RECOMMENDED LRFD GUIDELINES FOR THE SEISMIC DESIGN OF HIGHWAY BRIDGES

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Abstract

This paper provides an overview of the proposed seismic design provisions that have been developed to replace those currently in use throughout the United States. The proposed provisions include two-level design procedures, advanced analytical tools such as push-over, updated ground motion data, new site characterizations, simplified methods for lower seismicity regions, and more comprehensive liquefaction provisions. Many of the developments that have followed in the wake of recent earthquakes have been incorporated into the proposed provisions. The effort has been conducted and overseen by broad-based and nationally recognized teams. The proposed provisions are now being used in trial designs around the country and will be considered for adoption in Guide Specification form next year by AASHTO.

Background

In the fall of 1998, the AASHTO (American Association of State Highway and Transportation Officials)-sponsored National Cooperative Highway Research Program (NCHRP) initiated a project to develop a new set of seismic design provisions for highway bridges, compatible with the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 2000). NCHRP Project 12-49, which was conducted by a joint venture of the Applied Technology Council and the Multidisciplinary Center for Earthquake Engineering Research (the ATC/MCEER Joint Venture), had as its primary objectives the development of seismic design provisions that reflected the latest design philosophies and design approaches that would result in highway bridges with a high level of seismic performance.

NCHRP Project 12-49 was intended to reflect experience gained during recent damaging earthquakes and the results of research programs conducted in the United States and elsewhere over the prior 10 years. The primary focus of the project was on the development of design provisions which reflected the latest information regarding: design philosophy and performance criteria; seismic hazard representation, loads and displacements, and site effects; advances in analysis and modeling procedures; and requirements for component design and detailing. The new specification is intended to be nationally applicable with provisions for all seismic zones, and all bridge construction types and materials.

The current provisions contained in the AASHTO *LRFD Bridge Design Specifications* are, for the most part, based on provisions and approaches carried over from Division I-A, "Seismic Design," of the AASHTO *Standard Specifications for Highway Bridges* (AASHTO, 1996). The Division I-A seismic provisions were originally issued by AASHTO as a Guide Specification in 1983 and were subsequently incorporated with little modification into the *Standard Specifications* in 1991. Thus, the current *LRFD* (Load and Resistance Factor

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Design) provisions are based on seismic hazard, design criteria and detailing provisions, that are now considered at least 10 years and in many cases nearly 20 years out-of-date. Because AASHTO is in the process of transitioning from the *Standard Specifications* to the *LRFD* specification, it made sense to comprehensively update the seismic provisions.

NCHRP Project 12-49 developed a preliminary set of comprehensive specification provisions and commentary intended for incorporation into the AASHTO *LRFD* specifications. However, due to the amount of detail in the new provisions and the general view that the new provisions were significantly more complex than the existing provisions, the AASHTO Highway Subcommittee on Bridges and Structures recommended that the new provisions be adopted by AASHTO first as a Guide Specification (MCEER, 2001). This would then allow bridge designers the opportunity to become familiar with the proposed new specifications, and for any problems such as omissions and editorial or technical errors in the new provisions to be identified and rectified, prior to formal adoption into the AASHTO *LRFD* specifications.

Basic Concepts

The development of these specifications was predicated on the following basic concepts.

- Loss of life and serious injuries due to unacceptable bridge performance should be minimized.
- Bridges may suffer damage and may need to be replaced but they should have low probabilities of collapse due to earthquake motions.
- The function of essential (critical lifeline) bridges should be maintained even after a major earthquake.
- Upper level event ground motions used in design should have a low probability of being exceeded during the approximate 75-year design life of the bridge.
- The provisions should be applicable to all regions of the United States.
- The designer should not be restricted from considering and employing new and ingenious design approaches and details.

In comparison to the current AASHTO *Standard Specifications for Highway Bridges* and the AASHTO *LRFD Bridge Design Specifications,* the recommended Guide Specifications contain a number of new concepts and additions as well as some major modifications to the existing provisions. These are discussed in this paper.

New Seismic Hazard Maps

The national earthquake ground motion map used in the existing AASHTO provisions is a probabilistic map of peak ground acceleration (PGA) on rock that was developed by the U.S. Geological Survey (USGS) in 1990. The map provides contours of PGA for a probability of exceedance (PE) of 10% in 50 years, which corresponds to approximately 15% PE in the 75-year design life assumed by the *LRFD* specifications for a typical highway bridge.

