

Seismic Retrofit of Existing Bridges

Most of the nation's bridges were designed before modern seismic provisions existed. FHWA's Seismic Retrofitting Manual for Highway Structures: Part 1, Bridges (FHWA-HRT-06-032, January 2006, by Buckle, Friedland, Mander, Martin, Nutt and Power) gives owners a performance-based process for deciding which existing bridges need attention, how to evaluate them, and what to do about it. This course follows that process: the lessons of past earthquakes, the two-level ground motion and performance framework, service life and importance, seismic hazard levels and seismic retrofit categories, screening and prioritization with the indices method, evaluation methods from simple capacity checks to nonlinear analysis, the vulnerable details that cause most failures, and the retrofit approaches and measures available, including seat extensions, restrainers, column jackets and foundation work, with cost data the manual reports from California.

Learning objectives

- Identify vulnerable details
- Explain screening and evaluation
- Select retrofit measures
- Prioritize retrofit

Why retrofit, and what history taught

A new bridge can be designed to resist earthquakes from the start. An existing bridge carries whatever details were standard when it was built, and before the 1970s those details reflected almost no understanding of how bridges behave in strong shaking. FHWA-HRT-06-032, the Seismic Retrofitting Manual for Highway Structures, Part 1: Bridges, was written to give owners a systematic way to find and fix the most dangerous of those bridges.

The earthquake record. The manual draws on a series of U.S. earthquakes that each exposed different weaknesses:

- **1971 San Fernando.** Several freeway bridges collapsed, mainly from unseating of spans and from columns that lacked confinement and shear strength. This event launched the first generation of retrofit work: California's restrainer program, which tied adjacent spans together at hinges and joints to keep them from falling off their seats.
- **1987 Whittier Narrows.** Damage to columns highlighted their shear vulnerability and helped motivate column retrofit.
- **1989 Loma Prieta.** The double-deck Cypress Viaduct in Oakland collapsed, killing motorists. The manual notes that the structure had been fitted with restrainers, showing that restrainers alone could not prevent failure when the columns and joints were themselves deficient. Loma Prieta greatly expanded California's retrofit program to columns, joints and foundations.
- **1994 Northridge.** Bridges that had been retrofitted generally performed well, while several unretrofitted bridges failed. The manual cites this as evidence that retrofitting works.

The manual reports that about 2,200 bridges were retrofitted in California following these events. Other states with significant seismic hazard, and international experience from earthquakes such as Kobe (1995), also inform the manual's guidance.

Why it matters outside California. Moderate seismic hazard exists across much of the country, including the central U.S. near the New Madrid seismic zone and the Southeast near Charleston, South Carolina. Lower-probability events can still be damaging because older bridges in those regions were designed with even less seismic consideration. The manual is written

for all owners, with categories that scale effort to hazard, so a bridge in a low hazard region may need little more than a check of seats and connections.

The retrofit decision is a management decision. The manual emphasizes that retrofit is not just engineering. Owners have far more vulnerable bridges than they can fix, so they need a process to rank them, decide what level of performance is required, and choose between retrofitting, replacing, accepting the risk, or closing the bridge. The rest of this course walks through that process.

Scope. This course is a primer based solely on FHWA-HRT-06-032. Part 1 covers bridges; Part 2 of the series covers retaining structures, slopes, tunnels, culverts and roadways and is outside this course. BR-E01 covers new bridge design under the AASHTO specifications; many concepts, such as capacity design and ductile detailing, carry over directly.

Performance-based framework: ground motions, service life and importance

The 2006 manual introduced a performance-based framework. Instead of a single design earthquake, it uses two ground motion levels, and the required performance depends on the bridge's importance and anticipated remaining service life.

Two ground motions.

- **Lower level:** a ground motion with a 50% probability of exceedance in 75 years, a return period of about 100 years. This is an event the bridge will likely experience during its life.
- **Upper level:** a ground motion with a 7% probability of exceedance in 75 years, about 1,000 years. This matches the design event for new bridges described in BR-E01.

Anticipated service life. Retrofit investment should match how long the bridge will remain in service. The manual defines three categories:

- **ASL 1:** 0 to 15 years.
- **ASL 2:** 16 to 50 years.
- **ASL 3:** more than 50 years.

New bridges would generally be ASL 3. A bridge scheduled for replacement within 15 years is ASL 1.

Importance. Bridges are classified as standard or essential. The manual's criteria for an essential bridge include a bridge that is required to provide secondary life safety (for example, access to an emergency facility), a bridge whose loss would create a major economic impact, a bridge formally designated as critical in a local emergency plan, and a bridge on the Strategic Highway Network (STRAHNET). Classification is the owner's decision, often made in coordination with emergency planners.

