Section 8
SEISMIC DESIGN REQUIREMENTS (SDR)

8.1 GENERAL

Bridges classified as SDR in accordance with Table 3.7-2 of Article 3.7 shall conform to all of the requirements of this section.

8.2 DESIGN FORCES

8.2.1 Ductile Substructures (R>1) — Flexural Capacity

8.2.1.1 SDAP C

The sum of the capacities of all columns must satisfy Article 5.4.1.

8.2.1.2 SDAP D and E

Column design forces are the maximum of those obtained from an elastic analysis and reduced using the appropriate R-Factor as specified in Steps 2, 3 and 4 of Article 4.5 and combined in accordance with Article 3.6.

8.2.2 Capacity Protected Elements or Actions

The design provisions of Article 4.8 apply to capacity protected elements and actions.

Capacity design principles require that those elements not participating as part of the primary energy dissipating system (flexural hinging in columns), such as column shear, joints and cap beams, spread footings, pile caps and foundations be "capacity protected". This is achieved by ensuring the maximum overstrength moment and shear from plastic hinges in the columns can be dependably resisted by adjoining elements.

Exception: Elastic design of all substructure elements (Article 4.10), seismic isolation design (Article 8.10) and in the transverse direction of a column when a ductile diaphragm is used (Article 8.7.8.2).

8.2.3 Elastically Designed Elements

There may be instances where a designer chooses to design all of the substructure supports elastically (i.e., R=1.0 for all substructures) or in some cases a limited number of substructure elements are designed elastically. If so, the provisions of Article 4.10 apply.

8.2.4 Abutments and Connections

The seismic design forces for abutments are obtained by SDAP D or E when required and given in Article 8.5. The seismic design forces for connections are the lower of those obtained from Article 8.2.2 or the elastic forces divided by the appropriate R-Factor from Table 4.7-2.

8.2.5 Single Span Bridges

For single-span bridges, regardless of seismic zone and in lieu of a rigorous analysis, the minimum design force at the connections in the restrained direction between the superstructure and the substructure shall not be less than the product of $F_a S_S / 2.5$, and the tributary permanent load.

8.3 DESIGN DISPLACEMENTS

8.3.1 General

For this section, displacement is the displacement at the center of mass for a pier or bent in the transverse or longitudinal direction determined from the seismic analysis except in Article 8.3.2 where the displacement occurs at the bearing seat.

8.3.2 Minimum Seat Width Requirement

The seat width shall not be less than 1.5 times the displacement of the superstructure at the seat according to Equation (8.3.4-2) or:

$$N = \left[0.10 + 0.0017L + 0.007H + 0.05\sqrt{H} \cdot \sqrt{1 + \left(2\frac{B}{L}\right)^2} \right] \frac{1 + 1.25F_v S_1}{\cos\alpha} \quad (8.3.2\text{-}1)$$

where,

L = distance between joints in meters
H = tallest pier between the joints in meters
B = width of the superstructure in meters
α = skew angle

The ratio B/L need not be taken greater than 3/8.

8.3.3 Displacement Compatibility

All components that are not designed to resist seismic loads must have deformation capacity sufficient to transfer non-seismic loads.

8.3.4 P-Δ Requirements

The displacement of a pier or bent in the longitudinal and transverse direction must satisfy

$$\Delta \le 0.25 C_c H \quad (8.3.4\text{-}1)$$

where,

$$\Delta = R_d \Delta_e \quad (8.3.4\text{-}2)$$

$$R_d = \left(1 - \frac{1}{R}\right)\frac{1.25T_s}{T} + \frac{1}{R} \text{ for } T < 1.25T_s \quad (8.3.4\text{-}3)$$

where T_s is defined in Figure 3.4.1-1, otherwise $R_d = 1$,

Δ_e is the displacement demand from the seismic analysis, R is the ratio between elastic lateral force and the lateral strength of the pier or bent, C_c is the seismic coefficient based on the lateral strength of the pier or bent ($C_c = V/W$ where V is the lateral strength), and H is the height of the pier from the point of fixity for the foundation.

If a nonlinear time history seismic analysis is performed, the displacement demand, Δ, may be obtained directly from the analysis in lieu of Equation 8.3.4-2. However, the displacement Δ shall not be taken less than 0.67 of the displacement determined from an elastic response spectrum analysis.

8.3.5 Minimum Displacement Requirements for Lateral Load Resisting Piers and Bents

For SDAP E the maximum permitted displacement capacity from the Displacement Capacity Verification must be greater than the displacement demand according to the following requirement:

$$1.5\Delta \le \Delta_{capacity} \quad (8.3.5\text{-}1)$$

where the Δ is defined in Article 8.3.4 and $\Delta_{capacity}$ is the maximum displacement capacity per Article 5.4.3.

When a nonlinear dynamic analysis is performed the displacement demand may not be taken less than 0.67 times the demand from a elastic response spectrum analysis, nor may the displacement capacity be taken greater than the capacity from the Displacement Capacity Verification.

8.4 FOUNDATION DESIGN REQUIREMENTS

8.4.1 Foundation Investigation

8.4.1.1 General

A subsurface investigation, including borings and laboratory soil tests, shall be conducted in accordance with the provisions of Appendix B to provide pertinent and sufficient information for the determination of the Site Class of Article 3.4.2.1. The type and cost of foundations should be considered in the economic, environmental, and aesthetic studies for location and bridge type selection.

8.4.1.2 Subsurface Investigation

Subsurface explorations shall be made at pier and abutment locations, sufficient in number and depth, to establish a reliable longitudinal and transverse substrata profile. Samples of material encountered shall be taken and preserved for future reference and/or testing. Boring logs shall be prepared in detail sufficient to locate material strata, results of penetration tests, groundwater, any artesian action, and where samples were taken. Special attention shall be paid to the detection of narrow, soft seams that may be located at stratum boundaries.

8.4.1.3 Laboratory Testing

Laboratory tests shall be performed to determine the strength, deformation, and flow characteristics of soils and/or rocks and their suitability for the foundation selected. In areas of higher seismicity (e.g., SDR 3 or 4), it may be appropriate to conduct special dynamic or cyclic tests to establish the liquefaction potential or stiffness and material damping properties of the soil at some sites, if unusual soils exist or if the foundation is supporting a critical bridge.

8.4.2 Spread Footings

The design of spread footing foundations located in SDR 4 shall be based on column moments and shears developed using capacity design principles as described in Section 4.8.

Foundation flexibility (Article 5.3.4) shall be modeled for Soil Types C, D, and E if foundation flexibility results in more than a 20 percent change in response (Article C5.3.4). For Soil Types A and B, soil flexibility does not need to be considered because of the stiffness of the soil or rock. The potential for and effects of liquefaction and dynamic settlement shall also be determined for spread footing foundations subject to SDR 4. Normally, spread footings shall not be located at sites within SDR 4 where liquefaction is predicted to occur, unless:

- The foundation is located below the liquefiable layer.
- It can be demonstrated by special studies that liquefaction and its effects are very limited, or
- The ground will be improved such that liquefaction will not occur.

Owner approval shall be obtained before proceeding with a spread footing design at a site where liquefaction is predicted to occur.

8.4.2.1 Spring Constants for Footing (Nonliquefiable Sites)

When required to represent foundation flexibility, spring constants shall be developed for spread footing using equations given in Tables 8.4.2.1-1 and 8.4.2.1-2. Alternate procedures given in FEMA 273 (1997) are also suitable for estimating spring constants. These computational methods are appropriate for sites that do not liquefy or lose strength during earthquake loading. See Article 8.4.2.3 for sites that are predicted to liquefy.

The shear modulus (G) used to compute the stiffness values in Table 8.4.2.1-1 shall be determined by adjusting the low-strain shear modulus (G_{max}) for the level of shearing strain using the following strain adjustment factors, unless other methods are approved by the Owner.

$\underline{F_v S_1 \leq 0.40}$

- $G/G_{max} = 0.50$ for 50% in 75-year event
- $G/G_{max} = 0.25$ for 3% in 75-year event

$\underline{F_v S_1 > 0.40}$

- $G/G_{max} = 0.25$ for 50% in 75-year event
- $G/G_{max} = 0.10$ for 3% in 75-year event

Uplift shall be allowed for footings subject to SDR 4. The following area adjustment factors (R_a) shall be applied to the equivalent area to account for geometric nonlinearity introduced by uplift, unless the Owner approves otherwise.

$\underline{F_v S_1 \leq 0.40}$

- $R_a = 1.0$ for the 50% in 75-year event
- $R_a = 0.75$ for the 3% in 75-year event

Table 8.4.2.1-1. Surface Stiffnesses for a Rigid Plate on a Semi-Infinite Homogeneous Elastic Half-Space (adapted from Gazetas, 1991)[1]

Stiffness Parameter	*Rigid Plate Stiffness at Surface, Ki'*
Vertical Translation, K_z'	$\frac{GL}{1-\nu}\left[0.73+1.54\left(\frac{B}{L}\right)^{0.75}\right]$
Horizontal Translation, K_y' (toward long side)	$\frac{GL}{2-\nu}\left[2+2.5\left(\frac{B}{L}\right)^{0.85}\right]$
Horizontal Translation, K_x' (toward short side)	$\frac{GL}{2-\nu}\left[2+2.5\left(\frac{B}{L}\right)^{0.85}\right]-\frac{GL}{0.75-\nu}\left[0.1\left(1-\frac{B}{L}\right)\right]$
Rotation, $K_{\theta x}'$ (about x axis)	$\frac{G}{1-\nu}\ I_X^{0.75}\ \left(\frac{L}{B}\right)^{0.25}\left(2.4+0.5\frac{B}{L}\right)$
Rotation, $K_{\theta y}'$ (about y axis)	$\frac{G}{1-\nu}\ I_Y^{0.75}\ \left[3\left(\frac{L}{B}\right)^{0.15}\right]$

1. See Figure 8.4.2.1-1** for definitions of terms

Table 8.4.2.1-2. Stiffness Embedment Factors for a Rigid Plate on a Semi-Infinite Homogeneous Elastic Half-Space (adapted from Gazetas, 1991)[1]

Stiffness Parameter	*Embedment Factors, ei*
Vertical Translation, e_z	$\left[1+0.095\frac{D}{B}\left(1+1.3\frac{B}{L}\right)\right]\left[1+0.2\left(\frac{(2L+2B)}{LB}\ d\right)^{0.67}\right]$
Horizontal Translation, e_y (toward long side)	$\left[1+0.15\left(\frac{2D}{B}\right)^{0.5}\right]\left\{1+0.52\left[\frac{\left(D-\frac{d}{2}\right)16\left(L+B\right)d}{BL^2}\right]^{0.4}\right\}$
Horizontal Translation, e_x (toward short side)	$\left[1+0.15\left(\frac{2D}{L}\right)^{0.5}\right]\left\{1+0.52\left[\frac{\left(D-\frac{d}{2}\right)16\left(L+B\right)d}{LB^2}\right]^{0.4}\right\}$
Rotation, $e_{\theta x}$ (about x axis)	$1+2.52\frac{d}{B}\left(1+\frac{2d}{B}\left(\frac{d}{D}\right)^{-0.20}\left(\frac{B}{L}\right)^{0.50}\right)$
Rotation, $e_{\theta y}$ (about y axis)	$1+0.92\left(\frac{2d}{L}\right)^{0.60}\left(1.5+\left(\frac{2d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.60}\right)$

Note. Embedment factors multiplied by spring

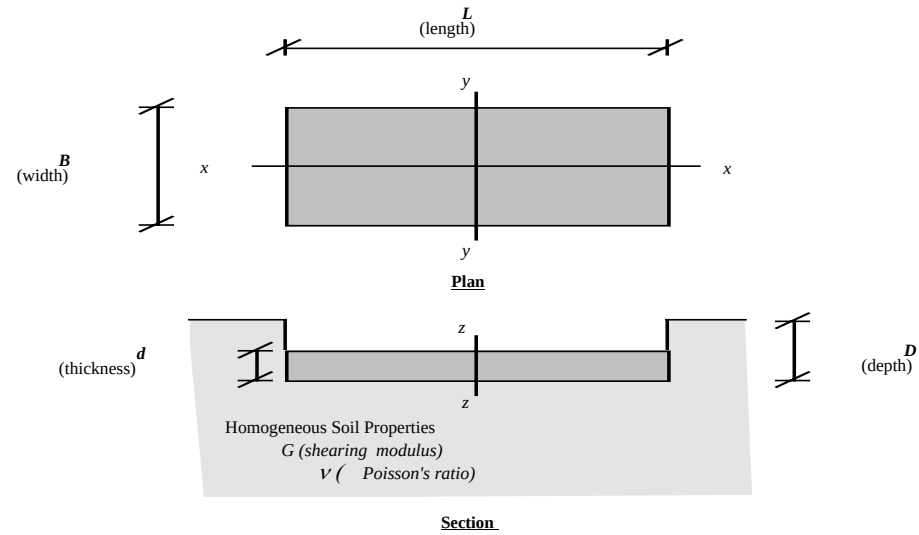


Figure 8.4.2.1-1. Properties of a Rigid Plate on a Semi-infinite Homogeneous Elastic Half-Space for Stiffness Calculations

$F_vS_1 > 0.40$

- R_a = 0.75 for the 50% in 75-year event
- R_a = 0.5 for the 3% in 75-year event

Values of G_{max} shall be determined by seismic methods (e.g., crosshole, downhole, or SASW), by laboratory testing methods (e.g., resonant column with adjustments for time), or by empirical equations (Kramer, 1996). The uncertainty in determination of G_{max} shall be considered when establishing strain adjustment factors.

No special computations are required to determine the geometric or radiation damping of the foundation system. Five percent system damping shall be used for design, unless special studies are performed and approved by the Owner.

8.4.2.2 Moment-Rotation and Shear-Displacement Relationships for Footing (Nonliquefiable Sites)

The moment and shear capacity of the foundation shall be confirmed for design loads given in Article 4.8. Moment-rotation and shear force-displacement relationships shall be developed as required by Article 5.3.4. Unless approved otherwise by the Owner, the moment-rotation curve for SDAP E shall be represented by a bilinear, moment-rotation curve. The initial slope of the bi-linear curve shall be defined by the rotational spring constant given in Article 8.4.2.1.

The maximum resisting force (i.e., plastic capacity) on the force-deformation curve shall be defined for the best-estimate case. The footing liftoff shall be no more that 50 percent at peak displacement during the push-over analysis, unless special studies are performed and approved by the Owner. A bilinear force displacement relationship shall also be developed for the shear component of resistance.

This approach shall not be used at sites that will liquefy during seismic loading. See Article 8.4.2.3 for sites that liquefy.

8.4.2.3 Liquefaction and Dynamic Settlement

An evaluation of the potential for liquefaction within near-surface soil shall be made in accordance with requirements given in Article 8.6 and Appendix D of these Specifications. If liquefaction is predicted to occur under the design ground motion, spread footings foundations shall not be used unless

- the footing is located below the liquefiable layer
- ground improvement is performed to mitigate the occurrence of liquefaction, or
- special studies are conducted to demonstrate that the occurrence of liquefaction will not be

detrimental to the performance of the bridge support system.

The Owner's approval shall be obtained before initiating ground improvement or special studies.

8.4.3 Driven Piles

8.4.3.1 General

Resistance factors for pile capacities shall be as specified in Table 10.5.4-2 of the AASHTO LRFD provisions, with the exception that resistance factors of 1.0 shall be used for seismic loads.

For the effect of settling ground and downdrag loads, unfactored load and resistance factors (γ = 1.0; ϕ = 1.0) shall be used, unless required otherwise by the Owner.

Batter piles shall not be used where downdrag loads are expected unless special studies are performed.

For seismic loading the groundwater table location shall be the average groundwater location, unless the Owner approves otherwise.

8.4.3.2 Design Requirements

The design of driven pile foundations shall be based column loads determined by capacity design principles (Article 4.8) or elastic seismic forces, whichever is smaller. Both the structural and geotechnical elements of the foundation shall be designed for the capacity design forces of Article 4.8.

Foundation flexibility (Article 5.3.4) shall be incorporated into design for Soil Profile Types C, D, and E, if the effects of foundation flexibility contribute more than 20 percent to the displacement of the system. For SDAP E foundations flexibility shall be included in the push-over analysis whenever it is included in the dynamic analysis.

Liquefaction shall be considered when applicable during the development of spring constants and capacity values for these seismic design and analysis procedures.

8.4.3.3 Axial and Rocking Stiffness for Driven Pile/Pile Cap Foundations (Nonliquefiable Sites)

The axial stiffness of the driven pile foundations shall be determined for design cases in which foundation flexibility is included. For many applications, the axial stiffness of a group of piles can be estimated within sufficient accuracy using the following equation:

$$K_{sv} = \Sigma\ 1.25AE/L \qquad (8.4.3.2\text{-}1)$$

where

A = cross-sectional area of the pile

E = modulus of elasticity of the piles

L = length of the piles

N = number of piles in group and is represented by the summation symbol in the above equations.

