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Seismic Design Philosophies and Performance-Based Design Criteria

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37.1 Introduction

Seismic design criteria for highway bridges have been improving and advancing based on research findings and lessons learned from past earthquakes. In the United States, prior to the 1971 San Fernando earthquake, the seismic design of highway bridges was partially based on lateral force requirements for buildings. Lateral loads were considered as levels of 2 to 6% of dead loads. In 1973, the California Department of Transportation (Caltrans) developed new seismic design criteria related to site, seismic response of the soils at the site, and the dynamic characteristics of bridges. The American Association of State Highway and Transportation Officials (AASHTO) modified the Caltrans 1973 Provisions slightly, and adopted Interim Specifications. The Applied Technology Council (ATC) developed guidelines ATC-6 [1] for seismic design of bridges in 1981. AASHTO adopted ATC-6 [1] as the Guide Specifications in 1983 and later incorporated it into the Standard Specifications for Highway Bridges in 1991.

Since the 1989 Loma Prieta earthquake in California [2], extensive research [3-15] has been conducted on seismic design and retrofit of bridges in the United States, especially in California. The performance-based project-specific design criteria [16,17] were developed for important bridges. Recently, ATC published improved seismic design criteria recommendations for California bridges [18] in 1996, and for U.S. bridges and highway structures [19] in 1997, respectively. Caltrans published the new seismic Design Methodology in 1999. [20] The new Caltrans Seismic Design Criteria [43] is under development. Great advances in earthquake engineering have been made during this last decade of the 20th century.

This chapter first presents the bridge seismic design philosophy and the current practice in the United States. It is followed by an introduction to the newly developed performance-based criteria [17] as a reference guide.

37.2 Design Philosophies

37.2.1 No-Collapse-Based Design

For seismic design of ordinary bridges, the basic philosophy is to prevent collapse during severe earthquakes [21-26]. To prevent collapse, two alternative approaches are commonly used in design. The first is a conventional force-based approach where the adjustment factor Z for ductility and risk assessment [26], or the response modification factor R [23], is applied to elastic member forces obtained from a response spectra analysis or an equivalent static analysis. The second approach is a more recent displacement-based approach [20] where displacements are a major consideration in design. For more-detailed information, reference can be made to a comprehensive discussion in *Seismic Design and Retrofit of Bridges* by Priestley, Seible, and Calvi [15].

37.2.2 Performance-Based Design

Following the 1989 Loma Prieta earthquake, bridge engineers [2] have faced three essential challenges:

- Ensure that earthquake risks posed by new construction are acceptable.
- Identify and correct unacceptable seismic safety conditions in existing structures.
- Develop and implement a rapid, effective, and economic response mechanism for recovering structural integrity after damaging earthquakes.

In the California, although the Caltrans Bridge Design Specifications [26] have not been formally revised since 1989, project-specific criteria and design memoranda have been developed and implemented for the design of new bridges and the retrofitting of existing bridges. These revised or supplementary criteria included guidelines for development of site-specific ground motion estimates, capacity design to preclude brittle failure modes, rational procedures for joint shear design, and definition of limit states for various performance objectives [14]. As shown in Figure 37.1, the performance requirements for a specific project must be established first. Loads, materials, analysis methods, and detailed acceptance criteria are then developed to achieve the expected performance.

37.3 No-Collapse-Based Design Approaches

37.3.1 AASHTO-LRFD Specifications

Currently, AASHTO has issued two design specifications for highway bridges: the second edition of AASHTO-LRFD [23] and the 16th edition of the Standard Specifications [24]. This section mainly discusses the design provisions of the AASHTO-LRFD Specifications.

The principles used for the development of AASHTO-LRFD [23] seismic design specifications are as follows:

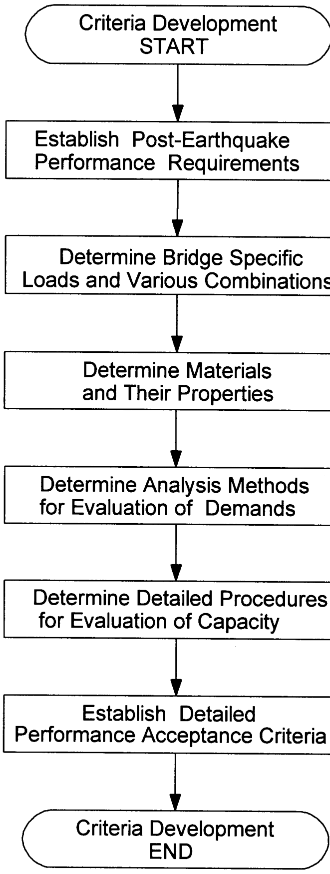

FIGURE 37.1 Development of performance-based seismic design criteria.

- Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage.
- Realistic seismic ground motion intensities and forces should be used in the design procedures.
- Exposure to shaking from a large earthquake should not cause collapse of all or part of bridges where possible; damage that does occur should be readily detectable and accessible for inspection and repair.

Seismic force effects on each component are obtained from the elastic seismic response coefficient C_{sm} and divided by the elastic response modification factor R. Specific detailing requirements are provided to maintain structural integrity and to ensure ductile behavior. The AASHTO-LRFD seismic design procedure is shown in Figure 37.2.

Seismic Loads

Seismic loads are specified as the horizontal force effects and are obtained by production of C_{sm} and the equivalent weight of the superstructures. The seismic response coefficient is given as:

$$C_{sm} = \begin{cases} \dfrac{1.25AS}{T_m^{2/3}} \le 2.5A & \\ A\left(0.8+4T_m\right) & \text{for Soil III, IV, and nonfundamental } T_m < 0.3s \\ 3AST_m^{0.75} & \text{for Soil III, IV and } T_m > 0.4s \end{cases} \tag{37.1}$$

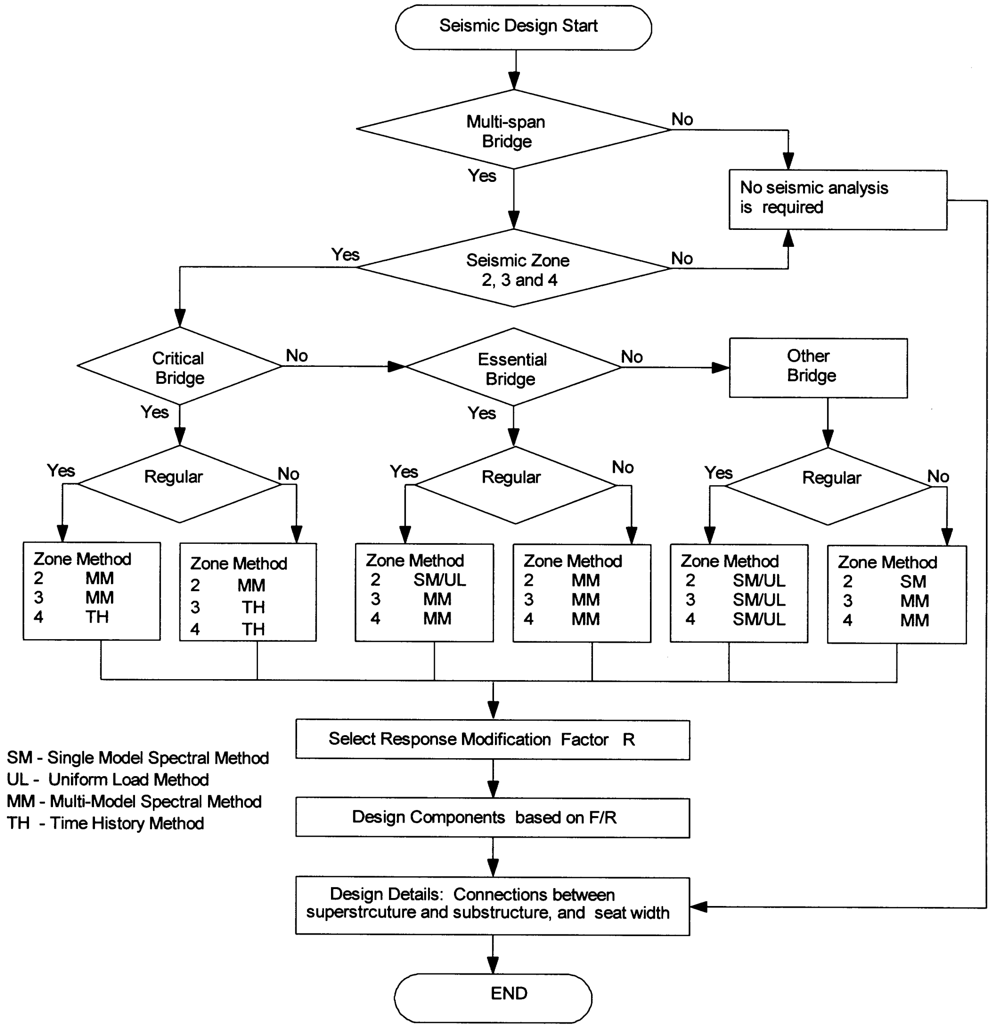

FIGURE 37.2 AASHO-LRFD seismic design procedure.

where A is the acceleration coefficient obtained from a contour map (Figure 37.3) which represents the 10% probability of an earthquake of this size being exceeded within a design life of 50 years; S is the site coefficient and is dependent on the soil profile types as shown in Table 37.1; T_m is the structural period of the mth mode in second.

Analysis Methods

Four seismic analysis methods specified in AASHTO-LRFD [23] are the uniform-load method, the single-mode spectral method, the multimode spectral method, and the time history method. Depending on the importance, site, and regularity of a bridge structure, the minimum complexity analysis methods required are shown in Figure 37.2. For single-span bridges and bridges located seismic Zone 1, no seismic analysis is required.

The importance of bridges is classified as critical, essential, and other in Table 37.2 [23], which also shows the definitions of a regular bridge. All other bridges not satisfying the requirements of Table 37.2 are considered irregular.

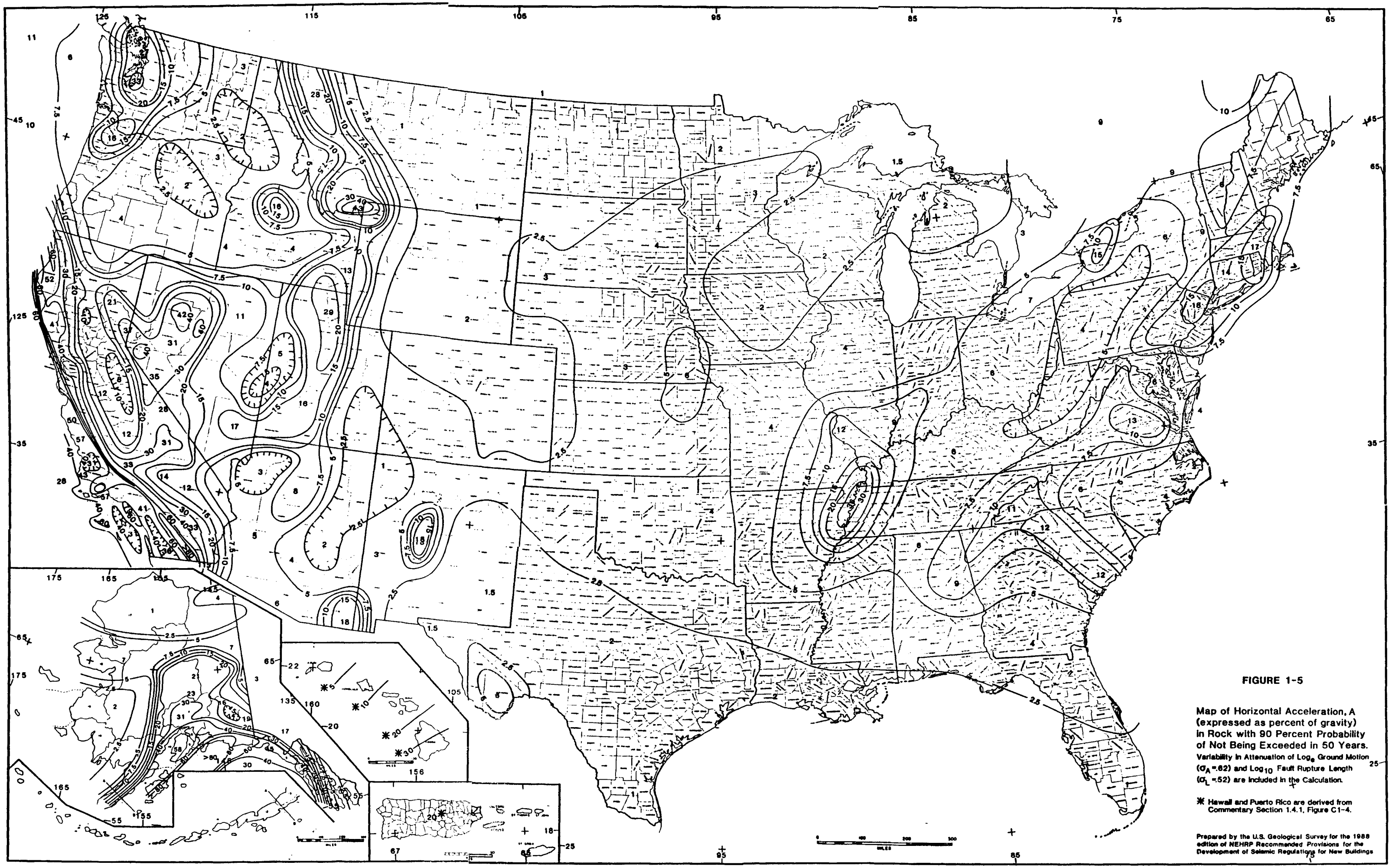

FIGURE 37.3 AASHTO-LRFD seismic contour map.

