September 2024 ERRATA for *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, 3rd Edition (LRFDSEIS-3)

September 2024

Dear Customer:

AASHTO has issued an erratum, which includes technical revisions, for *the AASHTO Guide Specifications for LRFD Seismic Bridge Design*, 3rd Edition (LRFDSEIS-3).

If you need to download this file again, it can be found on AASHTO's website at:

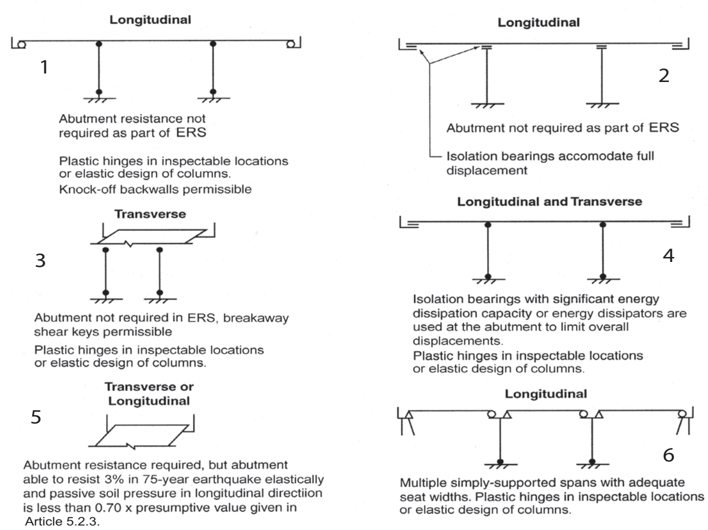
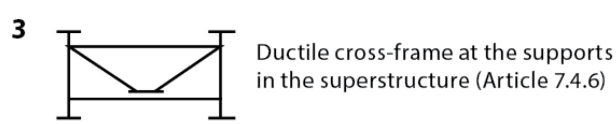
https://downloads.transportation.org/LRFDSEIS-3-Errata.pdf

The changes in this erratum are detailed in the table. All pages with corrections have a gray box in the page header reading as follows:

September 2024 Errata

AASHTO staff sincerely apologizes for any inconvenience.

This page intentionally left blank.

Original Page	Section	Existing Text/ Details of Change	Corrected Text
3-4	C3.3	The third paragraph of Article C3.3 should instead be the second paragraph, directly following the first.	Common examples from each of the three ERS and ERE categories are shown in Figures 3.3-1a and 3.3-1b, respectively. Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. To this end, the identification of the lateral-force-resisting concept and the selection of the necessary elements to fulfill the concept should be accomplished in the conceptual design phase; or the type, size, and location phase; or the design alternative phase of a project. For SDC B, it is suggested that the ERS be identified. The displacement checks for SDC B are predicated on the existence of a complete lateral load resisting system; thus, the Designer should ensure that an ERS is present and that no unintentional weak links exist. Additionally, identifying the ERS helps the Designer ensure that the model used to determine displacement demands is compatible with the drift limit calculation. For example, pile–bent connections that transmit moments significantly less than the piles can develop should not be considered as fixed connections.
3-10	Figure 3.3-1a	Individual items in the figure should be numbered.	
3-13	Figure 3.3-2	Item 3 in the figure's label has been changed from "Ductile end diaphragms in superstructure" to "Ductile cross-frame at the supports in the superstructure".	

Original Page	Section	Existing Text/ Details of Change	Corrected Text
7-5	7.3	"Ductile end diaphragms" in the second paragraph changed to "ductile cross frames".	For SDCs C and D, ductile substructure elements and ductile cross frames, as defined in Article 7.4.6 inclusive through Article 7.5, shall be made of steels satisfying the requirements of:
8-10	Equation 8.6.2-3	"0.0032" should be "0.032".	$$v_c = 0.032\alpha'\left(1+\frac{P_u}{2A_g}\right)\sqrt{f_c'} \le \min\begin{cases} 0.11\sqrt{f_c'} \\ 0.047\alpha'\sqrt{f_{c'}} \end{cases}$$

For bridges in the Critical and Recovery category that have been designed using the *AASHTO Guidelines for Performance-Based Seismic Design of Highway Bridges*, designing for higher performance levels at the design response spectrum will decrease the probability of reaching incipient collapse during a 75-year life. As of yet, this reduction in probability has not been quantified.

