	1.00	0.94	1.00	1.00	1.00	0.93	1.00	1.00	1.00
	2	0	0.93	0.92	0.89	0.45	0.92	0.90	0.87	0.60	0.91	0.88	0.84	0.53	0.89	0.85	0.81	0.47
		0.5	0.93	0.95	0.93	0.76	0.93	0.92	0.90	0.65	0.92	0.89	0.87	0.64	0.92	0.89	0.86	0.60
		1.25	0.93	0.95	0.94	0.86	0.93	0.93	0.92	0.78	0.93	0.91	0.89	0.74	0.93	0.90	0.88	0.74
		2.5	0.94	0.99	1.00	1.00	0.94	0.98	0.98	0.92	0.94	0.97	0.97	0.87	0.94	0.96	0.96	0.88
		5	0.95	1.00	1.00	1.00	0.96	1.00	1.00	1.00	0.98	1.00	1.00	1.00	0.96	1.00	1.00	1.00
		10	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.99	1.00	1.00	1.00

10.6.3.1.2d—Considerations for Two-Layer Soil Systems—Critical Depth

Where the soil profile contains a second layer of soil with different properties affecting shear strength within a distance below the footing less than H_{crit}, the bearing resistance of the layered soil profile shall be determined using the provisions for two-layered soil systems herein. The distance H_{crit}, in feet, may be taken as:

$$H_{crit} = \frac{(3B)\ \ln\left(\frac{q_1}{q_2}\right)}{2\left(1+\frac{B}{L}\right)} \qquad (10.6.3.1.2\text{d-}1)$$

where:

q_1 = nominal bearing resistance of footing supported in the upper layer of a two-layer system, assuming the upper layer is infinitely thick (ksf)
q_2 = nominal bearing resistance of a fictitious footing of the same size and shape as the actual footing but supported on surface of the second (lower) layer of a two-layer system (ksf)
B = footing width (ft)
L = footing length (ft)

10.6.3.1.2e—Two-Layered Soil System in Undrained Loading

Where a footing is supported on a two-layered soil system subjected to undrained loading, the nominal bearing resistance may be determined using Eq. 10.6.3.1.2a-1 with the following modifications:

c_1 = undrained shear strength of the top layer of soil as depicted in Figure 10.6.3.1.2e-1 (ksf)
N_{cm} = N_m, a bearing capacity factor as specified below (dim)
N_{qm} = 1.0 (dim)

Where the bearing stratum overlies a stiffer cohesive soil, N_m, may be taken as specified in Figure 10.6.3.1.2e-2.

Where the bearing stratum overlies a softer cohesive soil, N_m may be taken as:

$$N_m = \left(\frac{1}{\beta_m} + \kappa s_c N_c\right) \le s_c N_c \qquad (10.6.3.1.2\text{e-}1)$$

in which:

$$\beta_m = \frac{BL}{2(B+L)H_{s2}} \qquad (10.6.3.1.2\text{e-}2)$$

C10.6.3.1.2e

Vesic' (1970) developed a rigorous solution for the modified bearing capacity factor, N_m, for the weak undrained layer over strong undrained layer situation. This solution is given by the following equation:

$$N_m = \frac{\kappa N_c^*(N_c^* + \beta_m - 1)A}{B\ \ C - (\kappa N_c^* + \beta_m - 1))(N_c^* + 1)} \qquad (\text{C}10.6.3.1.2\text{e-}1)$$

in which:

$$A = \left[(\kappa+1)N_c^{*2} + (1+\kappa\beta_m)N_c^* + \beta_m - 1\right] \qquad (\text{C}10.6.3.1.2\text{e-}2)$$

$$B = \left[\kappa(\kappa+1)N_c^* + \kappa + \beta_m - 1\right] \qquad (\text{C}10.6.3.1.2\text{e-}3)$$

$$C = \left[(N_c^* + \beta_m)N_c^* + \beta_m - 1\right] \qquad (\text{C}10.6.3.1.2\text{e-}4)$$

- For circular or square footings:

$$\beta_m = \frac{B}{4H_{s2}} \qquad (\text{C}10.6.3.1.2\text{e-}5)$$

$$\kappa = \frac{c_2}{c_1} \quad (10.6.3.1.2e\text{-}3)$$

where:

β_m = the punching index (dim)
c_1 = undrained shear strength of upper soil layer (ksf)
c_2 = undrained shear strength of lower soil layer (ksf)
H_{s2} = distance from bottom of footing to top of the second soil layer (ft)
s_c = shape correction factor determined from Table 10.6.3.1.2a-3
N_c = bearing capacity factor determined herein (dim)
N_{qm} = bearing capacity factor determined herein (dim)

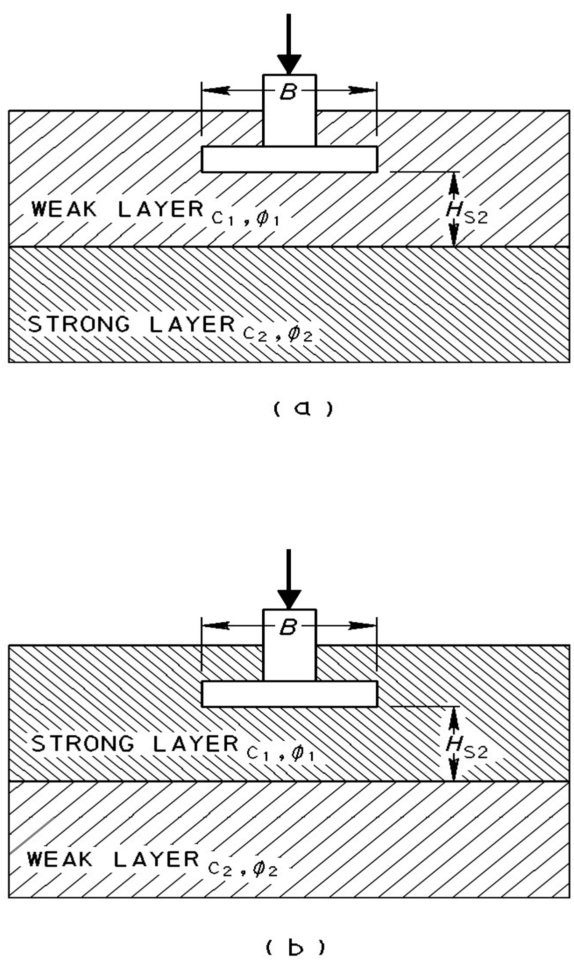


Figure 10.6.3.1.2e-1—Two-layer Soil Profiles

$$N_c^* = 6.17$$

- For strip footings:

$$\beta_m = \frac{B}{2H_{s2}} \quad (C10.6.3.1.2e\text{-}6)$$

$$N_c^* = 5.14$$

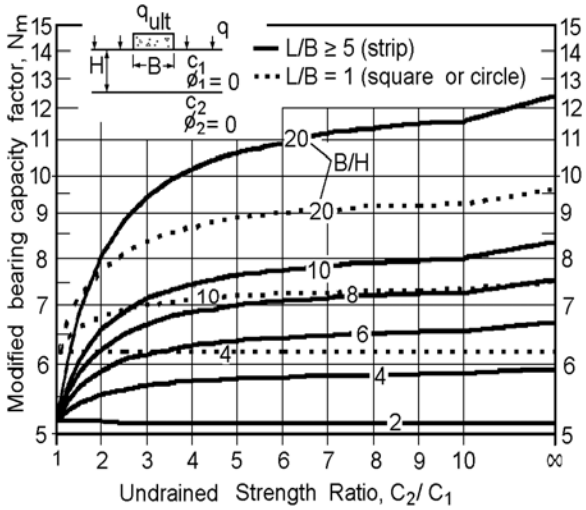


Figure 10.6.3.1.2e-2—Modified Bearing Factor for Two-Layer Cohesive Soil with Weaker Soil Overlying Stronger Soil (EPRI, 1983)

10.6.3.1.2f—Two-Layered Soil System in Drained Loading

Where a footing supported on a two-layered soil system is subjected to a drained loading, the nominal bearing resistance, in ksf, may be taken as:

$$q_n = \left[q_2 + \left(\frac{1}{K}\right)c_1' \cot\phi_1'\right] e^{2\left[1+\left(\frac{B}{L}\right)\right]K\tan\phi_1'\left(\frac{H_{s2}}{B}\right)} - \left(\frac{1}{K}\right)c_1' \cot\phi_1' \text{-} \qquad (10.6.3.1.2f\text{-}1)$$

in which:

$$K = \frac{1-\sin^2\phi_1'}{1+\sin^2\phi_1'} \qquad (10.6.3.1.2f\text{-}2)$$

where:

c'_1 = drained shear strength of the top layer of soil as depicted in Figure 10.6.3.1.2e-1 (ksf)

q_2 = nominal bearing resistance of a fictitious footing of the same size and shape as the actual footing but supported on surface of the second (lower) layer of a two-layer system (ksf)