In 1993, the USGS embarked on a major project to prepare updated national earthquake ground motion maps. The result of that project was a set of probabilistic maps first published in 1996 that cover several rock ground motion parameters and three different probability levels or return periods. The maps are available as large-scale paper maps, as small-scale paper maps obtained via the Internet, and as digitized values obtained from the Internet or a CD-ROM published by USGS (Frankel et al., 2000). Parameters of rock ground motions that have been contour mapped by USGS include peak ground acceleration (PGA) and elastic response spectral accelerations for periods of vibration of 0.2, 0.3, and 1.0 second. Contour maps for these parameters have been prepared for three different probabilities of exceedance (PE): 10% PE in 50 years, 5% PE in 50 years, and 2% PE in 50 years (approximately 3% PE in 75 years). In addition to these contour maps, the ground motion values at any specified latitude and longitude in the U.S. can be obtained via the Internet for the aforementioned three probability levels for PGA and spectral accelerations for periods of vibration of 0.2, 0.3, and 1.0 seconds. In addition, the published data contains not only the PGA and spectral acceleration values at three probability levels but also the complete hazard curves (i.e., relationships between the amplitude of a ground motion parameter and its annual frequency of exceedance at each grid point location). Therefore, the ground motion values for all of the aforementioned ground motion parameters can be obtained for any return period or probability of exceedance from the hazard curves. These maps formed the basis for seismic design using these new provisions. Upper bound limits of 1.5 times the median ground motions obtained by deterministic methods have been applied to limit probabilistic ground motions in the western United States.

Design Earthquakes and Performance Objectives

The existing AASHTO provisions have three implied performance objectives for small, moderate and large earthquakes with detailed design provisions for a 10% PE in 50 year event (approximately 15% PE in 75 year event) to achieve the stated performance objectives. The new provisions provide more definitive performance objectives and damage states for two design earthquakes with explicit design checks for each earthquake to ensure the performance objectives are met (Table 1). The upper-level event, termed the ‘rare earthquake’ or Maximum Considered Earthquake (MCE), describes ground motions that, for most locations, are defined probabilistically and have a probability of exceedance of 3% in 75 years. However, for locations close to highly active faults, the MCE ground motions are deterministically bounded so that the levels of ground motions do not become unreasonably high. Deterministic bound ground motions are calculated assuming the occurrence of maximum magnitude earthquakes on the highly active faults and are equal to 1.5 times median ground motions for the maximum magnitude earthquake but not less than 1.5g for the short-period spectral acceleration plateau and 0.6g for 1.0-second spectra acceleration. On the current MCE maps, deterministic bounds are applied in high-seismicity portions of California, in local areas along the California-Nevada border, along coastal Oregon and Washington, and in high-seismicity portions of Alaska and Hawaii. In areas where deterministic bounds are imposed, ground motions are lower than ground motions for 3% PE in 75 years. The MCE earthquake governs the limits on the inelastic deformation in the substructures and the design displacements for the support of the superstructure.

The lower level design event, termed the ‘expected earthquake’, has ground motions corresponding to 50% PE in 75 years. This event ensures that essentially elastic response is achieved in the substructures for the more frequent or expected earthquake. This design level is similar to the 100-year flood and has similar performance objectives. An explicit check on the strength capacity of the substructures is required. Parameter studies performed as part of the development of the provisions show that the lower level event will only impact the strength of the columns in parts of the western United States. Background on the choice of the two design events is provided in Appendix A of the Guide Specification.

Design Incentives

The provisions contain an incentive from a design and construction perspective for performing a more sophisticated “pushover analysis.” The R-Factor increases approximately 50% when a pushover analysis is performed, primarily because the analysis results will provide a greater understanding of the demands on the seismic resisting elements. The analysis results are assessed using plastic rotation limits on the deformation of the substructure elements to ensure adequate performance.

New Soil Factors

The site classes and site factors incorporated in the new provisions were originally recommended at a site response workshop in 1992 and subsequently were adopted in the Seismic Design Criteria of Caltrans (1999), the 1997 NEHRP Provisions (BSSC, 1998), the 1997 Uniform Building Code (UBC) (ICBO, 1997), and the 2000 International Building Code (IBC) (ICC, 2000). This is one of the most significant changes with regard to its impact on the level of seismic design forces. It should be noted that the recommended soil factors affect both the peak (flat) portion of the response spectra as well as the long-period descending portion of the spectra (Figure 1). The increase in site factors with decreasing accelerations is due to the nonlinear response effects of soils. Soils are more linear in their response to lower acceleration events and display more nonlinear response as the acceleration levels increase. The effects of soil nonlinearity are also more significant for soft soils than for stiff soils.

