

Seismic Bridge Design Basics

Earthquakes load a bridge in a way no other design case does: the demand is a displacement imposed through the ground, the design event is rare, and the economical answer is almost never to resist it elastically. This course walks through the basics of seismic bridge design as taught in FHWA's reference manual for the NHI course on LRFD seismic analysis and design (FHWA-NHI-15-004). It covers how U.S. practice evolved after the 1971 San Fernando earthquake, the 1,000-year design hazard and NEHRP site classes, how seismic zones and seismic design categories scale the design effort, the force-based approach of the AASHTO LRFD Bridge Design Specifications and the displacement-based approach of the AASHTO Guide Specifications for LRFD Seismic Bridge Design, earthquake resisting systems and global design strategies, capacity design and overstrength, minimum support lengths, and the detailing that turns a column into a ductile fuse.

Learning objectives

- Explain seismic design philosophy
- Describe seismic design categories
- Apply capacity design concepts
- Detail for ductility

Why seismic design is different

Most bridge load cases are forces: gravity acting on a mass, a truck axle, wind pressure on a projected area. A designer computes the force, multiplies by a load factor, and provides enough resistance. Earthquakes do not fit that pattern well. The ground moves, and the bridge, which has mass and stiffness, is forced to follow. The force that develops in the structure depends on how stiff and how strong the structure is. A very strong, stiff bridge attracts large inertial forces; a more flexible or weaker one attracts less force but moves more. FHWA-NHI-15-004 repeatedly frames seismic design around this trade between force and displacement.

Three facts drive the whole discipline.

The design event is rare. The design earthquake in current AASHTO practice, as described in the manual, has a 7% probability of exceedance in 75 years, which corresponds to a return period of roughly 1,000 years. A bridge with a 75-year design life will most likely never experience it. Designing every member to stay elastic for that event would be enormously expensive for a load that probably never arrives.

Elastic demands are very large. In moderate and high seismic regions, the elastic forces that the design spectrum produces in a typical bridge are often several times what the columns could economically resist. Designers therefore accept that some members will yield. The question becomes not whether there will be damage but where it will occur, whether it can be controlled and inspected, and whether the bridge can still carry its dead load afterward.

Collapse usually comes from details, not from global strength. The manual's history chapters and the earthquakes it describes show the same handful of failure modes again and again: spans falling off narrow seats, columns failing in shear before they can bend, lap splices in plastic hinge zones that unzip, poorly confined concrete that crushes, and joints and footings that fail because they were weaker than the columns they supported. None of these is solved by adding horizontal strength alone. They are solved by configuration, capacity protection and detailing.

For that reason, the stated performance objective of the standard AASHTO provisions, as the manual describes them, is life safety: no collapse in the design earthquake. Significant damage, and even loss of service, is accepted for ordinary bridges. The bridge should not fall down, and it should allow people to get off it. Higher performance objectives, such as keeping a critical bridge open immediately after a large event, require owner-specific criteria, which this course addresses only in outline.

What this course covers and does not. This is a one-hour primer for engineers who rate, inspect, manage or design bridges and need a working vocabulary of seismic design. It is not a substitute for the NHI course, the AASHTO specifications or a state's seismic design criteria. Where this course states a numerical provision, it is reported as FHWA-NHI-15-004 summarized it in 2014, and you must check the current AASHTO edition and any owner amendments before applying it. BR-E02 covers the companion topic of evaluating and retrofitting existing bridges, using FHWA's Seismic Retrofitting Manual.

How U.S. practice evolved

FHWA-NHI-15-004 traces modern U.S. bridge seismic design to a single event: the San Fernando earthquake of February 11, 1971. Several freeway structures in the Los Angeles area collapsed or were severely damaged. Before that earthquake, seismic loads on bridges were handled with small, simple lateral force coefficients that had little basis in observed behavior. San Fernando exposed the problems: inadequate seat lengths, columns with little confinement, and a design philosophy that did not consider how a structure would behave once it yielded.

Caltrans and the first AASHTO response. According to the manual, Caltrans developed new seismic criteria in 1973, and AASHTO adopted those criteria in 1975 as interim specifications. This was the first significant national change.

ATC-6 and the 1983 Guide Specification. FHWA funded the Applied Technology Council project that produced ATC-6, Seismic Design Guidelines for Highway Bridges. AASHTO adopted it as a Guide Specification in 1983. The manual identifies the key concepts it introduced: four seismic performance categories based on the acceleration coefficient and importance, response modification factors (R factors) to reduce elastic forces for design of ductile elements, and minimum support lengths. Those three ideas, scaled effort by hazard, explicit reliance on ductility, and minimum seats, remain the backbone of today's provisions.