The risk-targeted ground motions developed from incipient bridge column collapse are also applicable for geotechnical hazard assessments. Appendix C provides further information on this topic.

Allowable displacements for the risk-targeted design are constrained by geometric, structural, and geotechnical considerations. The most restrictive of these constraints will govern displacement capacity. These displacement constraints may apply to transient displacements as would occur during ground shaking, permanent displacements as may occur due to seismically induced ground failure, permanent structural deformations or dislocations, or a combination. The extent of allowable displacements depends on the desired performance level of the bridge design.

Geometric constraints generally relate to the usability of the bridge by traffic passing on or under it. Therefore, this constraint will usually apply to permanent displacements that occur as a result of the earthquake. The ability to repair such displacements or the desire not to be required to repair them should be considered when establishing displacement capacities. When uninterrupted or immediate service is desired, the permanent displacements under the risk-targeted ground motions should be small or nonexistent and should be at levels that are within an accepted tolerance for normally operational highways of the type being considered.

A bridge designed to a life safety performance level could be expected to be unusable after liquefaction, for example, and geometric constraints would have no influence. However, because life safety is at the heart of the design limit state, jurisdictions may consider establishing some geometric displacement limits for this performance level for important bridges or those with high average daily traffic (ADT). This can be done by considering the risk to highway users in the moments during or immediately following an earthquake. For example, an abrupt vertical dislocation of the highway of sufficient height could present an insurmountable barrier and thus result in a collision that could cause death or injury. Usually these types of geometric displacement constraints will be less restrictive than those resulting from structural considerations; for bridges on liquefiable sites, where forces from lateral flow or lateral spreading cannot be handled without large foundation displacements, it may not be economical to prevent significant displacements from occurring.

3.3—EARTHQUAKE-RESISTING SYSTEMS (ERS) REQUIREMENTS FOR SDCS C AND D

For SDC C or D (see Article 3.5), all bridges and their foundations shall have a clearly identifiable earthquake-resisting system (ERS) selected to achieve the life safety criteria defined in Article 3.2. For SDC B, identification of an ERS should be considered.

The ERS shall provide a reliable and uninterrupted load path for transmitting seismically induced forces into the surrounding soil and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements. All structural and foundation elements of the bridge shall be capable of achieving anticipated displacements consistent with the requirements of the chosen design strategy of seismic resistance and other structural requirements.

Design should be based on the following three Global Seismic Design Strategies used in these Guide Specifications based on the expected behavior characteristics of the bridge system:

- **Type 1**—Ductile Substructure with Essentially Elastic Superstructure: This category includes conventional plastic hinging in columns and walls and abutments that limits inertial forces by full mobilization of passive soil resistance. Also included are foundations that may limit inertial forces by in-ground hinging, such as pile bents and integral abutments on piles.
- **Type 2**—Essentially Elastic Substructure with a Ductile Superstructure: This category applies only to straight, nonskewed, steel I-Section composite girder superstructures with ductile cross-frames at the supports. The use of this strategy shall be approved by the Owner and based on a case-specific design criteria and methodology.
- **Type 3**—Elastic Superstructure and Substructure with a Fusing Mechanism between the two: This category includes seismically isolated structures and structures in which supplemental energy-dissipation devices, such as dampers, are used to control inertial forces transferred between the superstructure and substructure.

See also Article 7.2 for further discussion of performance criteria for steel structures.

C3.3

Common examples from each of the three ERS and ERE categories are shown in Figures 3.3-1a and 3.3-1b, respectively. Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. To this end, the identification of the lateral-force-resisting concept and the selection of the necessary elements to fulfill the concept should be accomplished in the conceptual design phase; or the type, size, and location phase; or the design alternative phase of a project.

For SDC B, it is suggested that the ERS be identified. The displacement checks for SDC B are predicated on the existence of a complete lateral load resisting system; thus, the Designer should ensure that an ERS is present and that no unintentional weak links exist. Additionally, identifying the ERS helps the Designer ensure that the model used to determine displacement demands is compatible with the drift limit calculation. For example, pile–bent connections that transmit moments significantly less than the piles can develop should not be considered as fixed connections.