Performance levels. The manual defines four:

- **PL0:** no minimum performance.
- **PL1:** life safety. Significant damage is sustained, and service is significantly disrupted, but life safety is preserved. The bridge may need to be replaced.
- **PL2:** operational. Damage sustained is minimal and full service for emergency vehicles should be available after inspection and clearance of debris. The bridge should be repairable with or without restrictions on traffic flow.
- **PL3:** fully operational. Damage sustained is negligible and full service is available for all vehicles after inspection and clearance of debris; any damage is repairable without interruption to traffic.

The manual defines its damage terms. **Minimal damage** includes minor inelastic response and narrow flexural cracking, with no apparent permanent deformation and repairs possible under non-emergency conditions (expansion joints excepted).

Significant damage includes permanent offsets and cracking, yielded reinforcement and major spalling that may require closure to repair; partial or complete column replacement may be needed; beams may be unseated from bearings, but no span should collapse.

Minimum performance levels (Table 1-2). The manual combines these to set the minimum performance for each case:

- ASL 1 bridges, standard or essential: PL0 for both ground motions. In practice the manual treats these bridges as exempt from retrofit requirements (discussed below).
- Standard bridges, ASL 2 or 3: PL3 for the lower-level event and PL1 for the upper-level event.
- Essential bridges, ASL 2: PL3 lower, PL1 upper.
- Essential bridges, ASL 3: PL3 lower, PL2 upper.

The pattern: every bridge that will remain in service more than 15 years should be fully operational after the frequent event and should not kill anyone in the rare event. Only long-life essential bridges must be operational after the rare event. Owners may choose higher levels.

Exempt bridges. The manual exempts bridges with an anticipated service life of 15 years or less, temporary bridges, and bridges closed to traffic with nothing active beneath them. An owner may still choose to evaluate them.

Why the lower level matters. Old seismic retrofit practice focused only on the large event. The manual's lower-level check asks a different question: will a frequent, moderate earthquake damage the bridge enough to close it? In moderate hazard regions, the lower-level event may produce small forces that many bridges can resist simply because of their wind and braking capacity, which leads to the screening check in a later section.

Seismic hazard levels and retrofit categories

Effort should scale with hazard. FHWA-HRT-06-032 does this with seismic hazard levels and seismic retrofit categories (SRC).

Seismic hazard levels (Table 1-5). The manual assigns one of four hazard levels using the site-adjusted spectral accelerations, SD1 (1.0 second) and SDS (short period):

- **Level I:** SD1 up to 0.15 or SDS up to 0.15.
- **Level II:** SD1 from 0.15 to 0.25, SDS from 0.15 to 0.35.
- **Level III:** SD1 from 0.25 to 0.40, SDS from 0.35 to 0.60.
- **Level IV:** SD1 above 0.40, SDS above 0.60.

The hazard level is evaluated separately for each ground motion level. A site might be Level I for the lower-level event and Level III for the upper-level event. Note that these boundaries are not the same as the Seismic Zone or SDC boundaries for new design described in BR-E01. They come from the retrofit manual and should not be mixed with the design provisions.

Seismic retrofit categories (Table 1-6). The hazard level and the required performance level together set the SRC, from A (least effort) to D (most):

- For the upper-level event: at PL0, every hazard level gives SRC A. At PL1 (life safety), hazard Level I gives A, Levels II and III give B, and Level IV gives C. At PL2 (operational), Levels I and II give B, Level III gives C and Level IV

gives D.

- For the lower-level event: at PL0, every hazard level gives A. At PL3 (fully operational), Levels I through III give C and Level IV gives D.

Minimum requirements by SRC (Table 1-7). The category then sets the minimum screening, evaluation and retrofit requirements. For the upper-level event, as tabulated in the manual: SRC A requires nothing (not required). SRC B screens seat widths, connections and liquefaction, evaluates with Method A1/A2 (minimum capacity checks, no analysis), and retrofits seats, connections and liquefaction if deficient. SRC C adds columns, walls and footings to screening and retrofit, with evaluation by Method B, C, D1 or D2. SRC D adds abutments, with evaluation by Method C, D1, D2 or E. For the lower-level event, SRC B is not used; SRC C and D screen seats, connections, columns, walls, footings and liquefaction (plus abutments in D), evaluate with Method C, and retrofit deficient components. The manual describes the lower-level process as a force-based approach in which displacements are assumed small, reasonable provided yielding does not occur, bearing restraints do not fail and soils do not soften.

