

Seismic Design Criteria for Highway Bridges

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The catastrophic collapse of the Cypress Street Viaduct during the Loma Prieta earthquake of October 17, 1989 and the loss of spans in both the Struve Slough Bridge and the San Francisco-Oakland Bay Bridge has emphasized, very forcibly, the importance of rigorous seismic design procedures for highway bridges. More than eighty bridges were damaged in this moderate earthquake; some were old, some were new, some were on soft soil and some on rock. Damage patterns confirm the seismic vulnerability of older structures on poor ground, but equally disturbing is the damage sustained by some new structures designed to so-called "modern" codes.

This paper first, reviews common bridge failures during earthquakes. It then summarizes the present situation regarding seismic codes for bridges in the United States, and finally it speculates on future developments. It is concluded that existing codes should be reexamined in the light of recent events and in the process, the following issues should be addressed: the design loads (acceleration coefficients, seismic hazard maps, and soil types); the response modification factors (redundancy and ductility); the seismic performance categories (importance issues); methods of analysis (curved bridges, single vs multimode methods, inelastic methods); seatwidths and design methods for reinforced concrete joints.

BRIDGE DAMAGE DUE TO EARTHQUAKES

Earthquakes damage civil structures every year and bridges are no exception. Historically, bridges have proven to be vulnerable to earthquakes, sustaining damage to substructures and foundations and in some cases being totally destroyed as superstructures collapse from their supporting elements. In 1964 nearly every bridge along the partially completed Copper River Highway in Alaska was seriously damaged or destroyed. Seven years later, the San Fernando earthquake damaged more than 60 bridges on the Golden State Freeway in California. This 1971 earthquake is estimated to have cost the State approximately \$100 million to repair and replace these bridges, including the indirect costs due to bridge closures. In 1989, the Loma Prieta earthquake in Northern California damaged more than 80 bridges in a five-county region, and caused the deaths of more than 40 people in bridge-related collapses alone. The cost to repair these

structures has been estimated at between \$1.8 and \$2.0 billion; a figure which also includes the cost of completing the California Department of Transportation (Caltrans) bridge retrofit program but excludes the societal costs of bridge closures [1].

Common Bridge Failures

Earthquake damage to bridge structures may occur in the superstructure, the substructure or the approaches. Typical types of damage are summarized in the subsections below. Connection failures are the most common type of damage and these may take several forms. They include, for example, the failure of bearings and expansion joints which connect the superstructure to the substructure and those shear or flexural failures that occur within the substructure itself (e.g. in the column/footing joint or the column/capbeam joint). Connection failures have been identified as the principal reasons for the collapse of the Cypress Street Viaduct in Oakland and the link span of the San Francisco-Oakland Bay Bridge during the Loma Prieta earthquake (Figure 1).

Superstructures

Since earthquake loads are predominantly horizontal in-plane loads, and because bridge superstructures are inherently very strong in-their-own-plane, earthquake related structural damage to a bridge superstructure is very rare. On the other hand, loss of support due to gross horizontal movement of one or more segments of a superstructure is very common and this may lead to the partial or total collapse of one or more spans of the bridge.

Loss of support may be caused by a connection failure, a lack of continuity in the superstructure, inadequate support lengths for the girders, skew supports