ϕ'_1 = effective stress angle of internal friction of the top layer of soil (degrees)

C10.6.3.1.2f

If the upper layer is a cohesionless soil and ϕ' equals 25–50 degrees, Eq. 10.6.3.1.2f-1 reduces to:

$$q_n = q_2 e^{0.67\left[1+\left(\frac{B}{L}\right)\right]\frac{H}{B}} \qquad (C10.6.3.1.2f\text{-}1)$$

10.6.3.1.3—Semiempirical Procedures

The nominal bearing resistance of foundation soils may be estimated from the results of in-situ tests or by observed resistance of similar soils. The use of a particular in-situ test and the interpretation of test results should take local experience into consideration. The following in-situ tests may be used:

- Standard Penetration Test
- Cone Penetration Test

C10.6.3.1.3

In application of these empirical methods, the use of average *SPT* blow counts and *CPT* tip resistances is specified. The resistance factors recommended for bearing resistance included in Table 10.5.5.2.2-1 assume the use of average values for these parameters. The use of lower bound values may result in an overly conservative design. However, depending on the availability of soil property data and the variability of

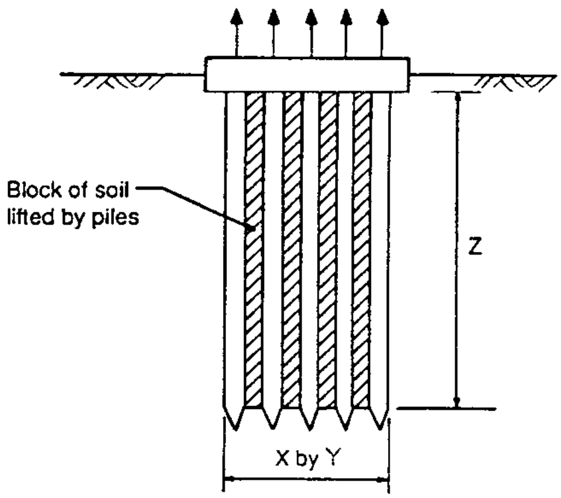


Figure 10.7.3.11-2—Uplift of Group of Piles in Cohesive Soils after Tomlinson (1987)

10.7.3.12—Nominal Lateral Resistance of Pile Foundations

The nominal resistance of pile foundations to lateral loads shall be evaluated based on both geomaterial and structural properties. The lateral soil resistance along the piles should be modeled using *P-y* curves developed for the soils at the site.

The applied loads shall be factored loads and they must include both lateral and axial loads. The analysis may be performed on a representative single pile with the appropriate pile top boundary condition or on the entire pile group. The *P-y* curves shall be modified for group effects. The *P*-multipliers in Table 10.7.2.4-1 should be used to modify the curves. If the pile cap will always be embedded, the *P-y* lateral resistance of the soil on the cap face may be included in the nominal lateral resistance.

The minimum penetration of the piles below ground (see Article 10.7.6) required in the contract should be established such that fixity is obtained. For this determination, the loads applied to the pile are factored as specified in Section 3, and a soil resistance factor of 1.0 shall be used as specified in Table 10.5.5.2.3-1.

If fixity cannot be obtained, additional piles should be added, larger diameter piles used if feasible to drive them to the required depth, or a wider spacing of piles in the group should be considered to provide the necessary lateral resistance. Batter piles may be added to provide the lateral resistance needed, unless downdrag is anticipated. If downdrag is anticipated, batter piles should not be used. The design procedure, if fixity cannot be obtained, should take into consideration the lack of fixity of the pile.

Lateral resistance of single piles may be determined by static load test. If a static lateral load test is to be performed, it shall follow the procedures specified in ASTM D3966.

C10.7.3.12

Pile foundations are subjected to lateral loads due to wind, traffic loads, bridge curvature, stream flow, vessel or traffic impact and earthquake. Batter piles are sometimes used but they are somewhat more expensive than vertical piles and vertical piles are more effective against dynamic loads.

Additional details regarding methods of analysis using *P-y* curves, both for single piles and pile groups, are provided in Article 10.7.2.4. As an alternative to *P-y* analysis, strain wedge theory may be used (see Article 10.7.2.4).

When this analysis is performed, the loads are factored since the strength limit state is under consideration, but the resistances as represented by the *P-y* curves are not factored since they already represent the ultimate condition.

The strength limit state for lateral resistance is only structural (see Sections 5 and 6 for structural limit state design requirements), though the determination of pile fixity is the result of soil-structure interaction. A failure of the soil does not occur; the soil will continue to displace at constant or slightly increasing resistance. Failure occurs when the pile reaches the structural limit state, and this limit state is reached, in the general case, when the nominal combined bending and axial resistance is reached.

If the lateral resistance of the soil in front of the pile cap is included in the lateral resistance of the foundation, the effect of soil disturbance resulting from construction of the pile cap should be considered. In such cases, the passive resistance may need to be reduced to account for the effects of disturbance.

For information on analysis and interpretation of load tests, see Article 10.7.2.4.

10.7.3.13—Pile Structural Resistance

10.7.3.13.1—Steel Piles

The nominal axial compression resistance in the structural limit state for piles loaded in compression shall be as specified in Article 6.9.4.1 for noncomposite piles and Article 6.9.5.1 for composite piles. If the pile is fully embedded, λ in Eq. 6.9.5.1-1 shall be taken as zero.

The nominal axial resistance of horizontally unsupported noncomposite piles that extend above the ground surface in air or water shall be determined from Eqs. 6.9.4.1.1-1 or 6.9.4.1.1-2. The nominal axial resistance of horizontally unsupported composite piles that extend above the ground surface in air or water shall be determined from Eqs. 6.9.5.1-1 or 6.9.5.1-2.

The effective length of laterally unsupported piles should be determined based on the provisions in Article 10.7.3.13.4.

The resistance factors for the compression limit state are specified in Article 6.5.4.2.

C10.7.3.13.1

Composite members refer to steel pipe piles that are filled with concrete.

The effective length given in Article C10.7.3.13.4 is an empirical approach to determining effective length. Computer methods are now available that can determine the axial resistance of a laterally unsupported compression member using a *P*-Δ analysis that includes a numerical representation of the lateral soil resistance (Williams et al., 2003). These methods are preferred over the empirical approach in Article C10.7.3.13.4.

10.7.3.13.2—Concrete Piles

The nominal axial compression resistance for concrete piles and prestressed concrete piles shall be as specified in Article 5.6.4.4.

The nominal axial compression resistance for concrete piles that are laterally unsupported in air or water shall be determined using the procedures given in Articles 5.6.4.3 and 4.5.3.2. The effective length of laterally unsupported piles should be determined based on the provisions in Article 10.7.3.13.4.

The resistance factor for the compression limit state for concrete piles shall be that given in Article 5.5.4.2 for concrete loaded in axial compression.

C10.7.3.13.2

Article 5.6.4 includes specified limits on longitudinal reinforcement, spirals and ties. Methods are given for determining nominal axial compression resistance but they do not include the nominal axial compression resistance of prestressed members. ~~Article C5.6.4.1 notes that~~ Compression members are usually prestressed only where they are subjected to high levels of flexure. Therefore, a method of determining nominal axial compression resistance is not given.

Article 5.6.4.5 specifically permits an analysis based on equilibrium and strain compatibility. Methods are also available for performing a stability analysis (Williams et al., 2003).

10.7.3.13.3—Timber Piles

The nominal axial compression resistance for timber piles shall be as specified in Article 8.8.2. The methods presented there include both laterally supported and laterally unsupported members.

The effective length of laterally unsupported piles should be determined based on the provisions in Article 10.7.3.13.4.

C10.7.3.13.3

Article 8.5.2.3 requires that a reduction factor for long term loads of 0.75 be multiplied times the resistance factor for Strength Load Combination IV.

10.7.3.13.4—Buckling and Lateral Stability

In evaluating stability, the effective length of the pile shall be equal to the laterally unsupported length, plus an embedded depth to fixity.