The rocking spring stiffness values about each horizontal pile cap axis can be computed assuming each axial pile spring acts as a discrete Winkler spring. The rotational spring constant (i.e., moment per unit rotation) is then given by

$$K_{srv} = \Sigma\ k_{vn}\ S_n^2 \qquad (8.4.3.2\text{-}2)$$

where

k_{vn} = axial stiffness of the nth pile

S_n = distance between the nth pile and the axis of rotation

The effects of group action on the determination of stiffness shall be considered if the center-to-center spacing of piles for the group in the direction of loading is closer than 3 pile diameters.

8.4.3.4 Lateral Stiffness Parameters for Driven Pile/Pile Cap Foundations (Nonliquefiable Sites)

The lateral stiffness parameters of driven pile foundations shall be estimated for design cases in which foundation flexibility is included. Lateral response of a pile foundation system depends on

the stiffness of the piles and, very often, the stiffness of the pile cap. Procedures for defining the stiffness of the pile component of the foundation system are covered in this article. Methods for introducing the pile cap stiffness are addressed in Article 8.4.3.5.

For preliminary analyses involving an estimate of the elastic displacements of the bridge, pile stiffness values can be obtained by using a series of charts prepared by Lam and Martin (1986). These charts are reproduced in Figures 8.4.3.4-1 through 8.4.3.4-6. The charts are applicable for mildly nonlinear response, where the elastic response of the pile dominates the nonlinear soil stiffness.

For push-over analyses the lateral load displacement relationship must be extended into the nonlinear range of response. It is usually necessary to use computer methods to develop the load-displacement relationship in this range, as both the nonlinearity of the pile and the soil must be considered. Programs such as LPILE (Reese et al., 1997), COM 624 (Wang and Reese, 1991), and FLPIER (Hoit and McVay, 1996) are used for this purpose. These programs use nonlinear "p-y" curves to represent the load-displacement response of the soil; they also can accommodate different types of pile-head fixity. Procedures for determining the "p-y" curves are discussed by Lam and Martin (1986) and more recently by Reese et al. (1997).

The effects of group action on lateral stiffness shall be considered if the center-to-center spacing of the piles is closer than 3 pile diameters.

8.4.3.5 Pile Cap Stiffness and Capacity

The stiffness and capacity of the pile cap shall be considered in the design of the pile foundation. The pile cap provides horizontal resistance to the shear loading in the column. Procedures for evaluating the stiffness and the capacity of the footing in shear shall follow procedures given in Article C8.4.2.2 for spread footings, except that the base shear resistance of the cap shall be neglected.

When considering a system comprised of a pile and pile cap, the stiffness of each shall be considered as two springs in parallel. The composite spring shall be developed by adding the reaction for each spring at equal displacements.

8.4.3.6 Moment and Shear Design (Nonliquefiable Sites)

The capacity of the structural elements of driven pile foundations shall be designed to resist the capacity design forces of Article 4.8 or the elastic design force within the column, whichever is smaller. Unfactored resistance (= 1.0) shall be used in performing the geotechnical capacity check. The leading row piles during overturning shall not exceed the plunging capacity of the piles. Separation between the pile tip and the soil (i.e. gapping) shall be allowed only in the most distant row of trailing piles. Forces on all other rows of piles shall either be compressive or not exceed the nominal tension capacity of the piles. The maximum shear force on the pile(s) shall be less than the structural shear capacity of the piles.

If the plunging capacity is exceeded or gapping of other than the trailing row of piles occurs, special studies shall be conducted to show that performance of the pile system is acceptable. Special studies shall be performed only with the prior consent of the Owner and require SDAP E.

8.4.3.7 Liquefaction and Dynamic Settlement Evaluations

If liquefaction is predicted to occur at the site, effects of liquefaction on the bridge foundation shall be evaluated. This evaluation shall consider the potential for loss in lateral bearing support, flow and lateral spreading of the soil, settlement below the toe of the pile, and settlement from drag loads on the pile as excess porewater pressures in liquefied soil dissipate. Procedures given in Appendix D shall be followed when making these evaluations.

If liquefaction causes unacceptable bridge performance, consideration should be given to the use of ground improvement methods to meet design requirements. In light of the potential costs of ground improvement, the Owner shall be consulted before proceeding with a design for ground improvement to review the risks associated with liquefaction relative to the costs for remediating the liquefaction potential.

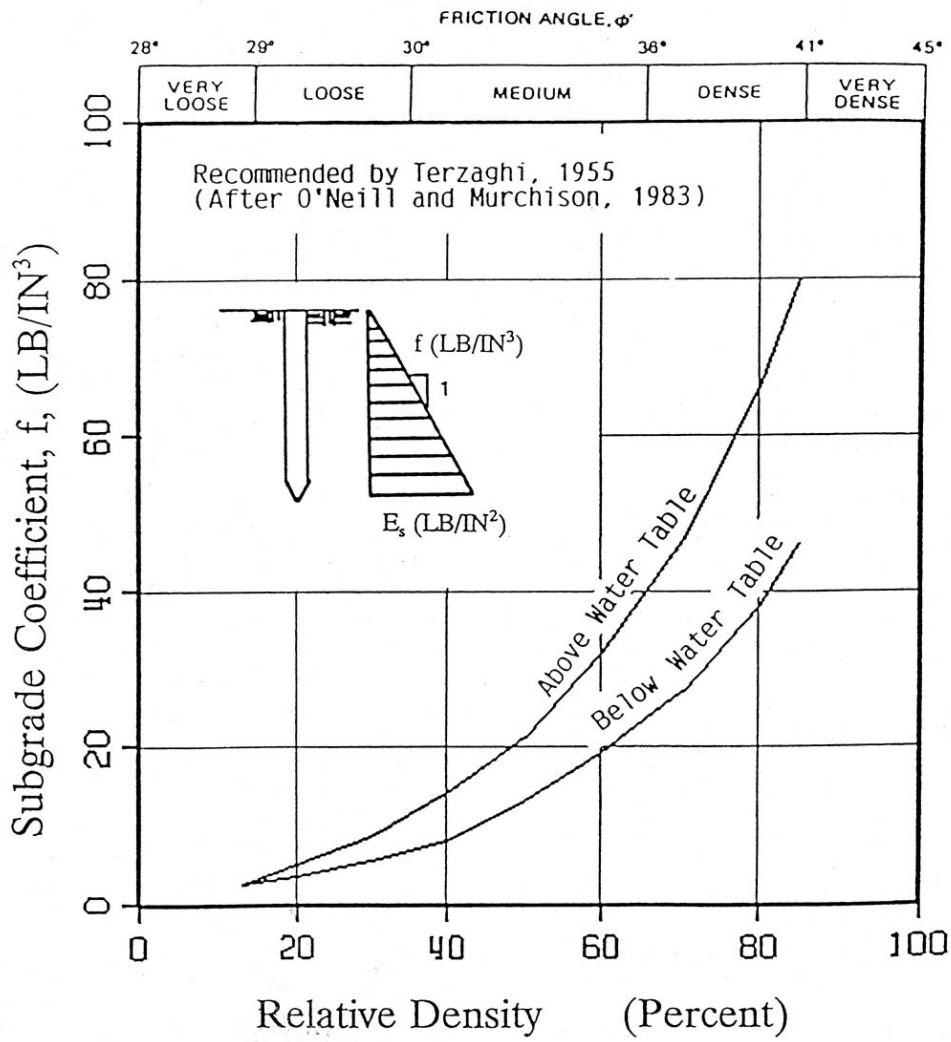


Figure 8.4.3.4-1. Recommendations for Coefficient of Variation in Subgrade Modulus with Depth for Sand (ATC, 1996)

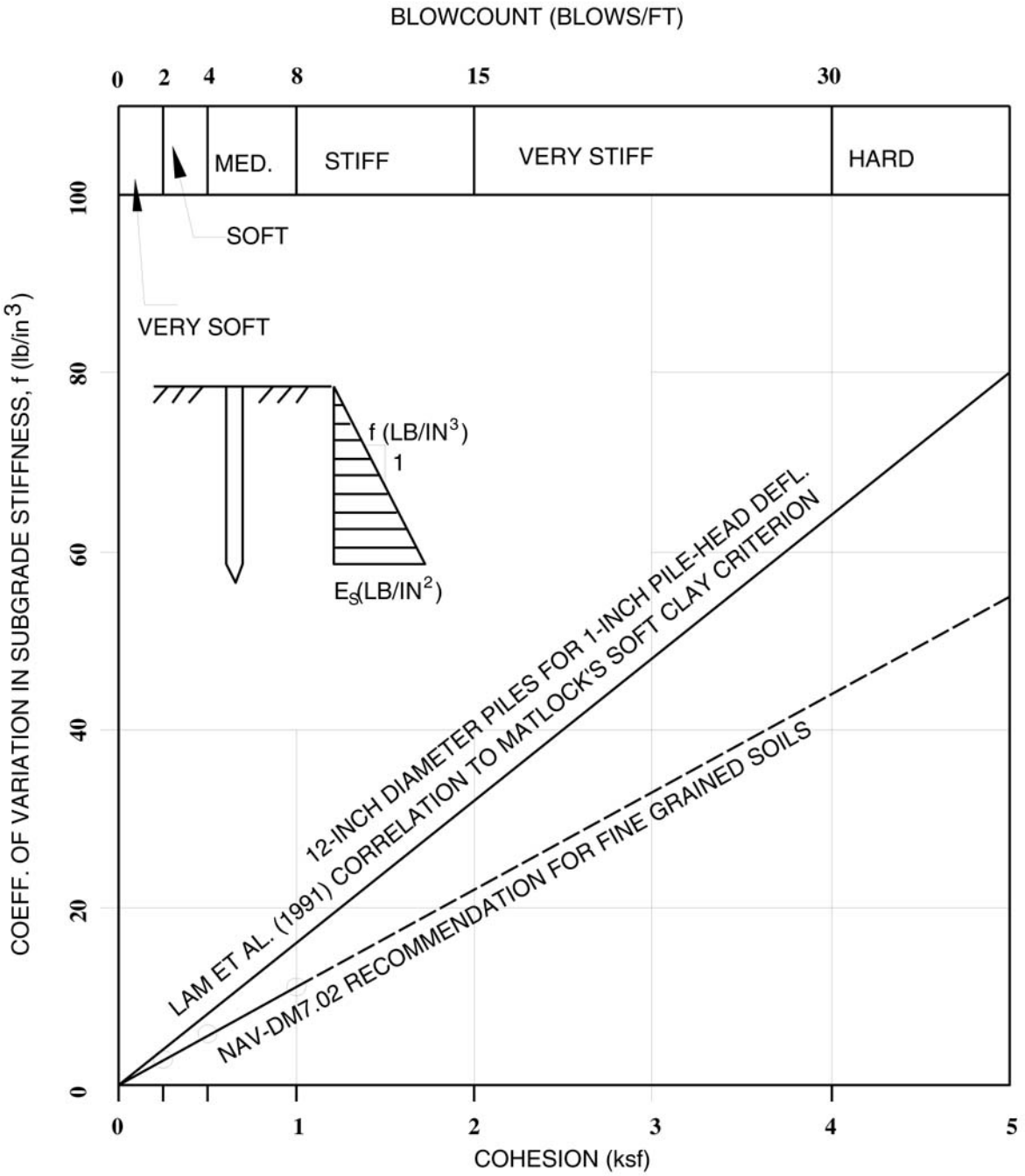


Figure 8.4.3.4-2. Recommendations for Coefficient of Variation in Subgrade Modulus with Depth for Clay (ATC, 1996)

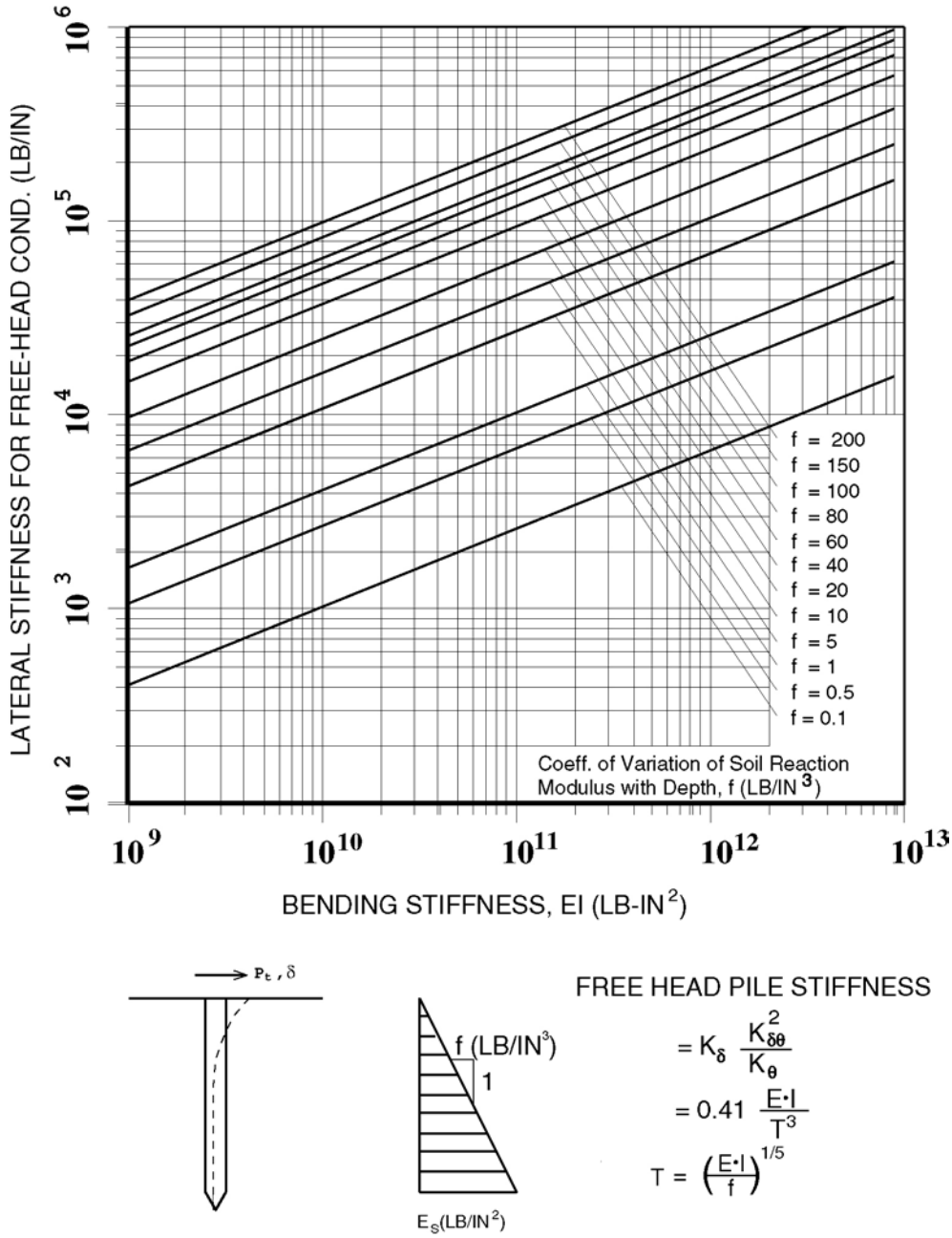


Figure 8.4.3.4-3. Coefficient of Lateral Pile Head Stiffness for Free-Head Pile Lateral Stiffness (ATC, 1996)

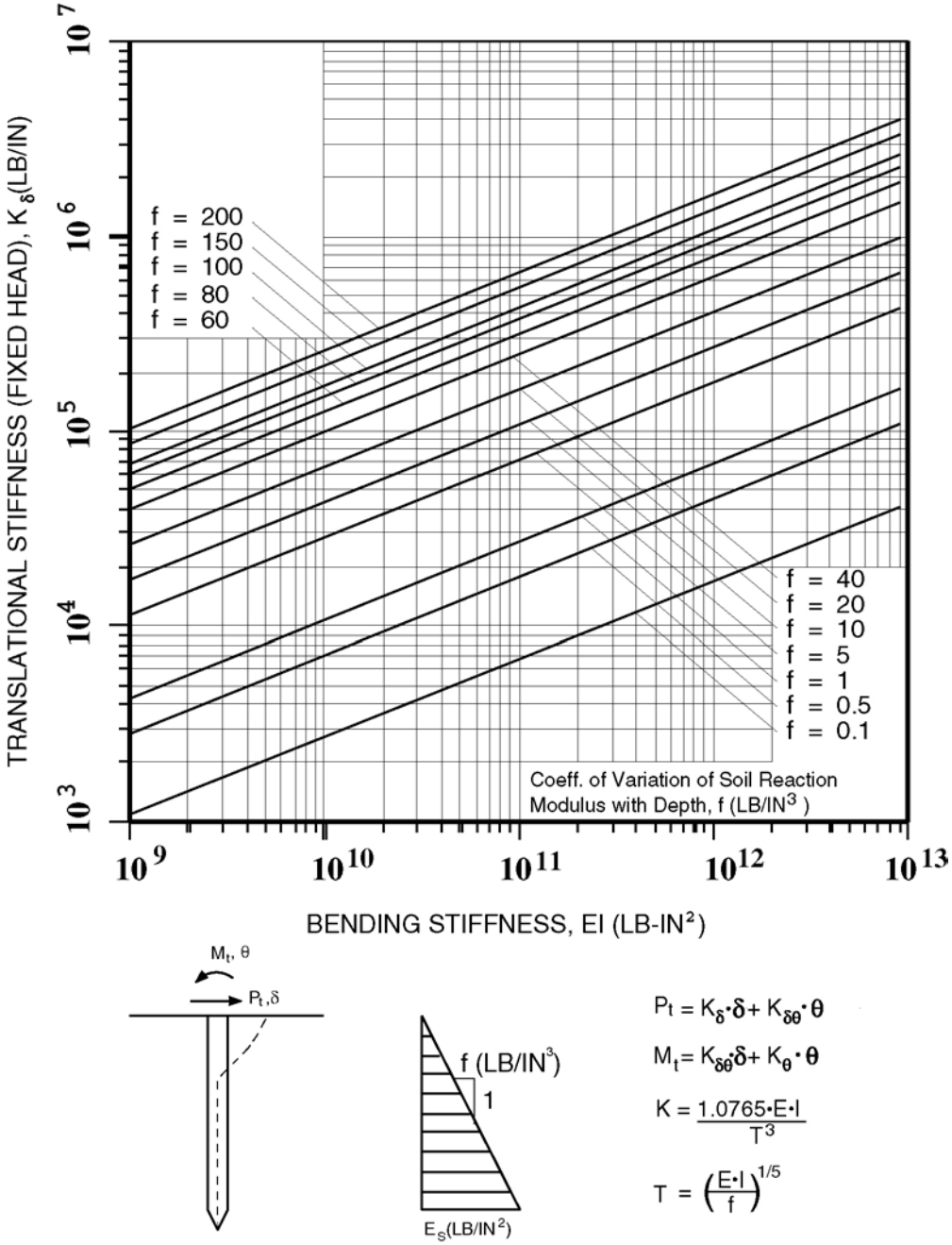


Figure 8.4.3.4-4 Coefficient for Lateral Pile-Head Stiffness for Fixed-Head Pile Lateral Stiffness (ATC, 1996)