The use of this strategy requires the Owner's approval and a case-specific design criteria and methodology because the design guidelines are under development and there is a lack of practical experience with ductile cross-frames.

to whether or not the abutments would be included and relied on in the ERS. Some states may require the design of a bridge in which the substructures are capable of resisting the entire lateral load without any contribution from the abutments. In this design approach, the abutments are included in a mechanism to provide an unquantifiable higher level of safety. Rather than mandate this design philosophy here, it was decided to permit two design alternatives. The first is where the ERS does not include the abutments and the substructures are capable of resisting all the lateral loads. In the second alternative, the abutments are an important part of the ERS and, in this case, a higher level of analysis is required.

If the abutment is included as part of the ERS, this design option requires a continuous superstructure to deliver longitudinal forces to the abutment. If these conditions are satisfied, the abutments can be designed as part of the ERS and become an additional source for dissipating the bridge's earthquake energy. In the longitudinal direction, the abutment may be designed to resist the forces elastically using the passive pressure of the backfill. In some cases, the longitudinal displacement of the deck will cause larger soil movements in the abutment backfill, exceeding the passive pressures there. This requires a more refined analysis to determine the amount of expected movement. In the transverse direction, the abutment is generally designed to resist the loads elastically. The design objective when abutments are relied on to resist either longitudinal or transverse loads is either to minimize column sizes or reduce the ductility demand on the columns, accepting that damage may occur in the abutment.

When the abutment is part of the ERS, the performance expectation is that inelastic deformation will occur in the columns as well as the abutments. If large ductility demands occur in the columns, then the columns may need to be replaced. If large movements of the superstructure occur, the abutment backwall may be damaged and there may be some settlement of the abutment backfill. Large movements of the superstructure can be reduced with use of energy dissipators and isolation bearings at the abutments and at the tops of the columns.

In general, the soil behind an abutment is capable of resisting substantial seismic forces that may be delivered through a continuous superstructure to the abutment. Furthermore, such soil may also substantially limit the overall movements that a bridge may experience. This is particularly so in the longitudinal direction of a straight bridge with little or no skew and with a continuous deck. The controversy with this design concept is the scenario of what may happen if there is significant abutment damage early in the earthquake ground motion duration and if the columns rely on the abutment to resist some of the load. This would be a problem in a long-duration, high-magnitude (greater than magnitude 7) earthquake. Another consideration is if a gap develops between the abutment and the soil after the first cycle of loading, due to the inelastic behavior of the soil when passive pressures are developed.

Unless shock transmission units (STUs) are used, a bridge composed of multiple simply supported spans cannot effectively mobilize the abutments for resistance to longitudinal force. It is recommended that simply supported spans not rely on abutments for any seismic resistance.

Because structural redundancy is desirable (Buckle et al., 1987), good design practice dictates the use of the design alternative in which the intermediate substructures, between the abutments, are designed to resist all seismic loads, if possible. This ensures that in the event abutment resistance becomes ineffective, the bridge will still be able to resist the earthquake forces and displacements. In such a situation, the abutments provide an increased margin against collapse.