Division I-A. In 1990, according to the manual, the Guide Specification provisions were incorporated into the AASHTO Standard Specifications for Highway Bridges as Division I-A, making them part of the standard rather than an alternative.

LRFD. When AASHTO adopted the LRFD Bridge Design Specifications in 1994, the seismic provisions carried over largely in the Division I-A form, with a force-based approach using R factors.

The 2007 change in hazard. The most consequential recent change described in the manual is the change in the design ground motion. Earlier AASHTO practice, dating to the 1981 guide, used a seismic hazard with roughly a 500-year return period (about 10% probability of exceedance in 50 years). In the revisions associated with the new Guide Specifications, the design hazard became 7% probability of exceedance in 75 years, about a 1,000-year return period, and the NEHRP site classification system was adopted for site effects. The manual notes these changes were made simultaneously in the LRFD Specifications and the Guide Specifications, because the two documents needed the same input ground motion to avoid an unacceptable inconsistency. FHWA's 2006 Seismic Retrofitting Manual and NCHRP Report 611 (2008) use the same 1,000-year hazard.

Why not 2,500 years? The manual explains that an NCHRP project (2001) proposed a two-level approach with a rare event at 3% in 75 years (about 2,500 years) and an expected event at 50% in 75 years (about 100 years). In developing the current

criteria, the two-level design approach was judged unjustified for most U.S. transportation facilities and was not adopted, and 2,500 years was considered too long a return period for the single design earthquake. The 1,000-year event was the compromise.

Two documents, one hazard. The result is that U.S. designers today, as of the manual's writing, have two acceptable AASHTO routes: the force-based method in the LRFD Specifications and the displacement-based method in the Guide Specifications. Many states have chosen one or the other, and some have their own seismic design criteria. The rest of this course compares the two as the manual does.

Seismic input: hazard, site class and the design spectrum

Every seismic design starts with the ground motion. FHWA-NHI-15-004 Chapter 2 explains that both AASHTO documents use the same input: a uniform-hazard design response spectrum for a 5%-damped single-degree-of-freedom system, built from mapped values at the 1,000-year hazard and adjusted for the soil at the site.

Mapped values. Three spectral values are taken from the AASHTO hazard maps (prepared with USGS) for a reference rock site, Site Class B: the peak ground acceleration (PGA), the short-period (0.2-second) spectral acceleration S_s , and the 1.0-second spectral acceleration S_1 . The manual describes obtaining these from the AASHTO maps, the AASHTO CD software, or the USGS web application, which at the time accepted a site location and site class and returned the values and the seismic design category. Engineers should confirm what tool is current; web tools change more often than specifications.

Site classes. Site effects are captured with the NEHRP site classes, defined by average properties in the top 100 feet. As tabulated in the manual:

- **A, hard rock:** shear wave velocity above 5,000 ft/s.
- **B, rock:** 2,500 to 5,000 ft/s.
- **C, very dense soil and soft rock:** 1,200 to 2,500 ft/s, or SPT N above 50, or undrained shear strength at least 2,000 psf.
- **D, stiff soil:** 600 to 1,200 ft/s, N from 15 to 50, or s_u from 1,000 to 2,000 psf.
- **E, soft soil:** below 600 ft/s, N below 15, or s_u below 1,000 psf; also any profile with more than 10 feet of soft clay meeting plasticity, moisture and strength criteria.
- **F:** soils requiring site-specific evaluation, such as liquefiable or very sensitive soils.

Site factors. Three site factors scale the mapped values: F_{PGA} for PGA, F_a for the short-period value and F_v for the long-period value. They depend on site class and the amplitude of shaking. The manual's F_v table illustrates the pattern: for Site Class B, F_v is 1.0 at every level; for Site Class D it ranges from 2.4 at S_1 of 0.10 or less down to 1.5 at S_1 of 0.50 or more; for Site Class E, from 3.5 down to 2.4. Soft soils amplify long-period motion strongly at low shaking levels, and the amplification decreases as shaking increases because soil behavior becomes more nonlinear. Site Class F requires site-specific analysis.

Design values. The site-adjusted values are $A_s = F_{PGA}$ times PGA, $SDS = F_a$ times S_s , and $SD_1 = F_v$ times S_1 . The design spectrum rises from A_s at zero period to the plateau SDS , holds that plateau, and then decays in proportion to SD_1 divided by period beyond the corner period T_s , where T_s is SD_1 divided by SDS . The manual notes a minor plotting error in the AASHTO CD software (A_s plotted as F_a times PGA rather than F_{PGA} times PGA), with the tabulated value correct: a reminder to read the numbers, not just the plot.