New Spectral Shapes

The long period portion of the current AASHTO acceleration response spectrum is governed by a spectrum shape that decays as $1/T^{2/3}$. During the development of this decay function for the existing provisions, there was considerable massaging of the factors that affect the long period portion of the spectra in order to produce a level of approximately 50% conservatism in the design spectra when compared to the ground spectra beyond a one-second period. The new provisions remove this conservatism and provide a more correct spectral shape that decays as 1/T for periods below three seconds. Guidance is also provided for the spectral shapes beyond a period of three seconds.

Earthquake Resisting Systems and Elements (ERS and ERE)

The provisions provide a mechanism to permit the use of some seismic resisting systems and elements that were not permitted for use in the current AASHTO provisions. Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. To this end, the identification of the lateral-force-resisting concept and the selection of the necessary elements to facilitate the concept should be accomplished in the conceptual design or Type, Selection, and Layout (TS&L) phase of the project. Seismic performance is typically better in systems with regular configurations and evenly distributed stiffness and strength. Thus, typical geometric configuration constraints, such as skew, unequal pier heights, and sharp curves, conflict, to some degree, with the seismic design goals. For this reason, it is advisable to resolve potential conflicts between configuration and seismic performance early in the design effort. The classification of ERS and ERE into the categories of (1) permissible, (2) permissible with owner's approval, and (3) not recommended is done to trigger due consideration of seismic performance that leads to the most desirable outcome — that is, seismic performance that ensures wherever possible post-earthquake serviceability. It is not the objective of this specification to discourage the use of systems that require owner approval. Instead, such systems may be used, but additional design effort and consensus between the designer and owner are required to implement such systems. Common examples from first two categories of systems are shown in the Figures 2 and 3.

No Analysis Design Concept

The no analysis design procedure is an important new addition to the recommended provisions. It applies to regular bridges in the lower seismic hazard areas, including the expanded areas now requiring more detailed seismic design. The bridge is designed for all non-seismic loads and does not require a seismic demand analysis. Capacity design procedures are used to determine detailing requirements in columns and in the connection forces of columns to the footing and superstructure. There are no seismic design requirements for abutments, except that integral abutments need to be designed for passive pressure.

Capacity Spectrum Design Procedure

The capacity spectrum design method is a new addition to the provisions and is conceptually the same as the new Caltrans' displacement design method. The primary difference is that the capacity spectrum design procedure begins with the non-seismic capacity of the columns and then assesses the adequacy of the resulting displacements. At this time, the capacity spectrum method may be used for very regular bridges that respond essentially as single-degree-of-freedom systems, although future research should expand the range of applicability. The capacity spectrum approach uses the elastic response spectrum for the site, and this is reduced to account for the dissipation of energy in the earthquake resisting elements. The advantage of the approach is that the period of vibration does not need to be calculated, and the designer sees the explicit trade-off between the design forces and displacements. The method is also quite useful as a preliminary design tool for bridges that may not satisfy the current regularity limitations of the approach.

Displacement Capacity Verification ("Pushover") Analysis

The pushover method of analysis has seen increasing use since the early 1990's, and is widely employed in the building industry and by some transportation departments including the Caltrans seismic retrofit program. This analysis method provides additional information on the expected deformation demands of columns and foundations and, as such, provides the designer with a greater understanding of the expected performance of the bridge. The method was used for two different purposes in these new provisions. First, it provided a mechanism under which the highest R-Factor for preliminary design of a column could be justified, because there are additional limits on the column plastic rotations that the results of the pushover analysis must satisfy. Second, it provided a mechanism to allow incorporation of earthquake resisting elements (ERE) that require owner's approval. The trade-off was the need for a more sophisticated analysis so that the expected deformations in critical elements could be assessed. The ERE could then be used, provided that the appropriate plastic deformation limits were met.

Foundations

The new provisions are an update of the existing AASHTO LRFD provisions incorporating explicit material that was referenced in the existing specifications and to incorporate recent research. The changes include specific guidance for the development of spring constants for spread footings and deep foundations (i.e., driven piles and drilled shafts.), as well as approaches for defining the capacity of the foundation system under overturning moments. The capacity provisions specifically address issues such as uplift and plunging (or yield) limits within the foundation. Procedures for including the pile cap in the lateral capacity and displacement evaluation are also provided. The implications of liquefaction of the soil, either below or around the foundation system, are also described. This treatment of liquefaction effects is a major technical addition to the provisions.