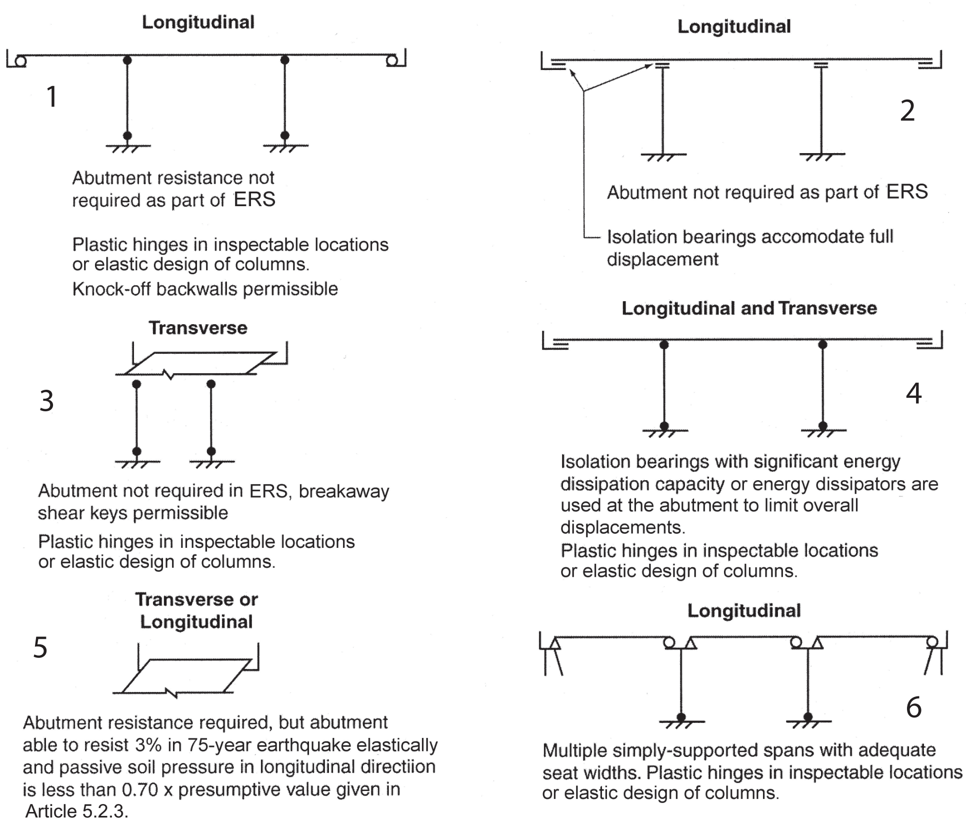


Figure 3.3-1a—Permissible Earthquake-Resisting Systems (ERSs)