Why SD_1 matters most. SD_1 includes both the regional hazard and the site amplification. The manual stresses that, because of this, a site on poor soil can fall in a higher seismic zone or category than a nearby site on firm soil, and that it is no longer

possible to map zones without considering soil conditions. The site class decision belongs to the geotechnical engineer, and it can change the entire design approach for a bridge.

Illustrative example. As an invented illustration, suppose a site has mapped $S_1 = 0.10$. On Site Class B rock, $F_v = 1.0$, so $SD_1 = 0.10$. On Site Class E soft soil, $F_v = 3.5$, so $SD_1 = 0.35$. Using the partition table in the next section, the same location moves from the lowest category to the third of four. Nothing about the earthquake changed; only the soil did.

Site-specific procedures. Both specifications allow a site-specific hazard or site response analysis instead of the general procedure, and require it in some cases (for example Site Class F). The manual's Chapter 2 discusses when these are warranted and the limits on reducing spectra below the general procedure.

Seismic zones and seismic design categories

Both AASHTO documents partition the country so that design effort matches hazard. FHWA-NHI-15-004 Section 8.2.2.1 explains that the partition uses SD_1 , the design spectral acceleration at 1.0 second. The LRFD Specifications call the partitions Seismic Zones 1 through 4; the Guide Specifications call them Seismic Design Categories (SDC) A through D. The boundaries are the same. As tabulated in the manual (Table 8-2):

- SD_1 up to 0.15: Zone 1, SDC A.
- 0.15 to below 0.30: Zone 2, SDC B.
- 0.30 to below 0.50: Zone 3, SDC C.
- 0.50 and above: Zone 4, SDC D.

The manual explains that SD_1 was chosen because one-second response is more representative of how bridges respond than the short-period plateau, and is consistent with a displacement-based approach. Using very short periods tends to produce high forces not reflective of the damage observed in past earthquakes.

What changes by category in the Guide Specifications. The manual's Table 8-10 summarizes the Guide Specification requirements by SDC. Paraphrased:

- **SDC A:** support connections and support length required; no demand analysis, capacity check, ductility detailing, capacity protection, P-delta check or liquefaction evaluation required.
- **SDC B:** global strategy and identification of the earthquake resisting system recommended; demand analysis and an implicit displacement capacity check required; SDC B ductility detailing; capacity protection and liquefaction evaluation recommended; minimum lateral strength required.
- **SDC C:** global strategy and ERS identification required; demand analysis and implicit capacity check required; SDC C detailing; capacity protection, P-delta, minimum lateral strength and liquefaction evaluation required.
- **SDC D:** as SDC C, except the displacement capacity must be determined by pushover analysis rather than implicit equations, and SDC D detailing applies.

Operational classification. The LRFD Specifications also classify bridges by importance, which the manual says used to be called Importance and is now Operational Classification. Critical bridges should remain open to all traffic after the design earthquake and be usable by emergency vehicles and for security and defense immediately after a large event (the manual gives a 2,500-year event as an example). Essential bridges should be open to emergency vehicles and for security and defense immediately after the design (1,000-year) earthquake. Other bridges are all the rest.

The manual is candid about the limits: the LRFD Specifications address Critical and Essential bridges mainly through more conservative R factors, and the Guide Specifications essentially cover only Other bridges. For a Critical or Essential bridge, the manual recommends developing site-specific and bridge-specific criteria with the owner so the desired performance is likely to be achieved. Caltrans' Seismic Design Criteria, from which the Guide Specifications' framework was adopted, likewise cover ordinary standard bridges.

Zone 1 and SDC A in practice. Much of the eastern and central U.S. is in Zone 1 or SDC A for firm sites. That does not mean seismic design is skipped. The manual describes two requirements that remain. First, minimum connection forces: for Zone 1, the horizontal connection between superstructure and substructure must be designed for a fraction of the tributary vertical reaction; the manual states 0.15 times the reaction where A_s is less than 0.05, and 0.25 otherwise. Second, minimum support lengths. These two provisions address the failure modes, unseating and connection failure, that have caused collapses even at modest shaking levels.

Two design methods: force-based and displacement-based

FHWA-NHI-15-004 Chapter 7 describes the two methods currently available. Both use the same hazard and both aim at life safety and no collapse. They differ in how they treat the yielding elements.