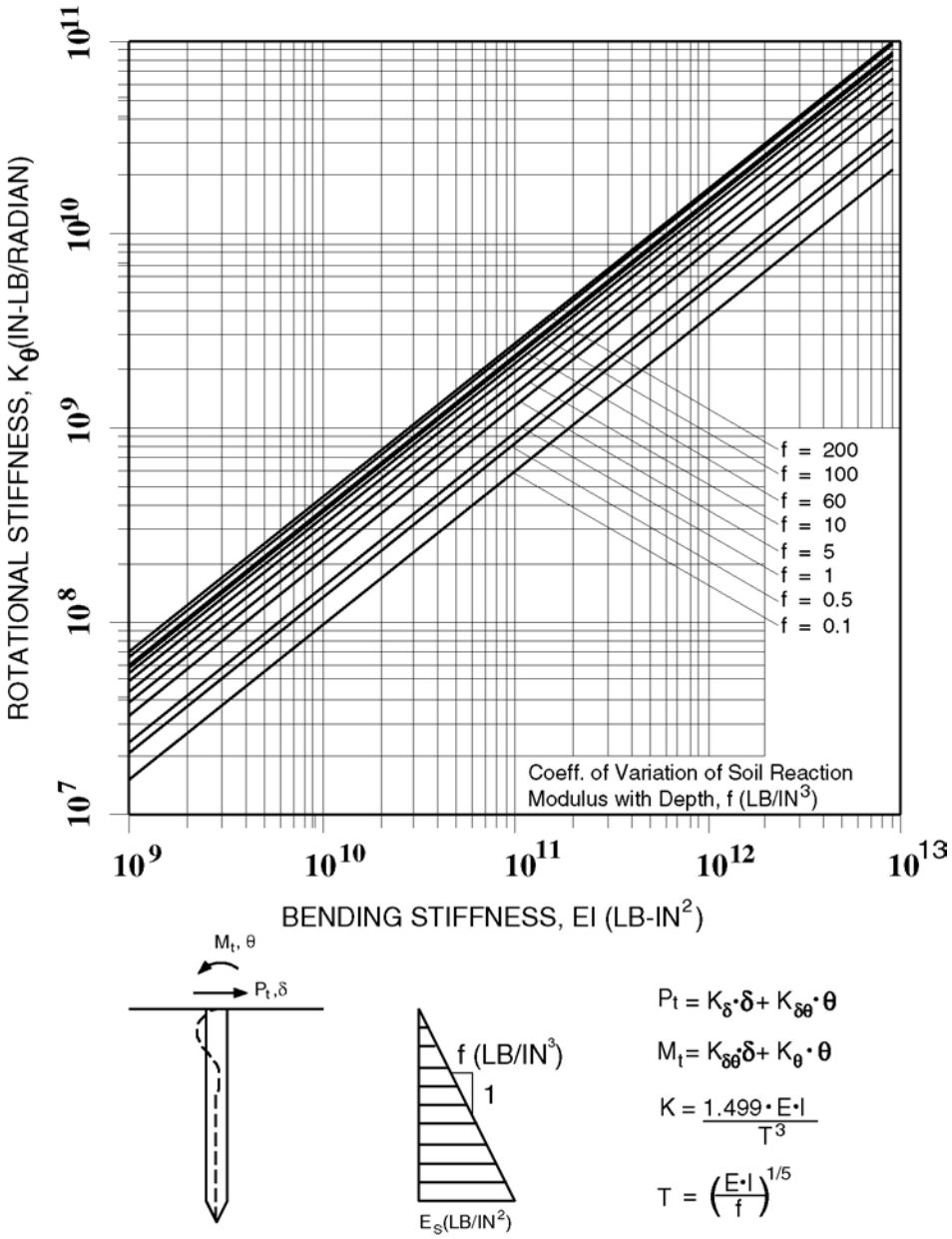


Figure 8.4.3.4-5 Coefficient for Pile Head Rotation (ATC, 1996)

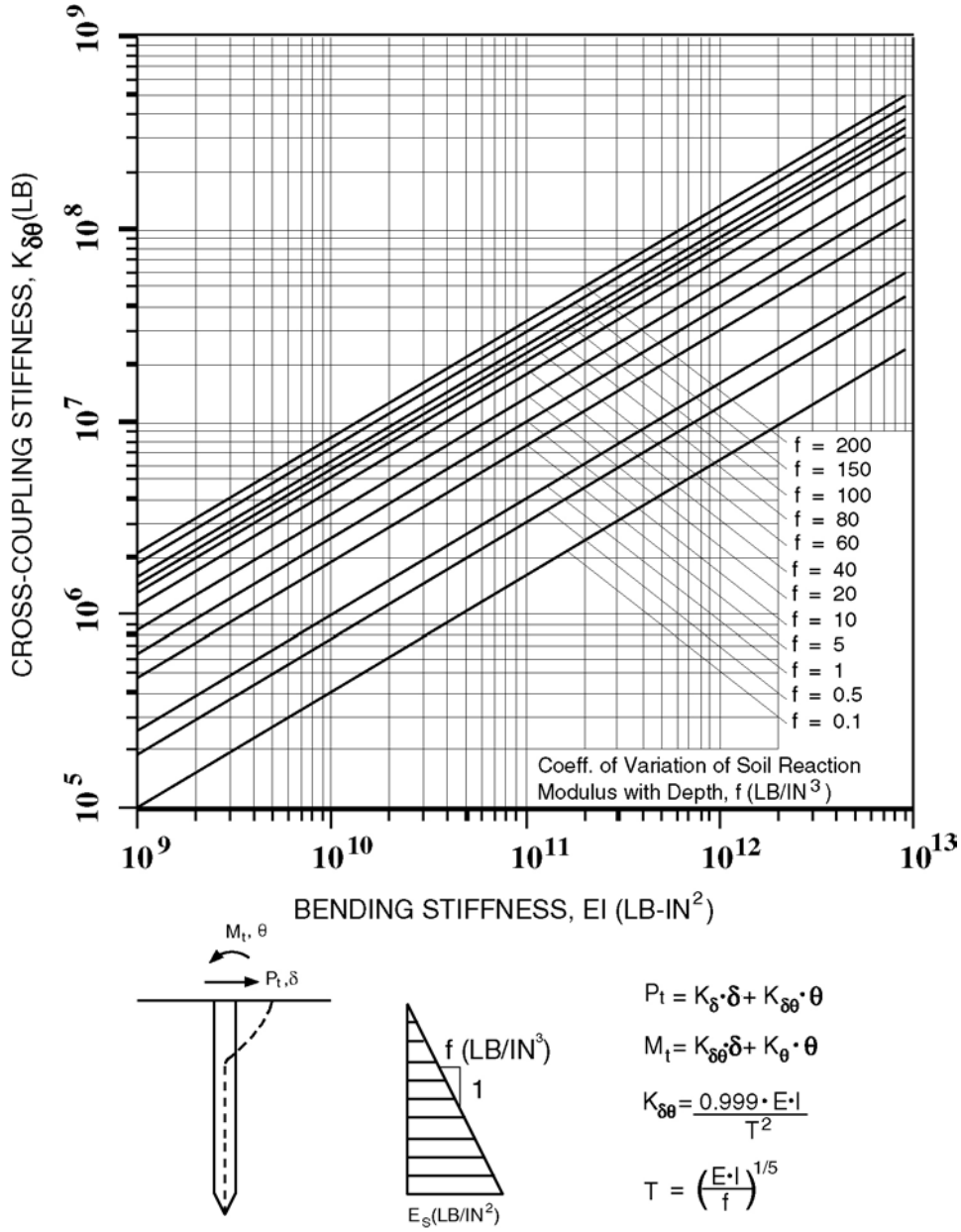


Figure 8.4.3.4-6. Coefficient for Cross-Coupling Stiffness Term (ATC, 1996)

8.4.4 Drilled Shafts

Procedures identified in Article 8.4.3, including liquefaction and dynamic settlement, generally apply with the exceptions that, (1) the ultimate capacity of single shaft foundations in compression and uplift shall not be exceeded under maximum seismic loads and (2) the flexibility of the drilled shaft shall be represented using either the estimated depth of fixity or soil springs in a lateral pile analysis.

Checks shall be conducted to confirm that minimum shaft lengths occur. The stable length can be determined by conducting nonlinear computer modeling or by using a length (L) > $\pi\lambda$ where

λ = $[EI_p/E_s]^{0.25}$ for cohesive soils

and

λ = $[EI_p/f]^{0.20}$ for cohesionless soils

and

E = Young's modulus of the shaft

I_p = moment of inertia of the shaft

f = coefficient of variation of subgrade modulus

E_s = subgrade modulus of soil = f x z

Z = embedded depth of the shaft

The nonlinear properties of the shaft shall be considered in evaluating the lateral response of the pile to lateral loads during a seismic event. Diameter adjustments shall be considered during lateral analyses of shafts with a diameter greater than 600 mm if the shaft is free to rotate, as in the case of a column extension (i.e., no pile cap). Contributions from base shear shall also be considered

8.5 ABUTMENT DESIGN REQUIREMENTS

8.5.1 General

The effect of earthquakes shall be investigated using the extreme event limit state of Table 3.2-1 with resistance factors φ = 1.0. Requirements for static design should first be met, as detailed in Articles 11.6.1 through 11.6.4 of the AASHTO LRFD provisions. Selection of abutment types prior to static design shall recognize type selection criteria for seismic conditions, as described in Articles 3.3, 3.3.1, Section 4, Table 3.3.1-1 and Figure C3.3.1-4.

8.5.1.1 Abutments and Wingwalls

The participation of abutment walls and wingwalls in the overall dynamic response of bridge systems to earthquake loading and in providing resistance to seismically induced inertial loads shall be considered in the seismic design of bridges, as outlined in these provisions. Damage to walls that is allowed to occur during earthquakes shall be consistent with the performance criteria. Abutment participation in the overall dynamic response of the bridge systems shall reflect the structural configuration, the load-transfer mechanism from the bridge to the abutment system, the effective stiffness and force capacity of the wall-soil system, and the level of expected abutment damage. The capacity of the abutments to resist the bridge inertial load shall be compatible with the structural design of the abutment wall (i.e., whether part of the wall will be damaged by the design earthquake), as well as the soil resistance that can be reliably mobilized. The lateral load capacity of walls shall be evaluated based on an applicable passive earth-pressure theory.

8.5.2 Longitudinal Direction

Under earthquake loading, the earth pressure action on abutment walls changes from a static condition to one of generally two possible conditions, depending on the magnitude of seismically induced movement of the abutment walls, the bridge superstructure, and the bridge/abutment configuration. For seat-type abutments where the expansion joint is sufficiently large to accommodate both the cyclic movement between the abutment wall and the bridge superstructure (i.e., superstructure does not push against abutment wall), the seismically induced earth pressure on the abutment wall would be the dynamic active pressure condition. However, when the gap at the expansion joint is not sufficient to accommodate the cyclic wall/bridge movements, a transfer of forces will occur from the superstructure to the abutment wall. As a result, the active earth pressure condition will not

be valid and the earth pressure approaches a passive pressure condition behind the backwall.

For stub or integral abutments, the abutment stiffness and capacity under passive pressure loading, are primary design concerns, as discussed in Articles 8.5.2.1 and 8.5.2.2. However, for partial depth or full depth seat abutment walls, earthquake-induced active earth pressures will continue to act below the backwall following separation of a knock-off backwall. These active pressures need to be considered in evaluating wall stability.

8.5.2.1 SDAP B and C

Abutments designed for service load conditions in these categories should resist earthquake loads with minimal damage with the exception of bridges in Seismic Hazard Level IV using SDAP C. For seat-type abutments, minimal abutment movement could be expected under dynamic active pressure conditions. However, bridge superstructure displacement demands could be 100 mm or more and potentially impact the abutment backwall. Where expected displacement demands are greater than a normal expansion gap of 25 to 50 mm, a knock-off backwall detail is recommended to minimize foundation damage, or alternatively, a cantilever deck slab to extend the seat gap should be provided, with a knock-off backwall tip.

In the case of integral abutments, sufficient reinforcing should be provided in the diaphragm to accommodate higher lateral pressures. For spread footing foundations, knock-off tabs or other fuse elements should be provided to minimize foundation damage. For pile-supported foundations, fuse elements should be used or connection detailing should ensure increased moment ductility in the piles.

8.5.2.2 SDAP D and E

For these design categories passive pressure resistance in soils behind integral abutment walls and knock-off walls for seat abutments will usually be mobilized due to the large longitudinal superstructure displacements associated with the inertial loads. For design purposes static passive pressures may be used without potential reductions associated with inertial loading in abutment backfill. Inclusion of abutment stiffness and capacity in bridge response analyses will reduce ductility demands on bridge columns as discussed in Article C3.3.

Case 1: To ensure that the columns are always able to resist the lateral loads, designers may choose to assume zero stiffness and capacity of abutments. In this case designers should check abutment damage potential and performance due to abutment displacement demand. Knock-off backwall details for seat abutments should be utilized to protect abutment foundations and increased reinforcing used in diaphragms or integral abutments to accommodate passive pressures.

Case 2: Where abutment stiffness and capacity is included in the design, it should be recognized that the passive pressure zone mobilized by abutment displacement extends beyond the active pressure zone normally adapted for static service load design, as illustrated schematically in Figure 8.5.2.2-1. Whether presumptive or computed passive pressures are used for design as described in the commentary paragraphs, backfill in this zone should be controlled by specifications unless the passive pressure that is used in less than 70% of the presumptive value.

Abutment stiffness and passive pressure capacity for either (1) SDAP D or (2) SDAP E two-step analysis methods should be characterized by a bi-linear relationship as shown in Figure 8.5.2.2-2. For seat type abutments, knock-off backwall details should be utilized with superstructure diaphragms designed to accommodate passive pressures, as illustrated in Figure C3.3.1-4. For integral abutments the end diaphragm should be designed for passive pressures, and utilize a stub pile footing or normal footing for support, with a sliding seat. Passive pressures may be assumed uniformly distributed over the height (H) of the backwall or diaphragm. Thus the total passive force is:

$$P_p = p_p * H \qquad (8.5.2.2\text{-}1)$$

where:

H = wall height in meters

p_p = passive pressure behind backwall

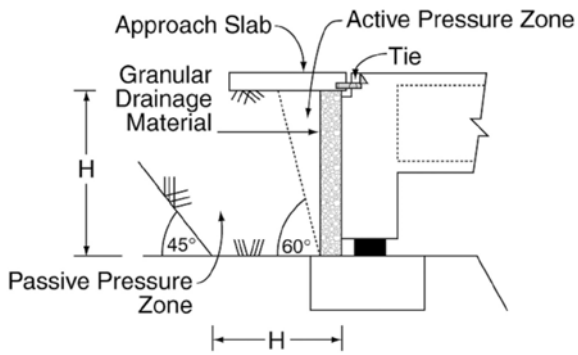


Figure 8.5.2.2-1 Design Passive Pressure Zone

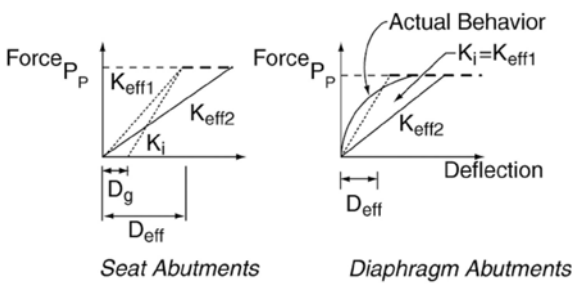


Figure 8.5.2.2-2 Characterization of Abutment Capacity and Stiffness

Calculation of Best-Estimate Passive Force P_p

If the strength characteristics of compacted or natural soils in the "passive pressure zone" (total stress strength parameters c and) are known, then the passive force for a given height H may be computed using accepted analysis procedures. These procedures should account for the interface friction between the wall and the soil. The properties used shall be those indicative of the entire “passive pressure zone” as indicated in Figure 8.5.2.2-1. Therefore the properties of backfill that is only placed adjacent to the wall in the active pressure zone may not be appropriate.