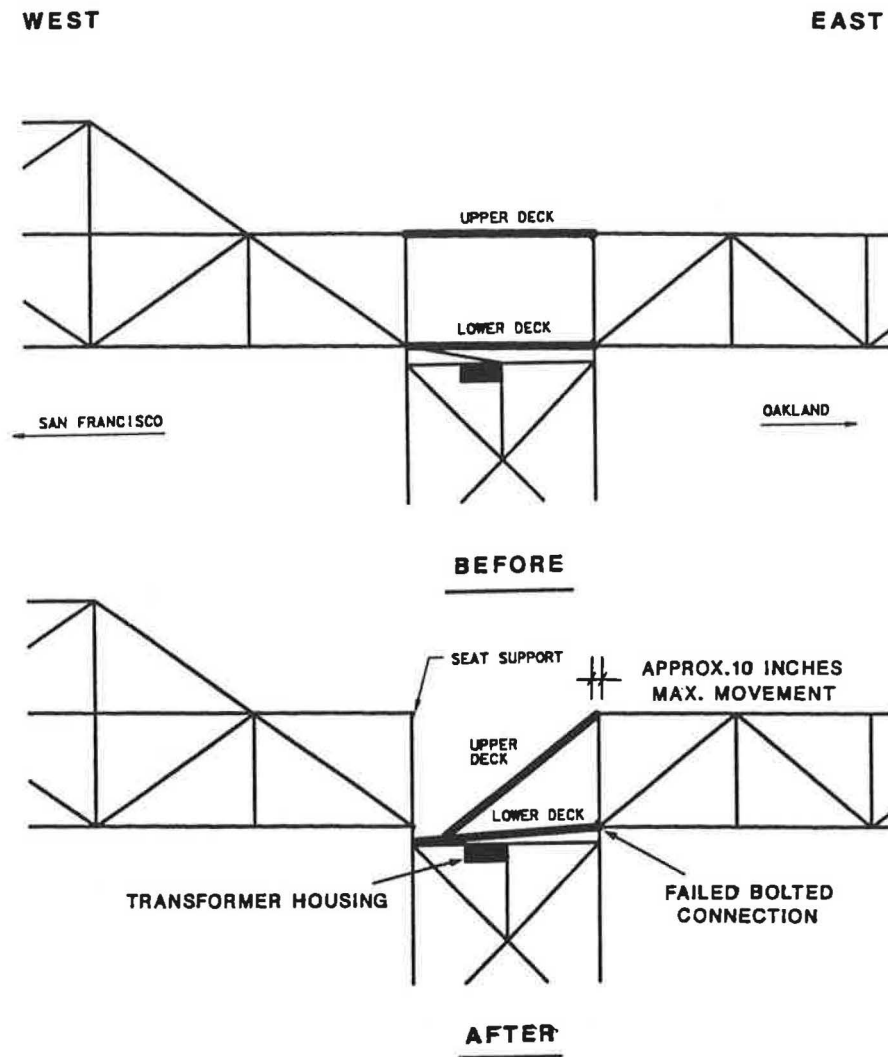


Figure 1. The link span at Bent E-9 in the San Francisco-Oakland Bay Bridge collapsed because of a failed bolted connection of the eastern truss to the Bent
 Source: Loma Prieta Earthquake Reconnaissance Report Supplement to Vol.6, Earthquake Spectra, May 1990 (EERI)

(which encourage rotation of the superstructure about a vertical axis), or gross movements at the superstructure supports due to some form of soil failure under the piers or abutments. Reduction of this type of failure has been the principal aim of Phase I of the California seismic retrofit program (the cable restrainer program).

Substructures

Substructure damage includes the structural failure of the columns, abutments and foundations (piles, footings). Column damage can be caused by flexural failure, shear failure, and anchorage failure of the longitudinal reinforcement. All of these types of failure modes may also cause collapse of the superstructure by removal of support for the superstructure.

Foundations

foundation failures may result from excessive ground deformation and/or the loss of stability and bearing capacity of the foundation soils. As a result, substructures may tilt, settle, slide, or even overturn, thus experiencing severe distortion or complete failure. It is also well established that soft soils can amplify bedrock ground motion and that relatively harmless rock accelerations can be intensified by overlying soil to become destructive events at the surface. Deep deposits of both natural and manmade fill are potential sites for soil amplification effects. Even if piles are used to support a bridge through soft material to bedrock (or similar) large horizontal movements may still occur in the superstructure because of the flexible nature of the soil/pile/substructure system.

Abutments

By virtue of their high lateral stiffness, abutments may attract the largest share of the seismic inertia forces developed in the superstructure. These forces can be very high and may cause severe failures, often of a brittle nature. The interaction of

the abutment with the backfill may also cause the wing walls to separate from the abutments. Backfill settlement resulting from compaction is often observed.

Vulnerable Bridges

Two factors determine bridge vulnerability to seismic effects: first, their ability to resist earthquake forces; and second, their ability to tolerate large superstructure movements. Earthquake forces are generally higher in bridges supported on stiff substructures (i.e. in short period bridges) and deflections are usually larger in the heavier decks on flexible substructures (i.e. in long period bridges).

Bridges with the greatest vulnerability for seismic damage are therefore multi-span structures that have one or more of the following

- : simply supported spans which have deficient bearings and inadequate seatwidths

- : continuous spans which have intermediate hinges with deficient bearings and inadequate seatwidths

- : nonductile substructures

- : under-reinforced footings

- : under-reinforced abutment backwalls and wingwalls

- : unusual geometry (severe curvature, severe skew, tall piers, piers of differing heights, long continuous spans, piers in deep water)

- : hazardous site conditions (near active faults, on or near unstable slopes, on liquefiable foundations, on deep soft soil sites).

On the other hand, bridges with the least seismic vulnerability include

- : single span bridges with either integral abutments or generous seatwidths and adequate connection details at the abutments

- : continuous bridges with either integral abutments or generous seatwidths and adequate connection details at the abutments, that have redundant substructures and no internal hinge seats

- : bridges with earthquake protective systems such as base isolation devices which reduce seismic forces and control large superstructure movements.

SEISMIC HAZARD TO BRIDGES

Seismicity of the United States

Although the earthquake "problem" is widely believed to be unique to California, it is in fact a national problem. This was recognized by Congress when Public Law 95-124 was passed in 1977 which identified 39 states as being exposed to a moderate level (and higher) of seismic threat.

Figure 2 shows the location of those earthquakes which have been reported in the United States since early colonial times through to the present day. Most of these are small magnitude events but a few have been very large and if they were to occur today they would be very destructive. The largest earthquakes to have been experienced in the continental U.S. were those in New Madrid on the Mississippi River during the winter of 1811 and 1812. Three magnitude 8 earthquakes occurred in a period of 3 months and were felt throughout the eastern United States. Figure 3 shows the extent of the felt area and compares it to that of similar magnitude events elsewhere in the U.S. It is clear that the affected areas are much larger in the east than in the west due, most probably, to the homogeneous nature of the basement rock in the east. The fact that large magnitude earthquakes have occurred in the East in the past and that they were felt over large areas of the continent, must be kept in mind when deciding seismic criteria for bridges in the East and Central United States.

Seismic Design Codes for Bridges

Two sets of provisions are commonly used for the seismic design of bridges outside of California. One is the AASHTO Standard Specification [2] and the other is the AASHTO Guide Specification for Seismic Design (1983) [3]. The California Department of Transportation uses a hybrid set of specifications, modified for local conditions [4]. The Guide Specifications are identical to the ATC-6 Guidelines prepared under contract to Federal Highway Administration in 1981 and are more rigorous than those provisions in the Standard Specifications.