Force-based method (FBM), LRFD Specifications. The designer performs an elastic analysis with the design spectrum to get elastic forces in the columns and piers. Those elastic moments are divided by a response modification factor R to get design moments, and the columns are designed for that reduced force. R represents the ductility the designer expects the column to supply. The manual reproduces the LRFD substructure R factors (its Table 8-7, from LRFD Table 3.10.7.1-1). For Other bridges: wall-type piers in the large dimension, 2.0; reinforced concrete pile bents with vertical piles only, 3.0, and with batter piles, 2.0; single columns, 3.0; steel or composite pile bents with vertical piles, 5.0, and with batter piles, 3.0; multiple column bents, 5.0. For Critical bridges every value is 1.5. Essential bridges fall between (for example 2.0 for single columns and 3.5 for multiple column bents). Connections use different factors (Table 8-8): 0.8 for superstructure to abutment and for expansion joints within a span; 1.0 for columns, piers or pile bents to cap beam or superstructure, and for columns or piers to foundations. Factors below 1.0 mean connections get more than the elastic force.

Because R is one-size-fits-all within a category, the manual notes, detailing must also be worst-case: the same transverse reinforcement rules apply wherever the R factor is used, because the method does not quantify how much ductility a particular column actually needs.

Displacement-based method (DBM), Guide Specifications. Here, the R factor for ductile concrete columns is replaced by a direct comparison of displacement demand with displacement capacity. The designer runs an elastic analysis (with cracked section properties) to estimate displacement demand, which by the equal-displacement observation for moderate-to-long period structures approximates the inelastic displacement. Displacement capacity is then found either from implicit equations (SDC B and C) or a pushover analysis based on moment-curvature behavior (SDC D). If capacity exceeds demand, the column works. The manual notes that in the lower SDCs, lower permissible demands allow more liberal transverse steel and detailing, while in SDC D a direct quantitative link is provided between transverse steel and permissible displacement: higher demands can be accepted, at the cost of more confinement.

Implicit capacity. The manual explains that the implicit displacement capacity equations, developed by Imbsen (2006) from Berry and Eberhard's (2003) column damage research on hundreds of tested columns, give a simple conservative estimate of displacement capacity for reinforced concrete columns as a function of column aspect ratio and fixity. They can serve as reasonableness checks regardless of which specification was used for design.

Ductility limits. Where local member ductility demand is checked, the manual lists (Chapter 7): ductility demand not exceeding 5 for single column bents, 6 for multiple column bents, and 4 where in-ground plastic hinging is anticipated. It describes full-ductility structures in high seismic regions as designed for demands in the range of 4 to 6, and limited-ductility structures in lower regions for demands of 4 or less.

Damping. The Guide Specifications permit damping above 5% under limited conditions. The manual lists them: total bridge length under 300 feet, abutments designed for sustained soil mobilization, supports normal or slightly skewed (less than 20 degrees), and a continuous superstructure without hinges or joints, or a demonstrated reliable load path to the abutments.

Which to use. Both are acceptable AASHTO approaches; the choice is usually set by the owner. The manual's Table 8-1 lists six steps that both share: determine the seismic input, establish design procedures, identify the earthquake resisting system and global design strategy, perform the demand analysis, design and check the earthquake resisting elements, and apply capacity protection.

Earthquake resisting systems and global design strategies

Before any analysis, the designer must decide how the bridge is supposed to behave in an earthquake. FHWA-NHI-15-004 Chapter 4 calls this the structural form: geometry, structural type, member strengths and articulation. The Guide Specifications formalize it with Earthquake Resisting Elements (ERE), Earthquake Resisting Systems (ERS) and Global Design Strategies. The manual stresses that although they first appeared in the Guide Specifications, the concepts apply to any bridge regardless of the specification used, and they are especially useful at the type, size and location (TS&L) stage.

Three ERE categories. The Guide Specifications list elements as Permissible, Permissible that Require Owner's Approval, or Not Recommended for New Bridges. The manual traces these listings to the 1986 FHWA manual on seismic design and retrofit (Buckle and others). The guiding principle is to keep potential damage where it can be inspected. Plastic hinges that form above ground are permissible. Hinges just below ground, accessible by reasonable excavation, are also permissible. Hinges that form below the ground line, such as in piles beneath a wall pier, piles supporting an integral abutment, or pile shafts with the same strength as the extension above ground, require owner approval. They are not prohibited, because in-ground hinging is sometimes unavoidable, but they are discouraged because the damage may be impossible to detect and likely impossible to repair. The owner needs to understand the post-earthquake inspection and repair consequences.