Abutments

The new provisions incorporate much of the research that has been performed on abutments over the past 10 years. Current design practice varies considerably on the use of the abutments as part of the ERS. Some agencies design a bridge so that the substructures are capable of resisting all of the seismic loads without any contribution from the abutment. Other agencies use the abutment as a key component of the ERS. Both design approaches are permitted in these provisions. The abutments can be designed as part of the ERS and become an additional source for dissipating the earthquake energy. In the longitudinal direction, the abutment may be designed to resist the forces elastically utilizing the passive pressure of the backfill or, in some cases, passive pressure at the abutment is exceeded, resulting in larger soil movements in the abutment backfill. This requires a more refined analysis to determine the amount of expected movement, and procedures are provided herein to incorporate this nonlinear behavior. In the transverse direction, the abutment is generally designed to resist loads elastically. These provisions therefore recognize that the abutment can be an important part of the ERS and considerable attention is given to abutment impacts on the global response of the bridge. For the abutments to be able to effectively contribute to the ERS, a continuous superstructure is required.

Liquefaction

Liquefaction has been one of the most significant causes of damage to bridge structures during past earthquakes. Most of the damage has been related to lateral movement of soil at the bridge abutments. However, cases involving the loss of lateral and vertical bearing support of foundations for central piers of a bridge have also occurred. Considerable research and development have occurred over the past decade in the areas of liquefaction potential and effects, and much of this information has been incorporated in these new provisions. For example, the new provisions outline procedures for estimating liquefaction potential using methods developed in 1997, as part of a national workshop on the evaluation of liquefaction. Procedures for quantifying the consequences of liquefaction, such as lateral spreading of approach fills and settlement and potential flow of liquefied soils, are also given. The provisions also provide specific reference to methods for treating deep foundations extending through soils that are spreading or flowing laterally as a result of liquefaction.

Consideration of liquefaction is based, in part, on the mean earthquake magnitude at a site, and mean magnitudes are found in the same USGS database that is used to obtain spectral accelerations. For sites with mean earthquake magnitudes less than 6.0, the effects of liquefaction on dynamic response can be neglected. When liquefaction occurs, vibration and permanent movement occur simultaneously during a seismic event. The recommended methodology in these provisions is to consider the two effects independently; i.e., de-coupled.

If lateral flow or spreading occurs, significant movement of the abutment and foundation systems can result and this can be a difficult problem to mitigate. The range of design options include (1) designing the piles for the flow forces to (2) an acceptance of the predicted lateral flow movements, provided inelastic hinge rotations in the piles remain within a specified limit. Figure 4 shows a case where spreading movements will cause yielding in both the abutment and pier foundations. The acceptance of plastic hinging in the piles is a deviation from past provisions in that damage to piles is accepted when lateral flow occurs, thereby acknowledging that the bridge may need to be replaced if this option is selected.

Assessment techniques for determining post-earthquake conditions of deep foundations, such as piles and drilled shafts are expected to be developed in the future. Currently, such techniques as down-hole inclinometers are available. Additionally, video assessment techniques are emerging, but are not in use in the U.S. If appropriate sensing devices and access types can be developed, then practical assessment damage to deep foundations can become a tool for engineers to evaluate foundation condition and the need for repair or replacement.

Structural or soil mitigation measures to minimize the amount of movement to meet higher performance objectives are also outlined in the new provisions. Due to the concerns about the potential cost impact of liquefaction coupled with the impact of higher level design events, two detailed case studies on the application of the recommended design methods for both liquefaction and lateral flow design were performed (NCHRP, 2001). The results are also summarized in Appendix H of the provisions. These examples demonstrated that for some soil profiles application of the new provisions would not be significantly more costly than the application of the more conservative current provisions.

Steel Design Requirements

The existing AASHTO Specifications do not have seismic requirements for steel bridges, except for the provision of a continuous load path to be identified and designed (for strength) by the engineer. Consequently a comprehensive set of special detailing requirements for steel components expected to yield and dissipate energy in a stable and ductile manner during earthquakes were developed, including provisions for ductile moment-resisting frame substructures, concentrically-braced frame substructures, and end-diaphragms for steel girder and truss superstructures. These provisions now provide a complete set of guidance on steel structures, drafts of which have been well reviewed by a wide range of engineers knowledgeable in steel design and construction practice.

Concrete Design Requirements

There are no major additions to the concrete provisions, but there are important updates for key design parameters based on research conducted over the past decade. The minimum amount of longitudinal steel was reduced from 1% to 0.8%, which will result in cost savings when used with the capacity design procedures. An implicit shear equation was also added where no seismic demand has been determined. Modifications to the explicit shear equation and confinement requirements were made, and a global buckling provision was added, as were plastic rotation limits for the pushover analysis.

Superstructure Design Requirements

Detailed design requirements are not included in the current AASHTO seismic design provisions, other than those required by the generic load path requirement. Therefore, for the higher hazard levels, explicit design requirements have been added since the current provisions result in a wide discrepancy in their application.

