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OVERVIEW OF SEISMIC DESIGN METHODS FOR BRIDGES IN DIFFERENT COUNTRIES AND FUTURE DIRECTIONS

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ABSTRACT

This short paper briefly reviews some of the important issues in the seismic design of highway bridges. By comparing code philosophy and design procedures in Europe, Japan, New Zealand and the United States, similarities and differences in these provisions become apparent. Issues discussed include design philosophy and performance criteria, seismic loads and site effects, analysis and modeling, and design requirements. Issues related to seismically isolated bridges and the retrofit of existing bridges are considered to be outside the scope of this paper. It is concluded that although there have been significant advances in recent years, there is room for improvement. Heavy damage and some catastrophic collapses to new bridges in recent earthquakes confirm the need for continuing refinements to code philosophy, design procedures and construction practices. In this regard, this paper also presents some possible future directions in seismic design which may shape the next generation of bridge design codes.

KEYWORDS

Bridges; seismic design; international codes; design philosophy; performance criteria; load characterization; site effects; modeling and analysis; design requirements; response modification factors; future directions

INTRODUCTION

It has been said that the history of seismic design is also the history of damaging earthquakes. It is certainly true that after each major earthquake in which bridge and building structures are seriously damaged, the codes-of-practice change. There is a strong correlation throughout the world between the occurrence of major earthquakes and advances in seismic design. Each earthquake has tested the knowledge-of-the-day and where it has been found deficient, the lessons learned have led to improvements in the state-of-the-art. But new discoveries and fresh insight have also come from the research community which in recent years has been particularly active at generating insight and understanding and communicating results to the practicing profession. Codes and design provisions seem to be under constant review and particularly so in recent years.

Earthquakes of particular significance for their impact on bridge design include Anchorage (1964), Niigata (1964), Inangahua (1968), San Fernando (1971), Guatemala (1975), Fruili (1976), Edgecumbe (1987), Loma Prieta (1989), Philippines (1990), Costa Rica (1991), Okushiri (1993), Northridge (1994), and Hanshin-Awaji (1995). In the last decade (and particularly since the Loma Prieta earthquake in Northern California) researchers and practitioners have been able to improve the state-of-the-art substantially and major code revisions have occurred, or are in the process of occurring, in such areas as design philosophy, performance criteria, ground motion characterization, geotechnical design (site effects), inelastic analysis and capacity design procedures for concrete and steel structures. Whereas much of this activity has been directed towards the design of new bridges, there has also been significant progress in the evaluation and retrofit of existing structures. This paper, however, focuses on codes and provisions for new structures.

Countries which have made significant contributions in this area include, in alphabetic order, Europe, Japan, New Zealand and the United States. Each country has either recently revised its seismic bridge code or is in the process of revising its code. These developments are noted briefly below.

In 1994, the European Committee for Standardization (CEN) approved, as a European Prestandard, Part 2 to Eurocode 8 entitled "Earthquake Resistance Design of Bridges". This prestandard is currently in use on an experimental basis and will be reviewed and revised as necessary in 1997 for issuance as a final standard at that time (CEN,1994).

Seismic design requirements in Japan are published by the Japan Road Association as Part V of the "Specifications for Highway Bridges: Seismic Design" (JRA,1990). The most recent edition of these specifications is dated 1990 but it is currently under review following the unsatisfactory performance of many bridges in the 1995 Hanshin-Awaji earthquake near Kobe. Pending completion of this review, a tentative set of revised design guidelines has been issued by the Subcommittee for Earthquake Countermeasures for Highway Bridges of the Japan Road Association. These guidelines were issued in June 1995 and are titled "Guide Specifications for Reconstruction and Repair of Highway Bridges Damaged by the Hyogo-ken Nanbu (Hanshin-Awaji) Earthquake" (JRA, 1995).

In New Zealand seismic design procedures are defined in Section 5 of the "Bridge Manual: Earthquake Resistant Design" (TNZ, 1994). Issued by Transit New Zealand in 1994 and updated in June 1995, this manual replaces earlier requirements published in the Highway Bridge Design Brief of the Ministry of Works and Development. Nevertheless the procedures are essentially the same except for new sections on evaluation and are based on the 1980 recommendations of the NZ National Society for Earthquake Engineering Committee on Seismic Design of Bridges (NZNSEE, 1980).

At the present time, there are two national specifications for bridge design in the United States each containing seismic design provisions. Both are published by the American Association of State Highway and Transportation Officials (AASHTO). The first is a working-stress specification titled "Standard Specifications for Highway Bridges"; seismic requirements are contained in Division I-A: Seismic Design (AASHTO, 1992). The second is a limit-state specification, and is titled "LRFD Bridge Design Specifications" (AASHTO, 1994). Seismic requirements are contained within various chapters alongside provisions for other extreme loads such as wind, scour, ice, debris flow, and ship collision. The first edition was published in 1994 with an update issued in 1995.