Permissible systems. The manual's Table 4-4 lists permissible earthquake resisting systems, for example: longitudinal response with plastic hinges in inspectable locations (or elastic columns), with the abutment not required as part of the ERS and knock-off backwalls permissible; isolation bearings accommodating the full displacement; multiple simply supported spans with adequate support lengths; transverse response with breakaway shear keys at abutments; or abutments required to resist the earthquake elastically, with longitudinal passive pressure less than 70% of the maximum that can be mobilized.

Global design strategies. The Guide Specifications list three:

- **Type 1**, ductile substructure with essentially elastic superstructure. This is the conventional approach: columns or pier walls form plastic hinges; the superstructure, cap beams, joints and foundations stay essentially elastic.
- **Type 2**, essentially elastic substructure with ductile superstructure. This applies mainly to steel superstructures where ductile end cross-frames or diaphragms yield.
- **Type 3**, elastic superstructure and substructure with a fusing interface. Isolation bearings or other fusing elements between the superstructure and substructure limit forces.

The manual describes a fourth approach: design the whole structure to remain elastic. In low seismic regions, resisting earthquake loads elastically without explicitly building in ductility is a valid approach, and the manual discusses it first because of its simplicity.

Do not mix strategies at a bent. As a general rule, the manual says, strategies are not mixed. A ductile superstructure would not be combined with a ductile substructure at the same bent, because the response becomes too complex for the elastic and nonlinear static methods commonly used. Under the right conditions, such as limited curvature, one strategy may be used longitudinally and another transversely.

Simplicity, integrity and symmetry. Chapter 4 also emphasizes conceptual design principles: a clear, justifiable load path; continuity so the bridge holds together; and balanced stiffness. Bents of very different stiffness in the same frame attract force to the short, stiff ones, which then yield first and may be overloaded. Good structural form makes analysis easier and performance more predictable, which is why the strategy decision belongs at TS&L, not after the plans are drawn.

Demand analysis

Demand analysis estimates the seismic response, forces and displacements, of the bridge under the design spectrum. FHWA-NHI-15-004 Chapter 5 and Section 8.4.4 describe the methods and modeling choices.

Analysis methods. The manual describes, in increasing rigor: the uniform load method and single-mode spectral method (equivalent static approaches suited to regular bridges whose response is dominated by one mode); multimode spectral analysis, which combines the responses of several modes and is required for irregular bridges in higher categories; and nonlinear time-history analysis, used for critical or unusual bridges. The specifications define regular bridges by limits on span count, span length ratios, stiffness ratios between bents and curvature; bridges outside those limits need more rigorous analysis. Requirements also depend on zone or SDC and operational class. Consult the current specification for the exact limits.

Global model. In the Guide Specifications, the manual explains, a global model should capture the response of the whole bridge system. For irregular geometry, especially curved and skewed bridges, actual geometry should be modeled. Multiple transverse expansion joints, massive substructures and foundations on soft soil can produce dynamic behavior that a separate subsystem analysis would miss. For bridges of multiple simple spans, displacement demands may come from an equivalent static analysis.

Long bridges. For long bridges, the manual cautions that a finite element model of the entire length may not be warranted and can give questionable results, partly because out-of-phase ground motion is expected while multimode analysis assumes equal input at every support. It recommends models of no more than five frames plus boundary frames or an abutment, with each multi-frame model overlapping the next by at least one usable frame, while noting this is a recommendation to be evaluated case by case.

Stiffness assumptions. Column stiffness in demand analysis uses cracked section properties, because a column near yield is cracked. Using gross properties overestimates stiffness, which underestimates displacement and generally overestimates force. Foundation flexibility, abutment stiffness and soil springs also matter; the Guide Specifications provide recommendations for modeling soil-structure interaction at abutments and foundations.

Combination of directions. Because the earthquake can come from any direction, results from the two horizontal directions are combined, conventionally with the 100% plus 30% rule, so each member is checked for the full response in one direction plus 30% of the response in the orthogonal direction, and vice versa.

The equal-displacement idea. For structures with moderate and long periods, research has shown that the peak displacement of a yielding system is about equal to that of an elastic system with the same initial stiffness. That observation is what allows an elastic analysis to estimate inelastic displacement demand. For short-period structures, inelastic displacement exceeds elastic displacement, and the specifications include a magnification factor for that case. The R-factor approach in the force-based method relies on the same idea: dividing elastic force by R is consistent with a ductility demand of about R for longer-period structures.