Bearing Design Requirements

One of the significant issues that arose during development of the steel provisions, and was subsequently endorsed by the NCHRP Project Panel and the ATC/MCEER Joint Venture Project Team (PT) and Project Engineering Panel (PEP), was the critical importance of bearings as part of the overall bridge load path. The 1995 Kobe, Japan earthquake (and other more recent earthquakes) clearly showed the very poor performance of some bearing types and the disastrous consequence that a bearing failure can have on the overall performance of the bridge. Three design options are included to address the issue; these are (1) testing of the bearings, (2) ensuring restraint of the bearings, and (3) a design concept that permits the girders to slide on a flat surface if the bearings fail.

Seismic Isolation Provisions

The *Guide Specifications for Seismic Isolation Design* were first adopted by AASHTO in 1991; they were significantly revised and reissued in 1999. Under the NCHRP 12-49 project, the 1999 Guide Specification provisions were incorporated into the recommended LRFD provisions. This resulted in the addition of a new chapter, Chapter 15, for the recommended NCHRP 12-49 LRFD provisions, based on issues related to seismic isolation design. That new recommended chapter is included in this Guide Specification as Section 15, and it is essentially the same as the 1999 AASHTO *Guide Specifications for Seismic Isolation Design* (AASHTO, 1999).

Cost Implications

A parameter study was performed as part of the project. In brief, the study shows that the net effect on the cost of a column and spread footing system is on the average 2% less than the current Division I-A provisions for multi-column bents and 16% less than Division I-A provisions for single column bents. These cost comparisons are based on the use of the more refined method for calculating overstrength factors and 2400 different column configurations including the seismic input of five different cities.

One factor that caused a cost increase in some of the lower period configurations was the short period modifier, which accounts for the increased ductility demands inherent in short period structures. Since this provision needs to be a part of any new code and is not part of the current Division I-A provisions, the cumulative effect of all the other charges (including the 3% PE in 75 year/1.5 mean deterministic event, new soil factors, new spectral shape, new R-Factors, new phi-factors, cracked section properties for analysis, etc.) would likely have resulted in lower average costs had the short period modifier been a part of the current specification, Division I-A.

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- Donald Anderson, CH2M Hill, Inc.
- Michel Bruneau, University at Buffalo
- Gregory Fenves, University of California at Berkeley
- John Kulicki, Modjeski and Masters, Inc.
- John Mander, University of Canterbury (formerly with the University at Buffalo)
- Lee Marsh, BERGER/ABAM Engineers
- Ronald Mayes, Bridge Consultant
- Geoffrey Martin, University of Southern California
- Andrzej Nowak, University of Michigan
- Richard Nutt, bridge consultant
- Maurice Power, Geomatrix Consultants, Inc.
- Andrei Reinhorn, University at Buffalo

The project also included a distinguished advisory committee through ATC (the Project Engineering Panel); Ian Buckle, of the University of Nevada at Reno, co-chaired this committee with Christopher Rojahn of ATC. The NCHRP Project Panel C12-49, under the direction of NCHRP Senior Program Officer David Beal and chaired by Harry Capers of the New Jersey Department of Transportation, also provided significant input and guidance during the project. Three drafts of the specifications and commentary were prepared and reviewed by these panels and the AASHTO Highway Subcommittee on Bridges and Structures seismic design technical committee (T-3), which was chaired by James Roberts of Caltrans.