TABLE 37.1 AASHTO-LRFD Site Coefficient — S

Soil Profile Type	Descriptions	Site Coefficient, S
I	• Rock characterized by a shear wave velocity > 765 m/s • Stiff soil where the soil depth < 60 m and overlying soil are stable deposits of sands, gravel, or stiff clays	1.0
II	Stiff cohesive or deep cohesionless soil where the soil depth > 60 m and the overlying soil are stable deposits of sands, gravel, or stiff clays	1.2
III	Stiff to medium-stiff clays and sands, characterized by 9 m or more soft to medium-stiff clays without intervening layers of sands or other cohesionless soils	1.5
IV	Soft clays of silts > 12 m in depth characterized by a shear wave velocity < 153 m/s	2.0

TABLE 37.2 AASHTO-LRFD Bridge Classifications for Seismic Analysis

Importance	Critical	• Remain open to all traffic after design earthquake • Usable by emergency vehicles and for security/defense purposes immediately after a large earthquake (2500-year return period event)						
	Essential	Remain open emergency vehicles and for security/defense purposes immediately after the design earthquake (475-year return period event)						
	Others	Not required as critical and essential bridges						
Regularity	Regular	Structural Features	Number of Span	2	3	4	5	6
		Maximum subtended angles for a curved bridge				90°		
		Maximum span length ratio from span to span		3	2	2	1.5	1.5
		Maximum bent/pier stiffness ratio from span to span excluding abutments		—	4	4	3	2
	Irregular	Multispan not meet requirement of regular bridges						

TABLE 37.3 Response Modification Factor, R

		Important Category		
	Structural Component	Critical	Essential	Others
Substructure	Wall-type pier — Large dimension	1.5	1.5	2.0
	Reinforced concrete pile bent			
	• Vertical pile only	1.5	2.0	3.0
	• With batter piles	1.5	1.5	2.0
	Single column	1.5	2.0	3.0
	Steel or composite steel and concrete pile bents			
	• Vertical pile only	1.5	3.5	5.0
	• With batter piles	1.5	2.0	3.0
	Multiple column bents	1.5	3.5	5.0
	Foundations		1.0	
Connection	Substructure to abutment		0.8	
	Expansion joints with a span of the superstructure		0.8	
	Column, piers, or pile bents to cap beam or superstructure		1.0	
	Columns or piers to foundations		1.0	

Component Design Force Effects

Design seismic force demands for a structural component are determined by dividing the forces calculated using an elastic dynamic analysis by appropriate response modification factor R

(Table 37.3) to account for inelastic behavior. As an alternative to the use of R factor for connection, the maximum force developed from the inelastic hinging of structures may be used for designing monolithic connections.

To account for uncertainty of earthquake motions, the elastic forces obtained from analysis in each of two perpendicular principal axes shall be combined using 30% rule, i.e., 100% of the absolute response in one principal direction plus 30% of the absolute response in the other.

The design force demands for a component should be obtained by combining the reduced seismic forces with the other force effects caused by the permanent and live loads, etc. Design resistance (strength) are discussed in Chapter 38 for concrete structures and Chapter 39 for steel structures.

37.3.2 Caltrans Bridge Design Specifications

The current Caltrans Bridge Design Specifications [26] adopts a single-level force-based design approach based on the no-collapse design philosophy and includes:

- Seismic force levels defined as elastic acceleration response spectrum (ARS);
- Multimodal response spectrum analysis considering abutment stiffness effects;
- Ductility and risk Z factors used for component design to account for inelastic effects;
- Properly designed details.

Seismic Loads

A set of elastic design spectra ARS curves are recommended to consider peak rock accelerations (A), normalized 5% damped rock spectra (R), and soil amplification factor (S). Figure 37.4 shows typical ARS curves.

Analysis Methods

For ordinary bridges with well-balanced span and bent/column stiffness, an equivalent static analysis with the ARS times the weight of the structure applied at the center of gravity of total structures can be used. This method is used mostly for hinge restrainer design. For ordinary bridges with significantly irregular geometry configurations, a dynamic multimodal response spectrum analysis is recommended. The following are major considerations in seismic design practice:

- A beam-element model with three or more lumped masses in each span is usually used [25-27].
- A larger cap stiffness is often used to simulate a stiff deck.
- Gross section properties of columns are commonly used to determine force demands, and cracked concrete section properties of columns are used for displacement demands.
- Soil–spring elements are used to simulate the soil–foundation–structure–interaction. Adjustments are often made to meet force–displacement compatibility, particularly for abutments. The maximum capacity of the soil behind abutments with heights larger than 8 ft (2.44 m) is 7.7 ksf (369 kPa) and lateral pile capacity of 49 kips (218 kN) per pile.
- Compression and tension models are used to simulate the behavior of expansion joints.

Component Design Force Effects

Seismic design force demands are determined using elastic forces from the elastic response analysis divided by the appropriate component- and period-based (stiffness) adjustment factor Z, as shown in Figure 38.4a to consider ductility and risk. In order to account for directional uncertainty of earthquake motions, elastic forces obtained from analysis of two perpendicular seismic loadings are combined as the 30% rule, the same as the AASHTO-LRFD [23].

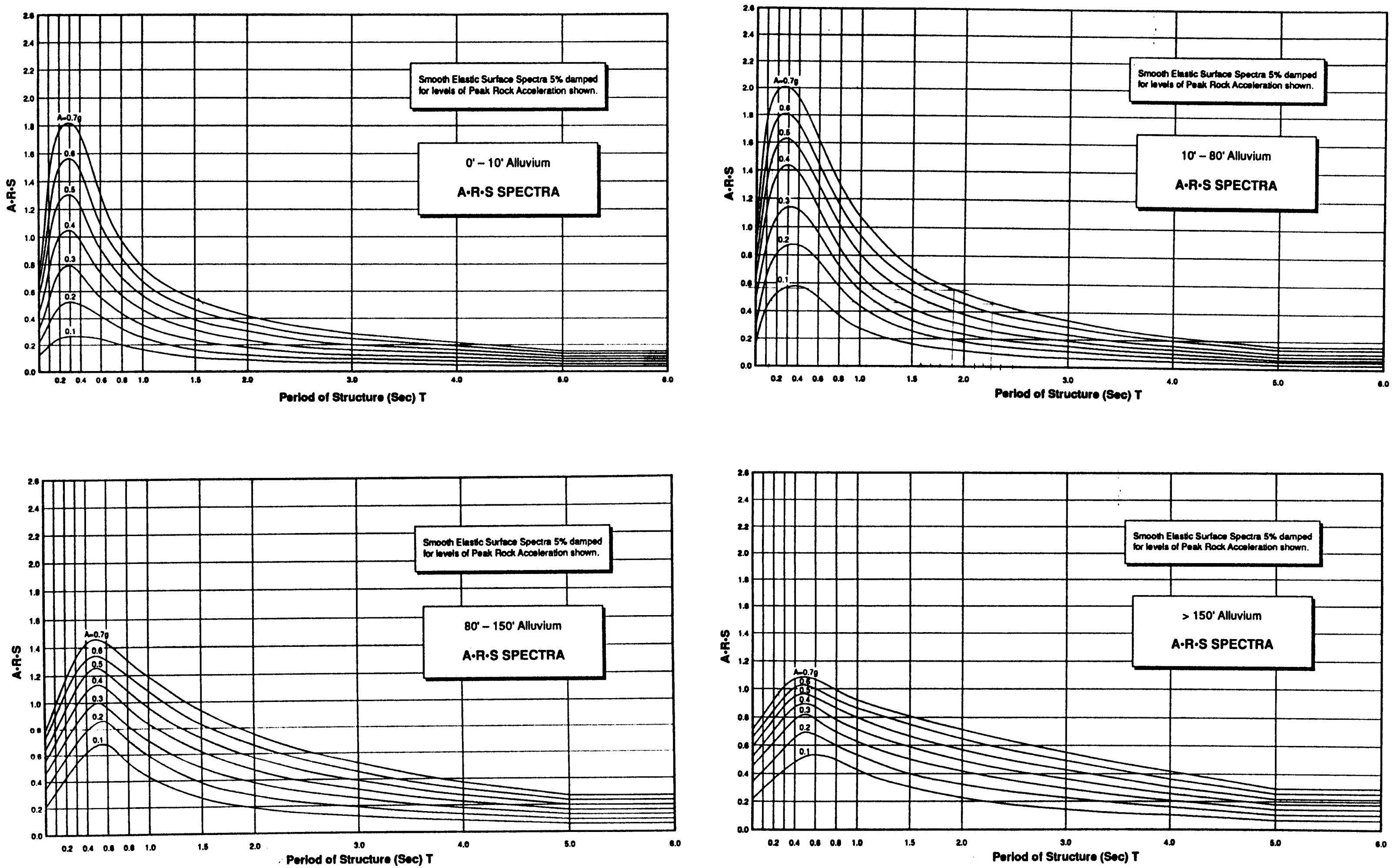

FIGURE 37.4 Caltrans ARS curves.

TABLE 37.4 Caltrans Seismic Performance Criteria

Ground motions at the site	Minimum (ordinary bridge) performance level	Important bridge performance level
Functional evaluation	Immediate service; repairable damage	Immediate service level; minimum damage
Safety evaluation	Limited service level; significant damage	Immediate service level; repairable damage

Definitions:
Important Bridge (one of more of following items present):
- Bridge required to provide secondary life safety
- Time for restoration of functionality after closure creates a major economic impact
- Bridge formally designed as critical by a local emergency plan

(*Ordinary Bridge*: Any bridge not classified as an important bridge.)
Functional Evaluation Ground Motion (*FEGM*): Probabilistic assessed ground motions that have a 40% probability of occurring during the useful lifetime of the bridge. The determination of this event shall be reviewed by a Caltrans-approved consensus group. A separate functionality evaluation is required for important bridges. All other bridges are only required to meet the specified design requirement to assure minimum functionality performance level compliance.

Safety Evaluation Ground Motion (*SEGM*): Up to two methods of defining ground motion may be used:
- Deterministically assessed ground motions from the maximum earthquake as defined by the Division of Mines and Geology Open-File Report 92-1 [1992].
- Probabilistically assessed ground motions with a long return period (approximately 1000–2000 years).

For important bridges both methods should be given consideration; however, the probabilistic evaluation should be reviewed by a Caltrans-approved consensus group. For all other bridges, the motions should be based only on the deterministic evaluation. In the future, the role of the two methods for other bridges should be reviewed by a Caltrans-approved consensus group.

Immediate Service Level: Full access to normal traffic available almost immediately (following the earthquake).
Repairable Damage: Damage that can be repaired with a minimum risk of losing functionality.
Limited Service Level: Limited access (reduced lanes, light emergency traffic) possible with in days. Full service restoration within months.
Significant Damage: A minimum risk of collapse, but damage that would require closure for repairs.

Note: Above performance criteria and definitions have been modified slightly in the proposed provisions for California Bridges (ACT-32, 1996) and the U.S. Bridges (ATC-18, 1997) and Caltrans (1999) MTD 20-1 (920).

37.4 Performance-Based Design Approaches

37.4.1 Caltrans Practice

Since 1989, the design criteria specified in Caltrans BDS [26] and several internal design manuals [20,25,27] have been updated continuously to reflect recent research findings and development in the field of seismic bridge design. Caltrans has been shifting toward a displacement-based design approach emphasizing capacity design. In 1994 Caltrans established the seismic performance criteria listed in Table 37.4. A bridge is categorized as an "important" or "ordinary" bridge. Project-specific two-level seismic design procedures for important bridges, such as the R-14/I-5 Interchange replacement [16], the San Francisco–Oakland Bay Bridge (SFOBB) [17], and the Benicia-Martinez Bridge [28], are required and have been developed. These performance-based seismic design criteria include site-specific ARS curves, ground motions, and specific design procedures to reflect the desired performance of these structures. For ordinary bridges, only one-level safety-evaluation design is required. The following section briefly discusses the newly developed seismic design methodology for ordinary bridges.

37.4.2 New Caltrans Seismic Design Methodology (MTD 20-1, 1999)

To improve Caltrans seismic design practice and consolidate new research findings, ATC-32 recommendations [18] and the state-of-the-art knowledge gained from the recent extensive seismic bridge design, Caltrans engineers have been developing the Seismic Design Methodology [20] and the Seismic Design Criteria (SDC) [43] for ordinary bridges.

Ordinary Bridge Category

An ordinary bridge can be classified as a "standard" or "nonstandard" bridge. An nonstandard bridge may feature irregular geometry and framing (multilevel, variable width, bifurcating, or highly horizontally curved superstructures, different structure types, outriggers, unbalanced mass and/or stiffness, high skew) and unusual geologic conditions (soft soil, moderate to high liquefaction potential, and proximity to an earthquake fault). A standard bridge does not contain nonstandard features. The performance criteria and the service and damage levels are shown in Table 37.4.