Illustrative example. As an invented illustration, a two-span bridge with a single-column pier has a cracked-section period of 0.9 s in the longitudinal direction. With $SD1 = 0.40$ and a corner period below 0.9 s, the spectral acceleration is $0.40/0.9$, or about 0.44 g. If the participating weight is 2,000 kips, the elastic base shear is about 890 kips. Under the force-based method, with $R = 3.0$ for a single column (Other bridge), the column is designed for about 300 kips of shear-equivalent moment demand. Under the displacement-based method, the designer would instead compute the elastic displacement from the same spectrum and compare it to the column's displacement capacity.

Capacity design and overstrength

Capacity design is the most important idea in modern seismic design. FHWA-NHI-15-004 describes it in Chapters 4, 7 and 8. The designer chooses where yielding will occur, details those locations for ductility, and then designs every other element in the load path to be stronger than the force the yielding elements can deliver. The yielding elements act like fuses: once they yield, the force in the rest of the structure cannot grow much further.

Capacity-protected elements. For a conventional Type 1 strategy, columns are the fuses. Cap beams, superstructure, joints, footings, pile caps, piles and shear in the columns themselves are capacity protected. The manual explains that capacity-protected members are designed to remain essentially elastic when the plastic hinge reaches its overstrength moment. All loads on the capacity-protected member are considered, including the axial forces from lateral overturning, such as tension in columns or uplift in piles.

Why overstrength. The actual moment a column can develop exceeds its nominal design strength. Reasons include actual material strengths above specified values, strain hardening of reinforcement at large curvature, and confinement increasing concrete strength. If the capacity-protected elements were designed only for the nominal column moment, the column could deliver more than they can carry, and the brittle element would fail first. Overstrength factors provide an upper bound to the force the fuse can transmit.

LRFD values. In the LRFD Specifications, as the manual describes for Zone 2 and higher, an estimate of column overstrength uses a factor of 1.3 for reinforced concrete columns and 1.25 for structural steel columns. In Zone 2, the LRFD Specifications take a simplified path: foundation design forces are the elastic forces divided by half of R, with $R/2$ not taken less than 1.0, which essentially doubles foundation forces relative to column forces. The manual notes this is generally conservative but warns that if seismic forces do not govern the column design, unintended column overstrength could exceed foundation capacity.

Guide Specification values. In the Guide Specifications, the manual explains, an overstrength magnifier is applied to the column's idealized plastic moment, which is itself based on expected (not specified) material properties: $M_{po} = \lambda_{mo}$ times M_p . The magnifier is 1.2 for ASTM A706 Grade 60 reinforcement and 1.4 for ASTM A615 Grade 60. A706 has controlled chemistry and limits on actual yield strength, so its overstrength is more predictable. Capacity-protected members use their expected nominal capacity, M_{ne} , computed from a strain compatibility moment-curvature analysis at a concrete strain of 0.003 with expected material strengths.

Column shear. A column that yields in flexure must not fail in shear first. The design shear is therefore the shear associated with the overstrength moment, not the elastic analysis shear. For a column fixed at both ends, that is approximately twice the overstrength moment divided by the clear height. Short, stiff columns attract the largest shears, which is one reason the manual's earthquake history includes so many short-column shear failures.

Illustrative example. As an invented illustration, a column reinforced with A706 bars has an idealized plastic moment of 4,000 kip-ft at top and bottom, with a clear height of 20 ft. The overstrength moment is 1.2 times 4,000, or 4,800 kip-ft. The capacity-protection shear is about 2 times 4,800 divided by 20, or 480 kips. The cap beam, joint and footing are designed for the 4,800 kip-ft moment delivered at the column face, plus their other concurrent loads, using expected capacities. If the bars were A615, the magnifier would be 1.4 and the overstrength moment 5,600 kip-ft, a 17% increase in the demand on everything around the column.

Minimum lateral strength and P-delta. The Guide Specifications also require a minimum lateral strength for columns and, in SDC C and D, a P-delta check that limits the moment from the dead load acting through the lateral displacement to a fraction of the column's plastic moment, so the column does not become unstable after it yields.

Support length and connections

The most common cause of bridge collapse in past earthquakes is a span falling off its support. FHWA-NHI-15-004 treats support length as a common feature of both specifications, required in every zone and category.

Terminology. The manual notes that "support length" replaced the older term "seat width." Seat width implies the full width of the supporting face; support length is the length of overlap available to support the span, measured normal to the centerline of bearing.

Empirical equation. The manual gives the LRFD empirical support length (its Eq. 8-1) as $N = (8 + 0.02L + 0.08H)(1 + 0.000125S^2)$, in inches, where L is the length of deck to the adjacent expansion joint or end of the deck in feet (for hinges within a span, the sum of distances to either side; for single spans, the deck length); H is the average height of columns supporting the deck to the next expansion joint for abutments, the column or pier height for piers, the average of the two adjacent columns or piers for in-span hinges, and zero for single-span bridges, in feet; and S is the skew in degrees.