Illustrative example. As an invented illustration: a standard bridge with ASL 2 at a site with upper-level SD1 of 0.30 and SDS of 0.50 is hazard Level III for the upper event. Required performance is PL1, so the upper-level SRC is B. If its lower-level SD1 is 0.06 and SDS 0.12, it is Level I for the lower event, and with PL3 required, the lower-level SRC is C, which in practice is handled by the screening check described in the next section. If the same bridge were essential with ASL 3, the upper-level requirement would become PL2, and the SRC would rise to C.

Why categories matter for programs. SRC A bridges, which may be a large share of an inventory in a low hazard state, need only seat and connection checks. That makes it practical to sweep a whole inventory for the cheapest, highest-value fixes: span unseating is the most common cause of collapse, and seat extensions and connection improvements are comparatively inexpensive.

Lower-level event screening

The lower-level ground motion has a return period of about 100 years. FHWA-HRT-06-032 provides a simple first check for it, based on the observation that a bridge already resists lateral forces from wind and braking.

The check. The manual estimates the lower-level seismic force as $F = \text{SDS} \times W$ (its Eq. 1-2), where W is the weight of the structure (or the part being checked) and SDS is the short-period spectral acceleration for the lower-level event. This is a conservative upper bound because it assumes the bridge responds on the spectral plateau. F is compared with the factored wind load in the transverse direction and the braking force in the longitudinal direction that the bridge was designed for. If the seismic force is less than those design forces, the bridge is assumed to satisfy the lower-level requirement without further analysis, since its lateral load path was already designed for a larger force.

Refinement. If the simple check fails, the manual allows refinement using the actual period of the bridge: longitudinal and transverse forces $FL = SaL \times W$ and $FT = SaT \times W$, where SaL and SaT are the spectral accelerations at the longitudinal and transverse periods. Flexible bridges with longer periods have lower spectral accelerations and may pass on refinement. If they still fail, a more detailed evaluation is needed.

Illustrative example. As an invented illustration: a simple-span bridge weighing 800 kips at a site with lower-level SDS = 0.05 has $F = 40$ kips. If the factored transverse wind force used in design was 60 kips, the transverse check passes. If the braking force used longitudinally was 30 kips, the longitudinal check fails, and the engineer would check the longitudinal period. If the longitudinal period is 1.2 s and the lower-level SD1 is 0.03, SaL is 0.025 and $FL = 20$ kips, which passes.

Limits of the check. Passing this screen does not mean the bridge is safe in the upper-level event. The screen only addresses the frequent event, and only for force demand on the lateral load path. It also assumes that the wind and braking load paths are actually present in the bridge as built, including bearings, keepers and anchor bolts in good condition. Deterioration found during inspection, such as corroded anchor bolts or frozen bearings, can undermine that assumption.

Where it fits. In low hazard regions, the lower-level check combined with the SRC A seat and connection checks may be all that is required. That gives owners in moderate and low seismic states a manageable first step: inventory-wide screening using data they already have.

Screening and prioritization

An owner may have thousands of bridges that need evaluation. FHWA-HRT-06-032 describes a screening process to rank them so the most vulnerable and important bridges are evaluated and retrofitted first.

Three prioritization methods. The manual presents three:

- **Indices method.** A simple ranking based on a vulnerability index and a seismic hazard index. This is the method most often used and is described below.
- **Expected damage method.** Uses fragility relationships to estimate the expected damage to each bridge, giving a damage-based ranking.
- **Risk assessment method.** Estimates the expected losses, including direct and indirect costs, from damage to the bridge, giving a ranking based on risk. This method can incorporate network effects and is the most data-intensive.

The indices method. The bridge rank is $R = V \text{ times } E$, where V is a vulnerability rating and E is a seismic hazard rating, each from 0 to 10. R therefore ranges from 0 to 100. The vulnerability rating is $V = \text{the larger of } V1 \text{ and } V2$:

- **V1** covers connections, bearings and seat widths: the likelihood of unseating. It is driven by bearing type, seat width compared with the required minimum, and connection details.
- **V2** covers columns, abutments, foundations and soils: $V2 = CVR + AVR + LVR$, not exceeding 10, where CVR is the column vulnerability rating, AVR the abutment vulnerability rating and LVR the liquefaction vulnerability rating.