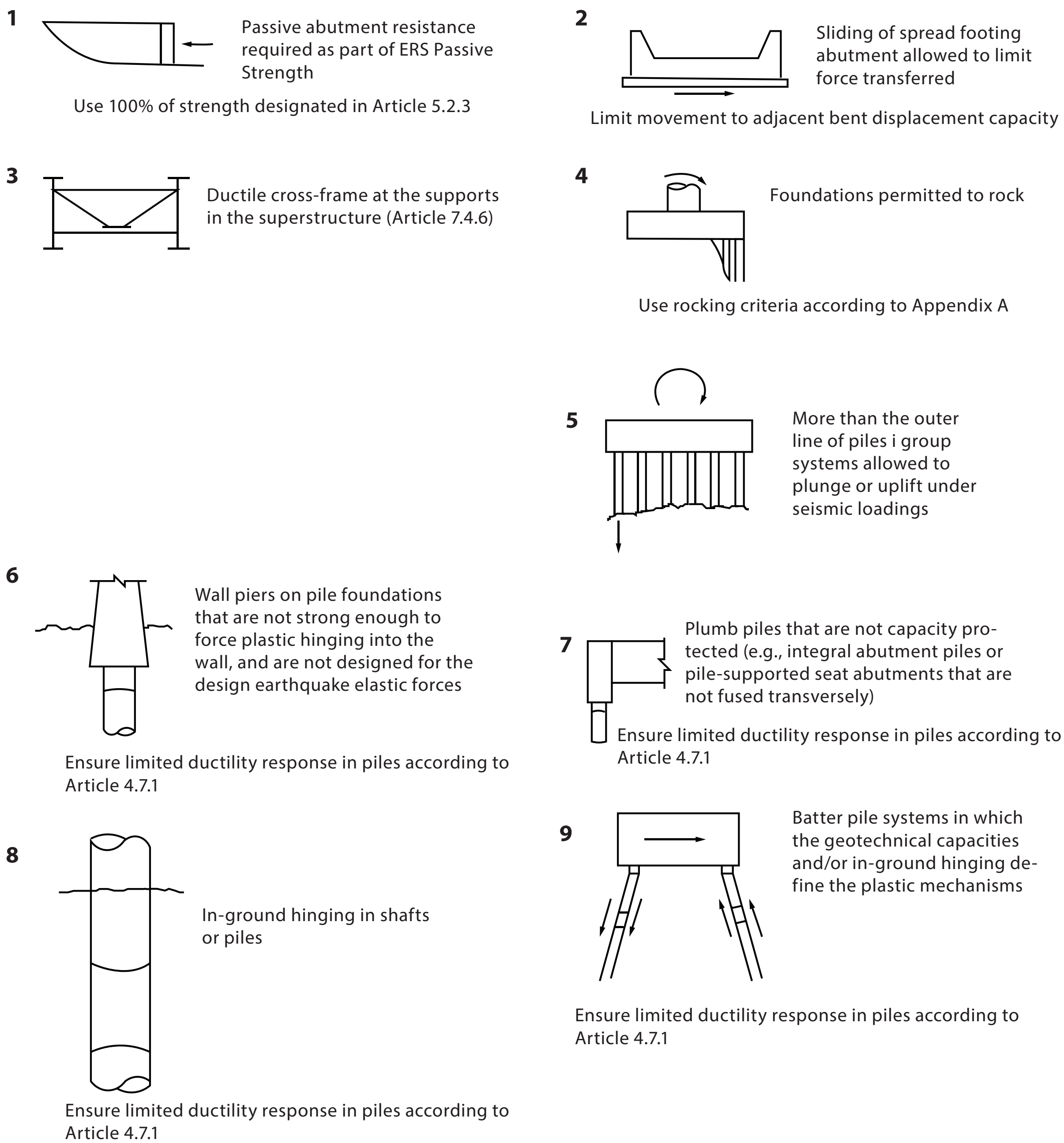


Figure 3.3-2—Permissible Earthquake-Resisting Elements that Require Owner's Approval

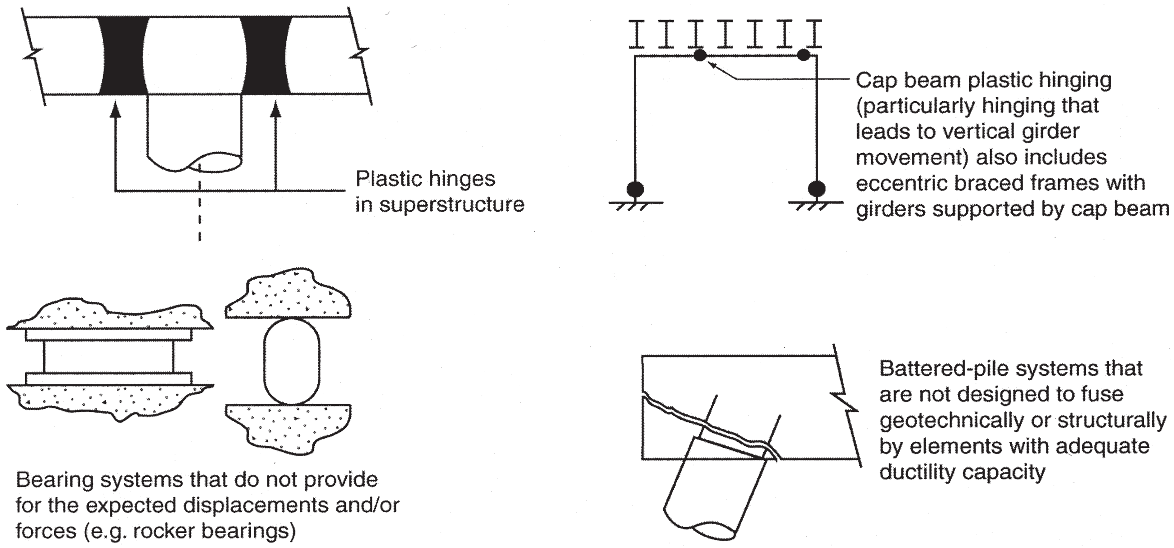


Figure 3.3-3—Earthquake-Resisting Elements that Are Not Recommended for New Bridges

3.4—SEISMIC DESIGN GROUND MOTION

The seismic design ground motion shall be characterized using a risk-targeted acceleration response spectrum. The risk-targeted acceleration response spectrum shall be determined in accordance with the general procedure of Article 3.4.1 or the site-specific procedure of Article 3.4.2.