Percentages by zone. In the LRFD Specifications, as tabulated in the manual (Table 8-5), N is modified: at least 75% for Zone 1 sites with A_s below 0.05; 100% for Zone 1 sites with A_s from 0.05 to 0.15; and 150% for Zones 2, 3 and 4. In other words, even the lowest-hazard bridges need most of the basic seat length.

What the equation does not include. The manual stresses that the actual overall support length must be larger than N by the movements due to prestress shortening, creep, shrinkage and thermal expansion and contraction. N is the seismic allowance on top of service movements.

Restrainers. Expansion bearings without restrainers, shock transmission units or dampers must meet the support length; otherwise, longitudinal restrainers are required. The Guide Specifications have their own support length provisions, including a displacement-based check in higher categories.

Illustrative example. As an invented illustration, an abutment supports a deck 240 ft long to the next joint, with average column height 25 ft and skew 30 degrees. The basic N is $(8 + 4.8 + 2.0)(1 + 0.000125 \text{ times } 900) = 14.8 \text{ times } 1.1125$, or about 16.5 inches. In Zone 1 with A_s of 0.08, 100% applies: 16.5 inches plus service movements. In Zone 3, 150% applies:

about 24.7 inches plus service movements. Skew increases the requirement because skewed spans tend to rotate in plan and walk off the acute corners.

Connections. Connections between superstructure and substructure must transfer the seismic force or be designed to fuse in a controlled way. In Zone 1, as described above, a minimum connection force of 0.15 or 0.25 times the tributary vertical reaction applies. In higher zones the LRFD connection R factors of 0.8 and 1.0 apply, and in capacity design, connections are designed for the force the fuse delivers. Shear keys at abutments may be designed as sacrificial (breakaway) elements, which is permitted in the Guide Specifications' permissible systems, but then the superstructure's restraint after the key breaks must be considered.

Inspection relevance. For inspectors and raters, support length is one of the few seismic features visible in the field. Narrow seats, missing or corroded restrainers, frozen or tipped rocker bearings, and severely skewed supports are the details that matter. BR-B01 covers bearings and BR-E02 covers screening existing bridges for these vulnerabilities.

Detailing for ductility

Ductility does not come from analysis. It comes from detailing. FHWA-NHI-15-004 Chapter 6 summarizes the large-scale laboratory testing that underpins the provisions and the local failure mechanisms that detailing must prevent. The key concrete column details are these.

Confinement. Closely spaced spirals, hoops or ties in the plastic hinge region confine the core concrete. Confinement raises the concrete's strain capacity, allowing the large curvatures that plastic hinging requires. Without it, cover spalls and the core crushes at small rotations. The Guide Specifications link the required confinement to the SDC; in SDC D, the manual explains, it is directly linked to the displacement demand.

Longitudinal bar buckling. Once cover spalls, longitudinal bars in compression can buckle between transverse bars. Close spacing of transverse reinforcement, expressed as a limit relative to the longitudinal bar diameter, prevents this. Berry and Eberhard's fragility work, cited in the manual, used cover spalling and bar buckling as the two damage states for the implicit displacement capacity equations.

Shear. Transverse reinforcement also provides shear resistance. Within the plastic hinge, concrete's shear contribution degrades with ductility demand, so more of the shear must be carried by steel.

Splices. Lap splices of longitudinal reinforcement in the plastic hinge zone are a classic failure mode, and current provisions restrict splicing in those regions. BR-E02 discusses how pre-1971 columns with starter-bar lap splices at the base are among the most common retrofit targets.

Anchorage. Column bars must be fully developed into footings and cap beams so the hinge can form at the face of the joint. Joints must be proportioned and reinforced to transfer the overstrength moments without failure; joint shear failures were observed in past earthquakes.

Plastic hinge length. The region over which confinement is required, often called the plastic hinge region, extends a specified distance from the face of the joint. Its length depends on the column dimension and height. Confinement must continue some distance into the adjacent joint.

Steel substructures. For steel yielding elements, the manual lists slenderness ratios, net section limits relative to gross section, welding details and filler metal as the equivalent detailing concerns. For Type 2 strategies, ductile end diaphragms are

detailed to yield in a controlled way.

Liquefaction and foundations. Detailing extends to foundations. In higher categories the Guide Specifications require evaluation of liquefaction and its consequences (loss of bearing, lateral spreading, downdrag), and the manual describes foundation and abutment design requirements. Liquefaction is listed in the SDC matrix as recommended in SDC B and required in C and D.