During the 1990 Annual Meeting of the AASHTO Bridge Subcommittee, it was agreed that the seismic provisions in the Standard Specifications be replaced by those from the Guide Specification. A formal ballot will be made of all members of the Subcommittee in the Fall of 1990 and if successful, the Guide Specifications will become the new national standard for seismic bridge design in the United States.

However, these guidelines are now 10 years old, and it is appropriate that they be reviewed in the light of experience with their use over the last decade and the performance of various bridges in the Loma Prieta earthquake.

Retrofit Guidelines have also been prepared by ATC under FHWA sponsorship [5] but these have not yet been formally adopted by AASHTO as a Guide Specification. FHWA has also funded the publication of a seismic design and retrofit manual [6] and an instructional short course for bridge designers [7] offered through the National Highway Institute.

Seismic Design Philosophy of the Guide Specification

The design earthquake motions and forces specified in the Guide Specification are based on a low probability of their being exceeded during the normal life expectancy of a bridge. Bridges and their components that are designed to resist these forces and that are constructed in accordance with the design details contained in the provisions may suffer damage, but should have low probability of collapse due to seismically induced ground shaking.

The principles used for the development of the provisions are [3]:

1. Small to moderate earthquakes should be resisted within the elastic range of the structural components without significant damage.
2. Realistic seismic ground motion intensities and forces are used in the design procedures.
3. Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair.

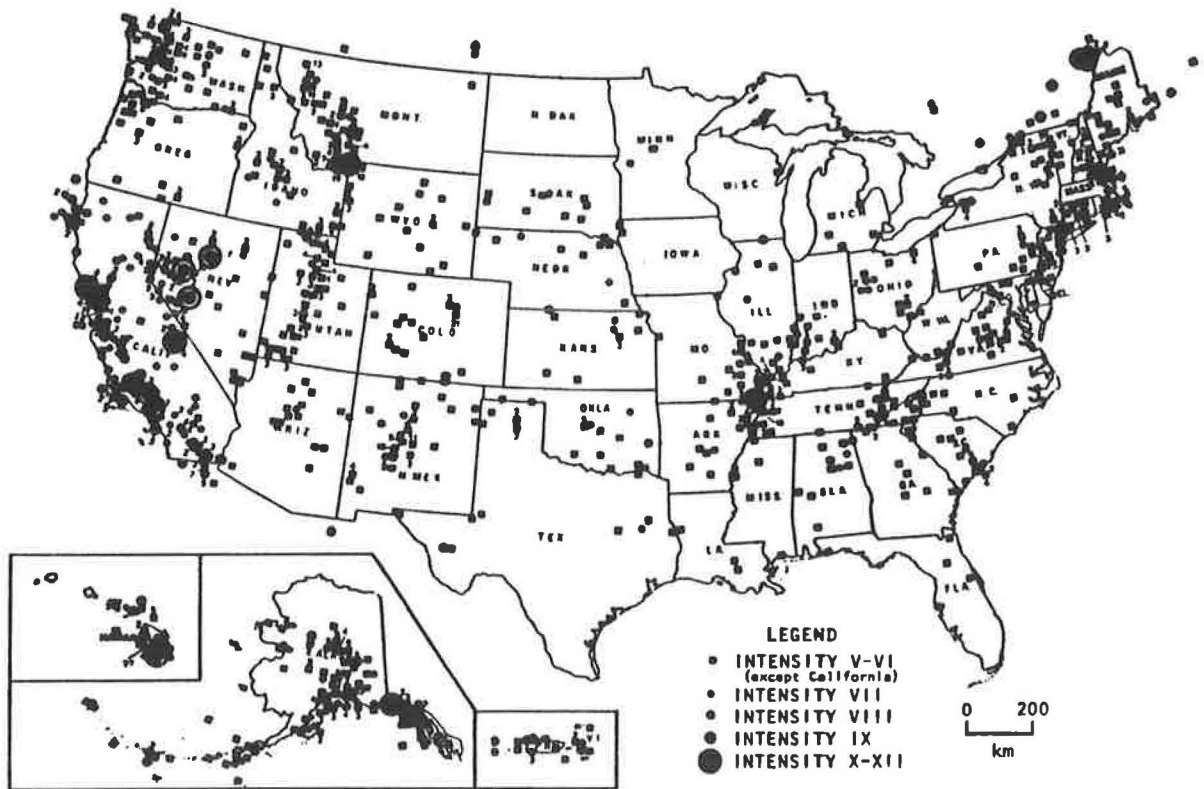


Figure 2. Earthquake with maximum Modified Mercalli intensities of V or above in the United States and Puerto Rico through 1976

Source: "An Introduction to the Seismicity of the United States", by S.T. Algermissen, EERI Monograph Series, 1983