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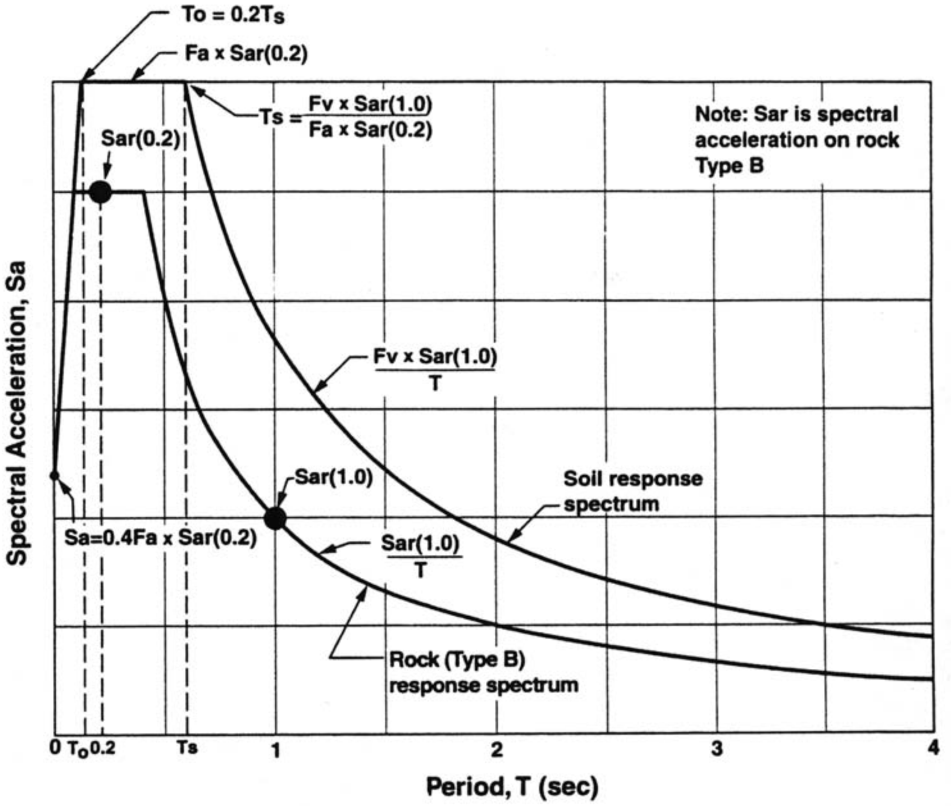
Table 1: Design Earthquakes and Seismic Performance Objectives

Probability of Exceedance For Design Earthquake Ground Motions		Performance Level: Life Safety	Performance Level: Operational
Rare Earthquake (MCE) 3% in 75 years	Service	Significant Disruption	Immediate
	Damage	Significant	Minimal
Expected Earthquake 50% in 75 years	Service	Immediate	Immediate
	Damage	Minimal	Minimal to None