Takeaway for reviewers. When reviewing plans for a bridge in a higher category, the most important sheets for seismic performance are often the column and joint details: spiral pitch, splice locations, bar anchorage, confinement extent and seat dimensions. These are also where field substitutions and RFIs can quietly undermine the design intent. Engineers reviewing shop drawings and change requests in seismic regions should treat changes to these details as design changes requiring the engineer of record's review.

Isolation, protective systems and putting it together

Seismic isolation. FHWA-NHI-15-004 devotes a chapter to seismic isolation, which corresponds to the Type 3 strategy. Isolation bearings, such as lead-rubber or friction pendulum bearings, lengthen the period of the bridge and add damping. Lengthening the period moves the bridge down the decaying part of the spectrum, reducing force; added damping reduces displacement. The substructure can then stay elastic. The cost is larger displacements at the isolation level, which must be accommodated by joints, seats and utilities. Isolation is most effective for stiff bridges on firm sites, where the unisolated period is short and the spectrum is high. On soft sites, where the spectrum may still be high at long periods, the benefit can be smaller. The manual's chapter on isolation describes the AASHTO Guide Specifications for Seismic Isolation Design; that document was not obtained for this course.

Putting the steps together. A designer working through a new bridge, following the manual's six steps, would typically:

1. Obtain the mapped values for the site and the site class from the geotechnical engineer; compute A_s , SDS and $SD1$; determine zone or SDC.
2. Confirm the operational class with the owner and which specification and state criteria apply.
3. Choose the global design strategy and ERS at TS&L: where the hinges will be, whether the abutments participate, whether isolation is worth considering.
4. Build the analysis model with cracked section properties and appropriate boundary conditions; perform the required level of demand analysis; combine directions.
5. Design and check the earthquake resisting elements: R-factor design (LRFD) or displacement capacity versus demand (Guide Specifications), with the required detailing.
6. Capacity protect everything else with overstrength forces; check support lengths, connections, P-delta and minimum strength; evaluate liquefaction if required.

Common errors. Based on the principles in the manual, frequent problems in seismic design include: using gross section stiffness and underestimating displacement; omitting the site class amplification or using the wrong site class; designing cap beams and footings for analysis forces rather than overstrength forces; forgetting the axial force from overturning in pile design; inadequate seats at in-span hinges; mixing ductile elements at the same bent; and specifying A615 bars in a design that assumed the A706 overstrength magnifier.

For raters and inspectors. Seismic design is not part of the routine load rating covered in BR-L01. However, the same bridge's seismic vulnerability may matter for asset management and risk planning; BR-M01 notes that federal asset

management rules require states to consider seismic activity in risk management and life-cycle planning. For existing bridges, FHWA's Seismic Retrofitting Manual provides a separate, performance-based process, covered in BR-E02.

Where to verify. This course has reported provisions as FHWA-NHI-15-004 summarized them in 2014. AASHTO revises its specifications periodically, and state DOTs, including those in moderate seismic regions such as South Carolina, may publish seismic design specifications that modify or replace the AASHTO provisions. Before using any value from this course, check the current AASHTO edition and the owner's criteria.

Key takeaways

- The current AASHTO design earthquake, as described in FHWA-NHI-15-004, has a 7% probability of exceedance in 75 years (about a 1,000-year return period), with NEHRP site classes used for site effects; the change from the earlier 500-year hazard came in 2007.
- Seismic Zones 1 to 4 (LRFD) and SDC A to D (Guide Specifications) both partition on SD1 at 0.15, 0.30 and 0.50, and SD1 includes site amplification, so soft soils can raise the category.
- The LRFD Specifications use a force-based method with R factors; the Guide Specifications use a displacement-based method comparing displacement demand to implicit or pushover displacement capacity.
- Choose a global design strategy and keep plastic hinges in inspectable locations; in-ground hinging requires owner approval.
- Capacity design protects everything around the fuse using overstrength (1.3 for RC columns in LRFD; 1.2 for A706 and 1.4 for A615 in the Guide Specifications).
- Minimum support length and connection forces apply in every zone, including Zone 1, because unseating is the most common collapse mode.
- Ductility comes from detailing: confinement, bar buckling restraint, shear capacity, splice location, anchorage and joint design.

References

1. FHWA-NHI-15-004, LRFD Seismic Analysis and Design of Bridges Reference Manual (Marsh, Buckle and Kavazanjian, October 2014)

Study text only; the final quiz is taken online. Verify all references against the edition your agency has adopted.