The seismic hazard rating includes both seismicity and site conditions through SD1: the manual's Eq. 4-8 sets $E = 10 \text{ times } SD1$, not more than 10. Where soils are not known well enough to set the site class, F_v is based on engineering judgment. In the manual's worked examples, a simply supported bridge ranked $R = 43$ and a continuous bridge $R = 33$; the difference was smaller than expected because the continuous bridge sat on steel bearings mounted on pedestals, from which the beams could come off despite continuity.

Other factors. The rank is not the whole decision. The manual lists other factors that owners consider in prioritization: importance (essential or standard), network redundancy (whether alternate routes exist), the age and condition of the bridge (a deteriorated bridge may be better replaced than retrofitted), and non-seismic deficiencies that might be addressed at the same time. The final priority combines the rank with these factors through engineering judgment and owner policy.

Illustrative example. As an invented illustration: Bridge 1 has steel rocker bearings on narrow seats ($V1 = 8$), well-confined columns ($V2 = 3$), and hazard $E = 5$ ($SD1 = 0.50$), giving $V = 8$ and $R = 40$. Bridge 2 has adequate seats ($V1 = 2$) but pre-1971 columns with lap splices in the hinge zone and potentially liquefiable soils ($V2 = 9$), with $E = 7$ ($SD1 = 0.70$), giving $R = 63$. Bridge 2 ranks higher. However, if Bridge 1 is essential and Bridge 2 has a nearby alternate route, the owner might

advance Bridge 1. Also, Bridge 1's deficiency might be fixed cheaply with seat extensions, while Bridge 2 may need a major foundation retrofit or replacement.

Data. Screening depends on inventory data: bearing types, seat widths, column details, foundation types and soil information. Much of this is not in the NBI. Owners often rely on plan review supplemented by field verification. Inspectors can contribute by documenting seat dimensions, bearing condition and tilt, and restrainer presence and condition. BR-B01 covers bearing inspection.

Evaluation methods

Once a bridge is selected, it is evaluated to determine whether it meets the required performance and, if not, which components are deficient. FHWA-HRT-06-032 offers several methods, with the minimum method set by the SRC.

Method A1 and A2: minimum capacity checks. No demand analysis is required; connection capacity and seat width are checked against minimum values. The manual describes the method as suitable for all single-span bridges and others in low hazard zones. Method A1 may be used where the short-period spectral ordinate S_s is less than 0.10 and requires connections to resist at least 10% of the tributary dead load reaction in their restrained directions; otherwise Method A2 applies, with 25%. Longitudinally, the tributary dead load at fixed bearings is the entire continuous segment; transversely it is the dead load reaction at each bearing. The minimum seat width is computed with the manual's Eq. 5-1b: $N = [4.0 + 0.02L + 0.08H + 1.1H \text{ times the square root of } (1 + (2B/L) \text{ squared})] \text{ times } (1 + 1.25 F_v S_1) \text{ divided by } \cos \alpha$, in inches, where L is the length of the deck to the next joint in feet, H the tallest pier between the joints in feet, B the width of the superstructure in feet (with B/L not taken greater than $3/8$), α the skew angle, and $F_v S_1$ the site-adjusted 1-second spectral acceleration. This equation differs from the new-design equation in BR-E01; it includes explicit terms for hazard level and for superstructure width relative to length.

Method B: component capacity checks. No demand analysis is required, but the relative strength of members and the adequacy of key details are checked against recommended minima. The manual describes it as suitable for regular bridges in SRC C, subject to restrictions on $F_v S_1$.

Method C: component capacity/demand ratios. The engineer performs an elastic demand analysis (often a multimode spectral analysis) and computes capacity/demand ratios for each vulnerable component: bearings, seats, columns (flexure, shear, splices, confinement, anchorage), joints, footings and soils. Ratios below 1.0 identify deficiencies. Column flexural ratios are adjusted for ductility: a column with good confinement can tolerate an elastic demand several times its yield capacity, while a column with lap splices in the hinge zone cannot.

Method D1: capacity spectrum. Demands come from simple models such as the uniform load method, and capacity is based on a simplified bilinear lateral strength curve. A capacity spectrum gives the capacity/demand ratio for the bridge for each limit state. The manual describes it as suitable for regular bridges in SRC C and D.

Method D2: structure capacity/demand (pushover). Demands come from elastic multimode or time history analysis; capacity comes from the displacement capacity of individual piers from a pushover analysis including nonlinear behavior. Capacity/demand ratios are computed for each pier, bearing and foundation for each limit state. It is suitable for all bridges in SRC C and D, and is also called the Nonlinear Static Procedure.

Method E: nonlinear time history. For complex or important bridges, nonlinear dynamic analysis with suites of ground motions provides the most complete picture.