The potential for buckling of unsupported pile lengths and the determination of stability under lateral loading should be evaluated by methods that consider soil-structure interaction as specified in Article 10.7.3.12.

C10.7.3.13.4

For preliminary design, the depth to fixity below the ground, in ft, may be taken as:

- For clays:

$$1.4\,[E_p\, I_w / E_s]^{0.25} \qquad \text{(C10.7.3.13.4-1)}$$

- For sands:

E_n = nominal thickness of steel reinforcement at construction (mil.) (11.10.6.4.2a)
E_s = sacrificial thickness of metal expected to be lost by uniform corrosion during service life (mil.) (11.10.6.4.2a)
e = eccentricity of load from centerline of foundation (ft) (11.10.8)
F_p = static lateral force due to a concentrated surcharge load (kips/ft) (11.6.5.1)
F_T = resultant force of active lateral earth pressure (kips/ft) (11.6.3.2)
F_v = site class adjustment factor for the 1-sec. spectral acceleration (dim.) (A11.5)
F_y = minimum yield strength of steel (ksi) (11.10.6.4.3a)
F^* = reinforcement pullout friction factor (dim.) (11.10.6.3.2)
G_u = distance from center of gravity of a horizontal segmental facing block unit, including aggregate fill, measured from the front of the unit (ft) (11.10.6.4.4b)
H = height of wall (ft) (11.6.5.1)
H_h = hinge height for segmental facing (ft) (11.10.6.4.4b)
H_u = segmental facing block unit height (ft) (11.10.6.4.4b)
H_1 = equivalent wall height (ft) (11.10.6.3.1)
h = vertical distance between ground surface and wall base at the back of wall heel (ft) (11.6.3.2) (11.10.7.1)
h_a = distance between the base of the wall, or the mudline in front of the wall, and the resultant active seismic earth pressure force (ft) (A11.3.1)
h_i = height of reinforced soil zone contributing horizontal load to reinforcement at level i (ft) (11.10.6.2.1)
h_p = vertical distance between the wall base and the static surcharge lateral force F_p (ft) (11.6.5.1)
i = backfill slope angle (degrees) (A11.3.1)
i_b = slope of facing base downward into backfill (degrees) (11.10.6.4.4b)
K = seismic passive pressure coefficient (dim.) (A11.3.1)
K_{AE} = seismic active pressure coefficient (dim.) (A11.3.1)
k_a = active earth pressure coefficient (dim.) (11.8.4.1)
k_{af} = active earth pressure coefficient of backfill (dim.) (11.10.5.2)
k_h = horizontal seismic acceleration coefficient (dim.) (11.8.6)
k_{h0} = horizontal seismic acceleration coefficient at zero displacement (dim.) (11.6.5.2)
k_v = vertical seismic acceleration coefficient (dim.) (11.6.5.3)
k_r = horizontal earth pressure coefficient of reinforced fill (dim.) (11.10.5.2)
k_y = yield acceleration in sliding block analysis that results in sliding of the wall (dim) (A11.5)
L = spacing between vertical elements or facing supports (ft); length of reinforcing elements in an MSE wall and correspondingly its foundation (ft) (11.8.5.2) (11.10.2)
L_a = length of reinforcement in active zone (ft) (11.10.2)
L_b = anchor bond length (ft) (11.9.4.2)
L_e = length of reinforcement in resistance zone (ft) (11.10.2)
L_{ei} = effective reinforcement length for layer i (ft) (11.10.7.2)
M = moment magnitude of design earthquake (dim.) (A11.5)
$MARV$ = minimum average roll value (11.10.6.4.3b)
M_{max} = maximum bending moment in vertical wall element or facing (kip-ft or kip-ft/ft) (11.8.5.2)
N = normal component of resultant on base of foundation or standard penetration resistance from SPT (kips/ft or blows/ft, respectively) (11.6.3.2) (A11.5)
n = total number of reinforcement layers in the wall (dim) (11.10.7.2)
P_{AE} = dynamic active horizontal thrust, including static earth pressure (kips/ft) (11.10.7.1)
P_a = resultant active earth pressure force per unit width of wall (kips/ft) (11.8.6.2)
P_b = pressure inside bin module (ksf) (11.10.5.1)
PGA = peak ground acceleration (dim.) (11.6.5.1)
P_H = lateral force due to superstructure or other concentrated loads (kips/ft) (11.10.10.1)
P_i = factored horizontal force per mm of wall transferred to soil reinforcement at level i; internal inertial force, due to the weight of the backfill within the active zone (kips/ft) (11.10.6.2.1) (11.10.7.2)
P_{IR} = horizontal inertial force (kips/ft) (11.10.7.1)
P_{ir} = horizontal inertial force caused by acceleration of reinforced backfill (kips/ft) (11.10.7.1)
P_{is} = internal inertial force caused by acceleration of sloping surcharge (kips/ft) (11.10.7.1)
P_{PE} = dynamic passive horizontal thrust, including static earth pressure (kips/ft) (11.8.6.2)
P_r = ultimate soil reinforcement pullout resistance per unit of reinforcement width (kips/ft) (11.10.6.3.2)
P_{seis} = total lateral force applied to a wall during seismic loading (kips/ft) (11.6.5.1)
P_v = load on strip footing (kips/ft) (11.10.10.1)
P'_v = load on isolated rectangular footing or point load (kips) (11.10.10.1)
PVG = peak ground velocity (in./sec.) (A11.5)

p = average lateral pressure, including earth, surcharge and water pressure, acting on the section of wall element being considered (ksf) (11.9.5.2)
Q_n = nominal (ultimate) anchor resistance (kips) (11.9.4.2)
Q_R = factored anchor resistance (kips) (11.9.4.2)
q_s = surcharge pressure (ksf) (11.10.5.2)
q_{max} = maximum unit soil pressure on base of foundation (ksf) (11.6.3.2)
R = resultant force at base of wall (kips/ft) (11.6.3.2)
R_{BH} = basal heave ratio (C11.9.3.1)
R_c = reinforcement coverage ratio (dim.) (11.10.6.3.2)
R_n = nominal resistance (kips or kips/ft) (11.5.4)
R_R = factored resistance (kips or kips/ft) (11.5.4)
RF = combined strength reduction factor to account for potential long-term degradation due to installation damage, creep and chemical/biological aging of geosynthetic reinforcements (dim.) (11.10.6.4.2b)
RF_c = combined strength reduction factor for long-term degradation of geosynthetic reinforcement facing connection (dim.) (11.10.6.4.4b)
RF_{CR} = strength reduction factor to prevent long-term creep rupture of reinforcement (dim.) (11.10.6.4.3b)
RF_D = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation (dim.) (11.10.6.4.3b)
RF_{ID} = strength reduction factor to account for installation damage to reinforcement (dim.) (11.10.6.4.3b)
S_h = horizontal reinforcement spacing (ft) (11.10.6.4.1)
S_t = spacing between transverse grid elements (in.) (11.10.6.3.2)
S_u = undrained shear strength (ksf) (11.9.5.2)
S_v = vertical spacing of reinforcements (ft) (11.10.6.2.1)
S_{rs} = ultimate reinforcement tensile resistance required to resist static load component (kips/ft) (11.10.7.2)
S_{rt} = ultimate reinforcement tensile resistance required to resist transient load component (kips/ft) (11.10.7.2)
S_1 = 1-sec. spectral acceleration coefficient (dim.) (A11.5)
T_{ac} = nominal long-term reinforcement/facing connection design strength (kips/ft) (11.10.6.4.1)
$T_{a\ell}$ = nominal long-term reinforcement design strength (kips/ft) (11.10.6.4.1)
T_{crc} = creep reduced connection strength per unit of reinforcement width determined from the stress rupture envelope at the specified design life as produced from a series of long-term connection creep tests (kips/ft) (11.10.6.4.4b)
T_{lot} = ultimate wide width tensile strength per unit of reinforcement width (ASTM D4595 or D6637) for the reinforcement material lot used for the connection strength testing (kips/ft) (11.10.6.4.4b)
T_{md} = factored incremental dynamic inertia force (kips/ft) (11.10.7.2)
$T_{ultconn}$ = ultimate connection strength per unit of reinforcement width (kips/ft) (11.10.6.4.4b)
T_{ult} = ultimate tensile strength of reinforcement (kips/ft) (11.10.6.4.3b)
T_{max} = applied load to reinforcement (kips/ft) (11.10.6.2.1)
T_o = factored tensile load at reinforcement/facing connection (kips/ft) (11.10.6.2.2)
t = thickness of transverse elements (in.) (11.10.6.3.2)
T_s = fundamental period of wall (sec.) (A11.5)
T_{total} = total load on reinforcement layer (static & dynamic) per unit width of wall (kips/ft) (11.10.7.2)
V_s = shear wave velocity of soil behind wall (ft/sec.) (A11.5)
V_1 = weight of soil carried by wall heel, not including weight of soil surcharge (kips/ft) (11.6.3.2)
V_2 = weight of soil surcharge directly above wall heel (kips/ft) (11.6.3.2)
W_s = weight of the soil that is immediately above the wall, including the wall heel (kips/ft) (11.6.5.1)
W_u = unit width of segmental facing (ft) (11.10.2.3.2)
W_w = weight of the wall (kips/ft) (11.6.5.1)
W_1 = weight of wall stem (kips/ft) (11.6.3.2)
W_2 = weight of wall footing or base (kips/ft) (11.6.3.2)
x = spacing between vertical element supports (ft) (11.9.5.2)
Z = depth below effective top of wall or to reinforcement (ft) (11.10.6.2.1)
Z_p = depth of soil at reinforcement layer at beginning of resistance zone for pullout calculation (ft) (11.10.6.2.1)
α = scale effect correction factor, or wall height acceleration reduction factor for wave scattering (dim.) (11.10.6.3.2) (A11.5)
β = inclination of ground slope behind face of wall (degrees) (11.5.5)
γ_{EQ} = load factor for live load applied simultaneously with seismic loads in Article 3.4.1 (dim.) (11.6.5)
γ_P = load factor for vertical earth pressure in Article 3.4.1 (dim.) (11.10.6.2.1)
γ_s = soil unit weight (kcf)