In the general procedure, the spectral response parameters shall be determined using the AASHTO–USGS Seismic Design Ground Motion Database, produced by the U.S. Geological Survey (USGS) motion and spectral response using the 2018 USGS National Seismic Hazard Model (NSHM). This database defines spectral acceleration coefficients at 5 percent damping for 22 periods between zero seconds and 10 seconds. These ground motions have a targeted risk of incipient bridge column collapse of 1.5 percent in 75 years using the notional fragility function.

In the site-specific procedure, spectral response parameters shall be determined using a site-specific, risk-targeted seismic hazard analysis, a site-specific ground motion response analysis using risk-targeted ground motions, or both. Performance of site-specific seismic hazard and seismic ground response analyses requires Owner approval and may require an independent peer review, depending on Owner requirements.

A site-specific, risk-targeted seismic hazard analysis shall be considered if any of the following apply:

C3.4

The methodology used to develop the design ground motions, commonly referred to as a risk-targeted approach, results in a uniform risk of reaching the design limit state across the country.

The notional fragility curve used in the development of the ground motions is a lognormal distribution with a lognormal standard deviation of 0.6, and a probability of exceedance at the design ground motion of 5 percent. The hazard curves utilized in the development of the risk-targeted ground motions are based on the USGS 2018 NSHM, and the targeted risk was set to 1.5 percent in 75 years approximately equivalent to a reliability index of 2.2.

The use of risk-targeted design motions is also implemented in geotechnical design. At present, candidate fragility representations that cover the range of geotechnical hazards (e.g., liquefaction triggering and its effects, seismic slope instability, and seismic earth pressures) are not available. It is anticipated that ongoing efforts will result in a native geotechnical approach to risk-targeted design, but that the risk-targeted motions developed herein are appropriate for use until more refined results are available. Appendix C provides additional information on this topic.

For most geotechnical hazard evaluations and design, the risk-targeted ground motion should be obtained from the AASHTO–USGS Seismic Design Ground Motion Database and used in the general procedure summarized in these Guide

- Lateral resistance of the frame action generated between the girders and deck, and
- Inertial forces of the girders and substructure.

Where a Type 2 strategy is used, the substructure shall be detailed such that it has the capacity to respond in a ductile manner in both the longitudinal and transverse directions.

Seismic demands on the substructure must consider all of the applicable force components. These forces include those associated with inelastic resistance of ductile cross-frames as well as inertial effects of the structure below the deck. Demands in the longitudinal direction, including potential inelastic demands, must also be considered. Because the design guidelines for a Type 2 strategy are still under development, ductile detailing of the substructure elements in both the transverse and longitudinal direction is required. The extent of the inelastic action in the substructure and subsequent ductile detailing requirements is developed in a case-specific manner.

7.2.3—Type 3

For Type 3 structures, the Designer shall assess the overstrength capacity for the fusing interface including shear keys and bearings, then design for an essentially elastic superstructure and substructure. The minimum lateral design force shall be calculated using an acceleration of 0.4*g* or the elastic seismic force, whichever is smaller. If isolation devices are used, the superstructure shall be designed as essentially elastic (see Article 7.8).

7.3—MATERIALS

The provisions of Section 6 of the *AASHTO LRFD Bridge Design Specifications* for structural steel that is designed to remain essentially elastic during the design seismic event shall apply as applicable.

For SDCs C and D, ductile substructure elements and ductile cross frames, as defined in Article 7.4.6 inclusive through Article 7.5, shall be made of steels satisfying the requirements of:

- ASTM A709 Grade 50,
- ASTM A709 Grade 50W,
- ASTM A992,
- ASTM A500 Grade B, and
- ASTM A501.

For ASTM A709 Grade 50 and Grade 50W and ASTM A992 steels, the expected yield stress, F_{ye}, shall be taken as 1.1 times the nominal yield stress, F_y.

For ASTM A500 Grade B and ASTM A501 steels, the expected yield stress, F_{ye}, shall be taken as 1.4 times the nominal yield stress.