The Department of Transportation of the State of California (Caltrans), has developed an independent set seismic specifications which are similar to, but not the same as, the AASHTO provisions.They are published as a subset of "Bridge Design Specifications" and supplemented by "Memos to Designers" on an as-needed basis (Caltrans, 1995). Following the unacceptable performance of a number of major bridges in the San Francisco Bay Area during the Loma Prieta earthquake in 1989, Caltrans requested the Applied Technology Council (ATC) to conduct an exhaustive review of its design criteria and procedures. This review is now complete and many of the recommendations made in the final report have already been adopted by Caltrans on a trial basis. The ATC report is expected to be released in the Spring of 1996 (ATC32, 1995). The Applied Technology Council has also reviewed worldwide trends in bridge seismic codes under contract to the Federal Highway Administration through the National Center for Earthquake Engineering Research. A draft report is currently under review (ATC18, 1995).

OVERVIEW

Design Philosophy and Performance Criteria

All of the above noted codes imply that bridges designed acccording to the stated provisions will satisfy a minimum level of performance. The difference between the various codes is the degree to which these performance criteria are explicitly defined and checked in the design process. Common to all of the codes is the concept of acceptable damage provided that collapse of all or part of the bridge does not occur even in strong shaking. Acceptable damage is defined in the AASHTO specifications to mean flexural yielding in the columns only (i.e. no shear failures) and even then it must be detectable (above ground and water lines), inspectable and repairable. All other damage (to foundations, abutments, shear keys, connections, bearings and superstructure girders and slabs) is unacceptable. This definition is generally used by the other codes, especially for the type of damage (flexural yielding) but some codes relax the requirements on location, particularly to allow yielding in pile shafts, pile bents, and abutment back walls. The definition of "strong shaking" is vague even in the AASHTO specification, but is taken to mean the design earthquake which, for the AASHTO codes, is a 475-year event. Performance under smaller but more frequently occuring earthquakes is required to be essentially elastic (damage-free) but no specific check on this performance is made during the design process. In other words, a design that meets the code requirements at the design earthquake level ("strong shaking") is assumed to automatically satisfy the requirements for the lower level event - by default.

Codes which require explicit two-level design, at least in part, include Japan, the Caltrans/ATC revised specification and the site-specific criteria developed by the Corridor Design Management Group (CDMG) for a privately financed toll road in Southern California. In each case two sets of seismic loads are defined for two different return periods and the bridge is explicitly checked for performance against predetermined criteria for these two load cases. In the Caltrans criteria, acceptable damage levels and service states are defined in qualitative terms for both important bridges and ordinary bridges for a functional evaluation earthquake (30 - 40% probability of occurrence) and a safety evaluation earthquake (maximum earthquake that can be expected at the site - a return period of perhaps 1000 years in California).

The basic design philosophy inherent in all but the Japan codes is one of capacity design. The presumption is made that it is uneconomic to design bridges in high seismic zones to behave elastically and that inelastic behavior is inevitable - particularly during moderate and larger earthquakes. Under this philosophy, the yielding of ductile elements is used to protect other less ductile members from unacceptable failure modes, even during great earthquakes. The ultimate (force) capacity of the structure is used to limit member forces to predetermined values, regardless of earthquake size. However the corresponding structure displacement demands may be high and the possible occurrence of extreme member ductility demands (eg plastic hinge rotations) requires the use of special design details in and near the plastic hinge zones, to prevent hinge failure and structure collapse. A feature of all capacity-based seismic codes is the care and attention paid to member details to ensure the integrity of the structure during excursions into the plastic deformation range. On the other hand the Japan specification expects elastic performance under the design earthquake which is one that can be expected to occur several times in the life of the bridge (and is thus a smaller event than this term implies elsewhere in the world). Columns are sized for this situation and then a ductility check is made assuming a much larger earthquake (one that may occur only once in the life of the bridge). The principles of capacity design are not explicitly used in Japan to protect non-ductile components and provide capacity to survive greater-than-expected events.