Basic Seismic Design Concept

The objective of seismic design is to ensure that all structural components have sufficient strength and/or ductility to prevent collapse — a limit state where additional deformation will potentially render a bridge incapable of resisting its self-weight during a maximum credible earthquake (MCE). Collapse is usually characterized by structural material failure and/or instability in one or more components.

Ductility is defined as the ratio of ultimate deformation to the deformation at first yield and is the predominant measure of structural ability to dissipate energy. Caltrans takes advantage of ductility and postelastic strength and does not design ordinary bridges to remain elastic during design earthquakes because of economic constraints and the uncertainties in predicting future seismic demands. Seismic deformation demands should not exceed structural deformation capacity or energy-dissipating capacity. Ductile behavior can be provided by inelastic actions either through selected structural members and/or through protective systems — seismic isolations and energy dissipation devices. Inelastic actions should be limited to the predetermined regions that can be easily inspected and repaired following an earthquake. Because the inelastic response of a concrete superstructure is difficult to inspect and repair and the superstructure damage may cause the bridge to be in an unserviceable condition, inelastic behavior on most bridges should preferably be located in columns, pier walls, backwalls, and wingwalls (see Figure 38.1).

To provide an adequate margin of strength between ductile and nonductile failure modes, capacity design is achieved by providing overstrength against seismic load in superstructure and foundations. Components not explicitly designed for ductile performance should be designed to remain essentially elastic; i.e., response in concrete components should be limited to minor cracking or limited to force demands not exceeding the strength capacity determined by current Caltrans SDC, and response in steel components should be limited to force demands not exceeding the strength capacity determined by current Caltrans SDC.

Displacement-Based Design Approach

The objective of this approach is to ensure that the structural system and its individual components have enough capacity to withstand the deformation imposed by the design earthquake. Using displacements rather than forces as a measurement of earthquake damage allows a structure to fulfill the required functions.

In a displacement-based analysis, proportioning of the structure is first made based on strength and stiffness requirements. The appropriate analysis is run and the resulting displacements are compared with the available capacity which is dependent on the structural configuration and rotational capacity of plastic hinges and can be evaluated by inelastic static push-over analysis (see Chapter 36). This procedure has been used widely in seismic bridge design in California since 1994. Alternatively, a target displacement could be specified, the analysis performed, and then design strength and stiffness determined as end products for a structure [29,30]. In displacement-based design, the designer needs to define criteria clearly for acceptable structural deformation based on postearthquake performance requirements and the available deformation capacity. Such criteria are based on many factors, including structural type and importance.

Seismic Demands on Structural Components

For ordinary bridges, safety-evaluation ground motion shall be based on deterministic assessment corresponding to the MCE, the largest earthquake which is capable of occurring based on current geologic information. The ARS curves (Figure 37.5) developed by ATC-32 are adopted as standard horizontal ARS curves in conjunction with the peak rock acceleration from the Caltrans Seismic Hazard Map 1996 to determine the horizontal earthquake forces. Vertical acceleration should be considered for bridges with nonstandard structural components, unusual site conditions, and/or close proximity to earthquake faults and can be approximated by an equivalent static vertical force applied to the superstructure.

For structures within 15 km of an active fault, the spectral ordinates of the appropriate standard ARS curve should be increased by 20%. For long-period structures ($T \geq 1.5$ s) on deep soil sites (depth of alluvium ≥ 75 m) the spectral ordinates of the appropriate standard ARS curve should be increased by 20% and the increase applies to the portion of the curves with periods greater than 1.5 s.

Displacement demands should be estimated from a linear elastic response spectra analysis of bridges with effective component stiffness. The effective stiffness of ductile components should represent the actual secant stiffness of the component near yield. The effective stiffness should include the effects of concrete cracking, reinforcement, and axial load for concrete components; residual stresses, out-of-straightness, and axial load for steel components; the restraints of the surrounding soil for pile shafts. Attempts should be made to design bridges with dynamic characteristics (mass and stiffness) so that the fundamental period falls within the region between 0.7 and 3 s where the equal displacement principle applies. It is also important that displacement demands also include the combined effects of multidirectional components of horizontal acceleration (for example, 30% rules).

For short-period bridges, linear elastic analysis underestimates displacement demands. The inability to predict displacements of a linear analysis accurately can be overcome by designing the bridge to perform elastically, multiplying the elastic displacement by an amplification factor, or using seismic isolation and energy dissipation devices to limit seismic response. For long-period ($T > 3$ s) bridges, a linear elastic analysis generally overestimates displacements and linear elastic displacement response spectra analysis should be used.

Force demands for essentially elastic components adjacent to ductile components should be determined by the joint–force equilibrium considering plastic hinging capacity of the ductile component multiplied by an overstrength factor. The overstrength factor should account for the variations in material properties between adjacent components and the possibility that the actual strength of the ductile components exceeds its estimated plastic capacity. Force demands calculated from a linear elastic analysis should not be used.

Seismic Capacity of Structural Components

Strength and deformation capacity of a ductile flexural element should be evaluated by moment–curvature analysis (see Chapters 36 and 38). Strength capacity of all components should be based on the most probable or expected material properties, and anticipated damages. The impact of the second-order P-Δ and P-δ effects on the capacity of all members subjected to combined bending and compression should be considered. Components may require re-design if the P-Δ and P-δ effects are significant.

Displacement capacity of a bridge system should be evaluated by a static push-over analysis (see Chapter 36). The rotational capacity of all plastic hinges should be limited to a safe performance level. The plastic hinge regions should be designed and detailed to perform with minimal strength degradation under cyclic loading.

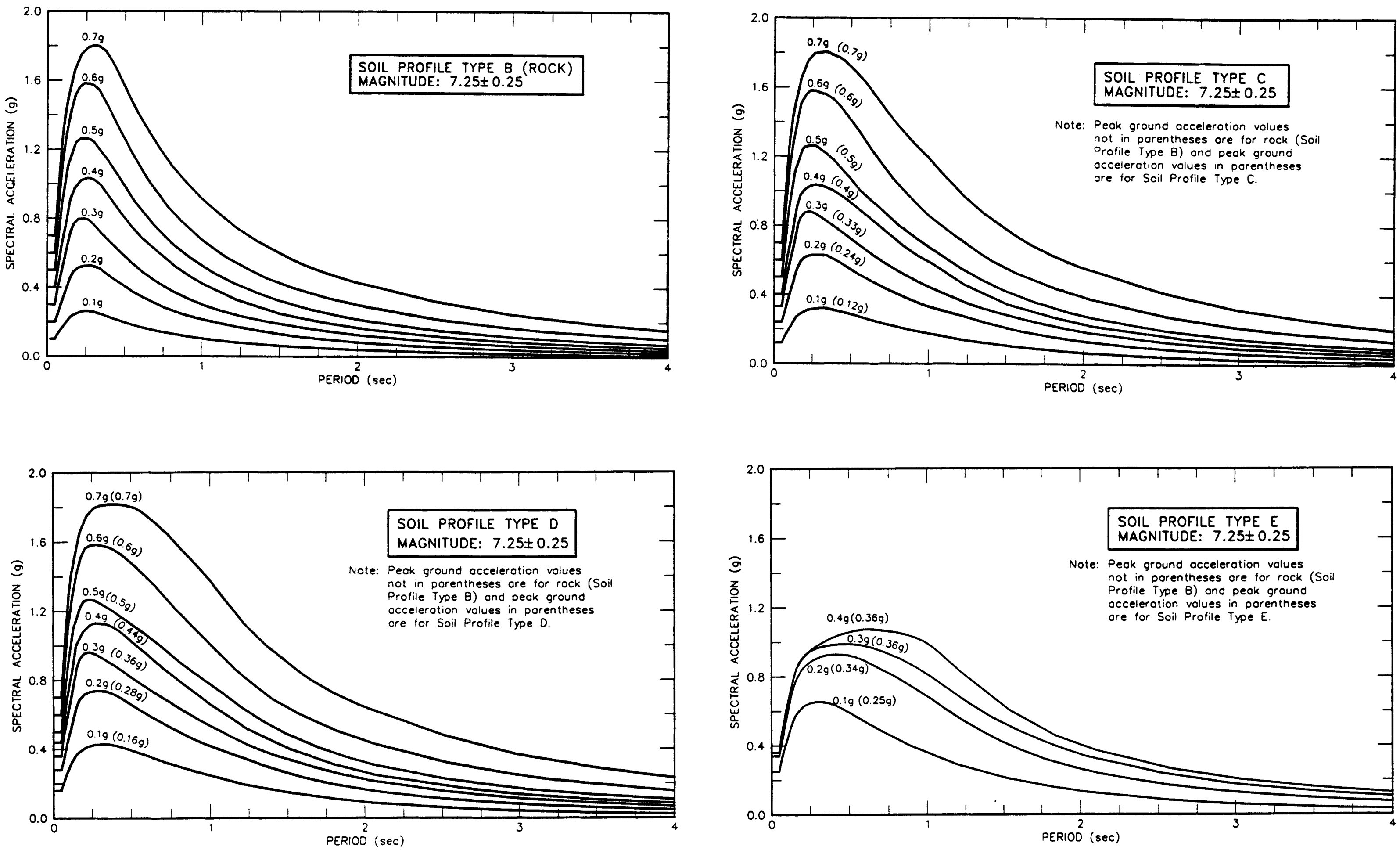

FIGURE 37.5 ATC-32 recommended ARS curves.

Seismic Design Practice

- Bridge type, component selection, member dimensions, and aesthetics should be investigated to reduce the seismic demands to the greatest extent possible. Aesthetics should not be the primary reason for producing undesirable frame and component geometry.
- Simplistic analysis models should be used for initial assessment of structural behavior. The results of more-sophisticated models should be checked for consistency with the results obtained from the simplistic models. The rotational and translational stiffness of abutments and foundations modeled in the seismic analysis must be compatible with their structural and geotechnical capacity. The energy dissipation capacity of the abutments should be considered for bridges whose response is dominated by the abutments.
- The estimated displacement demands under design earthquake should not exceed the global displacement capacity of the structure and the local displacement capacity of any of its individual components.
- Adjacent frames should be proportioned to minimize the differences in the fundamental periods and skew angles, and to avoid drastic changes in stiffness. All bridge frames must meet the strength and ductility requirements in a stand-alone condition. Each frame should provide a well-defined load path with predetermined plastic hinge locations and utilize redundancy whenever possible.
- For concrete bridges, structural components should be proportioned to direct inelastic damage into the columns, pier walls, and abutments. The superstructure should have sufficient overstrength to remain essentially elastic if the columns/piers reach their most probable plastic moment capacity. The superstructure-to-substructure connection for nonintegral caps may be designed to fuse prior to generating inelastic response in the superstructure. The girders, bent caps, and columns should be proportioned to minimize joint stresses. Moment-resisting connections should have sufficient joint shear capacity to transfer the maximum plastic moments and shears without joint distress.
- For steel bridges, structural components should be generally designed to ensure that inelastic deformation only occur in the specially detailed ductile substructure elements. Inelastic behavior in the form of controlled damage may be permitted in some of the superstructure components, such as the cross frames, end diaphragms, shear keys, and bearings. The inertial forces generated by the deck must be transferred to the substructure through girders, trusses, cross frames, lateral bracings, end diaphragms, shear keys, and bearings. As an alternative, specially designed ductile end-diaphragms may be used as structural mechanism fuses to prevent damage in other parts of structures.
- Initial sizing of columns should be based on slenderness ratios, bent cap depth, compressive stress ratio, and service loads. Columns should demonstrate dependable post-yield-displacement capacity without an appreciable loss of strength. Thrust–moment–curvature (P–M–Φ) relationships should be used to optimize the performance of a column under service and seismic loads. Concrete columns should be well proportioned, moderately reinforced, and easily constructed. Abrupt changes in the cross section and the capacity of columns should be avoided. Columns must have sufficient rotation capacity to achieve the target displacement ductility requirements.
- Steel multicolumn bents or towers should be designed as ductile moments-resisting frames (MRF) or ductile braced frames such as concentrically braced frames (CBF) and eccentrically braced frames (EBF). For components expected to behave inelastically, elastic buckling (local compression and shear, global flexural, and lateral torsion) and fracture failure modes should be avoided. All connections and joints should preferably be designed to remain essentially elastic. For MRFs, the primary inelastic deformation should preferably be columns. For CBFs, diagonal members should be designed to yield when members are in tension and to buckle inelastically when they are in compression. For EBFs, a short beam segment designated as a *link* should be well designed and detailed.

TABLE 37.5 ATC-32 Minimum Required Analysis

Bridge Type		Functional Evaluation	Safety Evaluation
Ordinary Bridge	Type I	None required	Equivalent static analysis or elastic dynamic analysis
	Type II	None required	Elastic dynamic analysis
Important Bridge	Type I	Equivalent static analysis or elastic dynamic analysis	
	Type II	Elastic dynamic analysis	Elastic dynamic analysis or inelastic static analysis or inelastic dynamic analysis

- Force demands on the foundation should be based on the most probable plastic capacity of the columns/piers with an appropriate amount of overstrength. Foundation elements should be designed to remain essentially elastic. Pile shaft foundations may experience limited inelastic deformation when they are designed and detailed in a ductile manner.
- The ability of an abutment to resist bridge seismic forces should be based on its structural capacity and the soil resistance that can be reliably mobilized. Skewed abutments are highly vulnerable to damage. Skew angles at abutments should be reduced, even at the expense of increasing the bridge length.
- Necessary restrainers and sufficient seat width should be provided between adjacent frames at all intermediate expansion joints, and at the seat-type abutments to eliminate the possibility of unseating during a seismic event.