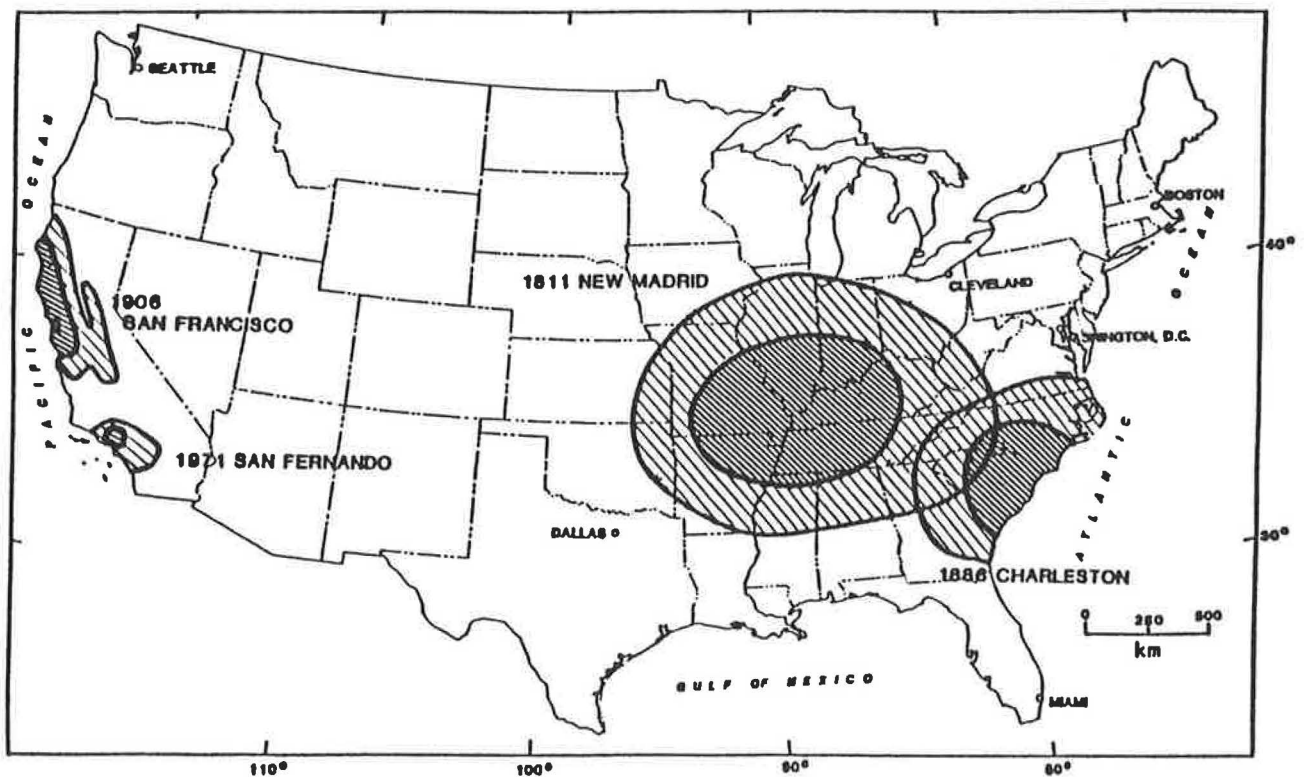


Figure 3. Regional areas affected by major earthquakes in the United States

Source: "Geology in the Siting of Nuclear Power Plants", Hatheway, A.W. and C.R. McClure, Eds., 1979. *Reviews in Engineering Geology*, Vol. 4, pages 67-94. Geological Society of America. Ed. note: Adapted from Nuttli, O.W. "Seismicity of the Central United States".

A basic premise in developing these seismic design guidelines was that they be applicable to all parts of the United States. The seismic risk varies from very small to high across the country. Therefore, for purposes of design, four Seismic Performance Categories (SPC) are defined on the basis of an acceleration coefficient (A) for the site, determined from the map provided (Figure 4), and the importance classification (IC). Different degrees of complexity and sophistication of seismic analysis and design are specified for each of the four Seismic Performance Categories.

An essential bridge is required to function during and after an earthquake. In areas with an acceleration coefficient greater than 0.29 essential bridges must meet additional requirements. A bridge is designated essential on the basis of social/survival and security/defense classifications presented in the commentary to the provisions [3].

One of the consequences of the 1989 Loma Prieta earthquake in California, has been a call for a reexamination of this philosophy. Accordingly, several efforts are underway at the time of writing (May 1990) including studies by Caltrans, the Governor's Board of Inquiry, the General Accounting Office and the Transportation Research Board through NCHRP Project 12-33.

The TRB/NCHRP activity is part of a much larger exercise sponsored by AASHTO through NCHRP to prepare a comprehensive bridge specification and commentary based on LRFD principles [8]. The Code Coordinating Committee for NCHRP 12-33 has formed an Earthquake Provisions Advisory Group to prepare the seismic provisions for the code. This paper summarizes the current thinking (as of May 1990) of this Group regarding some of the issues and concerns that need attention.

ISSUES AND CONCERNS

Loads: Acceleration Coefficient

A new map for the acceleration coefficient (A) has been prepared for the NEHRP'88 provisions for buildings [9] by the US Geological Survey. Higher acceleration coefficients are recommended in all parts

of the United States (for the same probability of exceedance as previously used) as shown in Figure 5.

At the 1990 AASHTO Annual Meeting of the Bridge Subcommittee, it was agreed to update the map in the Guide Specification (shown in Figure 4) by replacement with this new map (Figure 5). The revised map is much closer to the map used by Caltrans for bridges in California and it also reflects an improved understanding of the seismic risk in the East.

Notwithstanding these steps in the right direction, it is still worthwhile to consider the advantages and disadvantages of a two-level earthquake load requirement. The first level would be the existing design event (Figure 5), and be used for determining design forces; the second level would be the maximum credible event and be used for determining superstructure displacements (e.g. Figure 6). The Guide Specification uses the first level event (the design event). Caltrans uses the second level event (the maximum credible event). It will be recalled that Caltrans does not use either the Standard Specification or the AASHTO Guide Specification for seismic design but has adopted its own set of specifications based on higher input loads and more generous response modification factors.