Seismic Loads and Site Effects

All of the codes listed above except New Zealand, represent the seismic hazard by un-reduced, 5% damped, elastic response spectra. These smoothed acceleration spectra are not true spectra because they have been artificially scaled to account for uncertainties in the ground motion, particularly in the long period range. Also some of these design "spectra", such as those given by AASHTO and Japan, are capped in the short period range. Near field effects are not captured in these "spectra" and site specific studies are required if located near an active fault. Return periods are generally 475 years but Caltrans has historically designed its bridges to sustain maximum credible events, which in California may have return periods of 1000 years. In Japan the return period is not stated, but for the moderate event it may be of the order of 75 years and for the severe event it may be 500 years. The effect of soil conditions is generally included by either scaling the rock spectra by a uniform site factor or by plotting separate spectra for different soil conditions. Some large differences are apparent in the soft soil spectra with the Japanese spectra remaining "flat" to 2.0 seconds for the "severe" earthquake in these conditions. On the other hand AASHTO, Caltrans, the Eurocode and the New Zealand provisions show the spectra beginning to fall no later than 1.0 seconds, for the softest soil condition.

The exception to the use of elastic spectra is New Zealand specification which specifies inelastic spectra for two soil conditions and 6 structure ductility ratios. The implied return period is 150 years but adjustments are provided to accommodate different return periods and/or design life using Poisson's distribution.

Time history characterization of the ground motion is permitted in most of the codes but is required in the Eurocode to adequately represent the spatial variation in the ground motion for structures longer than 600 m or when abrupt changes in the soil types occur.

Analysis and Modeling

Equivalent static methods of elastic analysis are permitted by all of the codes for "regular" bridges with increasing rigor being required for bridges of increasing complexity and/or importance. In most cases this means using either a single or multi-mode method of analysis except for single span bridges which are generally exempt from dynamic analysis (but still subject to minimum design requirements). Multi-modal methods are not required in Japan unless special circumstances exist. Both elastic and inelastic time history methods of analysis are permitted in many codes provided the selection of input ground motions meets specified criteria. In the Eurocode, results from time history analyses may not be used to reduce values below those calculated from modal spectral methods, except for seismically isolated bridges. Of particular interest in the Caltrans/ATC specification, is the provision for incremental lateral strength analysis using monotonically increasing static loads, (i.e. push-over analysis) and this method is preferred (by Caltrans) to a time-history inelastic analysis. On the other hand the AASHTO LRFD specification encourages the use of time history methods especially for critical or unusual bridges.

Allowance for directional uncertainty in earthquake loading is generally made by using the maximum of $(X + 0.3Y)$ and $(0.3X + Y)$ in design where X and Y are effects from longitudinal and transverse earthquakes respectively. Exceptions are Japan and New Zealand in which the maximum of X and Y effects are used in design. Further, both Caltrans/ATC and the Eurocode have introduced the effects of vertical components in the ground motion and define three cases from which maximum values are to be selected for use in design. These are: (1) $X + 0.3Y + 0.3Z$; (2) $0.3X + Y + 0.3Z$; and (3) $0.3X + 0.3Y + Z$); where Z represents vertical earthquake effects.

Earthquake effects are generally combined with dead load effects and some second order load cases such as prestress shortening and earth pressure. Only the AASHTO LRFD code includes a provision to add live load to the same load group (as earthquake and dead load) but the fraction of live load to be considered is determined on a project-specific basis. Since the live load is not rigidly connected to the superstructure it should not be considered part of the inertial mass of the bridge and should not be included in the calculation of seismic effects; it is a gravity load-effect only.

Design Requirements

Since ductile (inelastic) performance is intended during the design earthquake, except in Japan, the substructure bending moments calculated by the elastic analysis methods, are reduced by factors which depend on the type of substructure, acceptable level of implied ductility (ie plastic deformation), bridge importance and expected peak ground acceleration.These force reduction factors are called: R-factors in the AASHTO specifications and range from 1.5 to 5; Z-factors in the Caltrans BDS and Caltrans/ATC recommendations and range up to 8 and 4 respectively; and q-factors in the Eurocode and range from 1 to 3.5. These reduction factors are not applied to the calculated (elastic) structure displacements.

Exceptions to the explicit use of reduction factors are Japan and New Zealand. The Japan specification expects elastic performance during "moderate" earthquakes and thus reduction factors are not required. But design forces are adjusted for ductility during "severe" earthquakes using a reduction factor that is a function of the structure ductility ratio (u). As noted above, the New Zealand specification uses inelastic spectra for different structure ductility ratios, to define the ground motions. Therefore, analytical results for member forces already include the effects of ductility and no further reductions are appropriate. However the New Zealand approach requires the displacement results to be scaled up by the structure ductility ratio (u) in order to estimate the actual displacements and include the effects of plastic deformation in the substructures. Both approaches give the same results for elastic-perfectly plastic, single degree-of-freedom systems for which R (or Z, or q) = u, i.e. that satisfy the so-called equal displacement rule.