Capacity of existing components. Evaluation uses realistic, not code-minimum, material properties and recognizes the specific weaknesses of older details. The manual provides guidance on: column flexural ductility as a function of confinement; shear strength degrading with ductility; lap splice failure in the hinge zone; anchorage of column bars into footings and caps; joint shear; footing flexure and shear; and pile capacity. A column that is adequate in flexure may still fail in shear or at its splice.

Foundations and soils. Evaluation includes liquefaction potential, the consequences of liquefaction (settlement, lateral spreading, loss of pile capacity), and abutment stability. Soil hazards can govern even when the structure itself is adequate.

Vulnerable details

FHWA-HRT-06-032 identifies a consistent set of details responsible for most bridge failures in past earthquakes. Recognizing these is the most useful seismic skill for anyone who reviews, inspects or rates older bridges.

Bearings. The manual describes steel rocker bearings as particularly vulnerable. Tall rockers can topple under longitudinal or transverse movement, dropping the girder and sometimes pulling it off the seat. Other bearing problems include sliding bearings without keepers, anchor bolts that are too small or corroded, and elastomeric pads that walk out. Bearing failure can be the first step in a chain that ends in unseating.

Seats, joints and hinges. Narrow seats at abutments, piers and in-span hinges are the most common cause of collapse. In-span hinges (for example, cantilever-suspended span hinges in older concrete box girder bridges) are especially vulnerable because they often have short seats and the adjacent frames can move out of phase. Skewed bridges tend to rotate in plan, walking off the acute corners of their supports.

Columns. Columns in pre-1971 bridges, as the manual describes, commonly have little transverse reinforcement, which leads to inadequate confinement and shear strength; lap splices of starter bars in the plastic hinge zone at the base; and insufficient anchorage of longitudinal bars into footings and caps. A typical failure sequence is spalling of cover, followed by splice slip or bar buckling, and then loss of axial capacity or a shear failure. Short columns, such as those with partial-height walls or stiff infill, attract large shears and fail in shear in a brittle manner.

Cap beams and joints. Older cap beams may lack the strength to develop the column's flexural capacity, and the beam-column joint may lack reinforcement to transfer forces. The Cypress Viaduct collapse involved joint and pedestal failures.

Footings and piles. Older footings often lack top reinforcement and shear reinforcement and may have insufficient pile capacity in tension (uplift) or poor pile-to-cap connections. Foundation retrofit is costly and often deferred unless the evaluation clearly shows the foundation will fail first.

Abutments. Abutments can rotate or slide, and backwalls can be damaged by the superstructure pounding against them. Wingwall damage is common but usually not collapse critical.

Soils. Liquefaction and lateral spreading can move abutments and piers, causing spans to unseat even if the structure itself is strong. Bridges on loose saturated sands near rivers and coastlines are most susceptible.

Inspection connection. Many of these details can be observed during routine inspections: rocker tilt, missing anchor bolt nuts, seat width and edge damage, restrainer cable condition, column cracking at the base, and evidence of past movement. Documenting these features with dimensions and photographs supports the screening process. BR-I08 covers substructure inspection and BR-B01 bearings.

Retrofit approaches

FHWA-HRT-06-032 frames retrofit as choosing an approach before choosing a measure. It lists seven approaches:

1. **Strengthening.** Increase the strength of deficient components so they can resist the demand, for example strengthening a column, cap or footing. Strengthening may increase stiffness and therefore increase force demand elsewhere, so the whole load path must be checked.
2. **Displacement capacity enhancement.** Improve a component's ability to deform without failure, for example confining a column so it can sustain plastic rotation, or extending seats so spans can move without falling.
3. **Force limitation.** Introduce a fuse that limits the force transmitted to vulnerable components, such as a sacrificial shear key or a limited-strength connection.
4. **Response modification.** Change the dynamic response of the bridge, for example with seismic isolation bearings or dampers, which lengthen the period and add damping, reducing forces in the substructure.
5. **Site remediation.** Improve the soil to reduce liquefaction or lateral spreading hazard, for example by densification, drainage or grouting.
6. **Damage acceptance or control.** Accept certain damage, provided it does not lead to collapse and is consistent with the required performance level. For a PL1 requirement, some damage is acceptable if life safety is maintained.
7. **Partial replacement.** Replace a deficient component, such as a bearing, a joint or a column.