37.4.3 ATC Recommendations

ATC-32 Recommendations to Caltrans

The Caltrans seismic performance criteria shown in Table 37.4 provide the basis for development of the ATC-32 recommendations [18]. The major changes recommended for the Caltrans BDS are as follows:

- The importance of relative (rather than absolute) displacement in the seismic performance of bridges is emphasized.
- Bridges are classified as either "important or ordinary." Structural configurations are divided into Type I, simple (similar to regular bridges), and Type II, complex (similar to irregular bridges). For important bridges, two-level design (safety evaluation and function evaluation) approaches are recommended. For ordinary bridges, a single-level design (safety evaluation) is recommended. Minimum analyses required are shown in Table 37.5.
- The proposed family of site-dependent design spectra (which vary from the current Caltrans curves) are based on four of six standard sites defined in a ground motion workshop [31].
- Vertical earthquake design loads may be taken as two thirds of the horizontal load spectra for typical sites not adjacent to active faults.
- A force-based design approach is retained, but some of the inherent shortcomings have been overcome by using new response modification factors and modeling techniques which more accurately estimate displacements. Two new sets of response modification factors Z (Figure 38.4b) are recommended to represent the response of limited and full ductile structural components. Two major factors are considered in the development of the new Z factors: the relationship between elastic and inelastic response is modeled as a function of the natural period of the structure and the predominate period of the ground motion; the distribution of elastic and inelastic deformation within a structural component is a function of its component geometry and framing configuration.

- P-Δ effects should be included using inelastic dynamic analysis unless the following relation is satisfied:

$$\frac{V_o}{W} \geq 4\frac{\delta_u}{H} \tag{37.2}$$

where V_o is base shear strength of the frame obtained from plastic analysis; W is the dead load; δ_u is maximum design displacement; and H is the height of the frame. The inequality in Eq. (37.2) is recommended to keep bridge columns from being significantly affected by P-Δ moments.
- A adjustment factor, R_d, is recommended to adjust the displacement results from an elastic dynamic analysis to reflect the more realistic inelastic displacements that occur during an earthquake.

$$R_d = \left(1 - \frac{1}{Z}\right)\frac{T}{T^*} + \frac{1}{Z} \geq 1 \tag{37.3}$$

where T is the natural period of the structure, T^* is the predominant period of ground motion, and Z is force-reduction coefficient defined in Figure 38.4b.
- Modification was made to the design of ductile elements, the design of nonductile elements using capacity design approach, and the detailing of reinforced concrete for seismic resistance based on recent research findings.
- Steel seismic design guidelines and detailing requirements are very similar to building code requirements.
- Foundation design guidelines include provisions for site investigation, determination of site stability, modeling and design of abutments and wing-walls, pile and spread footing foundations, drilled shafts, and Earth-retaining structures.

ATC-18 Recommendation to FHWA

The ATC recently reviewed current seismic design codes and specifications for highway structures worldwide and provided recommendations for future codes for bridge structures in the United States [19]. The recommendations have implemented significant changes to current specifications, most importantly the two-level design approach, but a single-level design approach is included. The major recommendations are summarized in Tables 37.6 and 37.7.

37.5 Sample Performance-Based Criteria

This section introduces performance-based criteria as a reference guide. A complete set of criteria will include consideration of postearthquake performance criteria, determination of seismic loads and load combinations, material properties, analysis methods, detailed qualitative acceptance criteria. The materials presented in this section are based on successful past experience, various codes and specifications, and state-of-the-art knowledge. Much of this section is based on the Seismic Retrofit Design Criteria developed for the SFOBB west span [17]. It should be emphasized that the sample criteria provided here should serve as a guide and are not meant to encompass all situations.

The postearthquake performance criteria depending on the importance of bridges specified in Table 37.4 are used. Two levels of earthquake loads, FEGM and SEGM, defined in Table 37.4 are required. The extreme event load combination specified by AASHTO-LRFD [23] should be considered (see Chapter 5).

TABLE 37.6 ATC-18 Recommendations for Future Bridge Seismic Code Development (Two-Level Design Approach)

Level		Lower Level Functional Evaluation	Upper Level Safety Evaluation
Performance Criteria	Ordinary bridges	Service level — immediate Damage level — repairable	Service level — limited Damage level — significant
	Important bridges	Service level — immediate Damage level — minimum	Service level — immediate Damage level — repairable
Design load		Functional evaluation ground motion	Safety evaluation ground motion
Design approach		• Continue current AASHTO seismic performance category • Adopt the two-level design approach at least for important bridges in higher seismic zones • Use elastic design principles for the lower-level design requirement • Use nonlinear analysis — deformation-based procedures for the upper-level design	
Analysis		Current elastic analysis procedures (equivalent static and multimodel)	Nonlinear static analysis
Design force	Ductile component	Remain undamaged	Have adequate ductility to meet the performance criteria
	Nonductile component	Remain undamaged	For sacrificial element — ultimate strength should be close to but larger than that required for the lower-level event For nonsacrificial element — based on elastic demands or capacity design procedure
	Foundation	Capacity design procedure — to ensure there is no damage	
Design displacement		Use the upper-level event Remain current seat width requirements Consider overall draft limits to avoid P-Δ effects on long-period structures	
Concrete and steel design		Use the capacity design procedure for all critical members	
Foundation design		• Complete geotechical analysis for both level events • Prevent structural capacity of the foundations at the lower level event • Allow damage in the upper-level event as long as it does not lead to catastrophic failure	

Functional Evaluation Ground Motion (*FEGM*): Probabilistic assessed ground motions that have a 72 ~ 250 year return period (i.e., 30 to 50% probability of exceedance during the useful life a bridge).
Safety Evaluation Ground Motion (*SEGM*): Probabilistic assessed ground motions that have a 950 or 2475 year return period (10% probability of exceedance for a design life of 100 ~ 250 years).
Immediate Service Level: Full access to normal traffic is available almost immediately (i.e., within hours) following the earthquake (It may be necessary to allow 24 h or so for inspection of the bridge).
Limited Service Level: Limited access (reduced lanes, light emergency traffic) is possible within 3 days of the earthquake. Full service restoration within months.
Minimum Damage: Minor inelastic deformation such as narrow flexural cracking in concrete and no apparent deformations.
Repairable Damage: Damage such as concrete cracking, minor spalling of cover concrete, and steel yield that can be repaired without requiring closure and replacing structural members. Permanent offsets are small.
Significant Damage: Damage such as concrete cracking, major spalling of concrete, steel yield that can be repaired only with closure, and partial or complete replacement. Permanent offset may occur without collapse.

37.5.1 Determination of Demands

Analysis Methods

For ordinary bridges, seismic force and deformation demands may be obtained by equivalent static analysis or elastic dynamic response spectrum analysis. For important bridges, the following guidelines may apply:

1. Static linear analysis should be used to determine member forces due to self-weight, wind, water currents, temperature, and live load.
2. Dynamic response spectrum analysis [32] should be used for local and regional stand-alone models and the simplified global model to determine mode shapes, periods, and initial estimates of seismic force and displacement demands. The analysis may be used on global models prior to a time history analysis to verify global behavior, eliminate modeling errors,

TABLE 37.7 ATC-18 Recommendations for Future Bridge Seismic Code Development (One-Level Approach)

Design philosophy		For lower-level earthquake, there should be only minimum damage For a significant earthquake, collapse should be prevented but significant damage may occur; damage should occur at visible locations The following addition to Item 2 is required if different response modification (R and Z) factors are used for important or ordinary bridges Item 2 as it stands would apply to ordinary bridges For important bridges, only repairable would be expected during a significant earthquake
Design load		Single-level — safety evaluation ground motion — 950 or 2475 year return period for the eastern and western portions of the U.S.
Design approach		• Continue current AASHTO seismic performance category • Use nonlinear analysis deformation-based procedures with strength and stiffness requirements being derived from appropriate nonlinear response spectra
Analysis		Nonlinear static analysis should be part of any analysis requirement At a minimum, nonlinear static analysis is required for important bridges Current elastic analysis and design procedure may be sufficient for small ordinary bridges Incorporate both current R-factor elastic procedure and nonlinear static analysis
Design Force	Ductile component	R-factor elastic design procedure or nonlinear static analysis
	Nonductile component	For sacrificial element, should be designed using a guideline that somewhat correspond to the design level of an unspecified lower-level event, for example, one half or one third of the force required for the upper-level event For nonsacrificial element, should be designed for elastic demands or capacity design procedure
	Foundation	Capacity design procedure — to ensure there is no damage
Design displacement		Maintain current seat width requirements Consider overall draft limits to avoid P-Δ effects on long-period structures
Concrete and steel design		Use the capacity design procedure for all critical members
Foundation design		• Complete geotechical analysis for the upper-level event • For nonessential bridges, a lower level (50% of the design acceleration) might be appropriate

and identify initial regions or members where inelastic behavior needs further refinement and inelastic nonlinear elements. In the analysis:

- Site-specific ARS curves should be used with 5% damping.
- Modal response should be combined using the complete quadratic combination (CQC) method and the resulting orthogonal responses should be combined using either the square root of the sum of the squares (SRSS) method or the "30%" rule as defined by AASHTO-LRFD [1994].

3. Dynamic Time History Analysis: Site-specific multisupport dynamic time histories should be used in a dynamic time history analysis [33].
   - Linear elastic dynamic time history analysis is defined as a dynamic time history analysis with consideration of geometric linearity (small displacement), linear boundary conditions, and elastic members. It should only be used to check regional and global models.
   - Nonlinear elastic dynamic time history analysis is defined as a dynamic time history analysis with consideration of geometric nonlinearity, linear boundary conditions, and elastic members. It should be used to determine areas of inelastic behavior prior to incorporating inelasticity into regional and global models.
   - Nonlinear inelastic dynamic time history analysis, level I, is defined as a dynamic time history analysis with consideration of geometric nonlinearity, nonlinear boundary conditions, inelastic elements (for example, seismic isolators and dampers), and elastic members. It should be used for final determination of force and displacement demands for existing structures in combination with static gravity, wind, thermal, water current, and live loads as specified in AASHTO-LRFD [23].

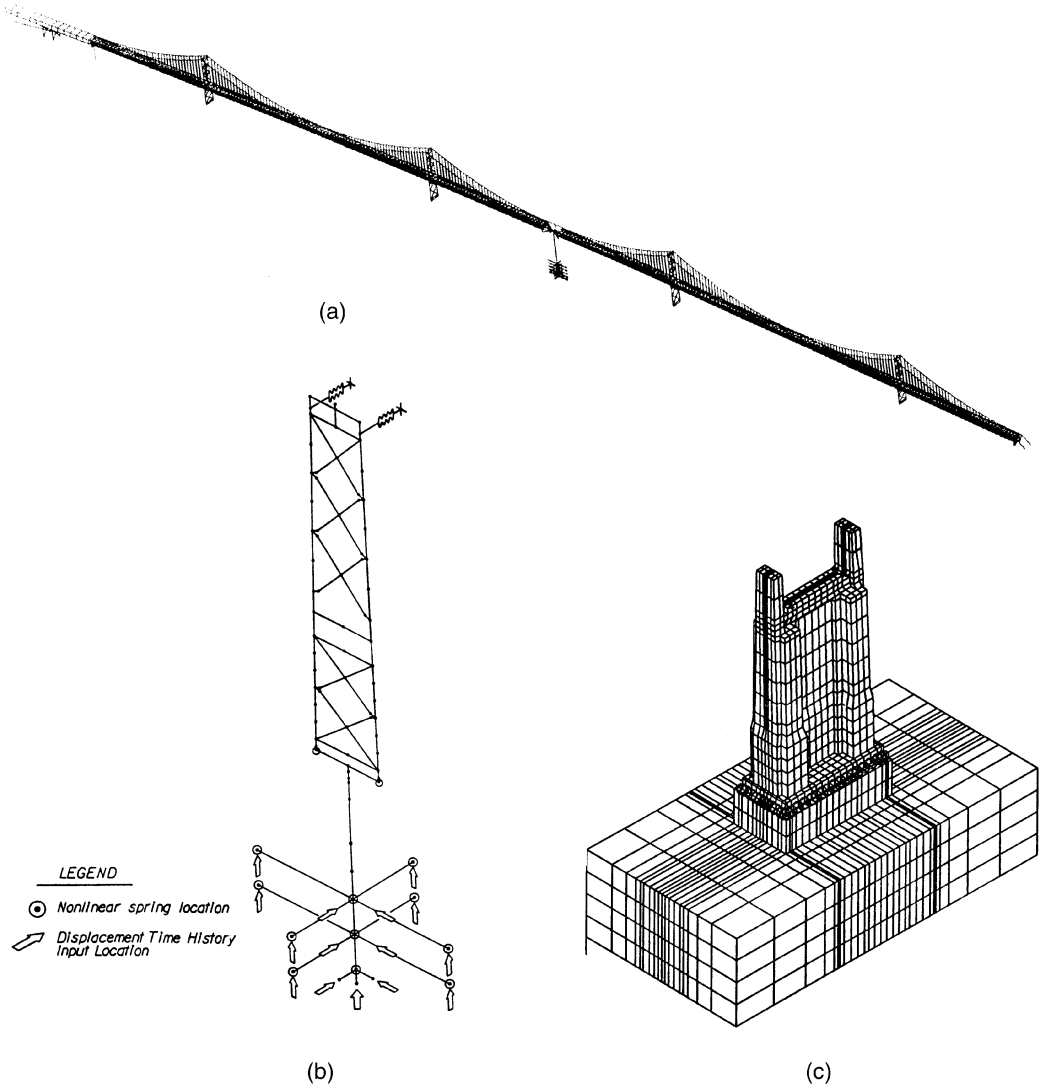

FIGURE 37.6 (a) Global, (b) Regional models for towers, and (c) local model for PW-1 for San Francisco–Oakland Bay Bridge west spans.