Almost all of the codes specify minimum displacements which are then used to specify minimum seat widths. In the United States these widths are a function of seismic zone, structure length, column height and angle of skew and range from 8 to 36 inches. Elsewhere minimum widths are of the order of 8 inches in New Zealand, 16 inches in Europe and 28 inches in Japan.

Minimum connection forces are also specified in every case but one (the Eurocode). In the United States these forces are simply the product of the acceleration coefficient and the effective weight (W) acting through the connection and thus they range from 0.01W to 0.4W.

Most of the codes contain detailed design requirements for bridge substructures (columns, piers, and foundations) although some, like the New Zealand provisions, refer to building standards for these provisions. Ultimate strength design is generally used for the design of the columns using both capacity reduction factors and material overstrength factors to ensure that flexural yield occurs before failure in shear or otherwise. Variations in these factors are evident as different codes strive for a balance between economy and performance. The exception to the use of ultimate strength design is the Japan specification which uses working stress design for the moderate design earthquake, with a factor of 1.5 on allowable stressses. Standard ductile details are however required in the columns to assure satisfactory (ductile) performance during a severe event.

Whereas ultimate strength provisions for concrete substructures are prescribed in some detail in many codes, steel columns receive less attention and are generally covered by performance-type requirements. The exception here is again in Japan where steel columns are commonly used and are well prescribed in the specification using working stress provisions supplemented by standard ductile details.

Foundation design generally uses ultimate soil strengths with capacity reduction factors. Limited uplift of footings (rocking) is permitted and pile connections to footings must have minimum pull-out capacities. Liquefaction effects are to be considered but few codes give specific requirements in this regard.

Displacement restrainers (or couplers) are required by all codes if minimum seat widths are not satisfied. Some codes require them regardless of seat width and for certain types of bridges such as the Eurocode requirements for base isolated bridges and those bridges designed for "limited ductility". Methods of restrainer design vary greatly.

FUTURE TRENDS

Design codes are continually evolving as new knowledge is acquired and experience with existing codes is gained during moderate-to-large earthquakes. Possible future trends in bridge codes are discussed below.

Design Philosophy and Performance Criteria

The trend towards the specification of more exact performance criteria is likely to continue in response to public demand that certain bridges perform to higher levels than others. The specification of probable service levels and damage states that are to be expected after an earthquake is a useful strategy to convey to the public and owner agency that all bridges are vulnerable to a certain degree. Less vulnerability can be achieved but at a higher cost. For certain important bridges this additional cost can be justified but for others it can not. The choice can then be made by the public rather than by the design engineer or in many cases today, by the code-writing authority. The approach used in California (by Caltrans and CDMG) is to define two levels of earthquake and for each earthquake, specify performance criteria for two classes of bridges ("important" and "ordinary"). There are at least three problems with such an approach and all need to be addressed before widespread adoption can be expected. These are listed below.
(1) *The qualitative nature of terms such as "immediate" and "limited" service level, "repairable" and "significant" damage states, and "important" and "ordinary" bridges.* These are imprecise terms and efforts to refine them by using, say, the number of days a bridge may be closed or be posted, are useful but could lead to legal difficulties if a bridge is closed for a day longer than the code allows, for example. Given the uncertainty in the design ground motions and the fact that most seismic loads are specified in a probabilistic manner, such an approach is going to be difficult to enforce. It may be an appropriate strategy in California where deterministic design loads are common practice, but this may not be so elsewhere.
(2) *The prescription of a design procedure which will deliver the performance level selected by the owner.* The state-of-the-art has progressed in recent years regarding the assessment of damage states but the correlation between component damage (eg curvature ductility demand) and serviceability of the structure (eg lane restrictions and/or repair times) is still ill-defined. Reducing the Response Modification Factor (R or Z or q) for important bridges is one strategy that has been adopted by Caltrans/ATC and AASHTO-LRFD but the relationship between R and damage state/service level is also poorly quantified. The uncertainty in the design ground motions also contributes to this problem.
(3) *The additional design effort.* Rigorous satisfaction of two-level performance criteria requires two separate designs of the bridge: one for each level of earthquake load. Designers who are experienced in two-level design, confirm that the effort, while higher, is not twice as high. Others are reluctant to expend the extra effort, except for important bridges. They assume that for ordinary bridges, if one level of performance is satisfied, the other is automatically satisfied - by default. Simplified design procedures and standardised checkpoints for satisfying the given criteria are required to ease the design effort, especially for "regular" bridges.

