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Seismic Performance of Bridges Learning from California's Experience

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ABSTRACT

In recent years, many bridges have been heavily damaged during strong earthquakes revealing the need to upgrade seismic performance requirements, establish improved design criteria and code provisions, advance performance-assessment methodologies, and develop effective retrofit measures. Because of the very large number of bridges operating in countries such as Canada and the United States, effective procedures for identifying, screening, and ranking those bridges which are hazardous to the public should be formulated. These and other issues, including research needs and the role of government in improving the seismic safety of bridges, are discussed in this paper.

BACKGROUND

Our understanding of the seismic performance of bridges has been advanced largely through post-earthquake investigations of the damages produced. In Japan, the 1923 Kanto earthquake was the first such event to cause major damage to modern facilities, including nearly two thousand bridges which suffered light to heavy damage due to fire and ground failures. Since then numerous other earthquakes have occurred in that country producing similar damages (Iwasaki, Penzien, and Clough 1972). The types of failures most often produced by ground failures have been (a) tilting, settlement, sliding, cracking, and overturning of substructures, (b) displacements, cracking, and complete dislodging of girders at supports, (c) shearing or pulling-out of anchor bolts and crushing of concrete at supports, (d) settlement of backfills, sliding and separation of wing walls from abutments, and failure of parapet walls. It should be noted that a relatively small amount of damage to Japan's bridges has been caused by seismically-induced structural vibrations.

Prior to 1971, the types of seismically-produced bridge failures in the United States were similar to those described above for Japan with few of them the direct result of structural vibrations. Most damages up to that time, even collapse in some cases, were caused by excessive differential movement resulting from ground failures. However, full awareness of the damaging effects of seismically-induced vibrations came with the 1971 San Fernando, California earthquake (M_w = 6.6; M_w is moment magnitude) which caused the collapse of five major freeway bridges and damaged many more (Iwasaki, et al., 1972; Jennings, 1971; Elliot and Nagai, 1973; Fung, et. al., 1971) It then became apparent that the equivalent-static seismic force levels previously used in design were much too low and that the

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detail-design provisions of the code were inadequate. These two factors had led to critical structural deficiencies including:

(1) Expansion joints – Insufficient seat widths and separation restraints allowing decks to fall off their supports,
(2) Columns – Inadequate ties, both in size and spacing, allowing shear and flexure type failures in columns,
(3) Column caps – Lack of reinforcing bars tying column caps to box-girder decks,
(4) Column foundations – Insufficient anchorage of column main reinforcing bars into the foundations, and
(5) Abutment and wing walls – Inadequate strength of abutments and wing walls to carry the backfill earth pressures and deck loadings under expected seismic conditions.

In addition to the above design deficiencies, member strengths were then much too low to accommodate high seismic excitations. Comparing the low seismic design forces used prior to 1971 with those currently used makes this inadequacy quite apparent.

The first specific lateral-load requirement introduced into the design specifications was in 1943. It required designing for an equivalent static lateral loading expressed as a percentage of the dead weight ranging from two to six depending on foundation conditions. Two percent was specified for bridges founded on rock, four percent for bridges founded on soils having bearing capacities exceeding 4 tons per square foot, and six percent for bridges founded on piles. In 1963, the then California Bridge Department adopted the Structural Engineers Association of California (SEAOC) building code provision requiring that the equivalent lateral seismic loading (EQ) be determined using the formula $EQ = KCD$ in which D is the dead loading of the structure, C is a seismic base-shear coefficient given by $0.05/T^{1/3}$ (T being the fundamental period of vibration of the structure) with 0.10 specified as its upper limit, and K is a coefficient reflecting the structures capacity to absorb energy. A K-value of 1.33 was specified for bridges having wall supports with height-to-length ratios of 2.5 or less; 1.00 was specified for bridges having single-column or pier supports with height-to-length ratios greater than 2.5; and 0.67 was specified for bridges supported on continuous frames. The design provision also specified that the product KC should never be less than 0.02.

Revolutionary changes to the seismic provisions of California's design specifications started in 1971 as a result of the San Fernando earthquake. The California Department of Transportation (Caltrans) immediately instituted changes to increase the 1963 code force levels by a factor of 2 for all bridges supported on spread footings and by a factor 2.5 for those on pile foundations. In addition to increasing the code force levels, detail design improvements were implemented to address the critical structural deficiencies described above. In 1977, use of elastic response spectra was introduced into the design specifications for the first time. Load reduction factors to be used in conjunction with these elastic spectra were specified for individual structural components, with their numerical values dependent on corresponding component toughnesses and on the fundamental period of the bridge structure.