A two-level earthquake approach could permit the introduction of fully elastic performance under the design event and inelastic response (but not collapse) under the maximum credible event. This may have particular appeal for critically important bridges. In this way it would also be possible to make a rational allowance for the difference in seismicity between the East and West. For example, the maximum credible event in the East is expected to be 3 times larger than the design event in the East, whereas in the West, this ratio is about 1.5. Ensuring satisfactory behavior during the maximum credible event in the East is therefore more difficult than in the West, especially if only a single design event is used.

An alternative procedure to the two-level event is to use the Caltrans' philosophy nationwide (i.e. use the maximum credible event to define the acceleration coefficient). At the same time more generous response modification factors are adopted so that the design forces will be

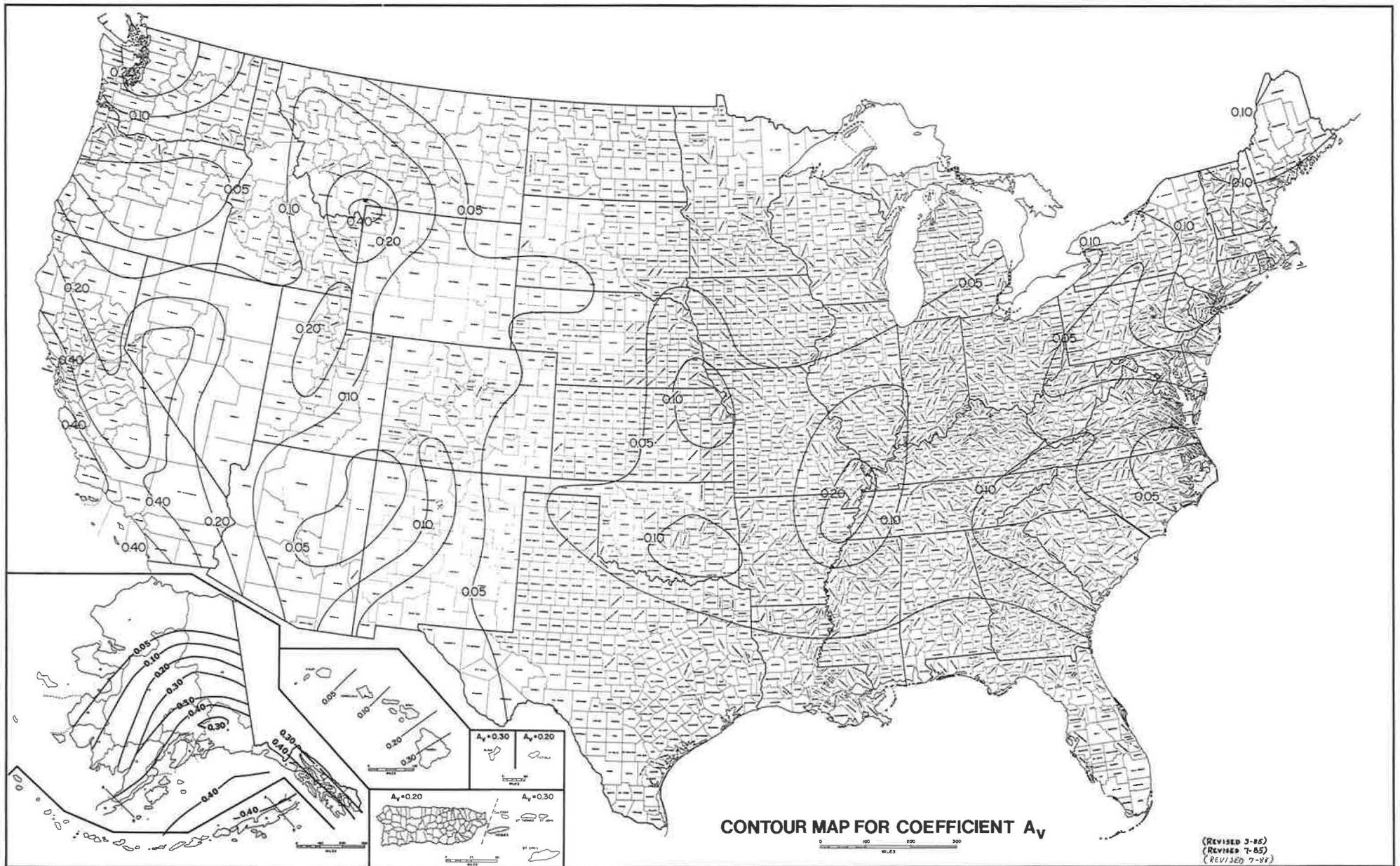


Figure 4. Acceleration Coefficient

Based on 10% probability of exceedance
in 50 years.

SOURCE: ATC 6 and AASHTO 1983 Guide Specifications
for the Seismic Design of Highway Bridges