If presumptive passive pressures are to be used for design, then the following criteria should apply:

(1) Soil in the "passive pressure zone" should be compacted to a dry density greater than 95 percent of the maximum per ASTM Standard Method D1557 or equivalent.

(2) For cohesionless, non-plastic backfill (fines content less than 30 percent), the passive pressure p_p may be assumed equal to H/10 MPa per meter of length of wall (2H/3 ksf per foot length of wall).

(3) For cohesive backfill (clay fraction > 15 percent), the passive pressure p_p may be assumed equal to 0.25 MPa (5 ksf) provided the estimated unconfined compressive strength is greater than 0.20 MPa (4 ksf).

The presumptive values given above apply for use in the " *Permissible with Owner’s Approval*" category, as defined in Article 3.3.1. If the design is based upon presumptive resistances that are no larger than 70 percent of the values listed above, then the structure may be classified in the "*Permissible"* category.

In all cases granular drainage material must be placed behind the abutment wall to ensure adequate mobilization of wall friction.

Calculation of Stiffness

For SDAP D one-step analyses and for the demand calculation of SDAP E analyses, an equivalent linear secant stiffness, K_{effn}, is required for analyses. For integral or diaphragm abutments, an initial secant stiffness (Figure 8.5.2.2-2) may be calculated as follows:

$$K_{effI} = P_p/0.02H \qquad (8.5.2.2\text{-}2)$$

If computed abutment forces exceed the capacity, the stiffness should be softened iteratively (K_{eff2} to K_{effn}) until abutment displacements are consistent (within 30 percent) with the assumed stiffness. For seat abutments the expansion gap should be included in the initial estimate of the secant stiffness. Thus:

$$K_{effI} = P_p/(0.02H + D_g) \qquad (8.5.2.2\text{-}3)$$

where:

D_g = gap width

For SDAP E two-step analyses, where push-over analyses are conducted, values of P_p and the initial estimate of K_{eff1} should be used to define a

bilinear load-displacement behavior of the abutment for the capacity assessment.

For partial depth or full-depth seat abutment walls, where knock-off backwalls are activated, the remaining lower wall design and stability check under the action of continuing earthquake-induced active earth pressures should be evaluated. For a no-collapse performance criteria, and assuming conventional cantilever retaining wall construction, horizontal wall translation under dynamic active pressure loading is acceptable. However, rotational instability may lead to collapse and thus must be prevented.

The design approach is similar to that of a free-standing retaining wall, except that lateral force from the bridge superstructure needs to be included in equilibrium evaluations, as the superstructure moves outwards from the wall. Earthquake-induced active earth pressures should be computed using horizontal accelerations at least equal to 50 percent of the site peak ground acceleration (i.e., $F_a S_s$ / 5.0). Using less than the expected site acceleration implies that limited sliding of the wall may occur during the earthquake. A limiting equilibrium condition should be checked in the horizontal direction. To ensure safety against potential overturning about the toe, a restoring moment of at least 50 percent more than the driving overturning moment should exist. If necessary, wall design (initially based on a static loading condition) should be modified to meet the above condition.

8.5.3 Transverse Direction

In general, abutments shall be designed to resist earthquake forces in the transverse direction elastically for the 50% PE in 75-year earthquake. For the 3% PE in 75-year/1.5 mean deterministic event, the abutment may either be designed to resist transverse forces elastically or a fuse shall be provided to limit the transverse force transfer at the abutment. If a fuse is used, then the effects of internal force redistribution resulting from fusing shall be taken into account in the design of the bridge. Limitations on the use of fusing for the various Seismic Design and Analysis Procedures are listed below.

In the context of these provisions, elastic resistance includes the use of elastomeric, sliding, or isolation bearings designed to accommodate the design displacements, soil frictional resistance acting against the base of a spread footing-supported abutment, pile resistance provided by piles acting in their elastic range, or passive resistance of soil acting at displacements less that 2 percent of the wall height.

Likewise, fusing includes: breakaway elements, such as isolation bearings with a relatively high yield force; shear keys; yielding elements, such as wingwalls yielding at their junction with the abutment backwall; elastomeric bearings whose connections have failed and upon which the superstructure is sliding; spread footings that are proportioned to slide in the rare earthquake; or piles that develop a complete plastic mechanism. Article 3.3.1 outlines those mechanisms that are permissible with the Owner's approval.

The stiffness of abutments under transverse loading may be calculated based on the procedures given in Article 8.4 for foundation stiffnesses. Where fusing elements are used, allowance shall be made for the reduced stiffness of the abutment after fusing occurs.

8.5.3.1 SDAP B and C

Connection design forces also apply to shear restraint elements such as shear keys.

8.5.3.2 SDAP D and E

For structures in these categories, either elastic resistance or fusing shall be used to accommodate transverse abutment loading. The elastic forces used for transverse abutment design shall be determined from an elastic demand analysis of the structure.

For short, continuous superstructure bridges (Length/Width < 4) with low skew angles (<20 degrees), low plan curvature (subtended angle < 30 degrees), and which also are designed for sustained soil mobilization in the transverse direction, the elastic forces and displacements for the transverse earthquake design may be reduced by 1.4 to account for increased damping provided by the soil at the abutments. Herein transverse earthquake is defined as acting perpendicular to a chord extending between the two abutments. Sustained soil mobilization requires resistance to be present throughout the range of cyclic motion.

Where combined mechanisms provide resistance, at least 50 percent of the total resistance must be provided by a sustained mechanism for the system to qualify for the 1.4 reduction.

The design of concrete shear keys should consider the unequal forces that may develop in a skewed abutment, particularly if the intermediate piers are also skewed. (This effect is amplified if intermediate piers also have unequal stiffness, such as wall piers.) The shear key design should also consider unequal loading if multiple shear keys are used. The use of recessed or hidden shear keys should be avoided if possible, since these are difficult to inspect and repair.

8.6 LIQUEFACTION DESIGN REQUIREMENTS

8.6.1 General

An evaluation of the potential for and consequences of liquefaction within near-surface soil shall be made in accordance with the following requirements: A liquefaction assessment is required unless one of the following conditions is met or as directed otherwise by the Owner.

- Mean magnitude for the 3% PE in 75-year event is less than 6.0 (Figures 8.6.1-1 to 8.6.1-4);
- Mean magnitude of the 3% PE in 75-year event is less than 6.4 and equal to or greater than 6.0, and the normalized Standard Penetration Test (SPT) blow count $[(N_1)_{60}]$ is greater than 20;
- Mean magnitude for the 3% PE in 75-year event is less than 6.4 and equal to or greater than 6.0, $(N_1)_{60}$ is greater than 15, and F_aS_s is between 0.25 and 0.375; or

If the mean magnitude shown in Figures 8.6.1-1 to 8.6.1-4 is greater than or equal to 6.4, or if the above requirements are not met for magnitudes between 6.0 and 6.4 or if for the 50% PE in 75 year event, F_aS_s is greater than 0.375, evaluations of liquefaction and associated phenomena such as lateral flow, lateral spreading, and dynamic settlement shall be evaluated in accordance with these Specifications.

8.6.2 Evaluation of Liquefaction Potential

Procedures given in Appendix D shall be used to evaluate the potential for liquefaction.

8.6.3 Evaluation of the Effects of Liquefaction and Lateral Ground Movement

Procedures given in Appendix D shall be used to evaluate the potential for and effects of liquefaction and liquefaction-related permanent ground movement (i.e., lateral spreading, lateral flow, and dynamic settlement). If both liquefaction and ground movement occur, they shall be treated as separate and independent load cases, unless agreed to or directed otherwise by the Owner.

8.6.4 Design Requirements if Liquefaction and Ground Movement Occurs

If it is determined from Appendix D that liquefaction can occur at a bridge site, then one or more of the following approaches shall be implemented in the design.

Bridges shall be supported on deep foundations unless (1) the footing is located below the liquefiable layer, (2) special design studies are conducted to demonstrate that the footing will tolerate liquefaction, or (3) the ground is improved so that liquefaction does not occur. If spread footings are being considered for use at a liquefiable site, Owner approval shall be obtained before beginning the design process.

If liquefaction occurs, then the bridge shall be designed and analyzed in two configurations as follows:

1. Nonliquefied Configuration: The structure shall be analyzed and designed, assuming no liquefaction occurs using the ground response spectrum appropriate for the site soil conditions.
2. Liquefied Configuration: The structure as designed in Nonliquefied Configuration above shall be reanalyzed and redesigned, if necessary, assuming that the layer has liquefied and the liquefied soil provides whatever residual resistance is appropriate (i.e., “p-y curves” or modulus of subgrade reaction values for lateral pile response analyses consistent with liquefied soil