Characterization of the Seismic Loads and Site Effects

The most common method for defining the design seimic load is to specify a normalised design "spectrum" which is then scaled uniformly by an acceleration coefficient. This acceleration coefficient represents the effective peak acceleration (EPA) on rock at the site and if overlying soils exist, a second scale factor for soil type is introduced. Most countries map the acceleration coefficient (or its equivalent) and may define seismic zones based on this map. Future trends in characterizing the seismic hazard may involve the mapping of the ordinates of the design spectrum (rather than the EPA) at say two or three characteristic periods (eg 0.3, 1.0 and 3.0 secs). This will allow the shape of the spectrum to be adjusted with EPA and with seismic source zone. Long period values could also be improved under such a plan and the conservatism believed to be inherent in the $T^{-2/3}$ rule (used in several codes but not Caltrans, Japan and New Zealand) would be reduced. Guidance on the characterization of spatial variation effects and near-field effects in the design ground motions, might also be expected in future codes.

The characterization of soil amplification effects using a single soil factor which is independent of period and intensity of earthquake shaking, is also likely to change in the next generation of codes. Already in some United States building codes, two soil factors are used, one for the constant acceleration part of the spectrum and the other for the constant velocity portion. Furthermore these factors are dependent on EPA and decrease for increasing EPA. Such a trend takes into account the high amplifications that occur during small earthquakes (when the soils remain elastic) and the lower amplifications that are characteristic of larger earthquakes (when the soils yield and are unable to amplify the ground motions from the rock below).

Analysis

Current elastic methods of analysis, such as the equivalent static methods and the modal spectral methods, are not likely to change and will remain valuable tools for the designer. However the range of applicable structures may be further refined and in particular methods for bridges with simply supported spans are likely to be improved to better account for the discontinuities in the superstructure and to more accurately estimate connection forces. Also the use of cracked versus uncracked section properties in these analyses may be resolved.

Inelastic methods will be more prominent in future codes and may even be required for important bridges when subject to the higher level earthquake (in a two-level design). This requirement would avoid the use of R-factors (Z- and q-factors) with their inherent difficulties. Equivalent static inelastic methods (push-over methods) may become the preferred method for all but the simplest of structures, but before doing so significant work is required to develop system push-over methodolgies that give consistently reliable results. Currently, push-over methods are used for pier-by-pier analyses and complete bridge analyses by this procedure are not routinely performed, except perhaps by researchers and expert practitioners. Capacity design "spectra" are also needed before code adoption of these methods can be expected.

Design

The use of a Response Modification Factor of some kind (eg R, Z or q) to obtain design forces from elastic analyses is expected to remain a key step in seismic design (except in New Zealand where inelastic spectra are used to obtain design forces). Improvements in the estimation of this factor are likely to continue and these may include dividing it into its component parts (substructure type, redundancy, importance...) and reaching agreement on its relationship to structure and "site" period. Factors for structures on flexible foundations or those with flexible bearings (such as base isolated bridges) also need attention. In some low-to-moderate seismic zones where simplified methods of capacity design are used (such as in Category B in the U.S.), improvements are expected to prevent column overstrengths placing greater-than-anticipated demands on the foundations. This appears to be a particular problem when non-sesismic loads (eg wind) govern the design of the column.

In many countries the seismic design of steel bridges is not as well advanced as for concrete bridges, possibly because few bridge designers use steel columns or piers. Nevertheless improvements are expected in the near future and in particular a revised set of R-factors are anticipated. These new factors are likely to encourage yield in certain superstructure members (such as steel crossbracing), a concept which is not currently permitted in many sesimic codes today (definitions of acceptable damage generally exclude yield in superstructure members). Also, P-delta effects are likely to be addressed in a more rigorous manner, not just for steel columns but for all substructures.

Minimum design values are likely to be increased, especially for restraint forces at guided and/or fixed bearings to account for construction tolerances and the possibility of unanticipated load distributions which exceed connection strengths. Design methods for restrainers (couplers) are also expected to be improved.

New methods of seismic design which are either displacement-based or energy-based or both, are under development at the present time and are likely to find their way into the codes as acceptable alternatives to conventional force-design methods in the near future.

Foundations, Abutments and Walls

Perhaps the area in which there is greatest uncertainty, and the potential for significant improvement, is in the area of foundations, abutments and retaining walls. In many countries research is in progress to improve the characterization of soil properties and the modeling, analysis and design of foundation elements. Such studies include lateral spreading due to liquefaction, soil-structure interaction, pile group effects, soil stiffness and strength in service and ultimate limit states, pile pullout strengths, foundation uplift (rocking), equivalent springs and damping models, abutment back-wall strengths, and active and passive pressures on walls (restrained and unrestrained). Improvements in all of these areas may be expected in future codes.

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