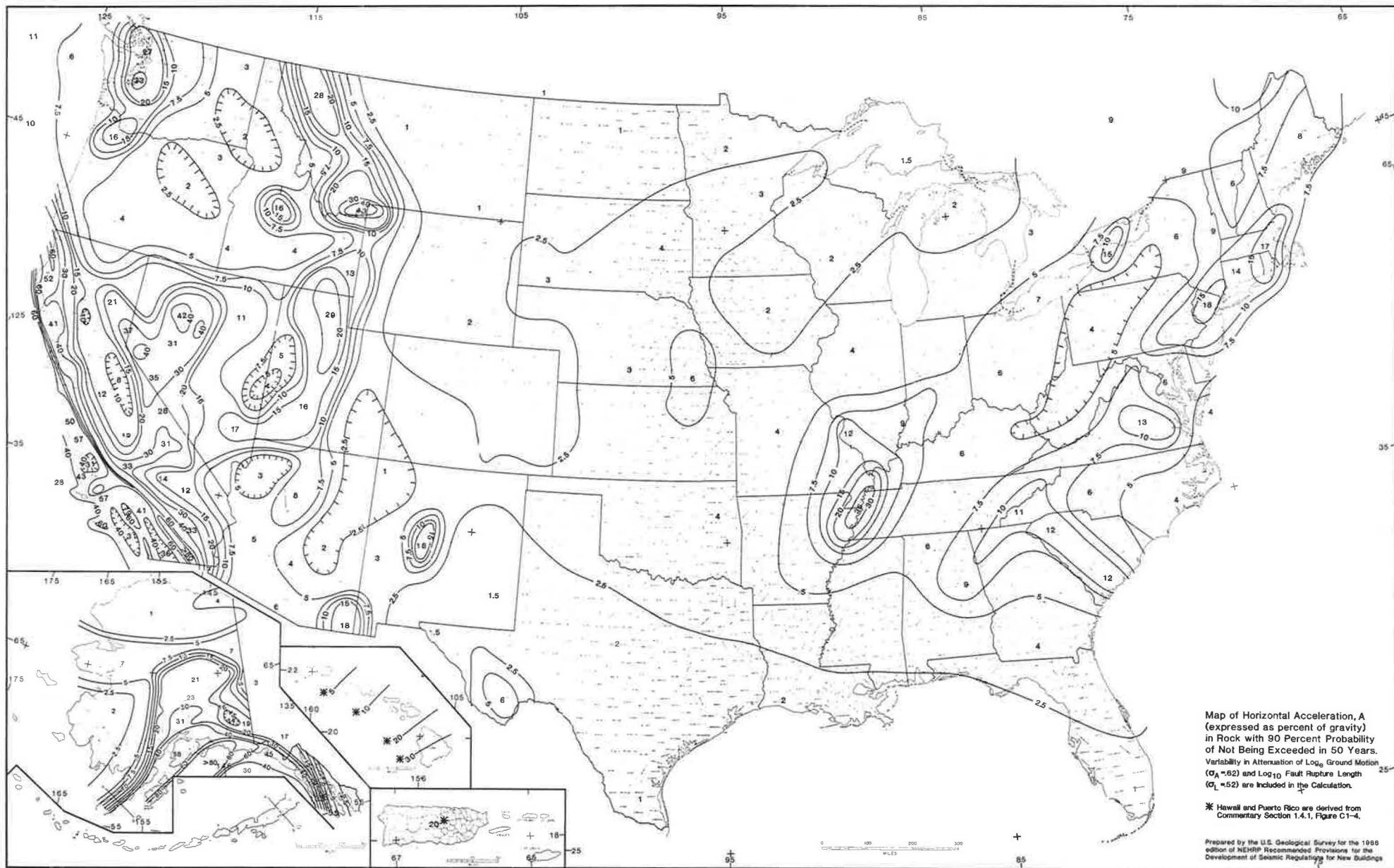


Figure 5. Revised Uniform Seismic Risk Map
 Based on 10% probability of exceedance in 50 years.
 SOURCE: USGS for 1988 Edition of NEHRP Provisions