Combinations. Approaches are often combined. A common California strategy, the manual describes, was to jacket columns (displacement capacity enhancement) and add restrainers or seat extensions (displacement capacity enhancement at the seats) while leaving foundations alone (damage acceptance), provided the foundation had enough capacity to resist the column's overstrength or that any foundation damage would not cause collapse.

Do nothing, replace or close. The manual recognizes that retrofit is not always the right answer. For a bridge that is nearing the end of its service life, or that has major non-seismic deficiencies, replacement may be more cost-effective. For a bridge with an alternate route and low importance, an owner may accept the risk. For exempt bridges, no action is required. These choices should be documented with their rationale.

Performance check. After the retrofit is designed, the bridge is re-evaluated to confirm it meets the required performance level. Retrofit often shifts the weak link: a jacketed column becomes stronger, which may make the footing or cap the new weak link. Capacity design principles from BR-E01 apply: decide where yielding should occur and protect everything else.

Retrofit measures

FHWA-HRT-06-032 describes a range of measures for each part of the bridge.

Restrainers. Cable or rod restrainers tie spans together at joints and hinges or tie the superstructure to the substructure. California's first-generation retrofit program installed restrainers widely after 1971. The manual explains that restrainers are effective in limiting relative movement, but they can fail if they are too weak or too stiff, and they do not address deficient columns (the Cypress lesson). Restrainer design must consider the relative movement of adjacent frames, and the manual's guidance includes a design method. Where restrainers would be needed to limit movement to a small fraction of the seat width, the manual favors seat extensions; it uses 67% of the seat width as the relevant threshold.

Seat extensions and catcher blocks. Seat extensions widen the support so a span can move without falling. They may be concrete corbels or steel brackets attached to the face of the abutment or pier. Catcher blocks are placed under the

superstructure to catch it if a bearing fails, limiting the drop. Both are simple and relatively inexpensive, and both address the most common collapse mode.

Bearing retrofit and replacement. Vulnerable bearings, especially steel rockers, may be replaced with elastomeric bearings or isolation bearings, or restrained with keeper plates and stronger anchor bolts. Isolation bearings, as a response modification measure, can reduce forces in the substructure enough that column and foundation retrofit is not required.

Column jackets. The manual describes steel jackets and fiber-reinforced polymer (FRP) jackets as the most widely used column retrofit. A steel jacket is a shell slightly larger than the column, with the gap filled with grout; it confines the concrete, clamps lap splices, and increases shear strength. FRP jackets provide similar confinement with less weight and installation effort. Concrete jackets with added reinforcement are also used, but increase column size and stiffness. The manual describes a vertical space of about 2 inches typically left between the end of a steel jacket and the footing or cap so the jacket does not bear and act as compression reinforcement at large drift, which could significantly increase flexural strength and overload adjacent members. BR-R01 covers FRP strengthening generally.

Shoring during retrofit. When a column retrofit requires removing concrete or the column is heavily damaged, temporary shoring is needed. The manual states that shoring must carry at least 50% of the column's shear capacity.

Cap beam and joint strengthening. Cap beams and joints may be strengthened with added reinforcement and concrete, external post-tensioning or steel elements. Joint retrofit is complex and often expensive.

Footings and piles. Footings can be strengthened by overlaying a concrete cap with top reinforcement, adding piles around the footing, or tying the footing to the column with dowels. Foundation retrofit is the most expensive category of work.

Abutments. Abutment retrofit may include adding anchors, increasing the backwall's resistance, or adding a catcher block.

Soil remediation. For liquefaction, the manual describes ground improvement methods such as vibro-compaction, stone columns, grouting and drainage. These can be very costly and are typically reserved for important bridges.

Superstructure. Superstructure retrofits include strengthening end diaphragms and cross-frames to carry seismic forces to bearings, and adding continuity across joints.

Column jackets in detail

Column jacketing is the signature retrofit of the post-Loma Prieta era. FHWA-HRT-06-032 Chapter 9 describes the main systems in detail.

Steel jackets for circular columns. The manual describes steel jacketing as originally developed for circular columns and, at the time of writing, Caltrans' preferred method for seismic retrofit of bridge columns. Two half-shells of steel plate, rolled to a radius equal to the column radius plus 0.5 to 1 inch for clearance, are placed around the portion of the column to be retrofitted and the vertical seams are welded. The gap is flushed with water and filled with pure cement grout. About 2 inches of vertical space is left between the jacket end and the footing or cap beam so the jacket does not bear against the supporting member at large drift and act as compression reinforcement; otherwise flexural strength could rise significantly and overload adjacent members. Handling stresses set a minimum plate thickness of 0.375 inch, and bending limits a maximum of 1 inch.