Fa – Acceleration-based Site Coefficient
Fv – Velocity-based Site Coefficient

Figure 1: Response Spectrum Construction

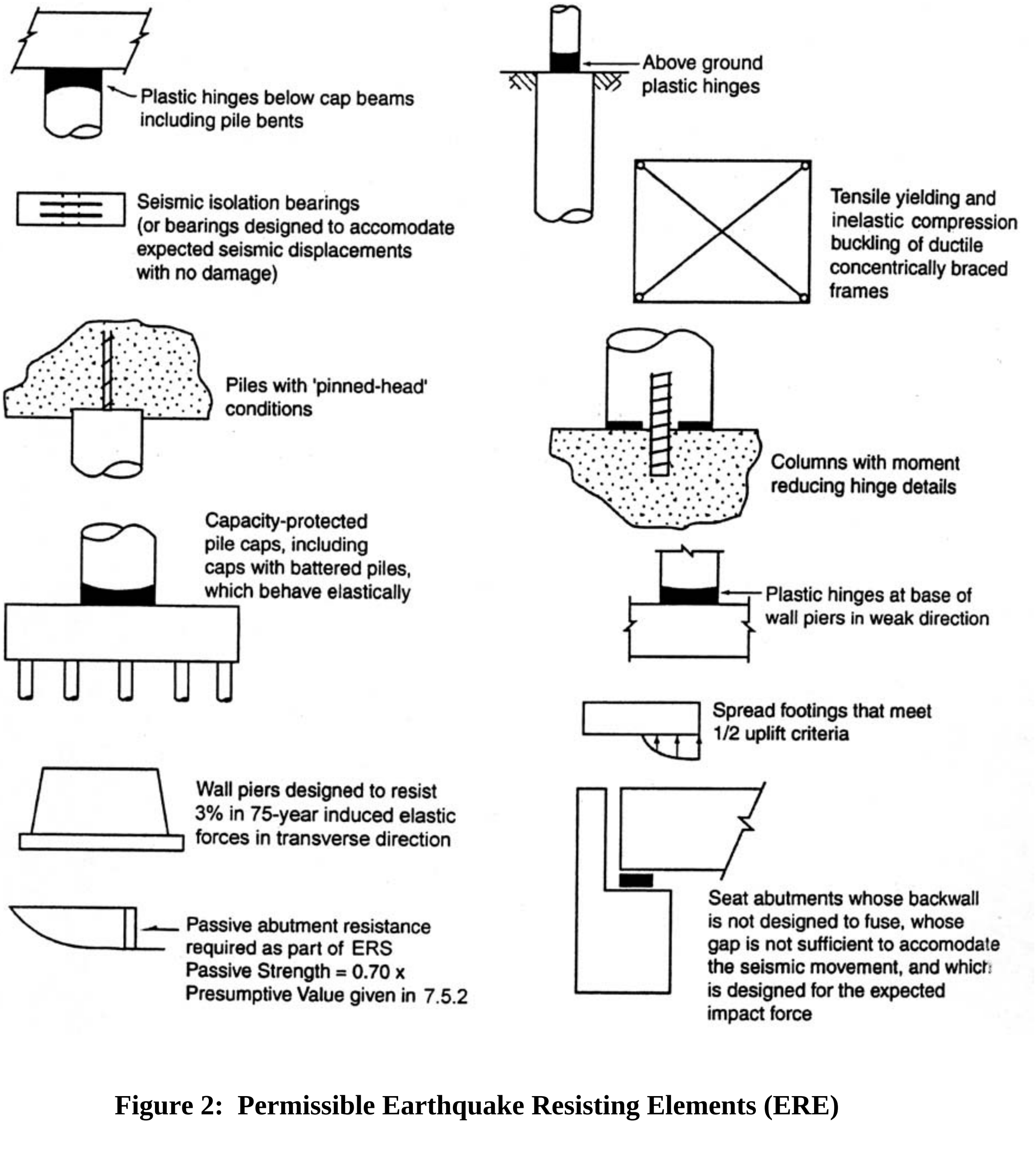


Figure 2: Permissible Earthquake Resisting Elements (ERE)