Reminders of the structural deficiencies of those bridges designed prior to 1971 came with the 1987 Whittier Narrows, 1989 Loma Prieta, and 1994 Northridge earthquakes, which occurred in California, of magnitudes M_w equal to 5.9, 7.0, and 6.7, respectively (Housner, 1990; Housner, 1994). During these seismic events, the damages to bridges of pre-1971 design were similar to those produced by the

1971 San Fernando earthquake. While Caltrans has aggressively upgraded the seismic design criteria and procedures since 1971, greatly mitigating the earthquake hazard posed by newly constructed bridges, nearly 2,000 out of the State's total inventory of approximately 24,000 bridges in use at this time (1995) remain hazardous, thus posing a threat to public safety. Needless to say, retrofitting hazardous bridges is a top priority activity in California.

With the above background in mind, certain issues related to enhancing the seismic performance of bridges are discussed subsequently, including upgrading seismic performance requirements, establishing improved design criteria and code provisions, advancing performance-assessment methodologies, developing retrofit measures, prioritizing bridges for retrofit, supporting bridge research and the role of government.

PERFORMANCE REQUIREMENTS

Bridge design codes in the United States and other countries have in the past been based primarily on the single performance requirement that life safety be preserved, i.e. heavy damage caused by a maximum credible seismic event has been considered acceptable provided it was short of any type of collapse which could result directly in injuries and possible loss of lives. Recently however, in recognition of other important factors, additional performance requirements are being specified which greatly limit acceptable damages to certain important bridges. For example, Caltrans bridges are now classified into two categories: *important* bridges and *minimum performance* bridges. The bridges in each category are expected to satisfy certain performance requirements when subjected to two different seismic intensity levels, a functional evaluation level and a safety evaluation level. The functional-evaluation ground motion is intended to have a 40 percent probability of exceedance during the expected lifetime of a bridge, while the safety evaluation ground motion is intended to have a mean return period in the range 1,000 to 2,000 years.

A bridge is classified as *important* if one or more of the following three conditions are met:

(1) The bridge is required to provide secondary life safety (example: access to an emergency facility).
(2) The time for restoration of the bridge's functionality after closure creates a major economic impact.
(3) The bridge is formally designated as critical by a local emergency plan.

The performance requirements of *important* bridges subjected to the lower functional-evaluation level of ground shaking allow minimal damage during the earthquake (the bridge performs in essentially an elastic manner); however, service must be restored immediately following an event of this intensity. When subjected to the higher safety-evaluation level of shaking, the damage must be repairable, with minimum risk of a bridge losing its functionality, and service must be restored immediately following such an event.

The performance requirements of all other bridges, i.e., those classified as *minimum performance* bridges, when subjected to the lower functional-evaluation level of shaking, are that the damage must be repairable and that full service must be restored immediately following the event. When subjected to the higher safety-evaluation level of shaking, minimum risk of collapse is, of course, a requirement; however, damage levels sufficient to cause closure for repairs, with only limited access for several days following the earthquake, and sufficient to require months for full restoration of service are acceptable.

It is clear that if new economical bridges are to meet the above performance requirements for the functional-evaluation and safety-evaluation seismic events defined above, it will be necessary to improve design criteria and code provisions and to advance performance-assessment methodologies. Further, if all existing bridges are to meet these same requirements for both design-level events, effective retrofit measures must be taken to upgrade those known to be hazardous.

SEISMIC DESIGN GROUND MOTIONS

The seismic input to each support of a bridge structure is usually specified through appropriate response spectrum curves representing the vertical and horizontal components of motion. In the past, it has been common practice to assume the shape of the response spectrum curve for vertical motion to be the same as that for horizontal motion and to assume the intensity of the vertical motion to be two-thirds that of the horizontal motion. Also, it has been common practice to use the same input to each bridge support, thus ignoring spatial variations in the motions. Recent developments, however allow these variations to be represented in a quantitative manner through the use of so-called "coherency" functions which make it possible to modify the phase angles and amplitudes of the full spectrum of harmonics in a component of ground motion at one point so that the resulting motion will be representative of the corresponding component of motion at another point.