- Nonlinear inelastic dynamic time history analysis, level II, is defined as a dynamic time history analysis with consideration of geometric nonlinearity, nonlinear boundary conditions, inelastic elements (for example, dampers), and inelastic members. It should be used for the final evaluation of response of the structures.

Modeling Considerations

1. *Global, Regional, and Local Models*
   The global models consider overall behavior and may include simplifications of complex structural elements (Figure 37.6a). Regional models concentrate on regional behavior (Figure 37.6b). Local models (Figure 37.6c) emphasize the localized behavior, especially complex inelastic and nonlinear behavior. In regional and global models where more than one foundation location is included in the model, multisupport time history analysis should be used.

2. *Boundary Conditions*
   Appropriate boundary conditions should be included in regional models to represent the interaction between the region and the adjacent structure. The adjacent portion is not explicitly modeled but may be simplified using a combination of springs, dashpots, and lumped masses. Appropriate nonlinear elements such as gap elements, nonlinear springs, seismic response modification devices (SRMDs), or specialized nonlinear finite elements should be included where the behavior and response of the structure is sensitive to such elements.
3. Soil–Foundation–Structure Interaction
   This interaction may be considered using nonlinear or hysteretic springs in global and regional models. Foundation springs to represent the properties of the soil at the base of the structure should be included in both regional and global models (see Chapter 42).
4. Damping
   When nonlinear material properties are incorporated in the model, Rayleigh damping should be reduced (perhaps 20%) from the elastic properties.
5. Seismic Response Modification Devices
   The SRMDs should be modeled explicitly with hysteretic characteristics determined by experimental data. See Chapter 41 for a detailed discussion of this behavior.

37.5.2 Determination of Capacities

Limit States and Resistance Factors

The *limit state* is defined as that condition of a structure at which it ceases to satisfy the provisions for which it was designed. Two kinds of limit state corresponding to SEGM and FEGM specified in Table 37.4 apply for seismic design and retrofit. To account for unavoidable inaccuracies in the theory, variation in the material properties, workmanship, and dimensions, nominal strength of structural components should be modified by a resistance factor ϕ specified by AASHTO-LRFD [23] or project-specific criteria to obtain the design capacity or strength (resistance).

Nominal Strength of Structural Components

The strength capacity of structural members should be determined in accordance with specified code formula [23,26, Chapters 38 and 39], or verified with experimental and analytical computer models, or project-specific criteria [19].

Structural Deformation Capacity

Structural deformation capacity should be determined by nonlinear inelastic analysis and based on acceptable damage levels as shown in Table 37.4. The quantitative definition of the damage corresponding to different performance requirements has not been specified by the current Caltrans BDS [26], AASHTO-LRFD [23], and ATC recommendations [18,19] because of the lack of consensus. As a starting point, Table 37.8 provides a quantitative strain and ductility limit corresponding to the three damage levels.

The displacement capacity should be evaluated considering both material and geometric nonlinearities. Proper boundary conditions for various structures should be carefully considered. A static push-over analysis (see Chapter 36) may be suitable for most bridges. A nonlinear inelastic dynamic time history analysis, Level II, may be required for important bridges. The available displacement capacity is defined as the displacement corresponding to the most critical of (1) 20% load reduction from the peak load or (2) the strain limit specified in Table 37.8.

Seismic Response Modification Devices

SRMDs include energy dissipation and seismic isolation devices. Energy dissipation devices increase the effective damping of the structure, thereby reducing reaction forces and deflections. Isolation devices change the fundamental mode of vibration so that the response of the structure is lowered; however, the reduced force may be accompanied by an increased displacement.

TABLE 37.8 Damage Levels, Strain, and Ductility

Damage level	Strain		Ductility	
	Concrete	Steel	Curvature μ_ϕ	Displacement μ_Δ
Significant	ε_{cu}	ε_{sh}	8 ~ 10	4 ~ 6
Repairable	Larger $\begin{cases} 0.005 \\ \dfrac{2\varepsilon_{cu}}{3} \end{cases}$	Larger $\begin{cases} 0.08 \\ \dfrac{2\varepsilon_{y}}{3} \end{cases}$	4 ~ 6	2 ~ 4
Minimum	Larger $\begin{cases} 0.004 \\ \varepsilon_{cu} \end{cases}$	Larger $\begin{cases} 0.03 \\ 15\varepsilon_{y} \end{cases}$	2 ~ 4	1 ~ 2

ε_{cu} = ultimate concrete compression strain depending of confinement (see Chapter 36)
ε_y = yield strain of steel
ε_{sh} = hardening strain of steel
μ_ϕ = curvature ductility (ϕ_u/ϕ_y)
μ_Δ = displacement ductility (Δ_u/Δ_y) (see Chapter 36)

The properties of SRMDs should be determined by the specified testing program. References are made to AASHTO [34], Caltrans [35], and Japan Ministry of Construction (JMC) [36]. Consideration of following items should be made in the test specifications:

- Scales — at least two full-scale test specimens are required;
- Loading (including lateral and vertical) history and rate;
- Durability — design life;
- Deterioration — expected levels of strength and stiffness.

37.5.3 Performance Acceptance Criteria

To achieve the performance objectives in Table 37.4, various structural components should satisfy the acceptable demand/capacity ratios (DC_{accept},) specified in this section. The form of the equation is:

$$\frac{\text{Demand}}{\text{Capacity}} \le DC_{\text{accept}} \tag{37.4}$$

where *demand,* in terms of factored moments, shears, and axial forces, and displacement and rotation deformations, should be determined by a nonlinear inelastic dynamic time history analysis, level I, for important bridges, and dynamic response spectrum analysis for ordinary bridges defined in Section 37.5.1, and *capacity,* in terms of factored strength and deformation capacities, should be obtained according to Section 37.5.2.

Structural Component Classifications

Structural components are classified into two categories: *critical* or *other.* It is the aim that other components may be permitted to function as *fuses* so that the critical components of the bridge system can be protected during the functionality evaluation earthquake (FEE) and the safety evaluation earthquake (SEE). As an example, Table 37.9 shows structural component classifications and their definition for a suspension bridge.

TABLE 37.9 Structural Component Classification

Component Classification	Definition	Example (SFOBB West Spans)
Critical	Components on a critical path that carry bridge gravity load directly The loss of capacity of these components would have serious consequences on the structural integrity of the bridge	Suspension cables Continuous trusses Floor beams and stringers Tower legs Central anchorage A-Frame Piers W-1 and W2 Bents A and B Caisson foundations Anchorage housings Cable bents
Other	All components other than Critical	All other components

Note: Structural components include members and connections.

Steel Structures

1. *General Design Procedure*
   Seismic design of steel members should be in accordance with the procedure shown in Figure 37.7. Seismic retrofit design of steel members should be in accordance with the procedure shown in Figure 37.8.
2. *Connections*
   Connections should be evaluated over the length of the seismic event. For connecting members with force *D*/*C* ratios larger than 1.0, 25% greater than the nominal capacity of the connecting members should be used in connection design.
3. *General Limiting Slenderness Parameters and Width–Thickness Ratios*
   For all steel members (regardless of their force *D*/*C* ratios), the slenderness parameter for axial load dominant members (λ_c) and for flexural dominant members (λ_b) should not exceed the limiting values ($0.9\lambda_{cr}$ or $0.9\lambda_{br}$ for *critical*, λ_{cr} or λ_{br} for *Others*) shown in Table 37.10.
4. *Acceptable Force* D/C *Ratios and Limiting Values*
   Acceptable force *D*/C ratios, DC_{accept} and associated limiting slenderness parameters and width–thickness ratios for various members are specified in Table 37.10. For all members with *D*/*C* ratios larger than 1.0, slenderness parameters and width–thickness ratios should not exceed the limiting values specified in Table 37.10. For existing steel members with *D*/*C* ratios less than 1.0, width–thickness ratios may exceed λ_r specified in Table 37.11 and AISC-LRFD [37].

The following symbols are used in Table 37.10. M_u is the factored moment demand; P_u is the vactored axial force demand; M_n is the nominal moment strength of a member; P_n is the nominal axial strength of a member; λ is the width–thickness (b/t or h/t_w) ratio of a compressive element; $\lambda_c = (KL/r\pi)\sqrt{F_y/E}$, the slenderness parameter of axial load dominant members; $\lambda_b = L/r_y$, the slenderness parameter of flexural moment dominant members; $\lambda_{cp} = 0.5$, the limiting column slenderness parameter for 90% of the axial yield load based on AISC-LRFD [37] column curve; λ_{bp} is the limiting beam slenderness parameter for plastic moment for seismic design; $\lambda_{cr} = 1.5$, the limiting column slenderness parameter for elastic buckling based on AISC-LRFD [37] column curve; λ_{br} is the limiting beam slenderness parameter for elastic lateral torsional buckling;

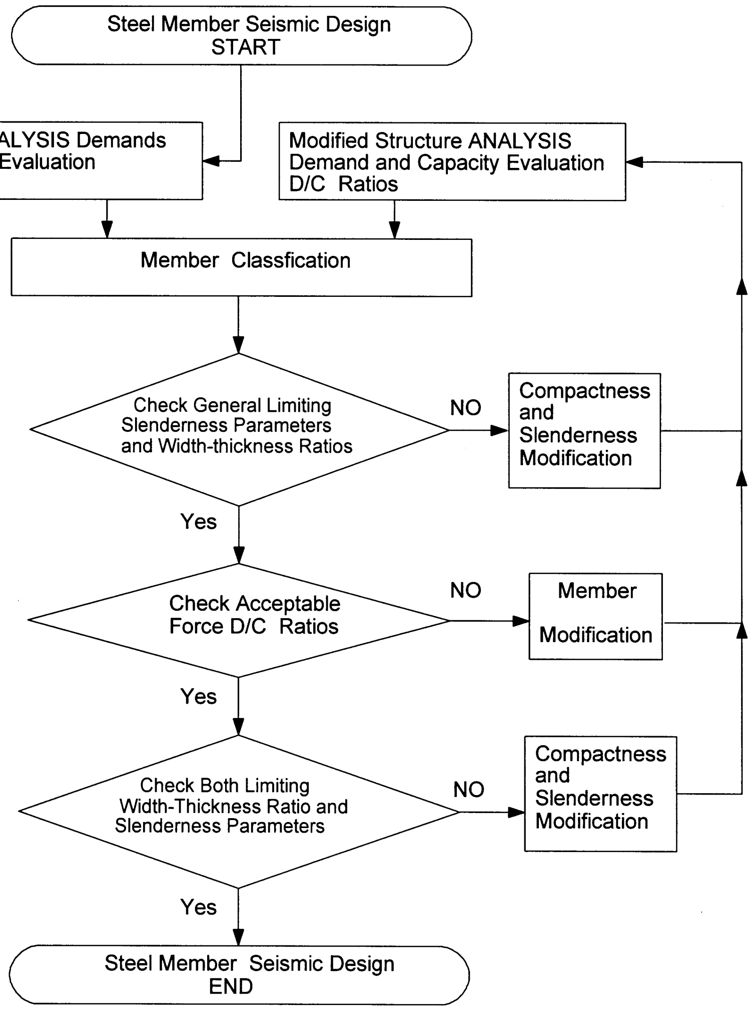

FIGURE 37.7 Steel member seismic design procedure.

$$\lambda_{br} = \begin{cases} \dfrac{57,000\sqrt{JA}}{M_r} & \text{for solid rectangular bars and box sections} \\[2ex] \dfrac{X_1}{F_L}\sqrt{1+\sqrt{1+X_2F_L^2}} & \text{for doubly symmetric I-shaped members and channels} \end{cases}$$

$$M_r = \begin{cases} F_L\, S_x & \text{for I - shaped member} \\ F_{yf}\, S_x & \text{for solid rectangular and box section} \end{cases}$$

$$X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}} \qquad X_s = \frac{4C_w}{I_y}\left(\frac{S_x}{GJ}\right) \qquad F_L = \text{smaller}\begin{cases} F_{yw} \\ F_{yf} - F_r \end{cases}$$