The resistance factors in Table 11.5.7-1, in combination with the load factor for horizontal active earth pressure (Table 3.4.1-2), are consistent with what would be required based on allowable stress design, for preliminary design of anchors for pullout (Sabatini et al., 1999). These resistance factors are also consistent with the results of statistical calibration of full scale anchor pullout tests relative to the minimum values of presumptive ultimate unit bond stresses shown in Tables C11.9.4.2-1 through C11.9.4.2-3. Use of the resistance factors in Table 11.5.7-1 and the load factor for apparent earth pressure for anchor walls in Table 3.4.1-2, with values of presumptive ultimate unit bond stresses other than the minimum values in Tables C11.9.4.2-1 through C11.9.4.2-3 could result in unconservative designs unless the Engineer has previous experience with the particular soil or rock unit in which the bond zone will be established.

Presumptive bond stresses greater than the minimum values shown in Tables C11.9.4.2-1 through C11.9.4.2-3 should be used with caution, and be based on past successful local experience, such as a high percentage of passing proof tests in the specified or similar soil or rock unit at the design bond stress chosen, or anchor pullout test results in the specified or similar soil or rock unit. Furthermore, in some cases the specified range of presumptive bond stresses is representative of a range of soil conditions. Soil conditions at the upper end of the specified range, especially if coupled with previous experience with the particular soil unit, may be considered in the selection of anchor bond stresses above the minimum values shown. Selection of a presumptive bond stress for preliminary anchor sizing should consider the risk of failing proof tests if the selected bond stress was to be used for final design. The goal of preliminary anchor design is to reduce the risk of having a significant number of production anchors fail proof or performance tests as well as the risk of having to redesign the anchored wall to accommodate more anchors due to an inadequate easement behind the wall, should the anchor capacities predicted during preliminary design not be achievable. See Article 11.9.8.1 for guidance on anchor testing.

Significant increases in anchor capacity for anchor bond lengths greater than approximately 40.0 ft cannot be achieved unless specialized methods are used to transfer load from the top of the anchor bond zone towards the end of the anchor. This is especially critical for strain sensitive soils, in which residual soil strength is significantly lower than the peak soil strength.

The anchor load shall be developed by suitable embedment outside of the critical failure surface in the retained soil mass.

Determination of the unbonded anchor length, inclination, and overburden cover shall consider:

- The location of the critical failure surface furthest from the wall,

Anchor inclination and spacing will be controlled by soil and rock conditions, the presence of geometric constraints and the required anchor capacity. For tremie-grouted anchors, a minimum angle of inclination of about 10 degrees and a minimum overburden cover of about 15.0 ft are typically required to assure grouting of the entire bonded length and to provide sufficient ground cover above the anchorage zone. For pressure-grouted anchors, the angle of inclination is generally not

- The minimum length required to ensure minimal loss of anchor prestress due to long-term ground movements,
- The depth to adequate anchoring strata, as indicated in Figure 11.9.1-1, and
- The method of anchor installation and grouting.

The minimum horizontal spacing of anchors should be the larger of three times the diameter of the bonded zone, or 5.0 ft. If smaller spacings are required to develop the required load, consideration may be given to differing anchor inclinations between alternating anchors.

critical and is governed primarily by geometric constraints, and the minimum overburden cover is typically 6.0–15.0 ft. Steep inclinations may be required to avoid anchorage in unsuitable soil or rock. Special situations may require horizontal or near horizontal anchors, in which case proof of sufficient overburden and full grouting should be required.

The minimum horizontal spacing specified for anchors is intended to reduce stress overlap between adjacent anchors.

Anchors used for walls constructed in fill situations, i.e., bottom-up construction, should be enclosed in protective casing to prevent damage during backfill placement, compaction and settlement.

Selection of anchor type depends on anticipated service life, soil and rock conditions, ground water level, subsurface environmental conditions, and method of construction.

11.9.4.3—Passive Resistance

The provisions of Articles 11.6.3.5, 11.6.3.6, and 11.8.4.1 shall apply.

C11.9.4.3

It is recommended in Sabatini et al. (1999) that methods such as the Broms Method or the Wang and Reese method be used to evaluate passive resistance and the wall vertical element embedment depth needed. However, these methods have not been calibrated for this application for LRFD as yet.

11.9.5—Safety Against Structural Failure

11.9.5.1—Anchors

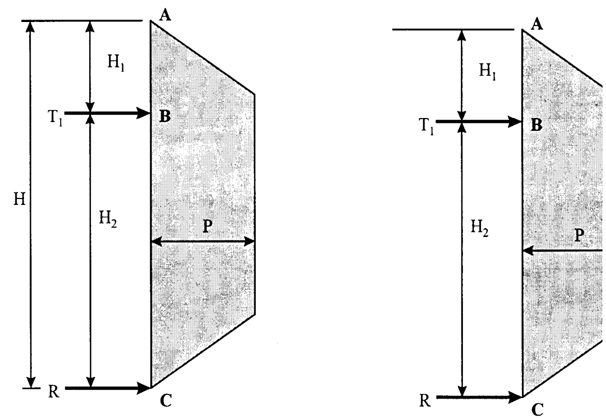
The horizontal component of anchor design force shall be computed using the provisions of Article 11.9.2 and any other horizontal pressure components acting on the wall in Article 3.11. The total anchor design force shall be determined based on the anchor inclination. The horizontal anchor spacing and anchor capacity shall be selected to provide the required total anchor design force.

C11.9.5.1

Anchor tendons typically consist of steel bars, wires or strands. The selection of anchor type is generally the responsibility of the contractor.

A number of suitable methods for the determination of anchor loads are in common use. Sabatini et al. (1999) provides two methods which can be used: the Tributary Area Method, and the Hinge Method. These methods are illustrated in Figures ~~C11.5.9.1-1~~ C11.9.5.1-1 and ~~C11.5.9.1-2~~ C11.9.5.1-2. These figures assume that the soil below the base of the excavation has sufficient strength to resist the reaction force R. If the soil providing passive resistance below the base of the excavation is weak and is inadequate to carry the reaction force R, the lowest anchor should be designed to carry both the anchor load as shown in the figures as well as the reaction force. See Article 11.8.4.1 for evaluation of passive resistance. Alternatively, soil-structure interaction analyses, e.g., beam on elastic foundation, can be used to design continuous beams with small toe reactions, as it may be overly conservative to assume that all of the load is carried by the lowest anchor.

In no case should the maximum test load be less than the factored load for the anchor.