Having specified appropriate response spectrum curves and coherency functions, response-spectrum-compatible and coherency-compatible time histories of ground motion can be generated for all support points of a bridge structure for use in carrying out linear and/or nonlinear time-history dynamic analyses. In doing so, it is important that each pair of orthogonal components of motion, whether representing components at one point or two points, be controlled to have low cross correlation so as to realistically represent actual field conditions during a seismic event. This requirement is easily satisfied by selecting three orthogonal components (two horizontal and vertical) of acceleration recorded at one station during a strong earthquake, modifying these recorded components so as to make them response-spectrum compatible (Lilhanand and Tseng, 1988), letting the modified accelerograms represent the three components of motion at one support point (control point), and then generating from these control motions the corresponding components at all other points so that the specified degree of coherency (or incoherency) between each pair of motions in one direction is satisfied (Hao, Oliveira, and Penzien, 1989; Deodatis, Shinozuka, and Papageogiou, 1990; Abrahamson, 1992; Geomatrix Consultants and Int. Civil Engr. Cons., Inc., 1992) and that all components of motion are compatible with their specified design response spectra. It is important for each time-history of acceleration generated that the corresponding velocity and displacement time-histories be generated to check that they are realistic in form. If not, better control of the longer periods of motion are required in the generation process. Similar checks should be made on the relative displacements between pairs of points.

Design Response Spectra

The design pseudo-acceleration response spectra specified in the current American Association of State Highway and Transportation Officials (AASHTO, 1992) Standard Specifications for Bridges are shown in Fig. 1 for three soil profile types defined as follows:

Soil Profile Type I: Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 ft/sec (762 m/sec);

or stiff soil conditions where the soil depth is less than 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravel, or stiff clays.

Soil Profile Type II: Deep cohesionless or stiff clay soil conditions, including sites where the soil depth exceeds 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravel, or stiff clays.

Soil Profile Type III: Soft to medium-stiff clays and sands, characterized by 30 ft (9.1 m) or more of soft to medium-stiff clay with or without intervening layers of sand or other cohesionless soils.

These normalized spectra were formulated using the Newmark/Hall approach based on statistical averaging of spectra for recorded horizontal motions; however, the averaged forms were smoothed to comply with the observation that the pseudo-acceleration, pseudo-velocity, and displacement spectral values are approximately constant amplifications of the peak ground acceleration (PGA), peak ground velocity, and peak ground displacement, respectively, within their connecting period ranges of primary control (Newmark and Hall, 1982). For example, the Soil Profile Type I spectrum in Fig. 1 is based on a normalized peak ground acceleration equal to 1g, a peak ground velocity equal to 24 in/sec, and acceleration and velocity amplification factors both equal to 2.5, thus resulting in the pseudo-acceleration spectral values being equal to 2.5g over the period range $0.15 < T < 0.39$ sec and the pseudo-velocity spectral values being equal to 60 in/sec over the range $0.39 < T < 3.00$ sec. The acceleration and velocity amplification factors for all three soil-profile types (I, II, and III) were specified to represent the 84-percentile levels; thus, the normalized AASHTO pseudo-acceleration response spectra in Fig. 1 were intended to represent mean-plus-one-standard-deviation ($mean + 1\sigma$) shapes.

The set of seven thin-line curves in Fig. 2 representing discrete values of PGA ranging from 0.1g to 0.7g are the specified 5%-damped elastic acceleration response spectra (ARS curves) in the current Caltrans Bridge Design Specifications (Caltrans, 1987) for hard-site conditions, i.e. 10 ft or less of alluvium over bedrock. Although not presented herein, Caltrans specifies two other sets of 5%-damped response spectra in the Design Specifications representing medium and soft site conditions, namely 10 to 80 ft and 80 to 150 ft of alluvium over bedrock, respectively. All three sets of spectra represent $mean + 1\sigma$ shapes.

It should be recognized that when selecting the appropriate response spectrum curve representing a particular PGA level from one of the above-described Caltrans' sets of spectra, or when specifying a particular PGA level for scaling one of the AASHTO spectra, that the mean return periods (measured in years) of the resulting spectral values in the period range of primary interest, $T > 0.5$ sec, will be much greater than the mean return period of the particular PGA value used (Penzien, 1993). For example, if the mean return period on the particular PGA value specified is 100 years, the mean return period on the $mean + 1\sigma$ response spectral value at $T = 1$ sec will be approximately 750 years.