For SDC B, ASTM A709 Grade 36 can be used. For ASTM A709 Grade 36 steel, the expected yield

C7.3

To ensure that the objective of capacity design is achieved, Grade 36 steel is not permitted for the components expected to respond in a significantly ductile manner. Grade 36 is difficult to obtain, and contractors often substitute it with Grade 50 steel. Furthermore, it has a wide range in its expected yield and ultimate strength and large overstrength factors to cover the anticipated range of property variations. The common practice of dual certification for rolled shapes, recognized as a problem from the perspective of capacity design following the Northridge earthquake, is now becoming progressively more common also for steel plates. As a result, only Grade 50 steels are allowed for structures in SDCs C and D.

In those instances when Grade 36 steel is permitted for use (SDC B), capacity design should be accomplished assuming an effective yield strength factor of 1.5.

The use of A992 steel is explicitly permitted. ASTM A992 steel, developed to ensure good ductile seismic performance, is specified to have both a minimum and maximum guaranteed yield strength and may be worthy of consideration for ductile energy-dissipating systems in steel bridges.

stress, F_{ye}, shall be taken as 1.5 times the nominal yield stress.

For SDC C and D, ductile concrete-filled steel pipe as defined in Article 7.6 shall be made of steels satisfying the requirements of:

- ASTM A53 Grade B
- API 5L X52

For ASTM A53 Grade B steel, the expected yield stress, F_{ye}, shall be taken as 1.5 times the nominal yield stress.

For API 5L X52 steel, the expected yield stress, F_{ye}, shall be taken as 1.2 times the nominal yield stress, F_y.

Because other steels may be used, provided that they are comparable to the approved Grade 50 steels, high-performance steel (HPS) Grade 50 would be admissible, but not HPS Grade 70W (or higher). Based on limited experimental data available, it appears that HPS Grade 70W has a lower rotational ductility capacity and may not be suitable for "ductile fuses" in seismic applications.

When other steels are used for energy-dissipation purposes, it is the responsibility of the Designer to assess the adequacy of material properties available and design accordingly. Other steel members expected to remain elastic during earthquakes should be made of steels conforming to Article 6.4 of the *AASHTO LRFD Bridge Design Specifications*.

The American Petroleum Institute (API) provides more stringent requirements for steel pipes. Other steel pipe materials are permitted with the Owner's approval.

The overstrength capacity shall be taken as the resistance of a member, connection, or structure based on the nominal dimensions and details of the final section(s) chosen. The overstrength capacity shall be determined using the expected yield stress, F_{ye}, and overstrength factor, λ_{mo}, as specified in Article 4.11.2.

The capacity design philosophy and the concept of capacity-protected elements are defined in Article 4.11.

Welding requirements shall be compatible with the *AASHTO/AWS D1.5M/D1.5 Bridge Welding Code*. Undermatched 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 ASTM Standard A709/A709M, Paragraph 10, "Fracture Critical (F) Tension Members, Zone 3." Weld metal connecting these members shall meet the toughness requirements specified in the *AASHTO/AWS D1.5M/D1.5 Bridge Welding Code* for Zone III.

Steel members and weld materials should have adequate notch toughness to perform in a ductile manner over the range of expected service temperatures. The A709/A709M S84 "Fracture-Critical Material Toughness Testing and Marking" requirement, typically specified when the material is to be used in a fracture-critical application as defined in the *AASHTO LRFD Bridge Design Specifications*, is deemed to be appropriate to provide the level of toughness sought for seismic resistance. For weld metals, the *AASHTO/AWS D1.5M/D1.5 Bridge Welding Code* requirement for Zone III, familiar to the bridge engineering community, is similar to the 20 ft-lbs at –20 degrees F requirement proposed by the SAC Joint Venture for weld metal in welded moment frame connections in building frames.

7.4—MEMBER REQUIREMENTS FOR SDCS C AND D

7.4.1—Limiting Slenderness Ratios

Bracing members shall have a slenderness ratio, *KL/r*, less than 120. The length of a member shall be taken between the points of intersection of members. An effective length factor, *K*, of 0.85 of compression

C7.4.1

In the ductile design of concentrically braced frames in buildings, the slenderness ratio limits for braces, up until the late 1990s, were approximately 75 percent of the value specified here. The philoso-

The minimum column cross section and reinforcing steel that meets the project requirements should be used. Using a larger column than necessary may increase size and cost of the connecting elements (e.g. foundations, cap beams, etc.).