Passive confinement. A steel jacket provides passive confinement: it is not engaged until the concrete dilates, through bulging under high axial strain, dilation around splices from vertical splitting cracks, or diagonal shear cracking. That is exactly when confinement is needed, so the passive system works well for splices, flexural ductility and shear.

Rectangular columns. For rectangular columns, the manual recommends an oval jacket, which develops continuous membrane confining action similar to a circular jacket; small aggregate is added to the grout because the gap is larger. Oval-jacketed rectangular columns performed very well in flexure and shear. Rectangular jackets have been less successful even when heavily stiffened, because they confine only through flexible plate bending of the sides. They can, however, eliminate brittle shear failure in shear-critical columns, after which the failure mode may shift to flexure, where the rectangular jacket provides limited help. Very flat portions of an oval may need braces or through-bolts.

Stiffness change. Retrofitting can change cracked-section stiffness, which must be reflected in analysis. Citing Priestley and Seible (1991), the manual reports average stiffness increases of 10 to 15% for partial-height steel jackets and about 30% for full-height jackets, while external prestressed confinement and fiber composite jackets have negligible influence on stiffness. A stiffer column attracts more force, which must be checked against the cap, joint and footing.

Fiber composite jackets. The manual describes proprietary systems using high-strength glass (E-glass), carbon or aramid fibers oriented mainly circumferentially in a polyester, vinyl ester or epoxy matrix. The anisotropic material improves flexural ductility, splice strength and shear strength without significantly affecting flexural strength or stiffness. Because properties depend on the specific fiber and resin combination, the manual advises using only components developed and thoroughly tested as a system, and points to ACI's guidance on externally bonded FRP systems. Current philosophy is passive confinement; the manual cautions against active (prestressed) composite wraps because composites creep under sustained load and can rupture even at relatively low stress. It notes that Illinois DOT used external prestressed steel cables, like steel hoops, to retrofit inadequate lap splices. For rectangular columns, composite shells over an elliptical built-up section increase ductility and shear strength; wraps applied directly to a rectangular surface increase shear resistance and force a flexural failure with increased but limited ductility if the section is not too large. Prefabricated composite shell systems also exist. BR-R01 covers FRP strengthening more broadly.

Illustrative example. As an invented illustration: a 1965 bridge has 4-foot-diameter circular columns with starter-bar lap splices at the base and #4 hoops at 12 inches. Evaluation finds the columns would fail at the splice well before reaching the displacement demand. A steel jacket with 2-inch end gaps over the lower hinge zone clamps the splice and confines the core; the engineer updates the model for about a 10 to 15% stiffness increase and then checks the footing for the column's new, higher deliverable moment. If the footing lacks top reinforcement, the owner faces a choice between a footing overlay and accepting foundation damage consistent with PL1.

Restrainers, seats and the load path

FHWA-HRT-06-032 Chapter 8 addresses retrofit of superstructures, bearings and seats, the components whose failure produces the most dramatic collapses.

Purpose of longitudinal restrainers. Longitudinal joint restrainers limit relative displacement at expansion joints and so reduce the chance of loss of support. Where bearing anchor bolts and similar details cannot prevent loss of support at fixed bearings, restrainers can also transfer the superstructure's longitudinal inertia force into the substructure. Because most restrainers work only in tension, restrainers are then needed at both ends of the span.

Design principles. The manual states that restrainers should resist the maximum calculated force within their elastic range; two symmetric restrainers per joint minimize eccentric movement; an adequate gap must allow normal in-service movement; and at piers, restrainers should provide a direct, positive connection between superstructure and pier unless the cap is wide enough to prevent loss of support and the expected movement will not cause excessive damage.

Cables and bars. Steel cables and bars in direct tension have been the most frequently used restrainers. They are intended to remain elastic and dissipate little energy. The manual acknowledges they may allow girder ends to be damaged by pounding, but that damage is usually repairable and not enough to cause loss of support; they are not ideal, but they are simple and an economical means of preventing catastrophic failure. There is no established rule for choosing between them. Cables often have the economic advantage because shorter lengths work for a given joint movement, and they tolerate transverse and vertical movement; bars may require additional transverse and vertical restraint to prevent shear and flexural distortion.

The standard cable unit. The manual describes the most common restrainer cable as a galvanized 0.75-inch steel cable of the type used to anchor guardrail, with galvanized cold-swaged fittings and 1-inch ASTM A449 threaded studs at each end, and nuts secured with a thread-locking system so they do not work loose. Spring washers can maintain tension while allowing thermal movement, and cable yield indicators give post-earthquake evidence of yielding.