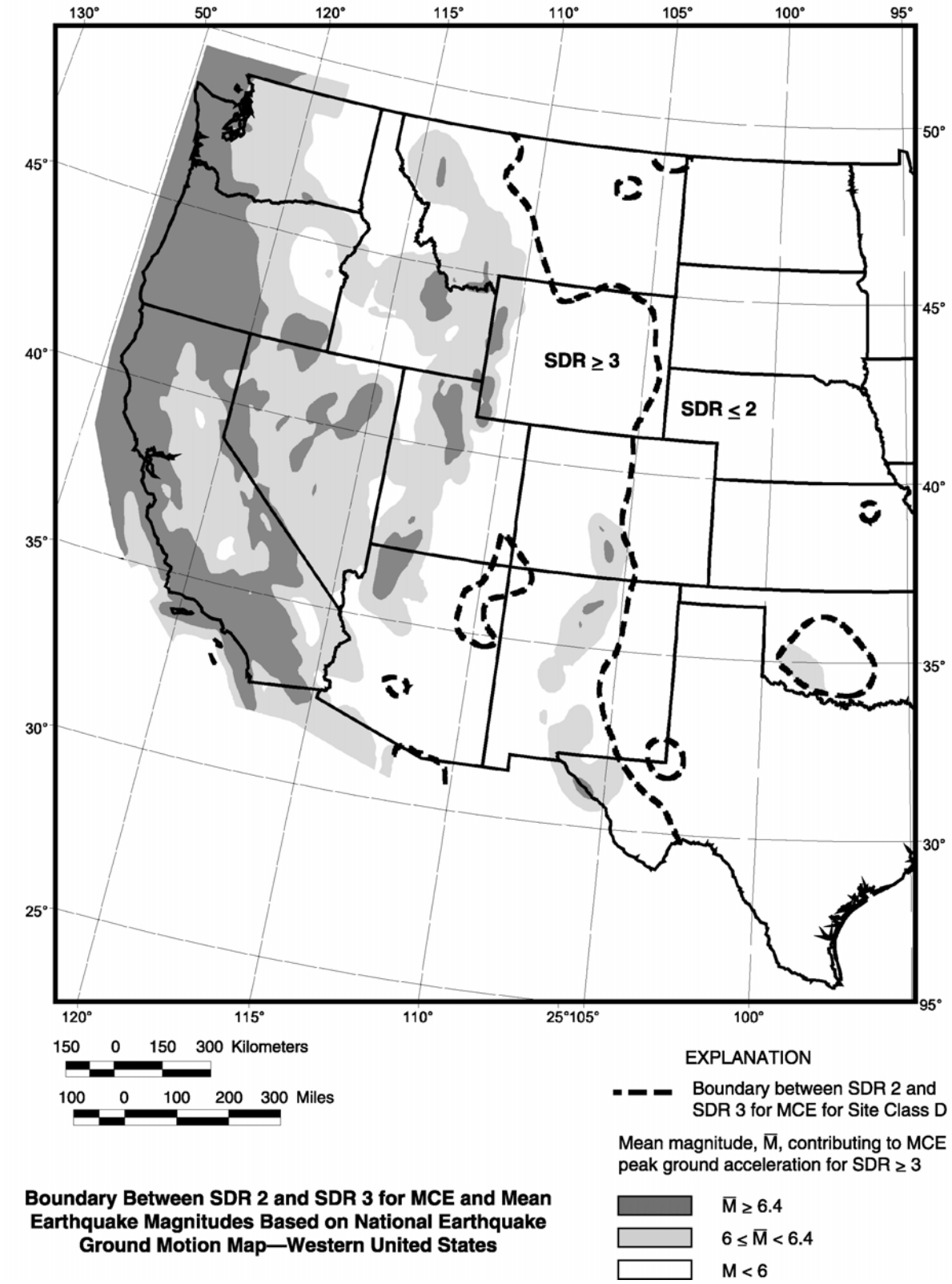


Figure 8.6.1-1 Mean Earthquake Magnitude Map for Western United States

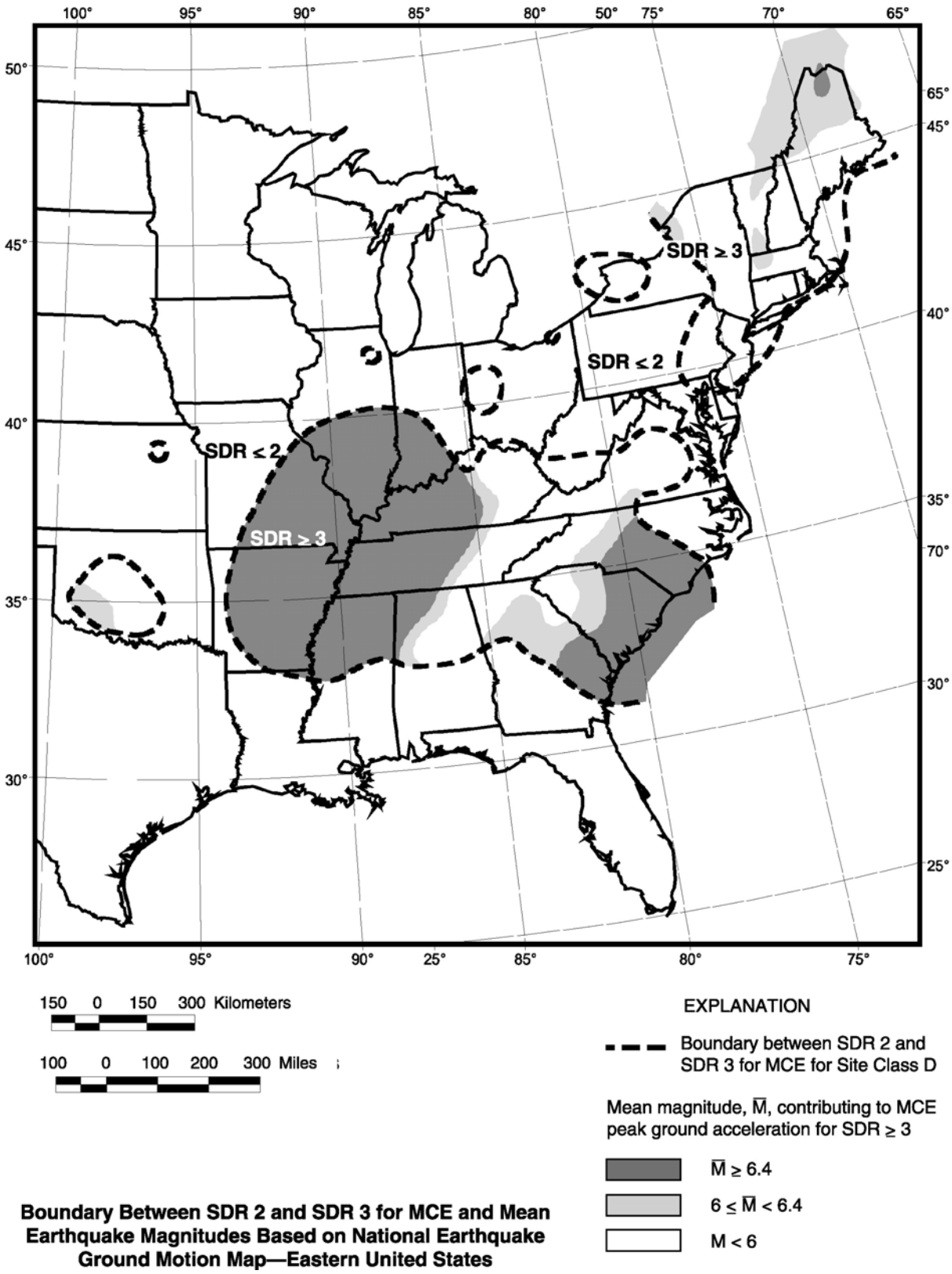


Figure 8.6.1-2 Mean Earthquake Magnitude Map for Eastern United States

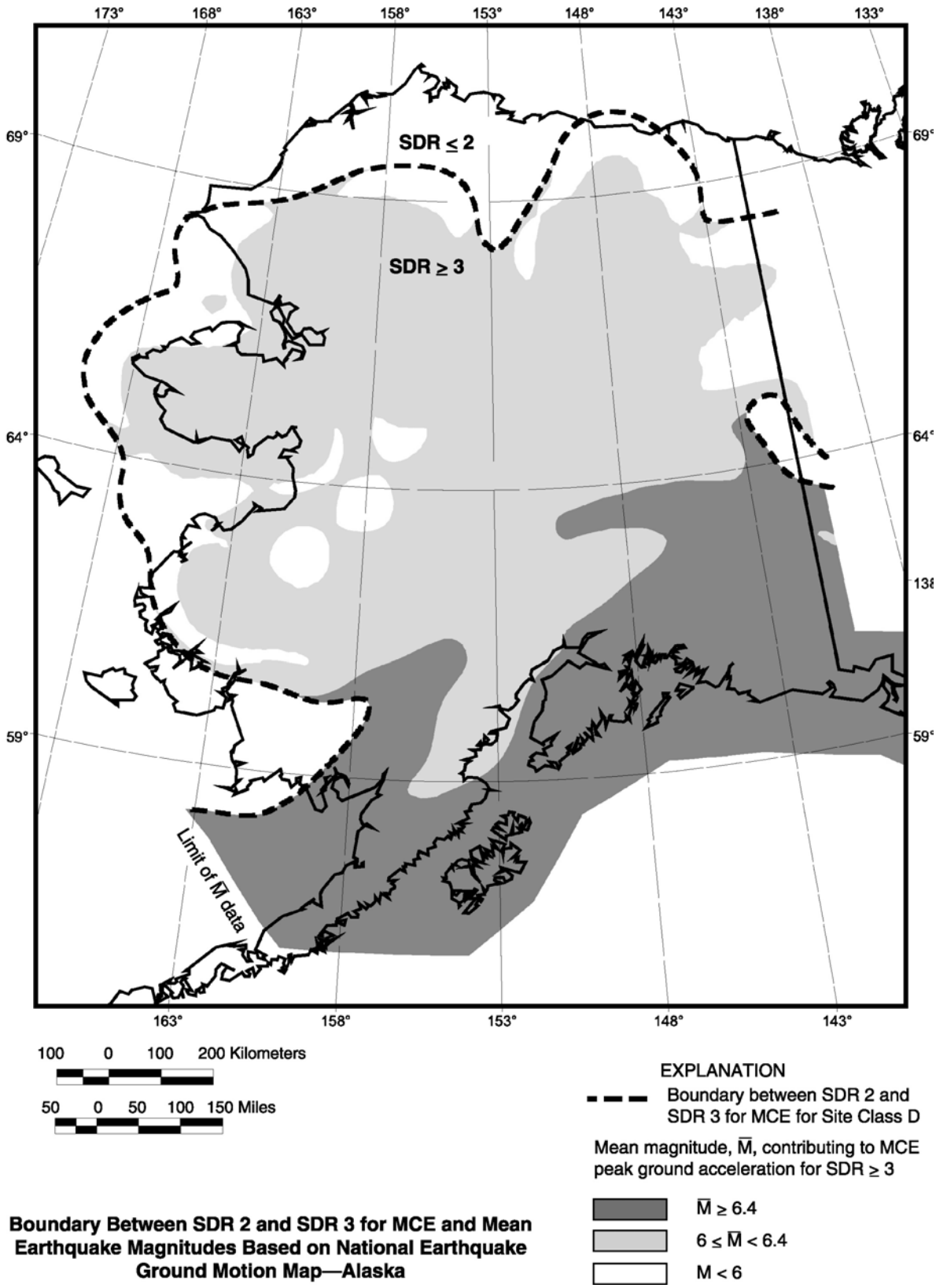
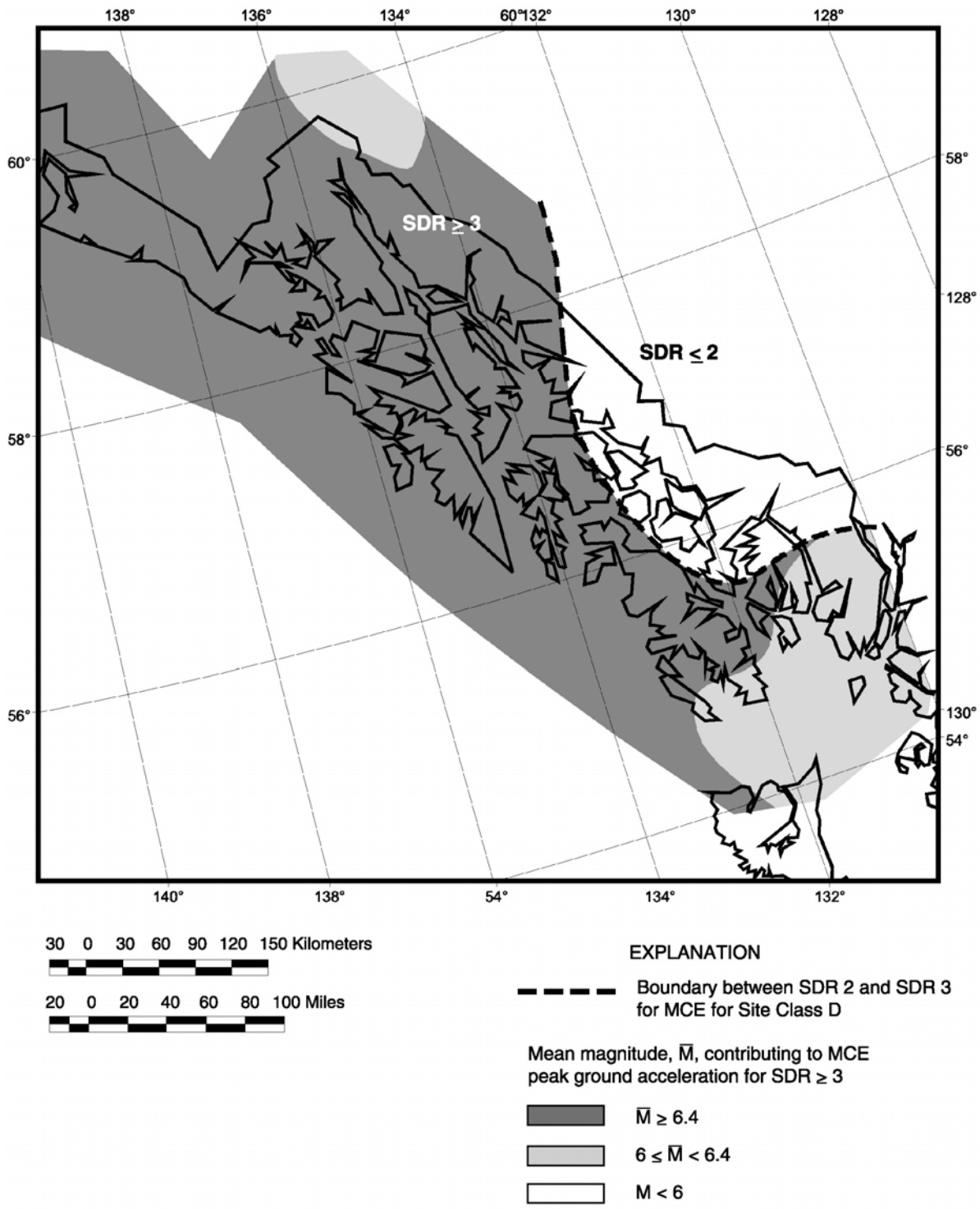


Figure 8.6.1-3 Mean Earthquake Magnitude Map for Alaska



Boundary Between SDR 2 and SDR 3 for MCE and Mean Earthquake Magnitudes Based on National Earthquake Ground Motion Map—Southeast Alaska

Figure 8.6.1-4 Mean Earthquake Magnitude Map for Southeast Alaska

conditions). The design spectra shall be the same as that used in Nonliquefied Configuration unless a site-specific response spectra has been developed using nonlinear, effective stress methods (e.g., computer program DESRA or equivalent) that properly account for the buildup in pore-water pressure and stiffness degradation in liquefiable layers. The reduced response spectra resulting from the site-specific nonlinear, effective stress analyses shall not be less than 2/3's of that used in Nonliquefied Configuration. The Designer shall provide a drawing of the load path and energy dissipation mechanisms in this condition as required by Article 3.3 since it is likely that plastic hinges will occur in different locations than for the non-liquefied case. Shear reinforcement given in Article 8.8.2.3 shall be used in all concrete and prestressed concrete piles to a depth of 3 pile diameters below the liquefied layer.

If lateral flow or lateral spreading occurs, the following options shall be considered.

1. Design the piles to resist the forces generated by the lateral spreading.
2. If the structure cannot be designed to resist the forces, assess whether the structure is able to tolerate the anticipated movements and meet the geometric and structural constraints of Table C3.2-1. The maximum plastic rotation of the piles is 0.05 radians as per Article 8.7.9 and 8.8.6.
3. If the structure cannot meet the performance requirements of Table 3.2-1, assess the costs and benefits of various mitigation measures to minimize the movements to a tolerable level to meet the desired performance objective. If a higher performance is desired so that the piles will not have to be replaced, the allowable plastic rotations in-ground hinges of Article 8.7.9.2 and 8.8.6.2 shall be met.

8.6.5 Detailed Foundation Design Requirements

Article 8.4 contains detailed design requirements for each of the different foundation types.

8.6.6 Other Collateral Hazards

The potential occurrence of collateral hazards resulting from fault rupture, landsliding, differential ground compaction, and flooding and inundation shall be evaluated. Procedures for making these evaluations are summarized in Appendix D.

8.7 STRUCTURAL STEEL DESIGN REQUIREMENTS

8.7.1 General

The provisions of this article shall apply only to a limited number of specially detailed steel components designed to dissipate hysteretic energy during earthquakes. This article does not apply to steel members that are designed to remain elastic during earthquakes.

For the few specially designed steel members that are within the scope of this article, the other requirements of Section 6 of the LRFD provisions are also applicable (unless superseded by more stringent requirements in this article).

Continuous and clear load path or load paths shall be assured. Proper load transfer shall be considered in designing foundations, substructures, superstructures and connections.

Welds shall be designed as capacity protected elements. Partial penetration groove welds shall not be used in ductile substructures.

Abrupt changes in cross sections of members in ductile substructures are not permitted within the plastic hinge zones defined in Article 4.9 unless demonstrated acceptable by analysis and supported by research results.

8.7.2 Materials

Ductile substructure elements and ductile end-diaphragms, as defined in Articles 8.7.4 through 8.7.8, shall be made of either:

(a) M270 (ASTM 709M) Grade 345 and Grade 345W steels

(b) ASTM A992 steel, or

(c) A500 Grade B or A501 steels (if structural tubing or pipe).

Other steels may be used provided that they are comparable to the approved Grade 345 steels.

Nominal resistance is defined as the resistance of a member, connection or structure based on the expected yield strength (F_{ye}), other specified material properties, and the nominal dimensions and details of the final section(s) chosen, calculated with all material resistance factors taken as 1.0.

Overstrength capacity is defined as the resistance of a member, connection or structure based on the nominal dimensions and details of the final section(s) chosen, calculated accounting for the expected development of large strains and associated stresses larger than the minimum specified yield values.

The expected yield strength shall be used in the calculation of nominal resistances, where expected yield strength is defined as $F_{ye} = R_y F_y$ where R_y shall be taken as 1.1 for the permitted steels listed above.

Welding requirements shall be compatible with AWS/AASHTO D1.5-96 Structural Bridge Welding Code. However, under-matched welds are not permitted for special seismic hysteretic energy dissipating systems (such as ductile substructures and ductile diaphragms).

Steel members expected to undergo significant plastic deformations during a seismic event shall meet the toughness requirements of A709/A709M Supplementary Requirement S84 (Fracture Critical). Welds metal connecting these members shall meet the toughness requirements specified in the AWS D1.5 Bridge Specification for Zone III.

8.7.3 Sway Stability Effects

The sway effects produced by the vertical loads acting on the structure in its displaced configuration shall be determined from a second-order analysis. Alternatively, recognized approximate methods for P-Δ analysis, or the provisions in Article 8.3.4, can be used.

8.7.4 Ductile Moment Resisting Frames and Single Column Structures

This article applies to ductile moment-resisting frames and bents, constructed with I-shape beams and columns connected with their webs in a common plane. Except as noted in Article 8.7.4-1, columns shall be designed as ductile structural elements, while the beams, the panel zone at column-beam intersections and the connections shall be designed as Capacity Protected Elements.

8.7.4.1 Columns

Width-to-thickness ratios of compression elements of columns shall be in compliance with Table 8.7.4-2. Full penetration flange and web welds are required at column-to-beam (or beam-to-column) connections.

The resistance of columns to combined axial load and flexure shall be determined in accordance with Article 6.9.2.2 of the AASHTO LRFD provisions. The factored axial compression due to seismic load and permanent loads shall not exceed $0.20A_gF_y$.

The shear resistance of the column web shall be determined in accordance with Article 6.10.7 of the AASHTO LRFD provisions.

The potential plastic hinge zones (Article 4.9), near the top and base of each column, shall be laterally supported and the unsupported distance from these locations shall not exceed $17250r_y/F_y$. These lateral supports shall be provided either directly to the flanges or indirectly through a column web stiffener or a continuity plate. Each column flange lateral support shall resist a force of not less than 2% of the nominal column flange strength (btF_y) at the support location. The possibility of complete load reversal shall be considered.

When no lateral support can be provided, the column maximum slenderness shall not exceed 60 and transverse moments produced by the forces otherwise resisted by the lateral bracing (including the second order moment due to the resulting column displacement) shall be included in the seismic load combinations.

Splices that incorporate partial joint penetration groove welds shall be located away from the plastic hinge zones as defined in Article 4.9 at a minimum distance equal to the greater of:

(a) one-fourth the clear height of column;
(b) twice the column depth; and
(c) one meter (39 inches).

8.7.4.2 Beams

The factored resistance of the beams shall be determined in accordance with Article 6.10.2 of the LRFD provisions. At a joint between beams

Table 8.7.4-1 Limiting Width-to-Thickness Ratios

Description of element	Width-to-thickness ratio (b/t)[1]	Limiting width-to-thickness ratio λ_p[2]	Limiting width-to-thickness ratio k[3]
Flanges of I-shaped sections and channels in compression	$\frac{b_f}{2t_f}$	$\frac{135}{\sqrt{F_y}}$	0.30
Webs in combined flexural and axial compression	$\frac{h_c}{t_w}$	**For** $\frac{P_u}{\Phi_b P_y} \le 0.125$ $\frac{1365}{\sqrt{F_y}}\left(1-\frac{1.54P_u}{\Phi_b P_y}\right)$ **For** $\frac{P_u}{\Phi_b P_y} > 0.125$ $\frac{500}{\sqrt{F_y}}\left(2.33-\frac{P_u}{\Phi_b P_y}\right) \ge \frac{665}{\sqrt{F_y}}$	**For** $\frac{P_u}{\Phi_b P_y} \le 0.125$ $3.05\left(1-\frac{1.54P_u}{\Phi_b P_y}\right)$ **For** $\frac{P_u}{\Phi_b P_y} > 0.125$ $1.12\left(2.33-\frac{P_u}{\Phi_b P_y}\right) \ge 1.48$
Hollow circular sections (pipes)	$\frac{D}{t}$	$\frac{8950}{F_y}$	$\frac{200}{\sqrt{F_y}}$
Unstiffened rectangular tubes	$\frac{b}{t}$	$\frac{300}{\sqrt{F_y}}$	0.67
Legs of angles	$\frac{b}{t}$	$\frac{145}{\sqrt{F_y}}$	0.32

1. Width-to-thickness ratios of compression elements – Note that these are more stringent for members designed to dissipate hysteretic energy during earthquake than for other members (Article 6.9.4.2).

2. Limits expressed in format to satisfy the requirement $\frac{b}{t} \le \lambda_p$

3. Limits expressed in format to satisfy the requirement $\frac{b}{t} \le k\sqrt{\frac{E}{F_y}}$

4. Note: In the above, b_f and t_f are respectively the width and thickness of an I-shaped section, h_c is the depth of that section and t_w is the thickness of its web.

and columns the sum of the factored resistances of the beams shall not be less than the sum of the probable resistances of the column(s) framing into the joint. The probable flexural resistance of columns shall be taken as the product of the overstrength factor (defined in Article 4.8) times the columns nominal flexural resistance determined either in accordance to Article 6.9.2.2 of the AASHTO LRFD provisions, or by

$$M_{nx} \quad 1.18 M_{px}\left[1-\frac{P_u}{AF_{ye}}\right] \le M_{px} \qquad (8.7.4.1\text{-}1)$$

unless demonstrated otherwise by rational analysis, and where M_{px} is the column plastic moment under pure bending calculated using F_{ye}.

8.7.4.3 Panel Zones and Connections

Column-beam intersection panel zones, moment resisting connections and column base connections shall be designed as Capacity Protected Elements.