8.6—SHEAR DEMAND AND CAPACITY FOR DUCTILE CONCRETE MEMBERS FOR SDCS B, C, AND D

8.6.1—Shear Demand and Capacity

The shear demand for a column, V_u, in SDC B shall be determined on the basis of the lesser of:

- The force obtained from a linear elastic seismic analysis, or
- The force, V_{po}, corresponding to plastic hinging of the column including an overstrength factor.

The shear demand for a column, V_u, in SDC C or D shall be determined on the basis of the force, V_{po}, associated with the overstrength moment, M_{po}, defined in Article 8.5 and outlined in Article 4.11.

The shear demand for a non-oversized shaft, V_u, in SDC B shall be determined on the basis of the lesser of:

- The force obtained from a linear elastic seismic analysis, or
- The force, V_{po}, corresponding to that determined using the procedure for SDC C or D.

The shear demand for a non-oversized pile shaft, V_u, in SDC C or D shall be determined on the basis of the greater of either the force reported in the soil–pile shaft interaction analysis when the in-ground hinges form, or on the basis of the shear calculated by dividing the overstrength moment capacity of the pile shaft by the length H_s. H_s shall be taken as the smaller of:

- $H' + 2D_c$, or
- The length of the column/pile shaft from the point of maximum moment in the pile shaft to the point of contraflexure in the column.

where:

H' = length of the pile shaft/column from the ground surface to the point of contraflexure in the column above the ground (in.)

D_c = diameter of pile shaft (in.)

C8.6.1

The requirements of this Article are, in part, intended to avoid column shear failure by using the principles of "capacity protection." For SDCs C and D, the design shear force is specified as a result of the overstrength plastic moment capacity, regardless of the elastic earthquake design forces. This requirement is necessary because of the potential for superstructure collapse if a column fails in shear.

In SDC B, either the elastic shear demand force or the plastic hinging shear force may be used for shear design of a column. It is recommended that the plastic hinging forces be used wherever practical.

A column may be loaded in either the longitudinal or transverse direction. The shear force corresponding to the maximum shear developed in either direction for noncircular columns should be used for the determination of the transverse reinforcement.

The column or non-oversized pile shaft shear strength capacity within the plastic hinge region as specified in Article 4.11.7 shall be calculated on the basis of the nominal material strength properties and shall satisfy:

$$\phi_s V_n \geq V_u \tag{8.6.1-1}$$

in which:

$$V_n = V_c + V_s \tag{8.6.1-2}$$

where:

φ_s = 0.90 for shear in reinforced concrete

V_n = nominal shear capacity of member (kips)

V_c = concrete contribution to shear capacity as specified in Article 8.6.2 (kips)

V_s = reinforcing steel contribution to shear capacity as specified in Article 8.6.3 (kips)

The nominal shear resistance for ductile concrete members outside the plastic hinge region as defined in Article 4.11.7 may be determined using the provisions of Articles 8.6.2 to 8.6.7 with the coefficient α' set equal to 3. This shall only be applicable to load cases that include seismic effects. Alternately, the provisions of *AASHTO LRFD Bridge Design Specifications* may be used.

8.6.2—Concrete Shear Capacity

The concrete shear capacity, V_c, of members designed for SDCs B, C, and D shall be taken as:

$$V_c = v_c A_e \tag{8.6.2-1}$$

in which:

$$A_e = 0.8 A_g \tag{8.6.2-2}$$

If P_u is compressive:

$$v_c = 0.032\alpha'\left(1+\frac{P_u}{2A_g}\right)\sqrt{f_c'} \leq \min\begin{cases}0.11\sqrt{f_c'}\\0.047\alpha'\sqrt{f_{c'}}\end{cases} \tag{8.6.2-3}$$

otherwise:

$$v_c = 0 \tag{8.6.2-4}$$

For circular columns with spiral or hoop reinforcing:

C8.6.2

The shear provisions in *AASHTO LRFD Bridge Design Specifications* are not applicable for sections that are expected to accommodate a significant amount of plastic deformation. The concrete shear strength within the plastic hinge region degrades as the ductility demand increases but is improved with increasing transverse confinement.