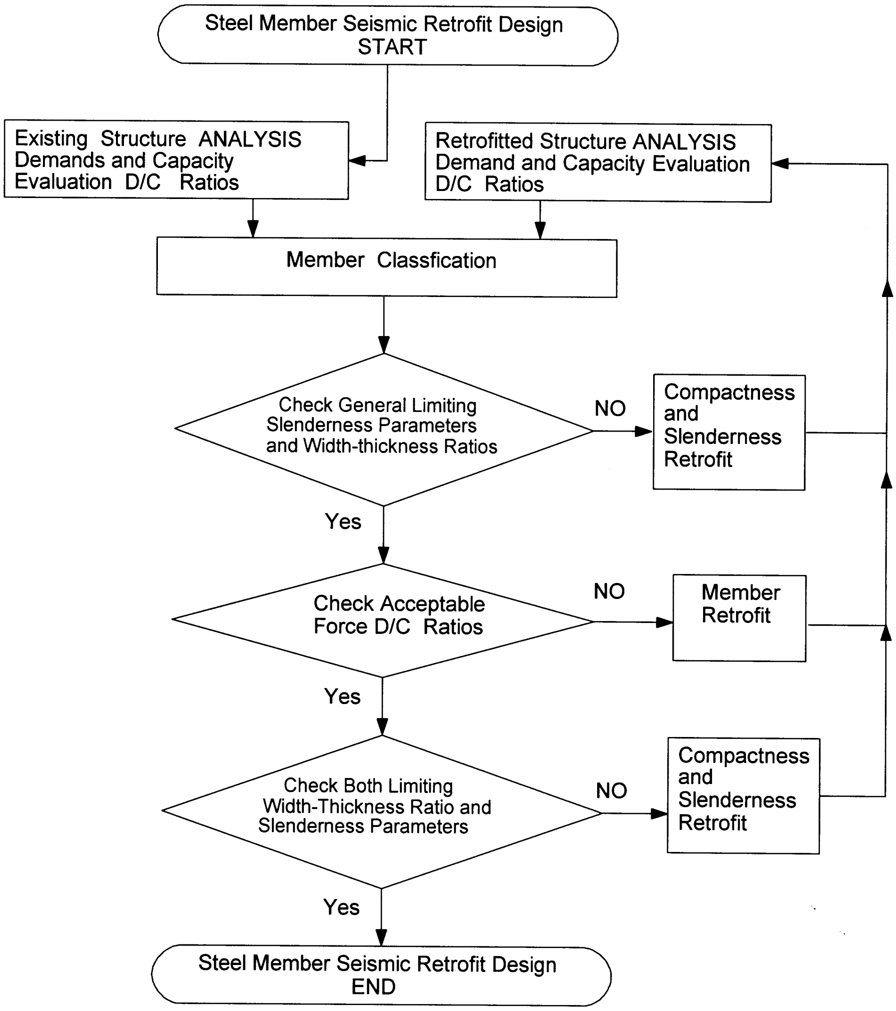

FIGURE 37.8 Steel member seismic retrofit design procedure.

where A is the cross-sectional area, in.^{2}; L is the unsupported length of a member; J is the torsional constant, in.^{4}; r is the radius of gyration, in.; r_y is the radius of gyration about minor axis, in.; F_y is the yield stress of steel; F_{yw} is the yield stress of web, ksi; F_{yf} is the yield stress of flange, ksi; E is the modulus of elasticity of steel (29,000 ksi); G is the shear modulus of elasticity of steel (11,200 ksi); S_x is the section modulus about major axis, in.^{3}; I_y is the moment of inertia about minor axis, in.^{4} and C_w is the warping constant, in.^{6} For doubly symmetric and singly symmetric I-shaped members with compression flange equal to or larger than the tension flange, including hybrid members (strong axis bending):

$$\lambda_{bp} = \begin{cases} \dfrac{\left[3600 + 2200\, M_1/M_2\right]}{F_y} & \text{for } \textit{other} \text{ members} \\[2ex] \dfrac{300}{\sqrt{F_{yf}}} & \text{for } \textit{critical} \text{ members} \end{cases} \tag{37.5}$$

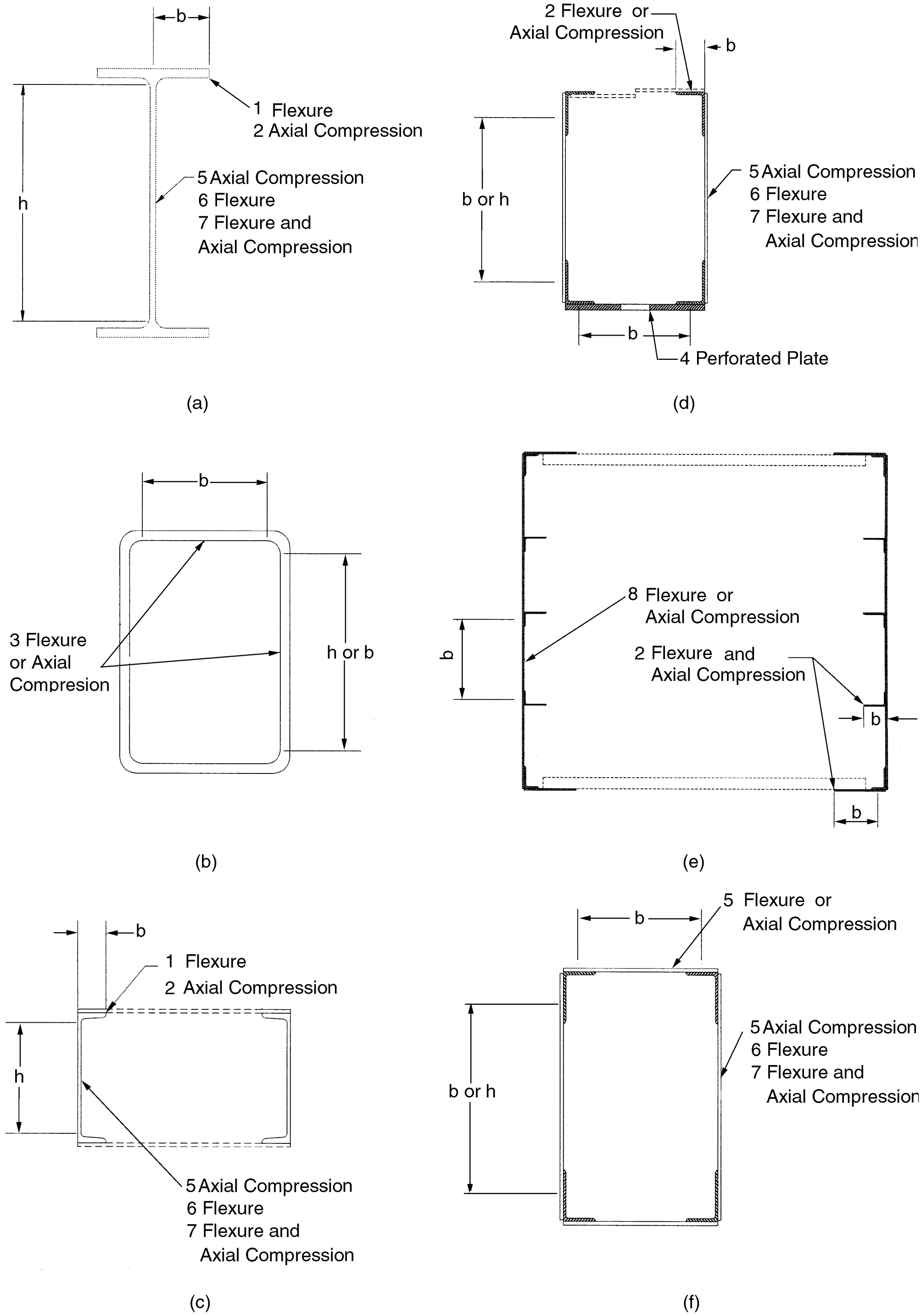

FIGURE 37.9 Typical cross sections for steel members: (a) rolled I section; (b) hollow structured tube; (c) built-up channels; (d) built-up box section; (e) longitudinally stiffened built-up box section; (f) built-up box section.

TABLE 37.10 Acceptable Force Demand/Capacity Ratios and Limiting Slenderness Parameters and Width/Thickness Ratios

Member Classification		Limiting Ratios: Slenderness Parameter (λ_c and λ_b)	Limiting Ratios: Width/Thickness λ (b/t or h/t_w)	Acceptable Force D/C Ratio DC_{accept}
Critical	Axial load dominant	$0.9\lambda_{cr}$	λ_r	$DC_r = 1.0$
	$P_u/P_n \geq M_u/M_n$	λ_{cpr}	λ_{pr}	$1.0 \sim 1.2$
		λ_{cp}	λ_p	$DC_p = 1.2$
	Flexural moment dominant	$0.9\lambda_{br}$	λ_r	$DC_r = 1.0$
	$M_u/M_n > P_u/P_n$	λ_{bpr}	λ_{pr}	$1.2 \sim 1.5$
		λ_{bp}	λ_p	$DC_p = 1.5$
Other	Axial load dominant	λ_{cr}	λ_r	$DC_r = 1.0$
	$P_u/P_n \geq M_u/M_n$	λ_{cpr}	λ_{pr}	$1.0 \sim 2.0$
		λ_{cp}	$\lambda_{p\text{-Seismic}}$	$DC_p = 2$
	Flexural moment dominant	λ_{br}	λ_r	$DC_r = 1.0$
	$M_u/M_n > P_u/P_n$	λ_{bpr}	λ_{pr}	$1.0 \sim 2.5$
		λ_{bp}	$\lambda_{p\text{-Seismic}}$	$DC_p = 2.5$

in which M_1 is larger moment at end of unbraced length of beam; M_2 is smaller moment at end of unbraced length of beam; (M_1/M_2) is positive when moments cause reverse curvature and negative for single curvature.

For solid rectangular bars and symmetric box beam (strong axis bending):

$$\lambda_{bp} = \begin{cases} \dfrac{5000 + 3000\left(M_1/M_2\right)}{F_y} \geq \dfrac{3000}{F_y} & \text{for } \textit{other} \text{ members} \\ \dfrac{3750}{M_p}\sqrt{JA} & \text{for } \textit{critical} \text{ members} \end{cases} \tag{37.6}$$

in which M_p is plastic moment $(Z_x F_y)$; Z_x is plastic section modulus about major axis; and λ_r, λ_p, $\lambda_{p\text{–Seismic}}$ are limiting width thickness ratios specified by Table 37.11.

$$\lambda_{pr} = \begin{cases} \left[\lambda_p + \left(\lambda_r - \lambda_p\right)\left(\dfrac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right)\right] & \text{for } \textit{critical} \text{ members} \\ \left[\lambda_{p\text{-Seismic}} + \left(\lambda_r - \lambda_{p\text{-Seismic}}\right)\left(\dfrac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right)\right] & \text{for } \textit{other} \text{ members} \end{cases} \tag{37.7}$$

For axial load dominant members $(P_u/P_n \geq M_u/M_n)$

$$\lambda_{cpr} = \begin{cases} \lambda_{cp} + \left(0.9\lambda_{cr} - \lambda_{cp}\right)\left(\dfrac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{critical} \text{ members} \\ \lambda_{cp} + \left(\lambda_{cr} - \lambda_{cp}\right)\left(\dfrac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{other} \text{ members} \end{cases} \tag{37.8}$$

For flexural moment dominant members $(M_u/M_n > P_u/P_n)$

TABLE 37.11 Limiting Width-Thickness Ratios

No	Description of Elements	Examples	Width-Thickness Ratios	λ_r	λ_p	$\lambda_{p\text{-}Seismic}$
			Unstiffened Elements			
1	Flanges of I-shaped rolled beams and channels in flexure	Figure 37.19a Figure 37.19c	b/t	$\frac{141}{F_y - 10}$	$\frac{65}{\sqrt{F_y}}$	$\frac{52}{\sqrt{F_y}}$
2	Outstanding legs of pairs of angles in continuous contact; flanges of channels in axial compression; angles and plates projecting from beams or compression members	Figure 37.19d Figure 37.19e Figure 37.19f	b/t	$\frac{95}{\sqrt{F_y}}$	$\frac{63}{\sqrt{F_y}}$	$\frac{32}{\sqrt{F_y}}$
			Stiffened Elements			
3	Flanges of square and rectangular box and hollow structural section of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds.	Figure 37.19b	b/t	$\frac{238}{\sqrt{F_y}}$	$\frac{190}{\sqrt{F_y}}$	F_y (tubes) F_y (others)
4	Unsupported width of cover plates perforated with a succession of access holes	Figure 37.19d	b/t	$\frac{317}{\sqrt{F_y}}$	$\frac{233}{\sqrt{F_y}}$	$\frac{132}{\sqrt{F_y}}$