Panel zones shall be designed such that the vertical shearing resistance is determined in accordance with Article 6.10.7.2 of the AASHTO LRFD provisions.

Beam-to-column connections shall have resistance not less than the resistance of the beam stipulated in Article 8.7.4.2.

Continuity plates shall be provided on both sides of the panel zone web and shall finish with total width of at least 0.8 times the flange width of the opposing flanges. Their b/t shall meet the limits for projecting elements of Article 6.9.4.2 of the AASHTO LRFD provisions. These continuity plates shall be proportioned to meet the stiffener requirements stipulated in Article 6.10.8.2 of the AASHTO LRFD provisions and shall be connected to both flanges and the web.

Flanges and connection plates in bolted connections shall have a factored net section ultimate resistance calculated by Equation 6.8.2.1-2, at least equal to the factored gross area yield resistance given by Equation 6.8.2.1-1, with A_g and A_n in Article 6.8.2.1 taken here as the area of the flanges and connection plates in tension. These referenced equations and article are from the AASHTO LRFD provisions.

8.7.4.4 Multi-tier Frame Bents

For multi-tier frame bents, capacity design principles as well as the requirements of Article 8.7.4.1 may be modified by the engineer to achieve column plastic hinging only at the top and base of the column, and plastic hinging at the ends of all intermediate beams. Column plastic hinging shall not be forced at all joints at every tier.

8.7.5 Ductile Concentrically Braced Frames

Braces are the Ductile Substructure Elements in ductile concentrically braced frames.

8.7.5.1 Bracing Systems

Diagonal braces shall be oriented such that a nearly identical ultimate strength is achieved in both sway directions, when considering only the strength contribution of braces in tension. To achieve this, it is required that, at any level in any planar frame, the sum of the horizontal components of the strength of the braces in tension when the frame sway in one direction, shall be within 30% of the same value for sway in the other direction.

Article 8.7.5 is only applicable to braced frames for which all braces' action lines meet at beam-to-column intersection points (such as X-braces).

8.7.5.2 Design Requirements for Ductile Bracing Members

Bracing members shall have a slenderness ratio, KL/r, less than $2600/\sqrt{F_y}$ or Article 6.9.3 of the AASHTO LRFD Provisions.

The width-to-thickness ratios of bracing members should be limited as indicated in Table 8.7.4-1. For back-to-back legs of double angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio shall not exceed $200/\sqrt{F_y}$ rather than the limit of Table 8.7.4-1

In built-up bracing members, the slenderness ratio of the individual parts between stitches shall be not greater than 0.4 times the slenderness ratio of the member as a whole. When it can be shown that braces will buckle without causing shear in the stitches, the spacing of the stitches shall be

such that the slenderness ratio of the individual parts does not exceed 0.75 times the slenderness ratio of the built-up member.

8.7.5.3 Brace Connections

The controlling overstrength capacity shall be taken as the axial tensile yield strength of the brace (A_gF_{ye}). Brace connections shall be designed as Capacity Protected Elements.

Connections must be designed to ensure that the bracing member is capable of yielding the gross section. Consequently, brace strength calculated based on tension rupture on the effective net section and block shear rupture, shall be greater that the design tensile strength of brace given by gross section yielding.

Eccentricities in bracing connections shall be minimized.

Brace connections including gusset plates shall be detailed to avoid brittle failures due to rotation of the brace when it buckles. This ductile rotational behavior shall be allowed for, either in the plane of the frame or out of it, depending on the slenderness ratios.

The design of gusset plates shall also include consideration of buckling.

Stitches that connect the separate elements of built-up bracing members shall, if the overall buckling mode induces shear in the stitches, have a strength at least equal to the design tensile strength of each element. The spacing of stitches shall be uniform and not less than two stitches shall be used. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

8.7.5.4 Columns, Beams and Other Connections

Columns, beams, beam-to-column connections and column splices that participate in the lateral-load-resisting system shall be designed as Capacity Protected Elements with the following additional requirements:

(a) Columns, beams and connections shall resist forces arising from load redistribution following brace buckling or yielding. The brace compressive resistance shall be taken as 0.3 $\phi_c P_n$ if this creates a more critical condition.

(b) Column splices made with partial penetration groove welds and subject to net tension forces due to overturning effects shall have Factored Resistances not less than 50% of the flange yield load of the smaller member at the splice.

8.7.6 Concentrically Braced Frames with Nominal Ductility

Braces are the Ductile Substructure Elements in nominally ductile concentrically braced frames.

8.7.6.1 Bracing Systems

Diagonal braces shall be oriented such that a nearly identical ultimate strength is achieved in both sway directions, when considering only the strength contribution of braces in tension. To achieve this, it is required that, at any level in any planar frame, the sum of the horizontal components of the strength of the braces in tension when the frame sway in one direction, shall be within 30% of the same value for sway in the other direction.

The categories of bracing systems permitted by this Article includes:

(a) tension-only diagonal bracing,
(b) chevron bracing (or V-bracing) and,
(c) direct tension-compression diagonal bracing systems of the geometry permitted in Article 8.7.5.1, but that do not satisfy all the requirements for ductile concentrically braced frames.

Tension-only bracing systems in which braces are connected at beam-to-column intersections are permitted in bents for which every column is fully continuous over the entire bent height, and where no more than 4 vertical levels of bracing are used along the bent height.

8.7.6.2 Design Requirements for Nominally Ductile Bracing Members

Bracing members shall have a slenderness ratio, KL/r, less than $3750/\sqrt{F_y}$ or Article 6.9.3 of the AASHTO LRFD provisions. This limit is waived for members designed as tension-only bracing.

In built-up bracing members, the slenderness ratio of the individual parts shall be not greater than 0.5 times the slenderness ratio of the member as a whole.

For bracing members having KL/r less than $2600/\sqrt{F_y}$ or Article 6.9.3 of the AASHTO LRFD Provisions, the width-to-thickness ratios of bracing members should be limited as indicated in Table 8.7.4-1. For bracing members that exceed that value, the width-to-thickness ratio limits can be obtained by linear interpolation between the values in Table 8.7.4-1 when KL/r is equal to $2600/\sqrt{F_y}$. and 1.3 times the values in Table 8.7.4-1 when KL/r is equal to $3750/\sqrt{F_y}$. .

For back-to-back legs of double angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio limit can be taken as $200/\sqrt{F_y}$.

No width-to-thickness ratio limit is imposed for braces designed as tension-only members and having KL/r greater than $3750/\sqrt{F_y}$.

8.7.6.3 Brace Connections

Brace connections shall be designed as Capacity Protected Elements. The controlling overstrength capacity shall be taken as the axial tensile yield strength of the brace (A_gF_{ye}).

For tension-only bracing the controlling probable resistance shall be multiplied by an additional factor of 1.10.

Connections must be designed to ensure that the bracing member is capable of yielding the gross section. Consequently, brace strength calculated based on tension rupture on the effective net section and block shear rupture, shall be less that the design tensile strength of brace given by gross section yielding.

Stitches that connect the separate elements of built-up bracing members shall, if the overall buckling mode induces shear in the stitches, have a strength at least equal to one-half of the design tensile strength of each element. The spacing of stitches shall be uniform and not less than two stitches shall be used. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

8.7.6.4 Columns, Beams and Other Connections

Columns, beams, and connections shall be designed as Capacity Protected Elements.

8.7.6.5 Chevron Braced and V-Braced Systems

Braces in chevron braced frames shall conform to the requirements of Article 8.7.6.2, except that bracing members shall have a slenderness ratio, KL/r, less than $2600/\sqrt{F_y}$. Tension-only designs are not permitted.

The beam attached to chevron braces or V-braces shall be continuous between columns and its top and bottom flanges shall be designed to resist a lateral load of 2% of the flange yield force ($F_yb_ft_{bf}$) at the point of intersection with the brace.

Columns, beams and connections shall be designed to resist forces arising from load redistribution following brace buckling or yielding, including the maximum unbalanced vertical load effect applied to the beam by the braces. The brace compressive resistance shall be $0.3\ \phi_cP_n$ if this creates a more critical condition.

A beam that is intersected by chevron braces shall be able to support its permanent dead and live loads without the support provided by the braces.

8.7.7 Concrete Filled Steel Pipes

Concrete-filled steel pipes use as columns, piers, or piles expected to develop full plastic hinging of the composite section as a result of seismic response shall be designed in accordance with Articles 6.9.2.2, 6.9.5, 6.12.3.2.2, of the AASHTO LRFD provisions as well as the requirements in this article.

8.7.7.1 Combined Axial Compression and Flexure

Concrete-filled steel pipe members required to resist both axial compression and flexure and intended to be ductile substructure elements shall be proportioned so that:

$$\frac{P_u}{P_r} + \frac{BM_u}{M_{rc}} \le 1.0 \qquad (8.7.7.1\text{-}1)$$

and

$$\frac{M_u}{M_{rc}} \le 1.0 \qquad (8.7.7.1\text{-}2)$$

where P_r is defined in Articles 6.9.2.1 and 6.9.5.1 of the AASHTO LRFD provisions, and M_{rc} is defined in Article 8.7.7.2

$$B = \frac{P_{ro} - P_{rc}}{P_{rc}} \qquad (8.7.7.1\text{-}3)$$

P_{ro} = factored compressive resistance (Articles 6.9.2.1 and 6.9.5.1 of the AASHTO LRFD provisions) with $\lambda = 0$

$$P_{rc} = \phi_c A_c f'_c \qquad (8.7.7.1\text{-}4)$$

M_u is the maximum resultant moment applied to the member in any direction, calculated as specified in Article 4.5.3.2.2 of the AASHTO LRFD provisions

8.7.7.2 Flexural Strength

The factored moment resistance of a concrete filled steel pipe for Article 8.7.7.1 shall be calculated using either of the following two methods:

(a) Method 1 – Using Exact Geometry

$$M_{rc} = \phi_f \ [C_r e + C'_r e'] \qquad (8.7.7.2\text{-}1)$$

where

$$C_r = F_y \beta \frac{Dt}{2} \qquad (8.7.7.2\text{-}2)$$

$$C'_r = f'_c \left[\frac{\beta D^2}{8} - \frac{b_c}{2}\left(\frac{D}{2} - a\right) \right] \qquad (8.7.7.2\text{-}3)$$

$$e = b_c \left[\frac{1}{(2\pi - \beta)} + \frac{1}{\beta} \right] \qquad (8.7.7.2\text{-}4)$$

$$e' = b_c \left[\frac{1}{(2\pi - \beta)} + \frac{b_c^2}{1.5\beta D^2 - 6b_c(0.5D - a)} \right] \qquad (8.7.7.2\text{-}5)$$

$$a = \frac{b_c}{2} \tan\left(\frac{\beta}{4}\right) \qquad (8.7.7.2\text{-}6)$$

$$b_c = D \sin\left(\frac{\beta}{2}\right) \qquad (8.7.7.2\text{-}7)$$

where β is in radians and found by the recursive equation:

$$\beta = \frac{A_s F_y + 0.25 D^2 f'_c \left[\sin(\beta/2) - \sin^2(\beta/2)\tan(\beta/4) \right]}{\left(0.125 \ D^2 f'_c + \ DtF_y\right)} \qquad (8.7.7.2\text{-}8)$$

(b) Method 2 – Using Approximate Geometry

A conservative value of M_{rc} is given by

$$M_{rc} = \phi_f \left[(Z - 2th_n^2)Fy + \left[\frac{2}{3}(0.5D - t)^3 - (0.5D - t)h_n^2 \right] f'_c \right] \qquad (8.7.7.2\text{-}9)$$

where

$$h_n = \frac{A_c f'_c}{2Df'_c + 4t(2F_y \quad f'_c)} \qquad (8.7.7.2\text{-}10)$$

and Z is the plastic modulus of the steel section alone.

For capacity design purposes, in determining the force to consider for the design of capacity protected elements, the moment calculated by this approximate method shall be increased by 10%.

8.7.7.3 Beams and Connections

Capacity-protected members must be designed to resist the forces resulting from hinging in the concrete-filled pipes calculated from Article 8.7.7.2.

8.7.8 Other Systems

This Article provides minimum considerations that must be addressed for the design of special systems.

8.7.8.1 Ductile Eccentrically Braced Frames

Ductile eccentrically braced frames for bents and towers may be used provided that the system, and in particular the eccentric link and link beam, can be demonstrated to remain stable up to the expected level of inelastic response. This demonstration of performance shall be preferably achieved through full-scale cyclic tests of specimens of size greater or equal to that of the prototype.

Seismic design practice for eccentrically braced frames used in buildings can be used to select width-to-thickness ratios, stiffeners spacing and size, and strength of the links, as well as to design diagonal braces and beams outside of the links, columns, brace connections, and beam-to-column connections.

Only the eccentric brace configuration in which the eccentric link is located in the middle of a beam is permitted.

8.7.8.2 Ductile End-Diaphragm in Slab-on-Girder Bridge

Ductile end-diaphragms in slab-on-girder bridges can be designed to be the ductile energy dissipating elements for seismic excitations in the transverse directions of straight bridges provided that:

(a) Specially detailed diaphragms capable of dissipating energy in a stable manner and without strength degradation upon repeated cyclic testing are used;
(b) Only ductile energy dissipating systems whose adequate seismic performance has been proven through cycling inelastic testing are used;
(c) Design considers the combined and relative stiffness and strength of end-diaphragms and girders (together with their bearing stiffeners) in establishing the diaphragms strength and design forces to consider for the capacity protected elements;
(d) The response modification factor to be considered in design of the ductile diaphragm is given by:

$$R = \left(\frac{\mu + \dfrac{K_{DED}}{K_{SUB}}}{1 + \dfrac{K_{DED}}{K_{SUB}}} \right) \qquad (8.7.8.2\text{-}1)$$

where μ is the ductility capacity of the end-diaphragm itself, and K_{DED}/K_{SUB} is the ratio of the stiffness of the ductile end-diaphragms and substructure; unless the engineer can demonstrated otherwise, μ should not be taken greater than 4;

(e) All details/connections of the ductile end-diaphragms are welded.
(f) The bridge does not have horizontal wind-bracing connecting the bottom flanges of girders, unless the last wind bracing panel before each support is designed as a ductile panel equivalent and in parallel to its adjacent vertical end-diaphragm.
(g) An effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the slab to the diaphragm.

Overstrength factors to be used to design the capacity-protected elements depend on the type of ductile diaphragm used, and shall be based on available experimental research results.

8.7.8.3 Ductile End Diaphragms in Deck Truss Bridges

Ductile end-diaphragms in deck-truss bridges can be designed to be the ductile energy dissipating elements for seismic excitations in the transverse directions of straight bridges provided that:

(a) Specially detailed diaphragms capable of dissipating energy in a stable manner and without strength degradation upon repeated cyclic testing are used;
(b) Only ductile energy dissipating systems whose adequate seismic performance has been proven through cycling inelastic testing are used;
(c) The last lower horizontal cross-frame before each support is also designed as a ductile panel equivalent and in parallel to its adjacent vertical end-diaphragm;

(d) Horizontal and vertical energy dissipating ductile panels are calibrated to have a ratio of stiffness approximately equal to their strength ratio;

(e) The concrete deck is made continuous between supports (and end-diaphragms), and an effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the deck to the diaphragms.;

(h) The response modification factor to be considered in design of the ductile diaphragm is given by:

$$R \quad \frac{\frac{K_{DED}}{K_{SUB}}}{1 \quad \frac{K_{DED}}{K_{SUB}}} \qquad (8.7.8.2\text{-}2)$$

where μ is the ductility capacity of the end-diaphragm itself, and K_{DED}/K_{SUB} is the ratio of the stiffness of the ductile end-diaphragms and substructure; unless the engineer can demonstrated otherwise, μ should not be taken greater than 4;

(i) All capacity-protected members are demonstrated able to resist without damage or instability the maximum calculated seismic displacements.

Overstrength factors to be used to design the capacity-protected elements depend on the type of ductile diaphragm used, and shall be based on available experimental research results.

8.7.8.4 Other Systems

Other framing systems and frames that incorporate special bracing, active control, or other energy absorbing devices, or other types of special ductile superstructure elements shall be designed on the basis of published research results, observed performance in past earthquakes, or special investigation, and provide a level of safety comparable to those in the AASHTO LRFD Specifications.

8.7.9 Plastic Rotational Capacities

The plastic rotational capacity shall be based on the appropriate performance limit state for the bridge. In lieu of the prescriptive values given below, the designer may determine the plastic rotational capacity from tests and/or a rational analysis.

8.7.9.1 Life Safety Performance

A conservative values of θ_p=0.035 radians may be assumed.

8.7.9.2 Immediate Use Limit State

To ensure the immediate use of the bridge structure following a design ground motion, the maximum rotational capacity should be limited to θ_p=0.005 radians.

8.7.9.3 In Ground Hinges

The maximum rotational capacity for in-ground hinges should be restricted to θ_p=0.01 radians.

8.8 REINFORCED CONCRETE DESIGN REQUIREMENTS

8.8.1 General

Reinforcing bars, deformed wire, cold-drawn wire, welded plain wire fabric, and welded deformed wire fabric shall conform to the material standards as specified in Article 9.2 of the AASHTO *LRFD Bridge Construction Specifications*.