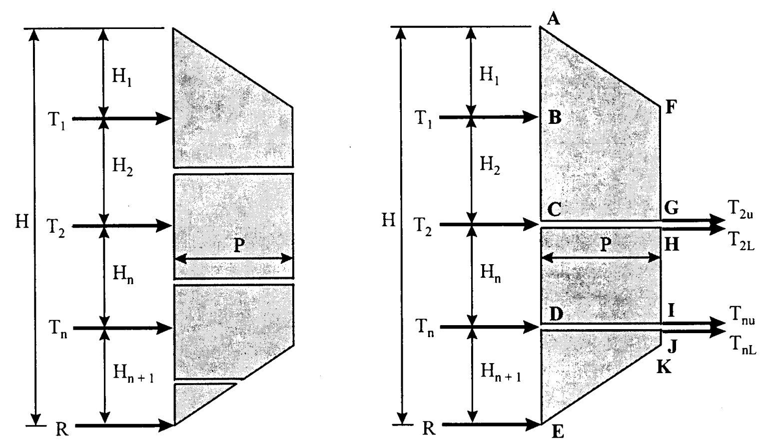
Tributary area method

T_1 = Load over length $H_1 + H_2/2$
R = Load over length $H_2/2$

Hinge method

T_1 Calculated from $\Sigma M_C = 0$
R = Total earth pressure – T_1

Figure C11.9.5.1-1—Calculation of Anchor Loads for One-Level Wall after Sabatini et al. (1999)



Tributary Area Method

T_1 = Load over length $H_1 + H_2/2$
T_2 = Load over length $H_2/2 + H_n/2$
T_n = Load over length $H_n/2 + H_{n+1}/2$
R = Load over length $H_{n+1}/2$

Hinge Method

T_1 Calculated from $\Sigma M_C = 0$
T_{2u} = Total earth pressure ($ABCGF$) – T_1
T_{2L} = Calculated from $\Sigma M_D = 0$
T_{nu} = Total earth pressure ($CDIH$) – T_{2L}
T_{nL} = Calculated from $\Sigma M_E = 0$
R = Total earth pressure – $T_1 - T_2 - T_n$
$\cancel{T_2 = T_{2u} = T_{2L}}$ $\underline{T_2 = T_{2u} + T_{2L}}$
$T_n = T_{nu} + T_{nL}$

Figure C11.9.5.1-2—Calculation of Anchor Loads for Multilevel Wall after Sabatini et al. (1999)

11.9.5.2—Vertical Wall Elements

Vertical wall elements shall be designed to resist all horizontal earth pressure, surcharge, water pressure, anchor, and seismic loadings, as well as the vertical component of the anchor loads and any other vertical loads. Horizontal supports may be assumed at each anchor location and at the bottom of the excavation if the vertical element is sufficiently embedded below the bottom of the excavation.

C11.9.5.2

Discrete vertical wall elements are continuous throughout their length and include driven piles, caissons, drilled shafts, and auger-cast piles, i.e., piles and built-up sections installed in preaugured holes and backfilled with structural concrete in the passive zone and lean concrete in the exposed section of the wall.

Continuous vertical wall elements are continuous throughout both their length and width, although vertical joints may prevent shear and/or moment transfer between adjacent sections. Continuous vertical wall elements include sheet piles, precast or cast-in-place concrete diaphragm wall panels, tangent-piles, and tangent caissons.

For structural analysis methods, see Section 4.

For walls supported in or through soft clays with $S_u < 0.15\gamma_s'H$, continuous vertical elements extending well below the exposed base of the wall may be required to prevent heave in front of the wall. Otherwise, the vertical elements are embedded approximately 3.0 ft or as required for stability or end bearing.

11.9.5.3—Facing

The provisions of Article 11.8.5.2 shall apply.

11.9.6—Seismic Design

The provisions of Article 11.8.6 shall apply except as modified in this Article.

The seismic analysis of the anchored retaining wall shall demonstrate that the anchored wall can maintain overall stability and withstand the seismic earth pressures induced by the design earthquake without exceeding the capacity of the anchors or the structural wall section supporting the soil. Limit equilibrium methods or numerical displacement analyses shall be used to confirm acceptable wall performance.

Anchors shall be located behind the limit equilibrium failure surface for seismic loading. The location of the failure surface for seismic loading shall be established using methods that account for the seismic coefficient and the soil properties (i.e., c and ϕ) within the anchored zone.

C11.9.6

See Article C11.8.6.

The seismic design of an anchored wall involves many of the same considerations as the nongravity cantilever wall. However, the addition of one or more anchors to the wall introduces some important differences in the seismic design check as identified in this Article.

The earth pressures above the excavation level result from the inertial response of the soil mass behind the wall. In contrast to a nongravity cantilever wall, the soil mass includes anchors that have been tensioned to minimize wall deflections under static earth pressures. During seismic loading, the bars or strands making up the unbonded length of the anchor are able to stretch under the imposed incremental seismic loads. In most cases, the amount of elastic elongation in the strand or bar under the incremental seismic load is sufficient to develop seismic active earth pressures but may not be sufficient to allow the horizontal seismic acceleration coefficient, k_{h0}, and associated earth pressure to be reduced to account for permanent horizontal wall displacement. The ability of the wall to deform laterally should be specifically investigated before reducing k_{h0} to account for horizontal wall displacement.

The passive pressure for the embedded portion of the soldier pile or sheet pile wall also plays a part in the stability assessment, as it helps provide stability for the portion of the wall below the lowest anchor. This passive pressure is subject to seismically induced inertial forces that will reduce the passive resistance relative to

The chemical properties of the native soil surrounding the mechanically stabilized soil backfill shall also be considered if there is potential for seepage of groundwater from the native surrounding soils to the mechanically stabilized backfill. If this is the case, the surrounding soils shall also meet the chemical criteria required for the backfill material if the environment is to be considered nonaggressive, or adequate long-term drainage around the geosynthetic reinforced mass shall be provided to ensure that chemically aggressive liquid does not enter into the reinforced backfill.

~~3)~~ *Polymer Requirements*: Polymers which are likely to have good resistance to long-term chemical degradation shall be used if a single default reduction factor is to be used, to minimize the risk of the occurrence of significant long-term degradation. The polymer material requirements provided in Table 11.10.6.4.2b-1 shall, therefore, be met if detailed product specific data as described in AASHTO R 69 and Elias, et al. (2009) is not obtained. Polymer materials not meeting the requirements in Table 11.10.6.4.2b-1 may be used if this detailed product specific data extrapolated to the design life intended for the structure are obtained.

For applications involving:

- Severe consequences of poor performance or failure,
- Aggressive soil conditions,
- Polymers not meeting the specific requirements set in Table 11.10.6.4.2b-1, or
- A desire to use an overall reduction factor less than the default reduction factor recommended herein,

then product-specific durability studies shall be carried out prior to product use to determine the product-specific long-term strength reduction factor, *RF*. These product-specific studies shall be used to estimate the short-term and long-term effects of these environmental factors on the strength and deformational characteristics of the geosynthetic reinforcement throughout the reinforcement design life.

Guidelines for product-specific studies to determine *RF* are provided in Elias et al. (2009) and AASHTO R 69, which is based on WSDOT Standard Practice T925 (WSDOT, 2009). Independent product-specific data from which *RF* may be determined can be obtained from the AASHTO National Transportation Product Evaluation Program (NTPEP) website at http://www.ntpep.org.

Table 11.10.6.4.2b-1—Minimum Requirements for Geosynthetic Products to Allow Use of Default Reduction Factor for Long-Term Degradation

Polymer Type	Property	Test Method	Criteria to Allow Use of Default *RF*
Polypropylene	UV Oxidation Resistance	ASTM D4355	Minimum 70% strength retained after 500 hrs. in weatherometer
Polyethylene	UV Oxidation Resistance	ASTM D4355	Minimum 70% strength retained after 500 hrs. in weatherometer
Polypropylene	Thermo-Oxidation Resistance	ENV ISO 13438:1999, Method A	Minimum 50% strength retained after 28 days
Polyethylene	Thermo-Oxidation Resistance	ENV ISO 13438:1999, Method B	Minimum 50% strength retained after 56 days
Polyester	Hydrolysis Resistance	Intrinsic Viscosity Method (ASTM D4603) and GRI Test Method GG8, or Determine Directly Using Gel Permeation Chromatography	Minimum Number Average Molecular Weight of 25000
Polyester	Hydrolysis Resistance	ASTM D7409	Maximum of Carboxyl End Group Content of 30
All Polymers	Survivability	Weight per Unit Area (ASTM D5261)	Minimum 270 g/m^2
All Polymers	% Post-Consumer Recycled Material by Weight	Certification of Materials Used	Maximum of 0%