What inspectors should look for. Because restrainers are tension devices that sit idle for decades, their condition matters: corroded or slack cables, missing or loose nuts, damaged swaged fittings, restrainers fouled by debris, and restrainers that have been cut or removed during later joint or bearing work. Yield indicators, where present, should be checked after any felt earthquake. Seat extensions and catcher blocks should be checked for cracking and anchorage condition.

Load path thinking. The Cypress lesson is that a restrainer ties two pieces of a bridge together but cannot make up for a weak column or joint. Retrofit must follow the whole load path: deck to bearing, bearing to seat, seat to cap, cap to column, column to footing, footing to soil. A restrainer that is much stronger than the pier it connects to may simply move the failure. That is why the SRC requirements add columns, walls, footings and, at SRC D, abutments to the scope as hazard and required performance increase.

Costs and program management

Cost data. FHWA-HRT-06-032 reports cost data from Caltrans for 165 bridges retrofitted in 1993 and 1994. Costs are expressed as a percentage of the cost of new construction:

- Superstructure retrofit only (such as restrainers and seat extensions): average 3.1%.
- Superstructure and substructure (adding column jackets): average 15.4%.
- Including foundation retrofit: average 28.8%.
- Overall average: 15.1%.

The manual also notes that engineering costs for retrofit are about twice those for a new bridge of comparable size, as a proportion of construction cost, because evaluation and retrofit design of an existing structure is more complex.

These are 1993-94 percentages from one state. They are useful as relative indicators: superstructure retrofit is inexpensive, column retrofit adds substantially, and foundation retrofit roughly doubles the cost again. Current costs must be estimated from current data.

Illustrative example. As an invented illustration: a bridge that would cost \$10 million to replace might be retrofitted with restrainers and seat extensions for about \$310,000 (3.1%), with columns for about \$1.5 million (15.4%) or with foundations for about \$2.9 million (28.8%), using the manual's average percentages as rough relative factors only. If the bridge also needs a deck replacement and has a remaining service life of 20 years, the owner might reasonably compare the foundation-level retrofit cost against replacement.

Program structure. A retrofit program typically proceeds:

1. Inventory and hazard: determine the hazard level for each bridge at both ground motion levels.
2. Classify: assign importance and anticipated service life; identify exempt bridges.
3. Determine SRC and minimum requirements.
4. Screen and prioritize: compute ranks with the indices method or another method; adjust for other factors.
5. Evaluate selected bridges with the required method.
6. Decide: retrofit, replace, accept risk or close, considering cost and other deficiencies.
7. Design and construct retrofit.
8. Document and update the inventory.

Asset management. Federal asset management rules require states to consider seismic risk in risk management and life-cycle planning for National Highway System assets (see BR-M01). Retrofit programs provide the data and decisions that feed those processes.

Post-earthquake inspection. Owners should also plan for post-earthquake inspection: which bridges to check first, who will inspect, and what criteria will be used to close or reopen bridges. Knowledge of vulnerable details from the screening process helps inspectors focus on likely damage locations. BR-I03 covers critical findings, which apply to post-event damage as well.

Where to verify. This course is based solely on the 2006 FHWA manual. States may have adopted their own retrofit criteria since then, and the AASHTO design specifications described in BR-E01 may affect evaluation approaches. Check current owner policy before applying any criteria.

Key takeaways

- FHWA-HRT-06-032 uses two ground motions: lower level (50% in 75 years, about 100 years) and upper level (7% in 75 years, about 1,000 years).
- Required performance depends on anticipated service life (ASL 1, 2, 3) and importance (standard or essential), from PL0 to PL3; bridges with 15 years or less of remaining life are exempt.
- Seismic hazard level (I to IV) and performance level set the seismic retrofit category (A to D), which sets the minimum evaluation method.
- The lower-level screen compares SDS times W with the design wind and braking forces.
- The indices method ranks bridges by $R = V \text{ times } E$, with V the larger of the bearing/seat rating and the column/abutment/liquefaction rating.
- The most vulnerable details are steel rocker bearings, narrow seats, pre-1971 columns with hinge-zone lap splices and little confinement, weak joints and footings, and liquefiable soils.
- Retrofit approaches include strengthening, displacement capacity enhancement, force limitation, response modification, site remediation, damage acceptance and partial replacement; steel and FRP jackets are the most widely used column measures.

References

1. FHWA, Seismic Retrofitting Manual for Highway Structures, FHWA-HRT-06-032, 2006

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