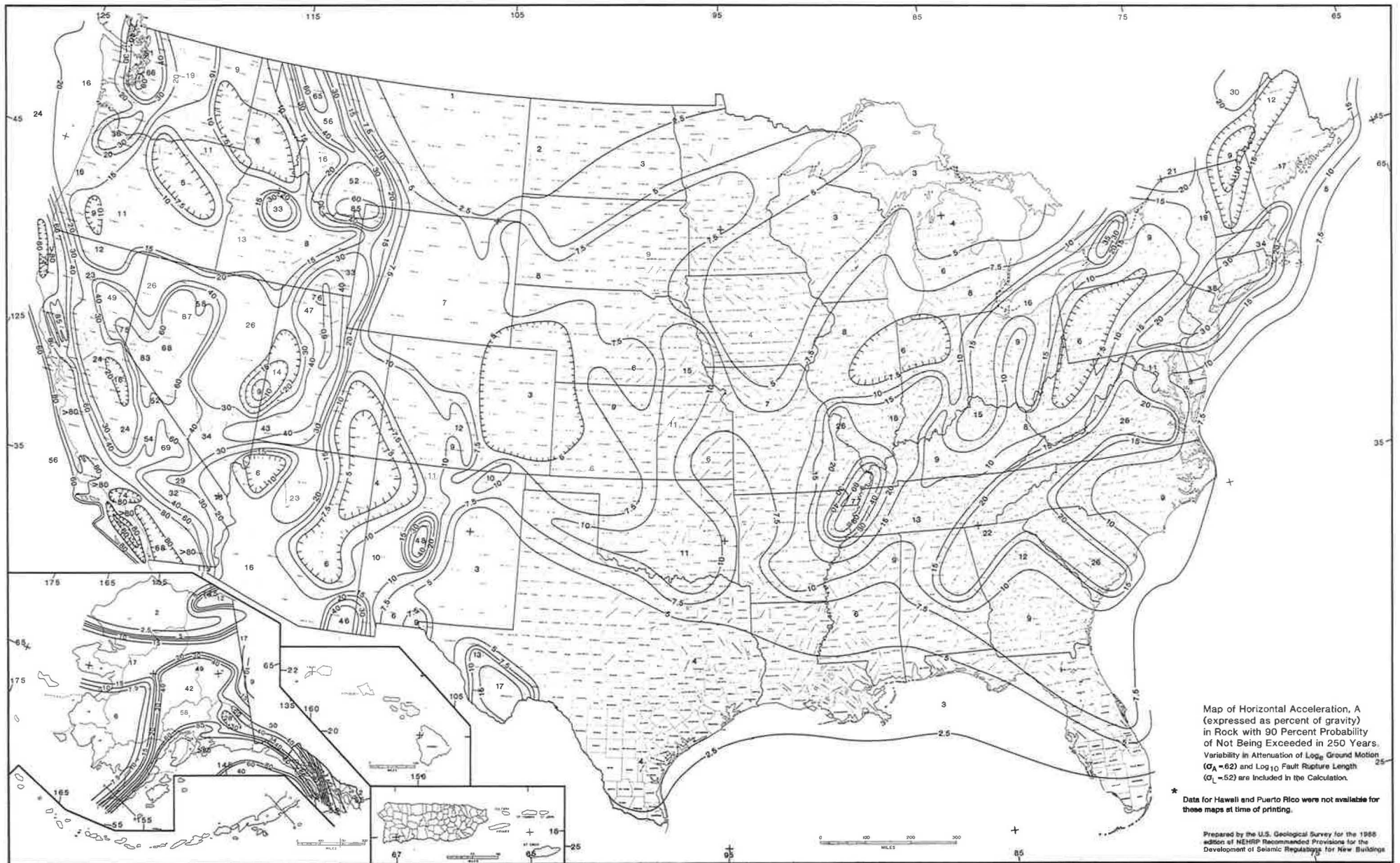


Figure 6. Revised Uniform Seismic Risk Map
 Based on 10% probability of exceedance in 250 years.
 SOURCE: USGS for 1988 Edition of NEHRP Provisions

about the same. However, the displacements will be significantly larger (2-3 times larger in the East than presently required in the East).

This will lead to greater seatwidth requirements in the East and Central U.S. than presently required. Such a move is necessary in order to prevent loss of girder support in a maximum credible event as required by the stated design philosophy. Seatwidths are further discussed in later section.

Loads: Vertical earthquake loads

Some substructure elements, including bearings; are thought to be susceptible to the vertical component in earthquake ground motion. Provisions that require analysis for these effects may be necessary. Further study is required.

Loads: Soil types and site coefficients

A fourth soil type for particularly soft materials has been adopted in the Uniform Building Code for buildings [10] and should be considered for bridges. The shape of the spectra for softer sites also needs to be defined. Provisions for the use of site-specific spectrum should also be made.

Response Modification Factors (R-Factors)

Present Response Modification Factors (R-factors) make allowance, in a somewhat arbitrary way, for redundancy and ductility in a bridge structure. The designer is permitted to reduce the elastic forces (obtained from a linear elastic analysis) by dividing by R to obtain the member design forces. For a multicolumn bent, $R=5$ (in the AASHTO Guide Specifications; $R=8$ for this same bent in the Caltrans Specifications). Because of the high degree of redundancy in a multicolumn bent, the inherent ductility in a modern, confined, reinforced concrete column and because the risk of total collapse is low, this factor is relatively high. Single column bents which are not redundant have R-factors as low as 3. Obviously, a great deal of professional judgement is required to select appropriate values for R. Note

that the displacements of the superstructure are not reduced by R to obtain the design displacements. (Alternatively, displacements may be calculated from inelastic time-history analyses).

There is some merit in clearly separating the two components that comprise the R-factor, particularly as research has now progressed to the point where analytical and experimental methods are available to give specific values for member deformation capacity and demand, leading to an identifiable ductility component in the R-factor.

If a numerical measure of redundancy could also be defined, it would help the designer to see more clearly the risk he/she is assuming and the benefits of redundancy in bridge substructures.

Since less risk should be taken for important structures, such a step would also lead to an improvement in the manner "importance" is included in the design process (see section below).

Seismic Performance Categories (SPC)

Importance is used, along with the acceleration coefficient (A), to define 4 seismic performance categories in the Guide Specifications [3] as shown in Table 1.

It is seen in this Table that importance affects only those bridges in locations where $A > .29$. Further, the only difference in the design requirements between SPC C and D is in the section on foundations and abutments. In general, more rigorous site inspections are required, soil degradation must be considered, and settlement or approach slabs must be provided at the abutments. All other design requirements are the same as for nonessential bridges (in the same seismic zone).

Recent experience with the Bay Bridge and the Cypress Street Viaduct in the San Francisco Bay Area would indicate that this treatment of importance is inadequate. It is therefore suggested that the "importance" requirements be moved and clearly identified in the R-factor Table and that SPC D be eliminated.

For example, for all elements in a critically essential bridge R should be essentially unity; for most other bridges the present R-factors may be adequate; for bridges in between these two extremes, the

TABLE 1: SEISMIC PERFORMANCE CATEGORIES (GUIDE SPECIFICATIONS)

Acceleration Coefficient	Importance	
	Essential	Other
< .09	A	A
< .19	B	B
< .29	C	C
> .29	D	C

TABLE 2: SELECTED R - FACTORS (PROPOSED)

Substructure	Importance		
	Other	Essential	Critical
Multicolumn bent	5	3	1.5
Single column bent	3	2	1.5
Wall pier	2	1.5	1.5

TABLE 3: METHODS OF ANALYSIS (GUIDE SPECIFICATIONS)

Seismic Performance Category	Geometry	
	Regular	Irregular
A	*	*
B	SM	SM
C	SM	MM
D	SM	MM

* = no seismic analysis is required regardless of geometry
 SM = single-mode method
 MM = multi-mode method

TABLE 4: METHODS OF ANALYSIS FOR MULTISPAN STRUCTURES (PROPOSED)

Seismic Zone	Importance					
	Other		Essential		Critical	
	regular	irregular	regular	irregular	regular	irregular
1	*	SMM	*	MM	MM	MM
2	SMM	SMM	SMM	MM	MM	IA
3	SMM	MM	MM	MM	MM	IA
4	SMM	MM	MM	MM	IA	IA

* = no seismic analysis required
 SMM = single-mode method (modified)
 MM = multi-mode method
 IA = inelastic analysis

R-factors might be about one-half of the present values. Table 2 illustrates this thought process. Of course if an inelastic method of analysis is used, the assignment of an R value is no longer necessary and for critically important structures this type of analysis should be encouraged. See section on inelastic methods.