11.10.6.4.3—Design Tensile Resistance

11.10.6.4.3a—Steel Reinforcements

The nominal reinforcement tensile resistance is determined by multiplying the yield stress by the cross-sectional area of the steel reinforcement after corrosion losses (see Figure 11.10.6.4.1-1). The loss in steel cross-sectional area due to corrosion shall be determined in accordance with Article 11.10.6.4.2a. The reinforcement tensile resistance shall be determined as:

$$T_{al} = \frac{A_c F_y}{b} \qquad (11.10.6.4.3a\text{-}1)$$

where:

$T_{a\ell}$ = nominal long-term reinforcement design strength (kips/ft)
F_y = minimum yield strength of steel (ksi)
A_c = area of reinforcement corrected for corrosion loss (Figure 11.10.6.4.1-1) ($in.^2$)
b = unit width of reinforcement (Figure 11.10.6.4.1-1) (ft)

11.10.6.4.3b—Geosynthetic Reinforcements

The nominal long-term reinforcement tensile strength shall be determined as:

C11.10.6.4.3b

$T_{a\ell}$ is the long-term tensile strength required to prevent rupture calculated on a load per unit of

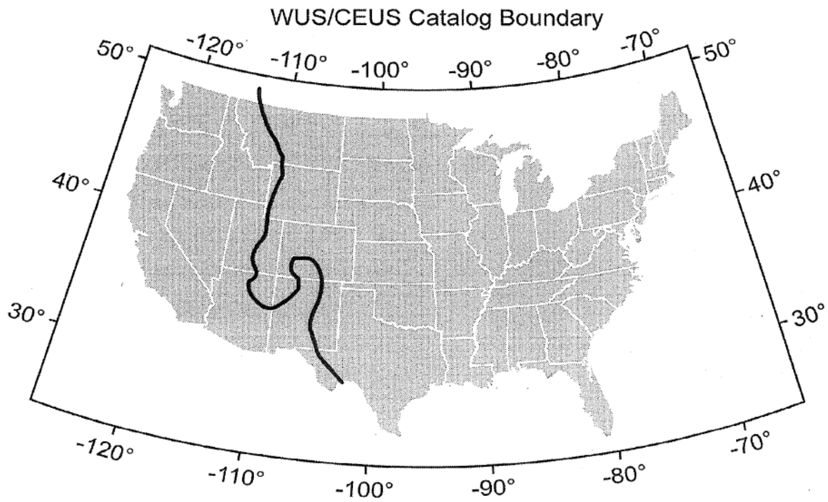


Figure A11.5.2-2—Boundary between WUS and CEUS Ground Motions

For all sites except CEUS rock sites (Categories A and B), the mean displacement (in.) for a given yield acceleration may be estimated as:

$$\log d = -1.51 - 0.74\log\left(\frac{k_y}{k_{h0}}\right) + 3.27\log\left(1-\frac{k_y}{k_{h0}}\right) - 0.80\log(k_{h0}) + 1.59\log(PGV) \quad \text{(A11.5.2-3)}$$

where:

k_y = yield acceleration

For CEUS rock sites (Categories A and B), this mean displacement (in.) may be estimated as:

$$\log d = -1.31 - 0.93\left(\log\frac{k_y}{k_{h0}}\right) + 4.52\log\left(1-\frac{k_y}{k_{h0}}\right) - 0.46\log\left(k_{h0}\right) + 1.12\log(PGV) \quad \text{(A11.5.2-4)}$$

Note that the above displacement equations represent mean values.

In Eqs. A11.5.2-3 and A11.5.2-4 it is necessary to estimate the peak ground velocity (*PGV*) and the yield acceleration (k_y). Values of *PGV* may be determined using the following correlation between *PGV* and spectral ordinates at 1 sec (S_1).

$$PGV\ \text{(in./sec)} = 38F_vS_1 \quad \text{(A11.5.2-5)}$$

where S_1 is the spectral acceleration coefficient at 1 sec and F_v is the site class adjustment factor.

The development of the *PGV*-S_1 correlation is based on a simplification of regression analyses conducted on an extensive earthquake database established from recorded and synthetic accelerograms representative of both rock and soil conditions for WUS and CEUS. The study is described in NCHRP Report 611 (Anderson et al., 2008). It was found that earthquake magnitude need not be explicitly included in the correlation, as its influence on *PGV* is captured by its influence on the value of S_1. The equation is based on the mean from the simplification of the regression analysis.

Values of the yield acceleration (k_y) can be established by computing the seismic coefficient for global stability that results in a capacity to demand (*C*/*D*) ratio of 1.0 (i.e., for overall stability of the wall/slope, the *FS* = 1.0). A conventional slope stability program is normally used to determine the yield acceleration. For these analyses, the total stress (undrained) strength parameters of the soil should usually be used in the stability analysis. See guidance on the use of soil cohesion for seismic analyses discussed in Article 11.6.5.3 and its commentary.

Once k_y is determined, the combined effect of wave scattering and lateral wall displacement d on k_h is determined as follows:

$$k_h = \alpha k_y \quad \text{(A11.5.2-6)}$$

A11.5.3—Bray et al. (2010), and Bray and Travasarou (2009)

The Bray et al. (2010) method (see also Bray and Travasarou, 2009) for estimating the value of k_h applied to the wall mass considers both the wave scattering and lateral deformation of the wall. The method was developed using 688 ground motion records. The method characterizes the ground motion using a spectral acceleration at five percent damping, the moment magnitude, M, as a proxy for duration of shaking, the fundamental period of the wall, T_s, and the lateral wall deformation allowed during shaking. In this method, k_h is determined as follows:

$$k_h = \exp\left(\frac{-a+\sqrt{b}}{0.66}\right) \quad \text{(A11.5.3-1)}$$

where:

a = $2.83 - 0.566ln(S_a)$
b = $a^2 - 1.33[ln(d) + 1.10 - 3.04ln(S_a) + 0.244(ln(S_a))^2 - 1.5T_s - 0.278(M - 7) - \varepsilon]$
S_a = the five percent damped spectral acceleration coefficient from the site response spectra
d = the maximum wall displacement allowed, in centimeters
M = the moment magnitude of the design earthquake
T_s = the fundamental period of the wall
ε = a normally distributed random variable with zero mean and a standard deviation of 0.66.

ε should be set equal to zero to estimate k_h considering D_a to be a mean displacement. To calculate the fundamental period of the wall, T_s, use the following equation:

$$T_s = 4H'/V_s \quad \text{(A11.5.3-2)}$$

where:

H' = 80 percent of the height of the wall, as measured from the bottom of the heel of the wall to the ground surface directly above the wall heel (or the total wall height at the back of the reinforced soil zone for MSE walls)
V_s = the shear wave velocity of the soil behind the wall

Note that V_s and H' must have consistent units. Shear wave velocities may be obtained from in-situ measurements or through the use of correlations to the Standard Penetration Resistance (*SPT*) or cone resistance (q_c). An example of this type of correlation for granular wall backfill materials is shown in Eq. A11.5.3-3 (Imai and Tonouchi, 1982).

$$V_s = 107N^{0.314} \quad \text{(A11.5.3-3)}$$

where:

N = the Standard Penetration Resistance (*SPT*) of the fill material, uncorrected for overburden pressure but corrected for hammer efficiency

The spectral acceleration, S_a, is determined at a degraded period of $1.5T_s$ from the five percent damped response spectra for the site (i.e., either the response spectra determined using the general procedure or using a site-specific response spectra).

To estimate lateral wall displacement for a given acceleration value, see Bray et al. (2010) and Bray and Travasarou (2009) for details.

A11.6—APPENDIX REFERENCES

Anderson, D. G., G. R. Martin, I. P. Lam, and J. N. Wang. 2008. *Seismic Analysis and Design of Retaining Walls, Slopes and Embankments, and Buried Structures*, NCHRP Report 611 National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, DC.

Bathurst, R. J. and Z. Cai, Z. 1995. "Psuedo-Static Seismic Analysis of Geosynthetic-Reinforced Segmental Retaining Walls," *Geosynthetics International*. International Geosynthetic Society, Jupiter, FL, Vol. 2, No. 5, pp. 787–830.

BURIED STRUCTURES AND TUNNEL LINERS

12.1—SCOPE

This Section provides requirements for the selection of structural properties and dimensions of buried structures, e.g., culverts, and steel plate used to support tunnel excavations in soil.

Buried structure systems considered herein are metal pipe, structural plate pipe, long-span structural plate, deep corrugated plate, structural plate box, reinforced concrete pipe, reinforced concrete cast-in-place and precast arch, box and elliptical structures, and thermoplastic pipe, and fiberglass pipe.