High strength high alloy bars, with an ultimate tensile strength of up to 1600 MPa, may be used for longitudinal column reinforcement for seismic loading providing it can be demonstrated through tests that the low cycle fatigue properties is not inferior to normal reinforcing steels with yield strengths of 520 MPa or less.

Wire rope or strand may be used for spirals in columns if it can be shown through tests that the modulus of toughness exceeds 100MPa.

In compression members, all longitudinal bars shall be enclosed by perimeter hoops. Ties shall be used to provide lateral restraint to intermediate longitudinal bars within the reinforced concrete cross section.

The size of transverse hoops and ties that shall be equivalent to or greater than:

- No. 10 bars for No. 29 or smaller bars,
- No. 16 bars for No. 32 or larger bars, and
- No. 16 bars for bundled bars.

The spacing of transverse hoops and ties shall not exceed the least dimension of the compression member or 300 mm. Where two or more bars larger than No. 36 are bundled together, the spacing shall not exceed half the least dimension of the member or 150 mm.

Deformed wire**,** wire rope or welded wire fabric of equivalent area may be used instead of bars.

Hoops and ties shall be arranged so that every corner and alternate longitudinal bar has lateral support provided by the corner of a tie having an included angle of not more than 135°. Except as specified herein, no bar shall be farther than 150 mm center-to-center on each side along the tie from such a laterally supported bar.

Where the column design is based on plastic hinging capability, no longitudinal bar shall be farther than 150 mm clear on each side along the tie from such a laterally supported bar. Where the bars are located around the periphery of a circle, a complete circular tie may be used if the splices in the ties are staggered.

Ties shall be located vertically not more than half a tie spacing above the footing or other support and not more than half a tie spacing below the lowest horizontal reinforcement in the supported member.

8.8.2 Column Requirements

For the purpose of this article, a vertical support shall be considered to be a column if the ratio of the clear height to the maximum plan dimensions of the support is not less than 2.5. For a flared column, the maximum plan dimension shall be taken at the minimum section of the flare. For supports with a ratio less than 2.5, the provisions for piers of Article 8.8.3 shall apply.

A pier may be designed as a pier in its strong direction and a column in its weak direction.

The piles of pile bents as well as drilled shaft and caissons shall be regarded as columns for design and detailing purposes.

If architectural flares or other treatments are provided to columns adjacent to potential plastic hinge zones, they shall be either “structurally isolated” in such a way that they do not add to the flexural strength capacity of the columns or the column and adjacent structural elements shall be designed to resist the forces generated by increased flexural strength capacity.

The size of the gap required for structural separation is 0.05 times the distance from the center of the column to the extreme edge of the flare, or 1.5 times the calculated plastic rotation from the pushover analysis times the distance from the center of the column to the extreme edge of the flare. Equation 8.8.6-4 provides an estimate of the reduced plastic hinge length at this location.

For oversized or architectural portions of piers or columns, minimum longitudinal and transverse reinforcement that complies with temperature and shrinkage requirements elsewhere in these specifications shall be provided.

8.8.2.1 Longitudinal Reinforcement

The area of longitudinal reinforcement shall not be less than 0.008 or more than 0.04 times the gross cross-section area A_g.

8.8.2.2 Flexural Resistance

The biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.6. The column shall be investigated for both extreme load cases (50% PE in 75 year and 3% PE in 75 year/1.5 mean deterministic as per Articles 4.4, 4.5 and 4.6. The resistance factors of Article 5.5.4.2 of the AASHTO LRFD provisions shall be replaced for both spirally and tied reinforcement columns by the value = 1.0, providing other member actions have been designed in accordance with the principles of capacity design.

8.8.2.3 Column Shear and Transverse Reinforcement

Provision of transverse reinforcement for shear shall be determined by one of the following

two methods: implicit approach or an explicit approach. The implicit approach may be used for all Seismic Hazard Levels. However, for Seismic Hazard Level IV with a two-step design (SDAP E), the shear strength shall be checked using the explicit approach.

8.3.2.3.1 Method 1: Implicit Shear Detailing Approach

(a) In potential plastic hinge zones (Article 4.9)

- For circular sections
- For rectangular sections

$$\rho_v = K_{shape}\Lambda\frac{\rho_\ell}{\phi}\frac{f_{su}}{f_{yh}}\frac{A_g}{A_{cc}}\tan\alpha\tan\theta \qquad (8.8.2.3\text{-}1)$$

in which

ϕ = 0.85

ρ_v = ratio of transverse reinforcement given by either equation 8.8.2.3-2 or 8.8.2.3-3.

- for rectangular sections

$$\rho_V = \frac{A_{sh}}{b_W s} \qquad (8.8.2.3\text{-}2)$$

and

- for circular columns

$$\rho_V = \frac{\rho_S}{2}\quad\frac{2A_{bh}}{sD''} \qquad (8.8.2.3\text{-}3)$$

where

A_{sh} = the area of the transverse hoops and cross-ties transverse to the axis of bending

A_{bh} = the area of one spiral bar or hoop in a circular section

S = the center-to-center spacing of hoopsets or the pitch the spiral steel

b_w = the web width resisting shear in a rectangular section D'' = center-to-center diameter of perimeter hoop or spiral

The terms in equation (8.8.2.3-1) are defined below:

K_{shape} = factor that depends on the shape of the section and shall be taken as

- for circular sections K_{shape} = 0.32
- for rectangular sections with 25 percent of the longitudinal reinforcement placed in each face K_{shape} = 0.375
- for walls with strong axis bending K_{shape} = 0.25
- for walls with weak axis bending K_{shape} = 0.5

Λ = fixity factor in the direction considered

Λ = 1 fixed-free (pinned one end)

Λ = 2 fixed-fixed

f_{su} = the ultimate tensile stress of the longitudinal reinforcement. If f_{su} is not available from coupon tests, then it shall be assumed that f_{su} = *1.5.* f_y.

θ = angle of the principal crack plane given by

$$\tan\theta = \left(\frac{1.6}{\Lambda}\frac{\rho_v}{\rho_\ell}\frac{A_v}{A_g}\right)^{0.25} \qquad (8.8.2.3\text{-}4)$$

with $\theta \geq 25^\circ$ and $\theta \geq \alpha$. θ may be taken as 45° as a default value.

α = geometric aspect ratio angle given by

$$\tan\alpha = \frac{D'}{L}$$

where D' = center-to-center diameter of the longitudinal reinforcement in a circular section, or the distance between the outer layers of the longitudinal steel in other section shapes.

A_v = shear area of concrete which may be taken as $0.8A_g$ for a circular section, or $A_v = b_w d$ for a rectangular section.

The spacing of the spirals or hoopsets shall not exceed 250mm or one-half the member width.

(b) Outside the Potential Plastic Hinge Zone

Outside the potential plastic hinge zone (Article 4.9) the transverse reinforcement may be reduced to account for some contribution of the concrete in shear resistance. The required amount of transverse reinforcement, outside the potential plastic hinge zone ρ_v^*, shall be given by

$$\rho_v^* = \rho_v - 0.17\frac{\sqrt{f_c'}}{f_{yh}} \qquad (8.8.2.3\text{-}5)$$

where

ρ_v = the steel provided in the potential plastic hinge zone.

ρ_v^* shall not be less than the minimum amount of transverse reinforcement required elsewhere in these specifications based on non-seismic requirements.

8.8.2.3.2 Method 2: Explicit Approach

The design shear force, V_u, on each principal axis of each column and pile bent shall be determined from considerations of the flexural overstrength being developed at the most probable locations of critical sections within the member, with a rational combination of the most adverse end moments.

In the end regions, the shear resisting mechanism shall be assumed to be provided by a combination of truss (V_s) and arch (strut) action (V_p) such that

$$V_s \geq V_u \qquad (V_p + V_c) \qquad (8.8.2.3\text{-}6)$$

where V_p = the contribution due to arch action given by

$$V_p = \frac{\Lambda}{2} P_e \tan\alpha \qquad (8.8.2.3\text{-}7)$$

where

$$\tan\alpha = \frac{D'}{L} \qquad (8.8.2.3\text{-}8)$$

P_e = compressive axial force including seismic effects

D' = center to center diameter of the longitudinal reinforcement in a circular column, or the distance between the outermost layers of bars in a rectangular column

L = column length

Λ = fixity factor defined above

V_c = the tensile contribution of the concrete towards shear resistance. At large displacement ductilities only a minimal contribution can be assigned as follows

$$V_c = 0.05\sqrt{f_c'}\, A_v \qquad (8.8.2.3\text{-}9)$$

Outside the plastic hinge zone

$$V_c = 0.17\sqrt{f_c'}\, A_v \qquad (8.8.2.3\text{-}10)$$

where

f'_c = concrete strength in MPa,

b_w = web width of the section, and

d = effective depth

V_s = the contribution of shear resistance provided by transverse reinforcement given by:

(i) for circular columns:

$$V_s = \frac{\pi}{2}\frac{A_{bh}}{s} f_{yh} D'' \cot\theta \qquad (8.8.2.3\text{-}12)$$

(ii) for rectangular sections

$$V_s = \frac{A_v}{s} f_{yh} D'' \cot\theta \qquad (8.8.2.3\text{-}13)$$

where

A_{bh} = area of one circular hoop/spiral reinforcing bar

A_{sh} = total area of transverse reinforcement in one layer in the direction of the shear force

f_{yh} = transverse reinforcement yield stress

D'' = center-to-center dimension of the perimeter spiral/hoops in the direction of loading

θ = principal crack angle/plane calculated as follows:

$$\tan\theta = \left(\frac{1.6\rho_v A_v}{\Lambda \rho_\ell A_g}\right)^{0.25} \geq \tan\alpha \qquad (8.8.2.3\text{-}14)$$

where

ρ_v = volumetric ratio of shear reinforcement given by

$$\rho_v = \frac{A_{sh}}{b_w s} \quad \text{for rectangular section}$$

$$\rho_v = \frac{\rho_s}{2} = \frac{2A_{bh}}{sD''} \quad \text{for circular columns.}$$

and $A_v =$ shear area of concrete which may be taken as $0.8A_v$ for a circular section, or $A_v = b_w d$ for a rectangular section.

8.8.2.3.3 Extent of Shear Steel

Shear steel shall be provided in all potential plastic hinge zones as defined in Article 4.9.

8.8.2.4 Transverse Reinforcement for Confinement at Plastic Hinges

The core concrete of columns and pile bents shall be confined by transverse reinforcement in the expected plastic hinge regions. The spacing shall be taken as specified in Article 8.8.2.6.

For a circular column, the volumetric ratio of spiral reinforcement, ρ_s, shall not be less than:

a) for circular sections

$$\rho_s = 0.008\frac{f'_c}{U_{sf}}\left[12\left(\frac{P_e}{f'_c A_g} + \rho_\ell \frac{f_y}{f'_c}\right)^2\left(\frac{A_g}{A_{cc}}\right)^2 - 1\right] \tag{8.8.2.4-1}$$

b) for rectangular sections

$$\frac{A_{sh}}{sB''} + \frac{A'_{sh}}{sD''} = 0.008\frac{f'_c}{U_{sf}}\left[15\left(\frac{P_e}{f'_c A_g} + \rho_\ell \frac{f_y}{f'_c}\right)^2\left(\frac{A_g}{A_{cc}}\right)^2 - 1\right] \tag{8.8.2.4-2}$$

where:

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (MPa)

f_y = yield strength of reinforcing bars (MPa)

P_e = factored axial load (N) including seismic effects. Seismic axial loads may consider the reduction due to the effect of plastic hinging.

U_{sf} = strain energy capacity (modulus of toughness) of the transverse reinforcement = 110 MPa.

$\rho_s = \frac{4A_b}{D's} =$ ratio of transverse reinforcement

where

A_{bh} = area of one circular hoop/spiral reinforcing bar.

D'' = center-to-center diameter of perimeter hoop or spiral. Within plastic hinge zones, splices in spiral reinforcement shall be made by full-welded splices or by full-mechanical connections.

s = vertical spacing of hoops, not exceeding 100 mm within plastic hinge zones

A_{cc} = area of column core concrete, measured to the centerline of the perimeter hoop or spiral (mm^2)

A_g = gross area of column (mm^2)

A_{sh} = total area of transverse reinforcement in the direction of the applied shear

A'_{sh} = total area of transverse reinforcement perpendicular to direction of the applied shear

B'' & D'' = center-to-center dimension of the transverse hoops of a tied column in the direction under consideration (mm)

Transverse hoop reinforcement may be provided by single or overlapping hoops. Cross-ties having the same bar size as the hoop may be used. Each end of the cross-tie shall engage a peripheral longitudinal reinforcing bar. All cross-ties shall have seismic hooks as specified in Article 5.10.2.2 of the AASHTO LRFD provisions.

Transverse reinforcement meeting the following requirements shall be considered to be a cross-tie:

- The bar shall be a continuous bar having a hook of not less than 135°, with an extension of not less than six diameters but not less than 75 mm at one end and a hook of not less than

90° with an extension not less than six diameters at the other end.
- Hooks shall engage all peripheral longitudinal bars.

Transverse reinforcement meeting the following requirements shall be considered to be a hoop:

- The bar shall be closed tie or continuously wound tie.
- A closed tie may be made up of several reinforcing elements with 135° hooks having a six diameter but not less than a 75 mm extension at each end.
- A continuously wound tie shall have at each end a 135° hook with a six diameter but not less than a 75 mm extension that engages the longitudinal reinforcement.

8.8.2.5 Transverse Reinforcement for Longitudinal Bar Restraint in Plastic Hinges

The longitudinal reinforcement in the potential plastic hinge zone shall be restrained by antibuckling steel as follows:

$$\text{(i) } s \le 6d_b \qquad (8.8.2.5\text{-}1)$$

where
d_b = diameter of longitudinal reinforcing bars being restrained by circular hoop or spiral

8.8.2.6 Spacing for Transverse Reinforcement for Confinement and Longitudinal Bar Restraint

Transverse reinforcement for confinement and longitudinal bar restraint (Articles 8.8.2.4 and 8.8.2.5 shall be provided at all plastic hinge zones as defined in Article 4.9 except that the requirements of Article 8.8.2.4 need not apply to the pile length from 3D to 10D below a buried pile cap.

The spacing of transverse reinforcement shall not be greater than:

$$\frac{M}{V}\left(1-\frac{M_y}{M_{po}}\right) \qquad (8.8.2.6\text{-}1)$$

The spacing of transverse reinforcement shall not exceed one-quarter of the minimum member dimension or 150 mm center-to-center outside plastic hinge zones.

8.8.2.7 Splices

The provisions of Article 5.11.5 of the AASHTO LRFD provisions shall apply for the design of splices.

Lap splices in longitudinal reinforcement shall be used only within the center half of column height, and the splice length shall not be less than 400mm or 60-bar diameters.

The spacing of the transverse reinforcement over the length of the splice shall not exceed one-quarter of the minimum member dimension.

Full-welded or full-mechanical connection splices conforming to Article 5.11.5 of the AASHTO LRFD provisions may be used, provided that not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section, and the distance between splices of adjacent bars is greater than 450mm measured along the longitudinal axis of the column.

8.8.2.8 Flexural Overstrength

Article 4.8 provides several alternate methods for calculating the flexural moment overstrength capacity (M_{po}) for columns/ piles/ drilled shafts that are part of the ERS. The plastic moment-axial load interaction formula of Equation C8.8.2.8-1 may be used to calculate the overstrength moment of a column or drilled shaft:

8.8.3 Limited Ductility Requirements for Wall Type Piers

These limited ductility provisions, herein specified, shall apply to the design for the strong direction of a pier. Providing ductile detailing is used, either direction of a pier may be designed as a column conforming to the provisions of Article 8.8.2, with the response modification factor for columns used to determine the design forces. If the pier is not designed as a column in either direction, then the limitations for factored shear resistance herein specified shall apply.

The minimum reinforcement ratio, both horizontally, ρ_h, and vertically, ρ_v, in any pier shall not be less than 0.0025. The vertical reinforcement ratio shall not be less than the horizontal reinforcement ratio.

Reinforcement spacing, either horizontally or vertically, shall not exceed 450 mm. The reinforcement required for shear shall be continuous and shall be distributed uniformly.

The factored shear resistance, V_r, in the pier shall be taken as the lesser of:

$$V_r = 0.253\sqrt{f_c'}bd \qquad (8.8.3\text{-}1)$$

$$V_r = \phi V_n \qquad (8.8.3\text{-}2)$$

for which:

$$V_n = \left[0.063\sqrt{f_c'} + \rho_h y_y\right] bd \qquad (8.8.3\text{-}3)$$

Horizontal and vertical layers of reinforcement should be provided on each face of a pier. Splices in horizontal pier reinforcement shall be staggered and splices in the two layers shall not occur at the same location.

8.8.4 Moment Resisting Connection Between Members (Column/Beam and Column/Footing Joints)

8.8.4.1 Implicit Approach: Direct Design

Flexural reinforcement in continuous, restrained, or cantilever members or in any member of a rigid frame shall be detailed to provide continuity of reinforcement at intersections with other members to develop the nominal moment resistance of the joint.

Joints shall be detailed to resist shears resulting from horizontal loads through the joint.