5	All other uniformly compressed stiffened elements, i.e., supported along two edges.	Figures 37.19a, c,d,f	b/t h/t_w	$\frac{253}{\sqrt{F_y}}$	$\frac{190}{\sqrt{F_y}}$	$\sqrt{F_y}$ (w/lacing) $\sqrt{F_y}$ (others)
6	Webs in flexural compression	Figures 37.19a,c,d,f	h/t_w	$\frac{970}{\sqrt{F_y}}$	$\frac{640}{\sqrt{F_y}}$	$\frac{520}{\sqrt{F_y}}$
7	Webs in combined flexural and axial compression	Figures 37.19a,c,d,f	h/t_w	$\frac{970}{\sqrt{F_y}}$ $\left(1 - \frac{0.74P}{\phi P_y}\right)$	For $P_u \le 0.125\phi_b P_y$ $\frac{640}{\sqrt{F_y}}\left(1 - \frac{2.75P_u}{\phi_b P_y}\right)$ For $P_u > 0.125\phi_b P_y$ $\frac{191}{\sqrt{F_y}}\left(2.33 - \frac{P_u}{\phi_b P_y}\right)$ $\ge \frac{253}{\sqrt{F_y}}$	For $P_u \le 0.125\phi_b P_y$ $\frac{520}{\sqrt{F_y}}\left(1 - \frac{1.54P_u}{\phi_b P_y}\right)$ For $P_u > 0.125\phi_b P_y$ $\frac{191}{\sqrt{F_y}}\left(2.33 - \frac{P_u}{\phi_b P_y}\right)$ $\ge \frac{253}{\sqrt{F_y}}$
8	Longitudinally stiffened plates in compression	Figure 37.19e	b/t	$\frac{113\sqrt{k}}{\sqrt{F_y}}$	$\frac{93\sqrt{k}}{\sqrt{F_y}}$	$\frac{73\sqrt{k}}{\sqrt{F_y}}$

Notes:
1. Width–thickness ratios shown in **bold** are from AISC-LRFD [1993] and AISC-Seismic Provisions [1997].
2. k = buckling coefficient specified by Article 6.11.2.1.3a of AASHTO-LRFD [AASHTO, 1994]
   for $n = 1$, $k = (8I_s/bt^3)^{1/3} \le 4.0$; for $n = 2, 3, 4,$ and 5, $k = (14.3I_s/bt^3n^4)^{1/3} \le 4.0$
   n = number of equally spaced longitudinal compression flange stiffeners
   I_s = moment of inertia of a longitudinal stiffener about an axis parallel to the bottom flange and taken at the base of the stiffener

$$\lambda_{bpr} \begin{cases} \lambda_{bp} + \left(0.9\lambda_{br} - \lambda_{bp}\right)\left(\dfrac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{critical} \text{ members} \\ \lambda_{bp} + \left(\lambda_{br} - \lambda_{bp}\right)\left(\dfrac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{other} \text{ members} \end{cases} \tag{37.9}$$

Concrete Structures

1. *General*
   For all concrete compression members (regardless of D/C ratios), the slenderness parameter (KL/r) should not exceed 60.

   For *critical* components, force $DC_{\text{accept}} = 1.2$ and deformation $DC_{\text{accept}} = 0.4$.
   For *other* components, force $DC_{\text{accept}} = 2.0$ and deformation $DC_{\text{accept}} = 0.67$.

2. *Beam–Column* (*Bent Cap*) *Joints*
   For concrete box-girder bridges, the beam–column (bent cap) joints should be evaluated and designed in accordance with the following guidelines [38,39]:
   a. Effective Superstructure Width: The effective width of superstructure (box girder) on either side of a column to resist longitudinal seismic moment at bent (support) should not be taken as larger than the superstructure depth.
      - The immediately adjacent girder on either side of a column within the effective superstructure width is considered effective.
      - Additional girders may be considered effective if refined bent–cap torsional analysis indicates that the additional girders can be mobilized.
   b. Minimum Bent–Cap Width: Minimum cap width outside column should not be less than $D/4$ (D is column diameter or width in that direction) or 2 ft (0.61 m).
   c. Acceptable Joint Shear Stress:
      - For existing unconfined joints, acceptable principal tensile stress should be taken as $3.5\sqrt{f_c'}$ psi $\left(0.29\sqrt{f_c'}\text{ MPa}\right)$. If the principal tensile stress demand exceeds this limiting value, the joint shear reinforcement specified in Item d should be provided.
      - For new joints, acceptable principal tensile stress should be taken as $12\sqrt{f_c'}$ psi ($1.0\sqrt{f_c'}$ MPa).
      - For existing and new joints, acceptable principal compressive stress shall be taken as f_c'.
   d. Joint Shear Reinforcement
      - Typical flexure and shear reinforcement (see Figures 37.10 and 37.11) in bent caps should be supplemented in the vicinity of columns to resist joint shear. All joint shear reinforcement should be well distributed and provided within $D/2$ from the face of column.
      - Vertical reinforcement including cap stirrups and added bars should be 20% of the column reinforcement anchored into the joint. Added bars shall be hooked around main longitudinal cap bars. Transverse reinforcement in the join region should consist of hoops with a minimum reinforcement ratio of 0.4(column steel area)/(embedment length of column bar into the bent cap)^2.
      - Horizontal reinforcement should be stitched across the cap in two or more intermediate layers. The reinforcement should be shaped as hairpins, spaced vertically at not more than 18 in. (457 mm). The hairpins should be 10% of column reinforcement. Spacing should be denser outside the column than that used within the column.
      - Horizontal side face reinforcement should be 10% of the main cap reinforcement including top and bottom steel.

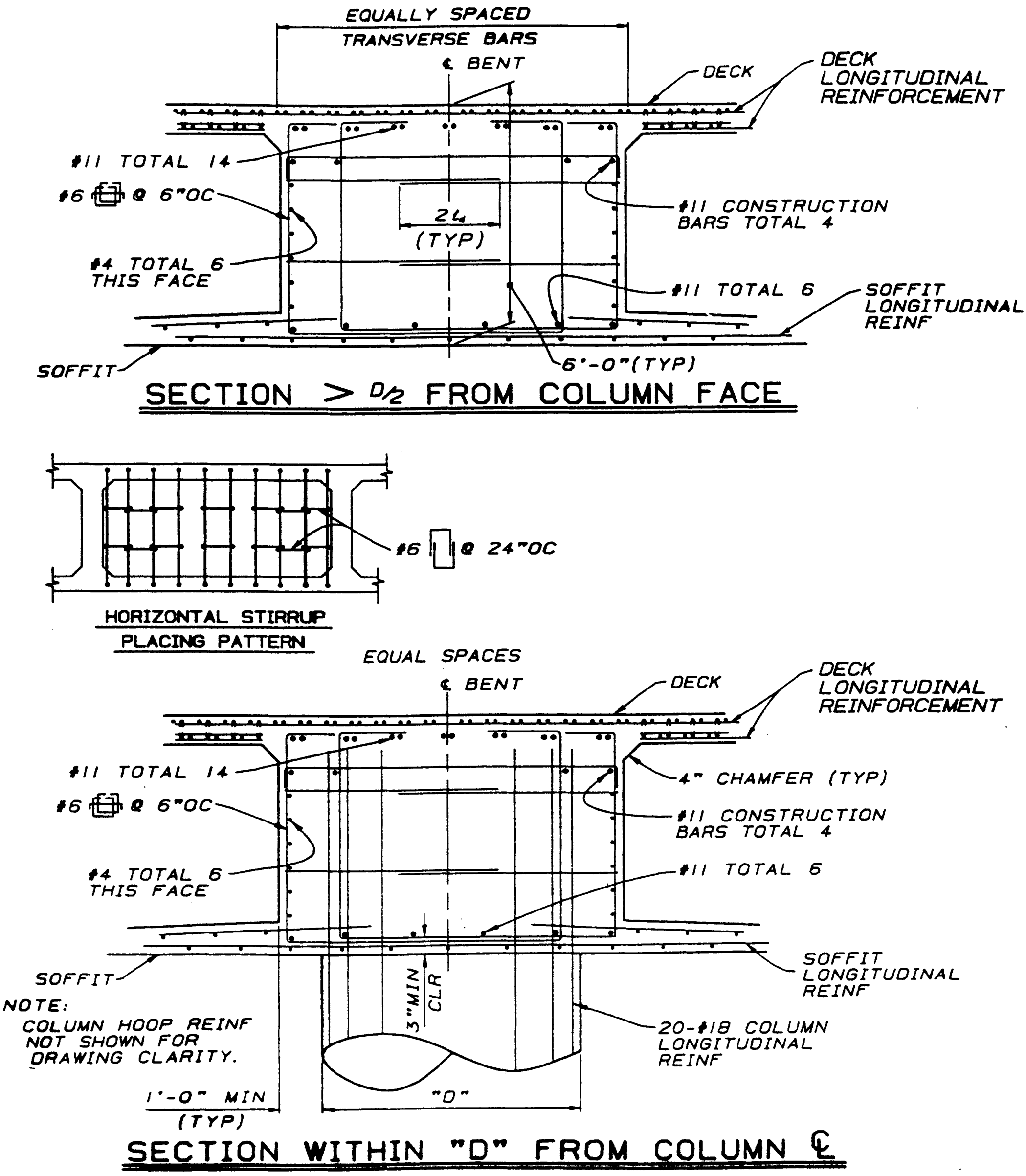

FIGURE 37.10 Example cap joint shear reinforcement — skews 0° to 20°.

- For bent caps skewed greater than 20°, the vertical J-bars hooked around longitudinal deck and bent cap steel should be 8% of column steel (see Figure 37.11). The J-bars should be alternatively 24 in. (600 mm) and 30 in. (750 mm) long and placed within a width of column dimension an either side of the column centerline.
- All vertical column bars should be extended as high as practically possible without interfering with the main cap bars.

Seismic Response Modification Devices

Analysis methods specified in Section 37.5.3 apply for determining seismic design forces and displacements on SRMDs. Properties or capacities of SRMDs should be determined by specified tests.

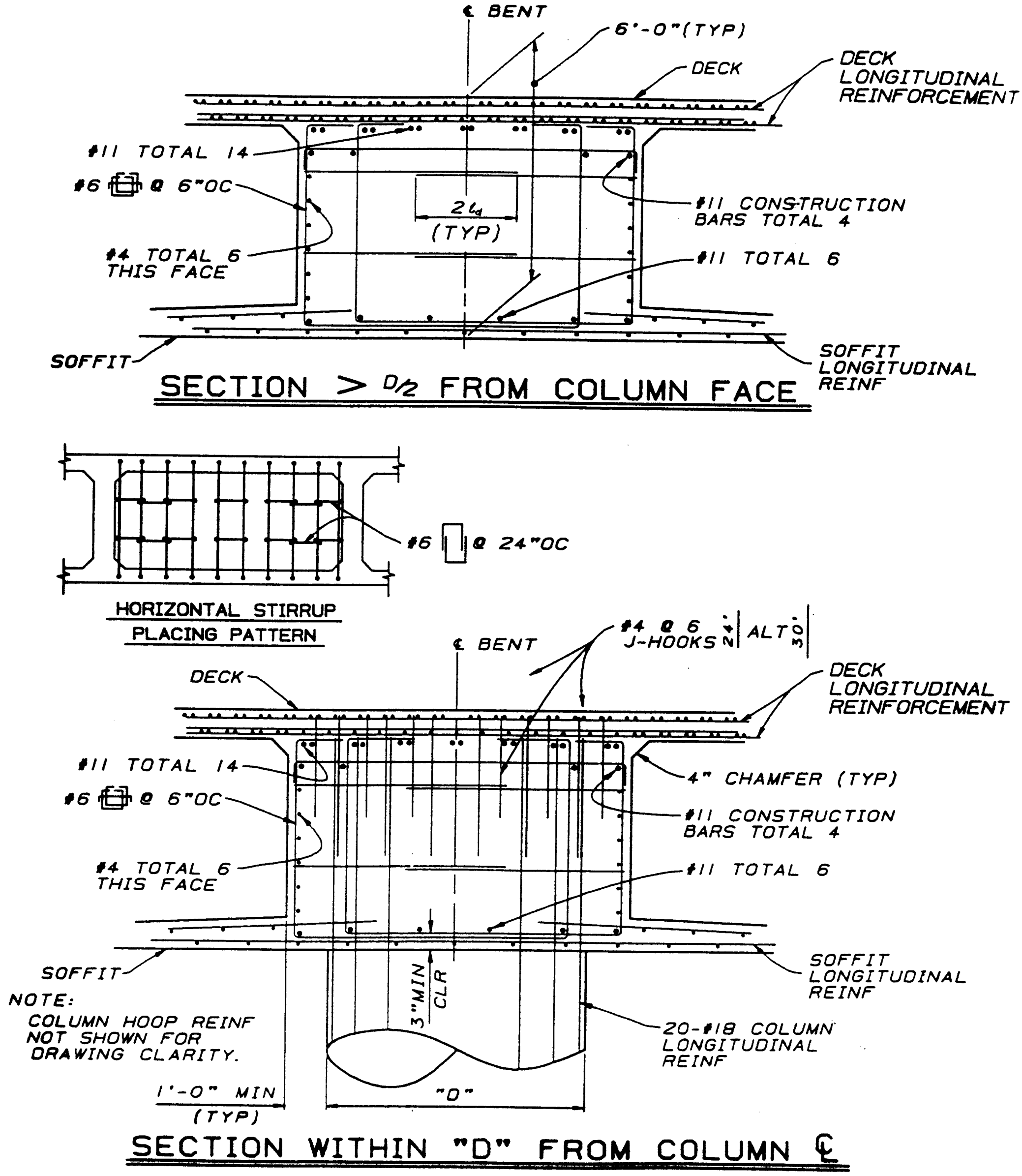

FIGURE 37.11 Example cap joint shear reinforcement — skews > 20°.