The type of liner plate considered is cold-formed steel panels.

C12.1

For buried structures, refer to Article 2.6.6 for hydraulic design considerations and FHWA (1985) for design methods related to location, length, and waterway openings.

Thermoplastic and fiberglass pipe are flexible plastic pipes that have similarities in installation and design; however, not all thermoplastic pipe design and installation specifications are applicable to fiberglass pipe. Fiberglass pipe is a smooth-walled thermoset resin pipe that relies on composite glass fiber within its wall for strength; thermoplastic pipe can have either solid or profile walls of homogenous material. The design specifications for fiberglass pipe are contained in Article 12.15 with reference to applicable sections from the thermoplastic pipe design specifications. Construction specifications for fiberglass pipe are included in the *AASHTO LRFD Bridge Construction Specifications*, Section 30, "Thermoplastic Pipe," with the provisions for thermoplastic pipe applicable to fiberglass pipe installations except as noted.

12.2—DEFINITIONS

Abrasion—Loss of section or coating of a culvert by the mechanical action of water conveying suspended bed load of sand, gravel, and cobble-size particles at high velocities with appreciable turbulence.

Buried Structure—A generic term for a structure built by embankment or trench methods.

Corrosion—Loss of section or coating of a buried structure by chemical and/or electrochemical processes.

Culvert—A curved or rectangular buried conduit for conveyance of water, vehicles, utilities, or pedestrians.

Deep Corrugated Plate—Structural Plate in AASHTO M 167 with a corrugation depth greater than 5.0 in.

FEM—Finite Element Method

Narrow Trench Width—The outside span of rigid pipe, plus 1.0 ft.

Projection Ratio—Ratio of the vertical distance between the outside top of the pipe and the ground or bedding surface to the outside vertical height of the pipe, applicable to reinforced concrete pipe only.

Side Radius—For deep corrugated plate structures, the side radius is the radius of the plate in the section adjacent to crown (top) section of the structure. In box shaped structures, this is often called the haunch radius.

Soil Envelope—Zone of controlled soil backfill around culvert structure required to ensure anticipated performance based on soil-structure interaction considerations.

Soil-Structure Interaction System—A buried structure whose structural behavior is influenced by interaction with the soil envelope.

Tunnel—A horizontal or near horizontal opening in soil excavated to a predesigned geometry by tunneling methods exclusive of cut-and-cover methods.

12.3—NOTATION

A	=	wall area (in.^{2}/ft) (12.7.2.3)
A_{eff}	=	effective wall area (in.^{2}/in.) (12.12.3.10.1b)
A_g	=	gross wall area within a length of one period (in.^{2}/in.); area of fiber-reinforced pipe wall per unit length of pipe (12.12.3.5) (12.15.6.3)
A_L	=	axle load, taken as 50 percent of all axle loads that can be placed on the structure at one time (kip); sum of all axle loads in an axle group (kip); total axle load on single axle or tandem axles (kip) (12.8.4.2) (12.9.4.2) (12.9.4.3)
A_s	=	tension reinforcement area on cross-section width, b (in.^{2}/ft) (~~C~~12.10.4.2.4a) ~~(C12.11.4) (C12.11.5)~~
A_{smax}	=	minimum flexural reinforcement area without stirrups (in.^{2}/ft) (12.10.4.2.4c)
A_T	=	area of the top portion of the structure above the springline (ft^2) (12.8.4.2)
A_{vr}	=	stirrup reinforcement area to resist radial tension forces on cross-section width, b in each line of stirrups at circumferential spacing, s_v (in.^{2}/ft) (12.10.4.2.6)
A_{vs}	=	required area of stirrups for shear reinforcement (in.^{2}/ft) (12.10.4.2.6)
B	=	width of culvert (ft) (C12.6.2.2.5)
B_c	=	outside diameter or width of the structure (ft) (12.6.6.3)
B'_c	=	out-to-out vertical rise of pipe (ft) (12.6.6.3)
B_d	=	horizontal width of trench at top of pipe (ft) (12.11.2.2)
B_{FE}	=	earth load bedding factor (12.10.4.3.1)
B_{FLL}	=	live load bedding factor (12.10.4.3.1)
B_1	=	crack control coefficient for effect of cover and spacing of reinforcement (C12.10.4.2.4d)
b	=	width of section (12.10.4.2.4c)
b_e	=	element effective width (in.) (12.12.3.10.1b)
C_A	=	constant corresponding to the shape of the pipe (12.10.4.3.2a)
C_c	=	load coefficient for positive pipe projection (12.10.4.3.2a)
C_d	=	load coefficient for trench installation (12.11.2.2)
C_{dt}	=	load coefficient for tunnel installation (12.13.2.1)
C_H	=	adjustment factor for shallow cover heights over metal box culverts (12.9.4.4)
C_L	=	width of culvert on which live load is applied parallel to span (ft); live load coefficient as specified in Article 12.12.3.5 (12.7.2.2) (12.12.2.2)
C_n	=	calibration factor to account for nonlinear effects (12.12.3.10.1e)
$C_{\ell\ell}$	=	live load adjusted for axle loads, tandem axles, and axles with other than four wheels; $C_1 C_2 A_L$ (kip) (12.9.4.2)
C_N	=	parameter that is a function of the vertical load and vertical reaction (12.10.4.3.2a)
C_s	=	construction stiffness for tunnel liner plate (kip/in.) (12.5.6.4)
C_1	=	1.0 for single axles and $0.5 + S/50 \leq 1.0$ for tandem axles; adjustment coefficient for number of axles; crack control coefficient for various types of reinforcement (12.9.4.2) (12.9.4.3) (C12.10.4.2.4d)
C_2	=	adjustment factor for number of wheels on a design axle as specified in Table 12.9.4.2-1; adjustment coefficient for number of wheels per axle (12.9.4.2) (12.9.4.3)
D	=	straight leg length of haunch (in.); pipe diameter (in.); required D-load capacity of reinforced concrete pipe (klf); diameter to centroid of pipe wall (in.) (12.9.4.1) (12.6.6.2) (12.10.4.3.1) (12.12.2.2)
D-$load$	=	resistance of pipe from three-edge bearing test load to produce a 0.01-in. crack (klf) (12.10.4.3)
D_f	=	shape factor (12.12.3.10.2b)
D_i	=	inside diameter of pipe (in.) (12.10.4.3.1)
D_L	=	deflection lag factor (12.12.2.2)
D_o	=	outside diameter of pipe (in.) (12.12.2.2)
d	=	required envelope width adjacent to the structure (ft); distance from compression face to centroid of tension reinforcement (in.) (12.8.5.3) (12.10.4.2.4a) (C12.11.4)
d'	=	width of warped embankment fill to provide adequate support for skewed installation (ft) (C12.6.8.2)
d_1	=	distance from the structure (ft) (12.8.5.3)
E	=	modulus of elasticity of the plastic (ksi); initial modulus of elasticity (ksi) (12.12.3.3) (12.12.3.6)
E_{cf}	=	circumferential flexural modulus (ksi) (12.15.5.2)
E_m	=	modulus of elasticity of metal (ksi) (12.7.2.4)
E_p	=	short- or long-term modulus of pipe material as specified in Table 12.12.3.3-1 (ksi) (12.12.2.2)
$E(x)$	=	lateral unbalanced distributed load on culvert below sloping ground and skewed at end wall (lbs.) (C12.6.2.2.5)
E_{50}	=	50-year modulus of elasticity (ksi) (12.12.3.3)
E_{75}	=	75-year modulus of elasticity (ksi) (12.12.3.3)
F	=	concentrated load acting at the crown of a culvert (kip) (C12.6.2.2.5)

12.8.2—Service Limit State

No service limit state criteria need be required.

C12.8.2

Soil design and placement requirements for long-span structures are intended to limit structure deflections. The contract documents should require that construction procedures be monitored to ensure that severe deformations do not occur during backfill placement and compaction.

12.8.3—Safety Against Structural Failure

With the exception of the requirements for buckling and flexibility, the provisions of Article 12.7 shall apply, except as described herein.

Dimensions and properties of structure cross-sections, minimum seam strength, mechanical and chemical requirements, and bolt properties for long-span structural plate sections shall be taken as specified in Appendix A12 or as described herein.