Note that if this concept is implemented, the term "Seismic Performance Category" has no useful purpose and could be removed from the jargon. Analysis and design requirements could be specified by seismic zone (for example: 1-4).

Definitions for importance will vary but factors influencing the importance classification of a bridge will include average daily traffic volume, available detour, use by emergency vehicles, replacement cost, and the nature and importance of the route being crossed by the bridge. It is possible that this classification will change with time and if it should increase (in importance), retrofit will be necessary. The owner should be involved in selecting the importance category.

Methods of Analysis

Curved bridges

All curved bridges should be analyzed by the multi-mode spectral analysis method but the definition of "curved" needs to be resolved. At present any bridge with a subtended angle (at the center of the arc) less than 90° may be analyzed by the single-mode method. This is too generous and leads to unconservative results. It is suggested that a "curved" bridge be defined as one for which the subtended angle is greater than or equal to 50°.

Single vs Multi-Mode Spectral analysis

The AASHTO Guide Specifications determine which method of analysis shall be used based on geometry, as in Table 3.

An irregular bridge is one for which there is an abrupt change in stiffness or mass from one bent to another and/or excessive curvature such that the subtended angle is greater than 90°. All other bridges are considered "regular".

The single-mode method can be performed manually for a simple structure, such as a uniform 2-span bridge. However, for all other bridge types, a hand solution is impractical and a computer-based solution becomes a necessity. The particular advantage of this simplified method is therefore lost and the retention of the single-mode method is consequently in doubt. If deleted, the multi-mode method could be used wherever the single-mode method was previously specified. However, there are good practical reasons for permitting a single-mode type of analysis for regular bridges in low seismic zones. In these cases, a "user friendly" version of the single-mode method would be particularly useful and should be investigated further. There is also a good argument to be made for requiring inelastic analysis for important, irregular bridges. Given these changes the table of recommended methods of analysis might be as shown in Table 4.

Inelastic Methods

Certain classes of irregular bridges in high seismic zones (especially those that are absolutely essential or monumental in size or both) should be analyzed using an inelastic, time history method of analysis. (If elastic methods are used, the R-factor should be, say, 1.5 for essentially elastic behavior, see Section on seismic performance categories.)

Bridges with intermediate hinges, soil nonlinearities, energy absorbers and/or base isolators are also candidates for inelastic analysis.

The method of analysis remains to be decided, and also the number and type of time histories (spectrum compatible or actual records?)

Special Requirements for the Various SPCs

Some changes are thought to be necessary to the various Special Requirements (for Analysis and Design) in the four Seismic Performance Categories.

These are summarized as follows:

General: No seismic analysis is required for any single span bridge regardless of seismic zone.

However minimum seatwidth requirements must be satisfied and all connections must be designed for a lateral force = $A \times$ dead load.

SPC A No analysis is required for regular bridges but minimum seatwidths must be satisfied and connection forces must be $0.2 \times$ dead load (except single span bridges).

SPC B Analysis is required; minimum seatwidths must be satisfied. Member design forces are calculated from elastic-analysis values divided by the R-factor. For foundation design forces, use $R/2$ values. No column hinging analysis is required. (To avoid undue conservatism here, the development of an approximate column-hinging analysis method would be a useful design aid.)

SPC C Analysis is required; minimum seatwidths must be satisfied. Member design forces are calculated from elastic analysis values divided by the R-factor for the substructure. Column hinging analysis is required; foundations must be designed for either the maximum shear corresponding to column hinging or the unreduced ($R=1$) elastic shear from the analysis whichever is the smaller.

SPC D As for SPC C, with additional requirements for foundations and abutments as presently required in the Guide Specifications.

Seatwidths

The seat width requirements for SPC A and B should be reconsidered. In regions of the US where these categories apply, the maximum credible earthquake is of the same order of magnitude as the design earthquake in SPC C and D. It follows therefore larger seatwidths than currently specified may be necessary to prevent girders being unseated in an event larger than the design

event and to satisfy the stated design philosophy as expressed in Section on seismic design philosophy. Attention must also be given to hinge seatwidths in both skewed and curved bridges.

Joint Shear

The shear reinforcement in the reinforced concrete joints of multicolumn bents needs to be reexamined. Requirements for joint steel vary from code to code, and from country to country. Damage sustained during the Loma Prieta earthquake have highlighted the need to change the design provisions for concrete joints, especially knee joints.

SUMMARY

This paper has identified some of the issues needing review in the current codes for seismic design of bridges. They include the design loads, the response modification factors, the seismic performance categories, methods of analysis, seatwidths and design methods for reinforced concrete joints. Current efforts are addressing these issues, with the expectation that improved seismic performance of bridges will be the end result.

ACKNOWLEDGEMENTS

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