SRMDs should be able to perform their intended function and maintain their design parameters for the design life (for example, 40 years) and for an ambient temperature range (for example, from 30 to 125°F). The devices should be accessible for inspection, maintenance, and replacement. In general, SRMDs should satisfy at least the following requirements:

- Strength and stability must be maintained under increasingly large displacement. Stiffness degradation under repeated cyclic load is unacceptable.
- Energy must be dissipated within acceptable design displacement limits, for example, a limit on the maximum total displacement of the device to prevent failure, or the device can be given a displacement capacity 50% greater than the design displacement.

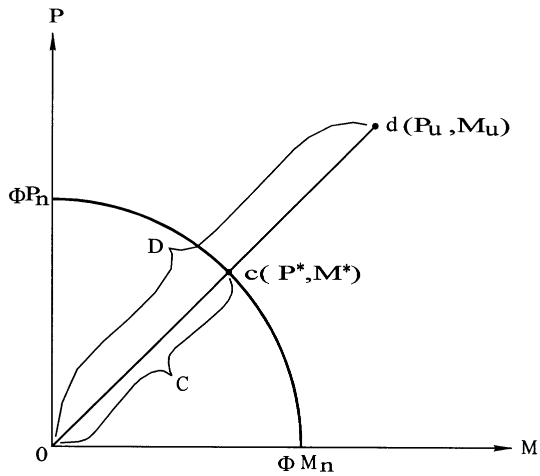

FIGURE 37.12 Definition of force D/C ratios for combined loadings.

- Heat builtup must be withstood and dissipated during “reasonable” seismic displacement time history.
- The device must survive subjected to the number of cycles of displacement expected under wind excitation during the life of the device and continue to function at maximum wind force and displacement levels for at least a given duration.

37.5.4 Acceptable Force *D/C* Ratios and Limiting Values for Structural Members

It is impossible to design bridges to withstand seismic forces elastically and the nonlinear inelastic response is expected. Performance-based criteria accept certain seismic damage in *other* components so the *critical* components will remain essentially elastic and functional after the SEE and FEE. This section presents the concept of acceptable force *D/C* ratios, limiting member slenderness parameters, and limiting width–thickness ratios, as well as expected ductility.

Definition of Force Demand/Capacity (D/C) Ratios

For members subjected to a single load, force demand is defined as a factored single force, such as factored moment, shear, or axial force. This may be obtained by a nonlinear dynamic time history analysis, level I, as specified in Section 37.5.1 and capacity is prescribed in Section 37.5.2.

For members subjected to combined loads, the force *D/C* ratio is based on the interaction. For example, for a member subjected to combined axial load and bending moment (Figure 37.12), the force demand *D* is defined as the distance from the origin point O(0, 0) to the factored force point $d(P_u, M_u)$, and capacity *C* is defined as the distance from the origin point O(0, 0) to the point $c(P^*, M^{*)}$ on the specified interaction surface or curve.

Ductility and Load–Deformation Curves

Ductility is usually defined as a nondimensional factor, i.e., the ratio of ultimate deformation to yield deformation [40,41]. It is normally expressed by two forms: (1) curvature ductility ($\mu_\phi = \phi_u/\phi_y$) and (2) displacement ductility ($\mu_\Delta = \Delta_u/\Delta_y$). Representing section flexural behavior, *curvature ductility* is dependent on the section shape and material properties and is based on the moment–curvature diagram. Indicating structural system or member behavior, *displacement ductility* is related to both the structural configuration and section behavior and is based on the load–displacement curve.

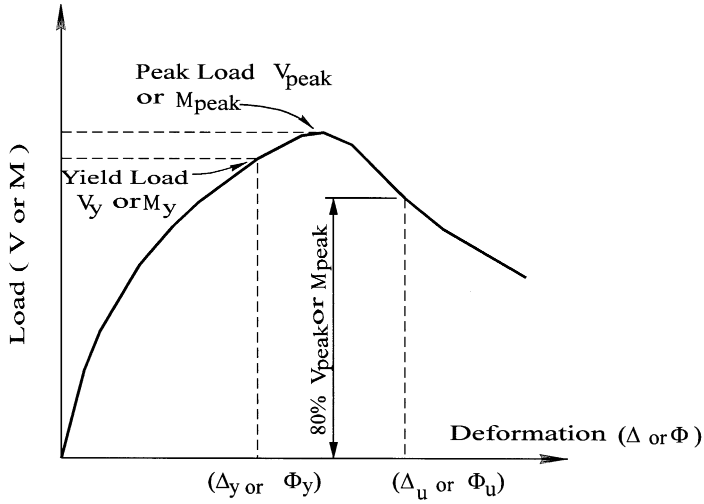

FIGURE 37.13 Load–deformation cCurves.

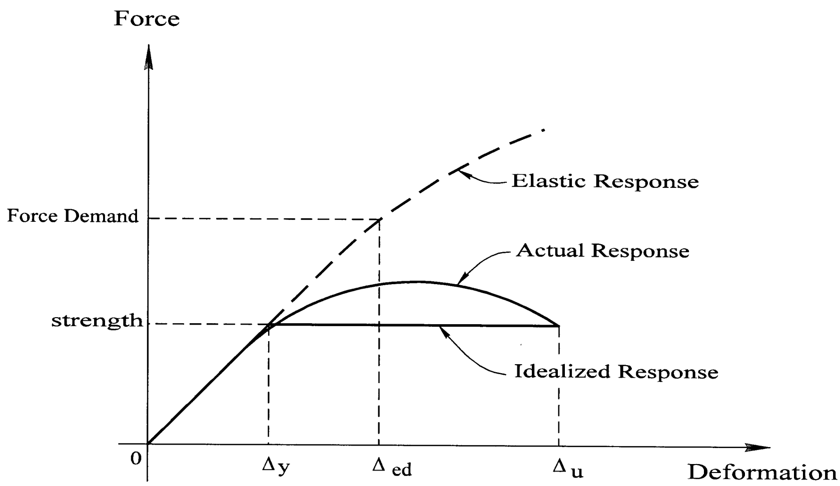

FIGURE 37.14 Response of a single-degree of freedom system.

A typical load–deformation curve, including both ascending and descending branches, is shown in Figure 37.13. The yield deformation (Δ_y or ϕ_y) corresponds to a loading state beyond which the structure responds inelastically. The ultimate deformation (Δ_u or ϕ_u) refers to the loading state at which a structural system or member can sustain without losing significant load-carrying capacity. Depending on performance requirements, it is proposed that the ultimate deformation (curvature or displacement) be defined as the most critical of (1) that deformation corresponding to a load dropping a maximum of 20% from the peak load or (2) that specified strain limit shown in Table 37.8.

Force D/C Ratios and Ductility

The following discussion will give engineers a direct measure of the seismic damage incurred by structural components during an earthquake. Figure 37.14 shows a typical load–response curve for a single-degree-of-freedom system. Displacement ductility is defined as

TABLE 37.12 Force D/C Ratio and Damage Index

Force D/C Ratio	Damage Index D_Δ	Expected System Displacement Ductility μ_Δ
1.0	No damage	No requirement
1.2	0.4	3.0
1.5	0.5	3.0
2.0	0.67	3.0
2.5	0.83	3.0

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} \tag{37.10}$$

A new term *damage index* is hereby defined as the ratio of elastic displacement demand to ultimate displacement capacity:

$$D_\Delta = \frac{\Delta_{ed}}{\Delta_u} \tag{37.11}$$

When the damage index $D_\Delta < 1/\mu_\Delta$ ($\Delta_{ed} < \Delta_y$), no damage occurs and the structure responds elastically; when $1/\mu_\Delta < D_\Delta < 1.0$, some damage occurs and the structure responds inelastically; when $D_\Delta > 1.0$, the structure collapses completely.

Based on the "equal displacement principle," the following relationship is obtained:

$$\frac{\text{Force Demand}}{\text{Force Capacity}} = \frac{\Delta_{ed}}{\Delta_y} = \mu_\Delta D_\Delta \tag{37.12}$$

It is seen from Eq. (37.12) that the force D/C ratio is related to both the structural characters in term of ductility $\mu_\Delta$and the degree of damage in terms of damage index D_Δ. Table 37.12 shows this relationship.

General Limiting Values

To ensure that important bridges have ductile load paths, general limiting slenderness parameters and width–thickness ratios are specified in Section 37.5.3.

For steel members, λ_{cr} is the limiting parameter for column elastic buckling and is taken as 1.5 from AISC-LRFD [37]; λ_{br} corresponds to beam elastic torsional buckling and is calculated by AISC-LRFD. For a *critical* member, a more strict requirement, 90% of those elastic buckling limits is proposed. Regardless of the force demand-to-capacity ratios, no members may exceed these limits. For existing steel members with D/C ratios less than 1, this limit may be relaxed. For concrete members, the general limiting parameter $KL/r = 60$ is proposed.

Acceptable Force D/C Ratios DC_{accept} for Steel Members

The acceptable force demand/capacity ratios (DC_{accept}) depends on both the structural characteristics in terms of ductility and the degree of damage acceptable to the engineer in terms of damage index D_Δ.

To ensure a member has enough inelastic deformation capacity during an earthquake and to achieve acceptable D/C ratios and energy dissipation, it is necessary, for steel member, to limit both the slenderness parameter and the width–thickness ratio within the ranges specified below.

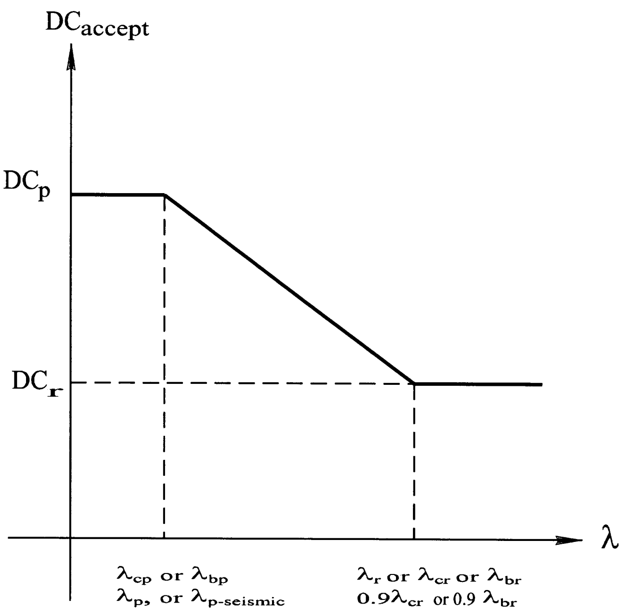

FIGURE 37.15 Acceptable D/C ratios and limiting slenderness parameters and width–thickness ratios.

Upper Bound Acceptable *D/C* Ratio DC_p

1. For *other* members, the large acceptable force *D/C* ratios (DC_p = 2 to 2.5) are listed in Table 37.10. The damage index is between 0.67 ~ 0.83 and more damage occurs in *other* members and great ductility is be expected. To achieve this,
   - The limiting width–thickness ratio was taken as $\lambda_{p\text{-Seismic}}$ from AISC-Seismic Provisions [42], which provides flexural ductility of 8 to 10.
   - The limiting slenderness parameters were taken as λ_{bp} for flexure-dominant members from AISC-LRFD [37], which provides flexural ductility of 8 to 10.
2. For *critical* members, small acceptable force *D/C* ratios (DC_p = 1.2 to 1.5) are proposed in Table 37.10, as the design purpose is to keep *critical* members essentially elastic and allow little damage (damage index values ranging from 0.4 to 0.5). Thus little member ductility is expected. To achieve this,
   - The limiting width–thickness ratio was taken as λ_p from AISC-LRFD [36], which provides a flexural ductility of at least 4.
   - The limiting slenderness parameters were taken as λ_{bp} for flexure dominant members from AISC [37], providing flexure ductility of at least 4.

3 For axial load dominant members the limiting slenderness parameter is taken as λ_{cp} = 0.5, corresponding to 90% of the axial yield load by the AISC-LRFD [37] column curve. This limit provides the potential for axial load dominant members to develop inelastic deformation.

Lower Bound Acceptable *D/C* Ratio DC_r

The lower-bound acceptable force D/C ratio DC_{rc} = 1 is proposed in Table 37.10. For DC_{accept} = 1, it is not necessary to enforce more strict limiting values for members and sections. Therefore, the limiting slenderness parameters for elastic global buckling specified in Table 37.10 and the limiting width–thickness ratios specified in Table 37.11 for elastic local buckling are proposed.

Acceptable *D/C* Ratios between Upper and Lower Bounds $DC_r < DC_{accept} < DC_p$

When acceptable force *D/C* ratios are between the upper and the lower bounds, $DC_r < DC_{accept} < DC_p$, a linear interpolation (Eqs. 37.7 to 37.9) as shown in Figure 37.15 is proposed to determine the limiting slenderness parameters and width–thickness ratios.

37.6 Summary

Seismic bridge design philosophies and current practice in the United States have been discussed. "No-collapse" based design is usually applied to ordinary bridges, and performance-based design is used for important bridges. Sample performance-based seismic design criteria are presented to bridge engineers as a reference guide. This chapter attempted to address only some of the many issues incumbent upon designers of bridges for adequate performance under seismic load. Engineers are always encouraged to incorporate to the best of their ability the most recent research findings and the most recent experimental evidence learned from past performance under real earthquakes.

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