C12.8.3

Most long-span culverts are designed for a larger load factor; however, the limit states of flexure and buckling are ignored for those structures. Considering these limit states reduces the uncertainty in the final design and permits use of a lower load factor. This is the same approach used for metal box culverts.

12.8.3.1—Section Properties

12.8.3.1.1—Cross-Section

The provisions of Article 12.7 shall apply, except as specified.

Structures not described herein shall be regarded as special designs.

Table A12-3 shall apply. Minimum requirements for section properties shall be taken as specified in Table 12.8.3.1.1-1. Covers that are less than that shown in ~~Table 12.8.3.1-1~~ Table 12.8.3.1.1-1 and that correspond to the minimum plate thickness for a given radius may be used if ribs are used to stiffen the plate. If ribs are used, the plate thickness may not be reduced below the minimum shown for that radius, and the moment of inertia of the rib and plate section shall not be less than that of the thicker unstiffened plate corresponding to the fill height. Use of soil cover less than the minimum values shown for a given radius shall require a special design.

Design not covered in Table 12.8.3.1.1-1 should not be permitted unless substantiated by documentation acceptable to the Owner.

C12.8.3.1.1

Sharp radii generate high soil pressures. Avoid high ratios when significant heights of fill are involved.

Table 12.8.3.1.1-1—Minimum Requirements for Long-Span Structures with Acceptable Special Features

Top Arc Minimum Thickness (in.)					
Top Radius (ft)	≤15.0	15.0–17.0	17.0–20.0	20.0–23.0	23.0–25.0
6" × 2" Corrugated Steel Plate—Top Arc Minimum Thickness (in.)	0.111	0.140	0.170	0.218	0.249
Geometric Limits					
The following geometric limits shall apply: • Maximum plate radius—25.0 ft • Maximum central angle of top arc—80.0° • Minimum ratio, top arc radius to side arc radius—2 • Maximum ratio, top arc radius to side arc radius—5					
Minimum Cover (ft)					
Top Radius (ft)	≤ 15.0	15.0–17.0	17.0–20.0	20.0–23.0	23.0–25.0
Steel thickness without ribs (in.)					
0.111	2.5	—	—	—	—
0.140	2.5	3.0	—	—	—
0.170	2.5	3.0	3.0	—	—
0.188	2.5	3.0	3.0	—	—
0.218	2.0	2.5	2.5	3.0	—
0.249	2.0	2.0	2.5	3.0	4.0
0.280	2.0	2.0	2.5	3.0	4.0

12.8.3.1.2—Shape Control

The requirements of Articles 12.7.2.4 and 12.7.2.6 shall not apply for the design of long-span structural plate structures.

12.8.3.1.3—Mechanical and Chemical Requirements

Tables A12-3, A12-8, and A12-10 shall apply.

12.8.3.2—Thrust

The factored thrust in the wall shall be determined by Eq. 12.7.2.2-1, except the value of S in the Equation shall be replaced by twice the value of the top arc radius, R_T.

12.8.3.3—Wall Area

The provisions of Article 12.7.2.3 shall apply.

12.8.3.4—Seam Strength

The provisions of Article 12.7.2.5 shall apply.

procedures were developed for the structural details as well.

Common types of MBJS are shown in Figures C14.5.6.9.1-1 through C14.5.6.9.1-3.

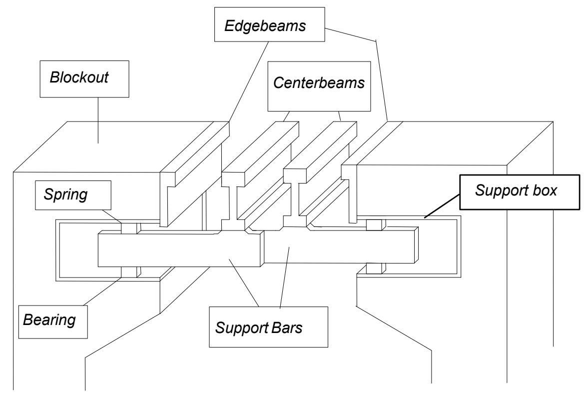


Figure C14.5.6.9.1-1—Cut-away View of Typical Welded-multiple-support-bar (WMSB) Modular Bridge Joint System (MBJS) Showing Support Bars Sliding within Support Boxes

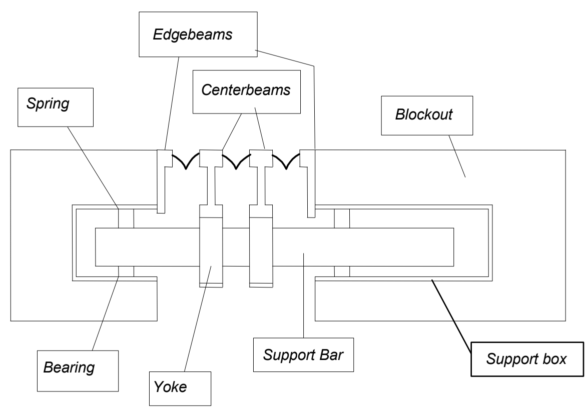


Figure C14.5.6.9.1-2—Cross-Section View of Typical Single-support-bar (SSB) Modular Bridge Joint System (MBJS) Showing Multiple Centerbeams with Yokes Sliding on a Single Support Bar

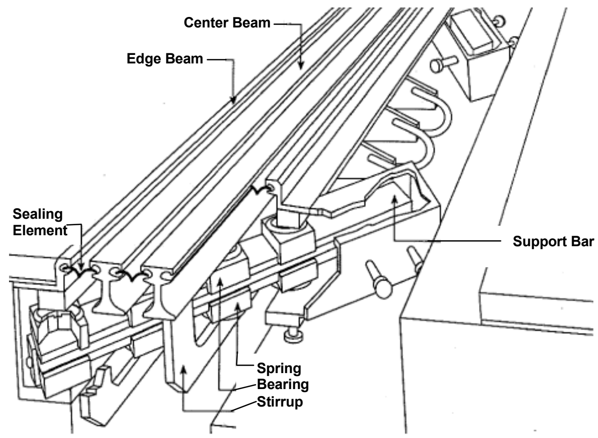


Figure C14.5.6.9.1-3—Cut-away View of a "Swivel Joint," i.e., a Special Type of Single-support-bar (SSB) Modular Bridge Joint System (MBJS) with a Swiveling Single Support Bar

14.5.6.9.2—Performance Requirements

The required minimum MBJS movement range capabilities for the six possible degrees of freedom given in Table 14.5.6.9.2-1 shall be added to the maximum movement and rotations calculated for the entire range of seals in the MBJS determined using the appropriate strength load combination specified in Table 3.4.1-1.

Table 14.5.6.9.2-1—Additional Minimum Movement Range Capability for MBJS

Type of Movement	Minimum Design Movement Range*
Longitudinal Displacement	Estimated Movement + 1.0 in.
Transverse Movement	1.0 in.
Vertical Movement	1.0 in.
Rotation around Longitudinal Axis	1°
Rotation Around Transverse Axis	1°
Rotation Around Vertical Axis	0.5°

* Total movement ranges presented in the table are twice the plus or minus movement.

C14.5.6.9.2

The MBJS should be designed and detailed to minimize excessive noise or vibration during the passage of traffic.

A common problem with MBJS is that the seals fill with debris. Traffic passing over the joint can work the seal from its anchorage by compacting this debris. MBJS systems can eject most of this debris in the traffic lanes if the seals are opened to near their maximum opening. Therefore, it is prudent to provide for additional movement capacity.

MBJS should permit movements in all six degrees of freedom, i.e., translations in all three directions and rotations about all three axes. While it is mandatory to provide at least 1.0 in. movement in the longitudinal direction, as shown in Table 14.5.6.9.2-1, no more than 2.0 in. should be provided in addition to the maximum calculated movement if feasible. Also, more than 1.0 in. should not be added if it causes a further seal to be used. In the five degrees of freedom other than the longitudinal direction, the MBJS should provide the maximum calculated movement in conjunction with providing for at least the minimum additional movement ranges shown in Table 14.5.6.9.2-1. Half of the movement range shall be assumed to occur in each direction about the mean position. Some bridges may require greater than the additional specified minimum values.

The designer should consider showing the total estimated transverse and vertical movement in each direction, as well as the rotation in each direction about the three principal axes on the contract plans. Vertical movement due to vertical grade, with horizontal bearings, and vertical movement due to girder and rotation may also be considered.

Further design guidelines and recommendations can be found in Chapter 19 of the *AASHTO LRFD Bridge Construction Specifications* and Dexter et al. (1997).