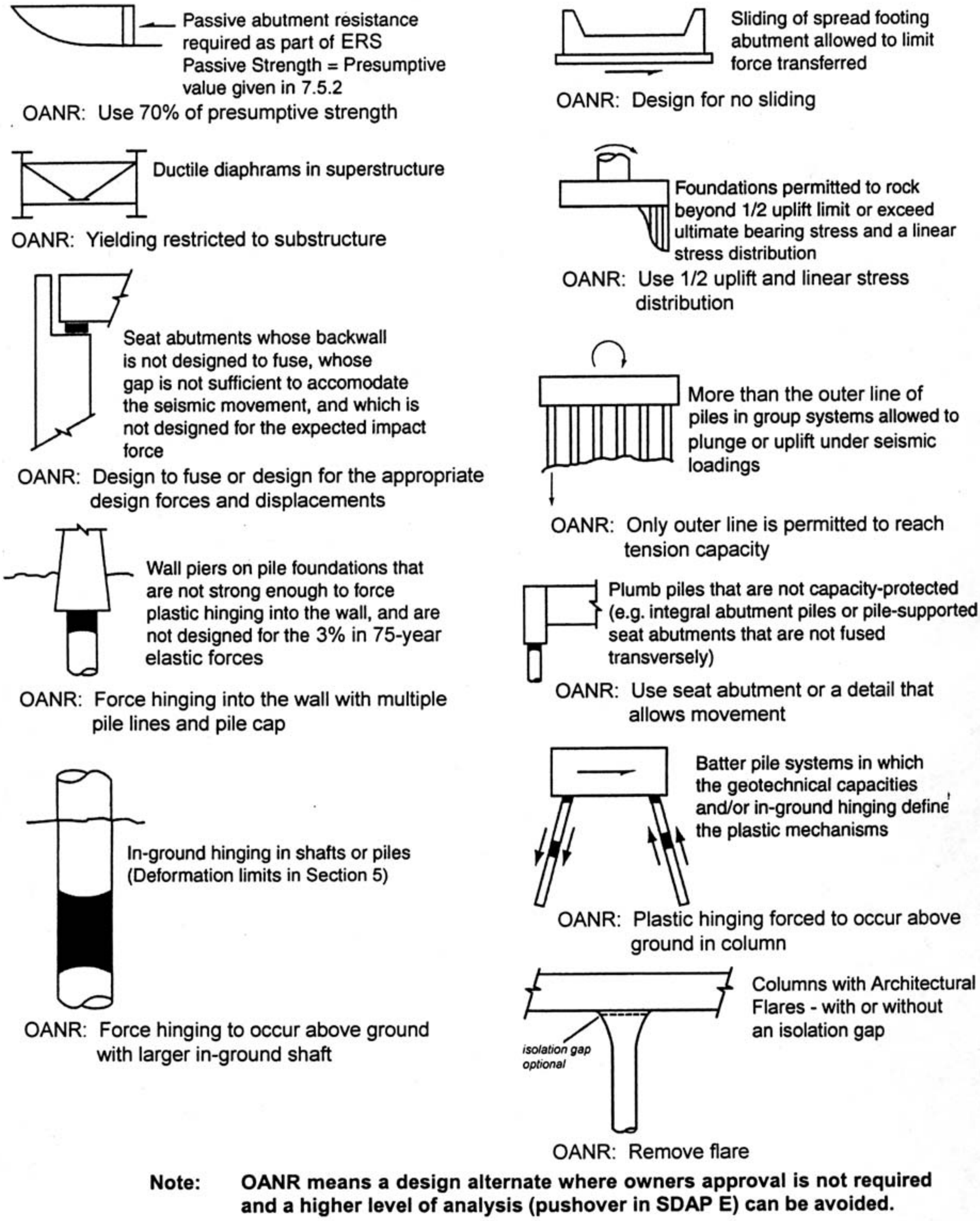


Figure 3: Earthquake Resisting Elements (ERE) that Require Owner's Approval

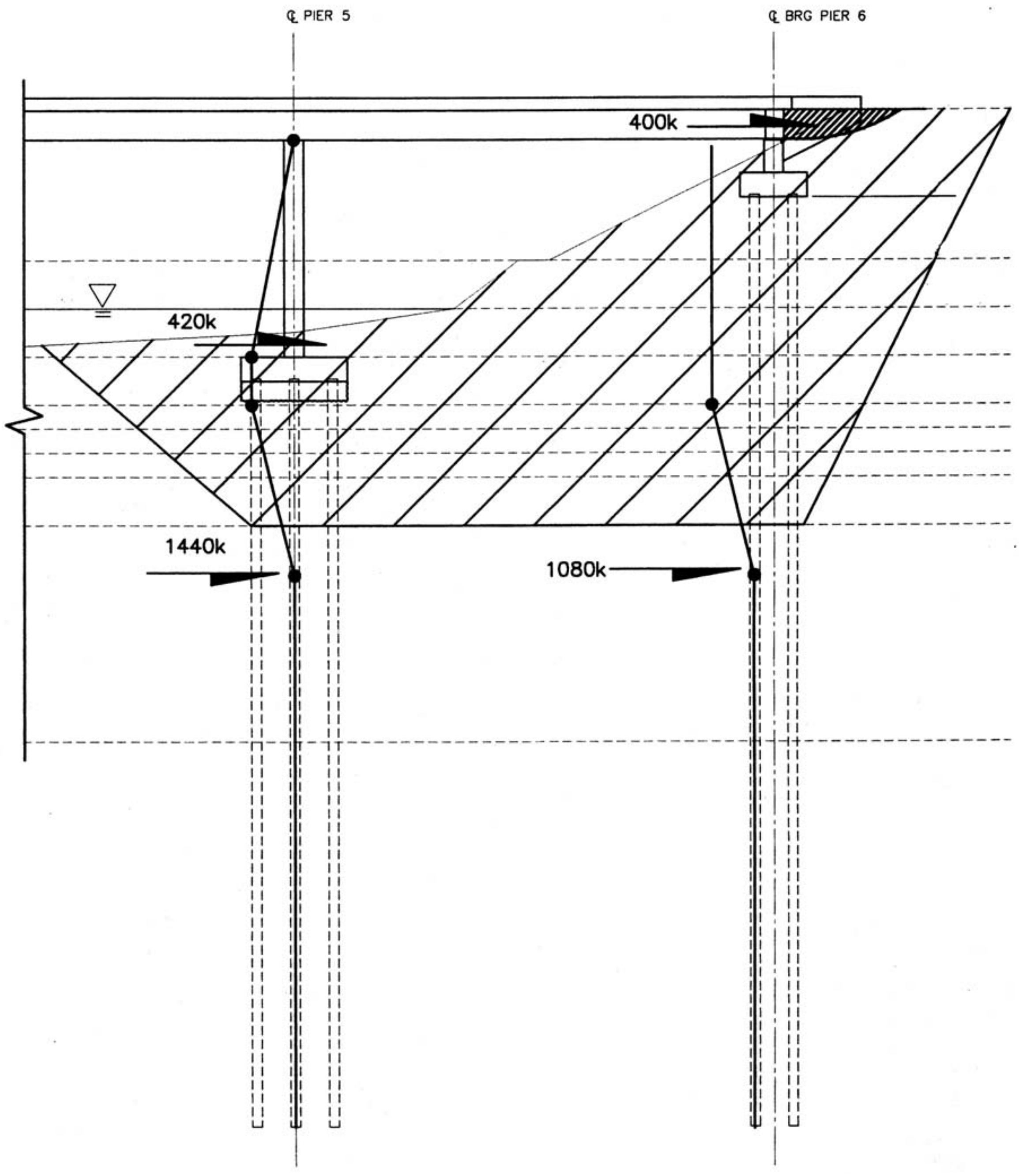


Figure 4: Foundation Movements and Resisting Forces from Lateral Spreading