Transverse reinforcement in cap beam-to-column or pile cap-to-column joints should consist of the greater of:

(a) Confinement reinforcement given in Article 8.8.2.4.

(b) Longitudinal bar restraint reinforcement given by Article 8.8.2.5; this article can be waived if the longitudinal bars framing into the joint are surrounded by sufficient concrete to inhibit bar buckling. For the purpose of waiving this article cover to the longitudinal steel shall be taken as the greater of 150 mm or 6 longitudinal bar diameters.

(c) Shear reinforcement given by Article 8.8.2.3 where the principal crack angle θ is given by the aspect ratio of the member and is defined by the joint dimensions as follows

$$\tan\theta = \tan\alpha = \frac{D}{H_c}$$

where

D = width or diameter of the column framing into the joint

H_c = the height of the cap beam/joint. Thus the joint shear horizontal (transverse) reinforcement is given by:

For circular columns with spirals or circular hoops

$$\rho_s \geq 0.76\frac{\rho_\ell}{\phi}\frac{f_{su}}{f_{yh}}\frac{A_g}{A_{cc}}\tan^2\alpha. \qquad (8.8.4.1\text{-}1)$$

for rectangular sections with rectilinear hoops and/or ties

$$\frac{A_{sh}}{sB''} \geq 1.2\frac{B'/D'+0.5}{2B'/D'+2}\frac{\rho_t}{\phi}\frac{f_{su}}{f_{yh}}\frac{A_g}{A_{cc}}\tan^2\alpha \qquad (8.8.4.1\text{-}2)$$

If the above equations lead to congested steel placement details, then alternative details may be adopted through the use of rational strut and tie models as given in Article 8.8.4.2.

where

ρ_s = ratio of transverse hoops/spirals $\left(\rho_s = \frac{4A_{bh}}{sD'}\right)$

ρ_ℓ = ratio of longitudinal reinforcement area to gross area of section

A_{sh} = area of transverse reinforcement in the direction of the applied shear

f_{su} = yield strength of transverse reinforcement

A_g = gross area of section

A_{cc} = confined core area (take as $0.8A_g$ for a circular section

ϕ = resistance factor for seismic shear (0.85)

B' = center-to-center dimension of extreme longitudinal column reinforcement bars in the direction perpendicular to applied shear

D' = center to center dimension of extreme longitudinal column reinforcement bars in the direction of applied shear

8.8.4.2 Explicit Approach: Detailed Design

8.8.4.2.1 Design Forces and Applied Stresses

Moment-resisting connections between members shall be designed to transmit the maximum forces applied by the connected members. Connection forces shall be based on the assumption of maximum plastic moment.

Forces acting on the boundaries of connections shall be considered to be transmitted by mechanisms involving appropriate contributions by concrete and reinforcement actions. Mechanisms shall be based on rational analysis of force-transfer within the connection, such as strut and tie models.

Principal stresses is any vertical plane within a connection shall be calculated in accordance with Eq. (8.8.4.2-1) and (8.8.4.2-2)

Principal tension stress is given by:

$$\rho_\ell = \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \qquad (8.8.4.2\text{-}1)$$

Principal compression stress is given by:

$$p_c = \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \qquad (8.8.4.2\text{-}2)$$

where

f_h and f_v = the average axial stresses in the horizontal and vertical directions within the plane of the connection under consideration as defined in Article C8.8.4.2.1 (compression stress positive) and

v_{hv} = the average shear stress within the plane of the connection.

8.8.4.2.2 Minimum Required Horizontal Reinforcement

When the principal tension stress is less than $P_\ell = 0.29\sqrt{f_c'}$ MPa, the minimum amount of horizontal joint shear reinforcement to be provided shall be capable of transferring 50 percent of the cracking stress resolved to the horizontal direction. For circular columns, or columns with intersecting spirals, the volumetric ratio of transverse reinforcement in the form of spirals or circular hoops to be continued into the cap or footing shall not be less than

$$\rho_s = \frac{0.29\sqrt{f_c'}}{f_{yh}} \qquad (8.8.4.2\text{-}3)$$

where

f_{yh} = yield stress of horizontal hoop/tie reinforcement in the joint.

8.8.4.2.3 Maximum Allowable Compression Stresses

Principal compression stress in a connection, calculated in accordance with Equation 8.8.4.2-2 shall not exceed $p_c = 0.25 f_c'$.

8.8.4.3 Reinforcement for Joint Force Transfer

8.8.4.3.1 Acceptable Reinforcement Details

Where the magnitude of principal tension stress values (calculated in accordance with Equation 8.8.4.2-1), exceed $\rho_\ell = 0.29\sqrt{f_c'}$ MPa, vertical and horizontal joint reinforcement, placed in accordance with Articles 8.8.4.3.2, 8.8.4.3.3 and 8.8.4.3.4.is required.

8.8.4.3.2 Vertical Reinforcement

Stirrups

On each side of the column or pier wall, the beam member or footing that is subject to bending forces shall have vertical stirrups, with a total area $A_{jv} = 0.16A_{st}$ located within a distance $0.5D$ or $0.5h$ from the column or pier wall face. These vertical stirrups shall be distributed over a width not exceeding $2D$.

where

A_{st} = total area of longitudinal steel

D = diameter of circular column

h = depth of rectangular column

Clamping Reinforcement

Longitudinal reinforcement contributing to cap beam or footing flexural strength (i.e., superstructure top reinforcement, cap top reinforcement, footing bottom reinforcement) shall be clamped into the joint by vertical bars providing a total area of $0.08A_{ST}$. These bars shall be hooked around the restrained longitudinal reinforcement and extend into the joint a distance not less than two-thirds of the joint depth. If more than 50 percent of the superstructure moment capacity and/or cap-beam moment capacity is provided by prestress, this reinforcement may be omitted, unless needed for the orthogonal direction of response.

8.8.4.3.3 Horizontal Reinforcement

Additional longitudinal reinforcement in the cap beam, superstructure, and footing of total amount $0.08A_{ST}$ over and above the required for flexural strength, shall be placed in the face adjacent to the column (i.e., bottom of cap beam or superstructure; top of footing), extending through the joint and for a sufficient distance to develop its yield strength at a distance of $0.5D$ from the column face, as shown in Figure 8.8.4.2-1

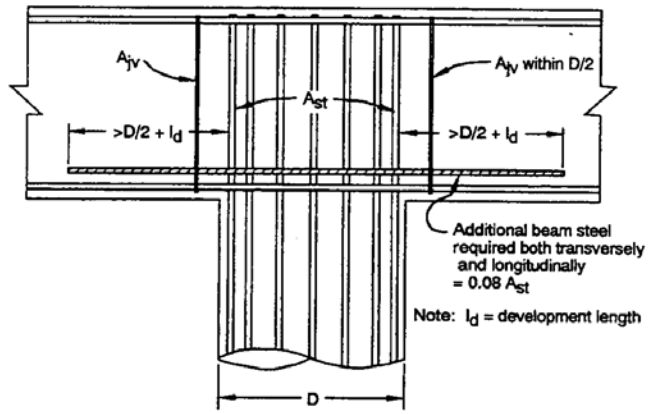


Figure 8.8.4.2-1 Additional cap beam bottom reinforcement for joint force transfer.

8.8.4.3.4 Hoop or Spiral Reinforcement

The required volumetric ratio of column joint hoop or spiral reinforcement to be carried into the cap or footing shall not be less than

$$\rho_s \geq \frac{0.4A_{ST}}{\ell_{ac}^2} \qquad (8.8.4.3\text{-}1)$$

where l_{ac} is the length of embedment of longitudinal column reinforcement. Hoop or spiral reinforcement shall be continued into the cap or footing for the full length of straight column reinforcement, or the straight portion of hooked column reinforcement unless a rational analysis

8.8.4.4 Footing Strength

In determining the flexural strength of footings resisting gravity plus seismic overloads, with monolithic column/footing connections, the effective width of the footing shall not be taken to be greater than the width of the column plus a tributary footing width, equal to the effective depth of the footing, on either side of the column.

The effective width for determining the shear strength of footings for gravity plus seismic overloads shall be as for flexural overstrength

When the nominal shear strength in footings arising from the maximum flexural overstrength, vertical stirrups or ties shall be provided to carry the deficit in shear strength. These stirrups shall be placed within the effective width as defined above.

8.8.5 Concrete Piles

8.8.5.1 Transverse Reinforcement Requirements

The upper end of every pile shall be reinforced and confined as a potential plastic hinge region as specified in Article 4.9, except where it can be established that there is no possibility of any significant lateral deflection in the pile. If an analysis of the bridge and pile system indicates that a plastic hinge can form at a lower level, the plastic hinge zone shall extend 3D below the point of maximum moment. The transverse reinforcement in the top 3D of the pile shall be detailed for the maximum of shear, confinement, and longitudinal bar restraint as for concrete columns described in Article 8.8.2. The top 10D of the pile shall be detailed for the maximum of shear and confinement as for concrete columns and described in Articles 8.8.2.3 and 8.8.2.4.

8.8.5.2 Volumetric Ratio of Transverse Reinforcement

In lieu of a precise soil structure interaction analysis to ascertain the shear demand, a value of α = 25 degrees may be assumed for use in the implicit shear design equations.

8.8.5.3 Cast-in-Place and Precast Concrete Piles

For cast-in-place and precast concrete piles, longitudinal steel shall be provided for the full length of the pile. In the upper two-thirds of the pile, the longitudinal steel ratio, provided by not less than four bars, shall not be less than 0.008**.**

8.8.6 Plastic Rotation Capacities

The plastic rotational capacity shall be based on the appropriate performance limit state for the bridge. In lieu of the prescriptive values given below, the designer may determine the plastic rotational capacity from tests and/or a rational analysis.

8.8.6.1 Life Safety Performance

The plastic rotational capacity of hinges shall be based on

$$\theta_p = 0.11\frac{L_p}{D'}\left(N_f\right)^{-0.5} \qquad rad \qquad (8.8.6\text{-}1)$$

in which

N_f = number of cycles of loading expected at the maximum displacement amplitude which may be estimated from

$$N_f = 3.5\left(T_n\right)^{-1/3} \qquad 2 \le N_f \le 10 \qquad (8.8.6\text{-}2)$$

where T_n = natural period of vibration of the structure.

For liquifiabile soils and piled foundation assessment, use $N_f = 2$

L_p = effective plastic hinge length give by

$$L_p = 0.08\frac{M}{V} + 4400\varepsilon_y d_b \qquad (8.8.6\text{-}3)$$

where
M/V = shear span of the member (M = end moment V = shear force)
ε_y = yield strain of the longitudinal reinforcement;

When an isolation gap of length L_g is provided between a structurally separated flare and an adjacent structural element, the plastic hinge length is given by

$$L_p = L_g + 8800\varepsilon_y d_b \qquad (8.8.6\text{-}4)$$

where L_g is the gap between the flare and the adjacent element.

D' = the distance between the outer layers of the longitudinal reinforcement on opposite faces of the member, equal to the pitch circle diameter for a circular section.

d_b = diameter of the main longitudinal reinforcing bars.

In lieu of the precise analysis given above, a conservative value of $\theta_p = 0.035 \quad rad$ shall be assumed.

For life-safety assessment of pile foundations that are potentially liquifiable, then $\theta_p = 0.05 rad$

8.8.6.2 Immediate Use Performance

To ensure the immediate use of the bridge structure following a design ground motion, the maximum rotational capacity should be limited to $\theta_p = 0.01 \quad rad$.

8.8.6.3 In-Ground Hinges

The maximum rotational capacity for in-ground hinges shall be restricted to $\theta_p = 0.02$ rad.

8.9 BEARING DESIGN REQUIREMENTS

There are three design or testing alternates for bearings that are not designed and tested as seismic isolation bearings as per Article 8.10. Alternate 1 requires both prototype and quality control testing of bearings as per Article 8.9.1. If testing of bearings is not performed for the required forces and displacements, then Alternate 2 provides a design option to provide a positive restraint system for the bearing. The restraint shall be capable of resisting the forces generated in the 3% PE in 75 year/1.5 mean deterministic event utilizing an analytical model that assumes that all bearings so designed are restrained. Alternate 3 provides a design option that permits a bearing to fail, provided there is a flat surface on which the girders can slide. The bearing or masonry plinth cannot impede the movement. The bridge must be analyzed in this condition and allowance for 150% of the calculated movement shall be provided.

If Alternate 3 is selected then a non-linear time history analysis is required using an appropriate coefficient of friction for the sliding surface to determine the amount of displacement that will result. The bearings shall be assumed to have failed early in the time history so a conservative value of the displacement is obtained.

8.9.1 Prototype and Quality Control Tests

Prototype Tests – each manufacturer shall perform a set of prototype tests on two full size bearings to qualify that particular bearing type and size for the rated forces or displacements of it's application. The sequence of tests shall be those given in Article 15.10.2 for the displacement or force for which it is to be qualified. For fixed bearings, the sequence of tests shall be performed for 110% of the lateral force capacity of the bearing where 110% of the force capacity replaces the total design displacement in Article 15.10.2. For bearings that permit movement, the total design displacement shall be 110% of the displacement for which they are to be qualified.

Quality Control Tests – a set of quality control tests shall be performed on 1 out of every 10 bearings of a given type and size. The tests shall be similar to those required for isolation bearings as specified in Articles 15.12.2, 15.14.2 and 15.15.6. For fixed bearings, the total design displacement shall be replaced by the lateral force capacity for which they are qualified.

8.10 SEISMIC ISOLATION DESIGN REQUIREMENTS

The design and testing requirements for the isolators are given in Articles 15.12 through 15.15

The analysis requirements for a seismically isolated bridge are given in Article 5.3.6 and Article 5.4.1.1 for the capacity spectrum method and Article 5.4.2.3 for a multi-mode analysis and Article 5.4.4 for a nonlinear time-history analysis. Other analysis and modeling issues are given in Article 15.4 and design properties of the isolators are given in Article 15.5. If an upper and lower bound analysis is performed as per Article 15.4, then the design forces and displacement shall be the maximum of those obtained from the upper and lower bound analyses respectively.

The supporting substructures may be all designed elastically using the provisions of Article 4.10. If an R of 1.5 as per Table 4.7-1 is used to design the substructure, all other elements connected to the column shall be designed using the Capacity Design procedures of Article 4.8. The design and testing of the isolator units is given in Article 15.10 and other design issues related to the isolators are given in Section 15.

8.11 SUPERSTRUCTURE DESIGN REQUIREMENTS

The provisions of this section apply in SDAP C, D and E for SDR 4. Unless noted otherwise these provisions apply to both levels of earthquake.

8.11.1 General

The superstructure shall either be capacity-protected, such that inelastic response is confined to the substructure or designed for the elastic seismic forces of the 3% PE in 75-year event/1.5 mean deterministic. If capacity protection is used, the overstrength forces developed in the piers and the elastic forces at the abutments shall be used to define the forces that the superstructure must resist. In addition to the earthquake forces, the other applicable forces for the Extreme Event combination shall be used. The combined action of the vertical loads and the seismic loads shall be considered. The superstructure shall remain essentially elastic using nominal properties of the members under the overstrength forces or elastic forces corresponding to the 3% PE in 75-year/1.5 mean deterministic earthquake, whichever are selected by the designer.

8.11.2 Load Paths

Load paths for resistance of inertial forces, from the point of origin to the points of resistance, shall be engineered. Positive connections between elements that are part of the earthquake resisting system (ERS) shall be provided. Bridges with a series of multi – simple spans cannot use the abutments to resist longitudinal forces from spans other than the two end spans. Longitudinal forces from interior spans may only be transferred to the abutments when the superstructure is continuous.

8.11.3 Effective Superstructure Width

The width of superstructure that is effective in resisting longitudinal seismic forces is dependent on the ability of the piers and abutments to effectively resist such forces. In the case of longitudinal moment transfer from the superstructure to the substructure, the pier cap beam shall be designed to resist forces transferred at the connection locations with the substructure. If such resistance is not provided along the cap beam, then a reduced effective superstructure width shall be used. This width shall be the sum of the column width along the transverse axis and the superstructure depth for open-soffit superstructures (e.g. I-girder bridges) or the column width plus twice the superstructure depth for box girders and solid superstructures. The effective width is to be taken transverse to the column at the pier and may be assumed to increase at a 45-degree angle as one moves along the superstructure until the full section becomes effective.

For superstructures with integral cap beams at the piers, the effective width of the cap beam may be as defined in Section 4.6.2.6 of the AASHTO LRFD provisions.

8.11.4 Superstructure to Substructure Connections

The provisions of this section apply in SDAP B, D, and E. These provisions apply to both levels of earthquake.

8.11.4.1 Connection Design Forces

The forces used for the design of connection elements shall be the lesser of the 3% PE in 75-year/1.5 mean deterministic elastic forces or the overstrength forces developed in the substructure below the connection as per Article 4.8.

8.11.4.2 Fuse Elements and Abutment Connections

Where connections or adjacent structure is designed to fuse (e.g. shear keys at abutments that might be intended to breakaway in the 3% in 75-year/1.5 mean deterministic earthquake), the design forces shall correspond to an upper-bound estimate of the force required to fuse the element. The materials and details used to create fuse elements shall be chosen such that reasonable predictability of the fuse strength is assured.