The set of six thick-line curves in Fig. 2 also represent response spectra for hard-site conditions. These spectra have been developed recently by the firm Geospectra for the Applied Technology Council under a Caltrans sponsored project ATC-32: Review and Revise Standards, Performance Criteria, Specifications and Practices for Design of New Bridge Structures (ATC, 1995). These ATC-32 curves are presently under review for possible adoption by Caltrans as a substitute for the current set of hard-site spectrum curves (thin-line curves in Fig. 2) when the controlling seismic event is judged to be in the magnitude range 6.5 ± 0.25. Two other sets of hard-site spectra have been developed by Geospectra for magnitude ranges 7.25 ± 0.25 and 7.9 ± 0.15. It should be pointed out that as the magnitude range increases to the highest level (7.9 ± 0.15), the corresponding set of response spectra compares reasonably well with the currently used magnitude-independent set represented by the thin-line curves in Fig. 2. This comparison may appear to be a contradiction since the magnitude-dependent set for M_w = 7.9 ± 0.15 have been developed to represent mean shapes, not $mean+1\sigma$ shapes as previously mentioned for the currently used magnitude-independent set of spectra. However, it is now known that the recorded strong-motion accelerograms from which the currently used set were derived were deficient in the longer-period components of motion expected to occur during large-magnitude events; also, it is believed that base-line corrections to the earlier recorded motions removed some of the longer period components which actually were present. In selecting the appropriate response spectrum curve from one of the magnitude-dependent sets, it should be kept in mind that its shape is intended to represent mean values in which case the mean return periods on the spectral values for $T > 0.5$ sec will be approximately the same as the mean return period of the PGA it represents. Comparisons of return periods for the different spectra as discussed here is important when selecting an appropriate set of response spectra and associated PGA levels for design purposes. The bridge performance requirements discussed earlier should be kept in mind when making this selection.

It is now being recognized that the effects of vertical seismic motions on bridge response can be significant; thus, more attention should be given to developing design response spectra representing these motions.

Uniform-Hazard Response Spectra

Since the present trend is to specify certain performance requirements for bridges when subjected to design seismic events having prescribed mean return periods, e.g. in California, prescribing a bridge's lifetime for the functional evaluation event and 1,000 to 2,000 years for the safety evaluation event, it would be consistent to use site-specific uniform-hazard spectra, i.e. response spectrum curves, each of which represents a uniform probability of exceedance over the entire period (or frequency) range of interest, say $0 < T < 10$ sec. The procedure for obtaining such site-specific uniform-hazard spectra is to first generate a set of seismic hazard curves for rock (or hard-site) conditions with each curve expressing an acceleration response spectral value for a specified discrete value of period (or frequency) and a specified discrete value of damping as a function of its annual probability of exceedance (or mean return period in years). The steps used to generate these hazard curves consist of the following (Cornell, 1968; McGuire, 1976; Der Küreghian and Ang, 1977; Sadigh, et al., 1989; Geomatrix Consultants and Int. Civil Engr. Cons., Inc., 1992; Clough and Penzien, 1993):

(1) The near-region around the site is divided into surface seismotectonic zones, each having similar tectonic characteristics over its area. Volume zoning below the surface may also be required to properly represent zones of high seismic activity, e.g. subduction zones. Uniform seismicity is usually assumed over each zone and geologically appropriate source (point and/or fault-rupture) models are specified. Nonuniform seismicity can, however, be assigned in the volume zones if sufficiently supported by field evidence. A conditional probability density function $p_i(r|m)$ is obtained for each source zone i ($i = 1, 2, \ldots n$) in which r and m denote source-to-site distance and magnitude, respectively.

(2) An annual magnitude/recurrence relation of the Gutenberg-Richter Type $logN(m) \equiv a - bm$ is established for each source zone. Constant b in this relation can be obtained by a

least-square fitting of an appropriate seismicity data subset obtained instrumentally, which is judged to be complete; constant a is then obtained using both historical and instrumental data over a long period of time, by specifying a relatively large value of m for which the corresponding value of N is believed to be known reasonably well and by taking into consideration geological factors. Having constants a and b for each source zone i, the corresponding probability density function on magnitude m, $p_i(m)$, can be obtained.

(3) An upper bound magnitude m_u is established for each source zone based on geological considerations and seismicity data but supplemented by the subjective judgement of specialists.

(4) Median (or mean) attenuation relations appropriate to the region of the site are established, expressing acceleration response spectral values $S_a(T, \xi)$, for discrete values of period T and damping ξ, as functions of source-to-site distance r for discrete values of magnitude m; the corresponding variances of the S_a-values about their mean levels are evaluated; and the conditional probability density functions for S_a, $p(S_a|r, m)$, are assumed to be of the lognormal form. Knowing the median values and variances for the spectral values S_a as functions of r and m, the probability of exceedance functions $P(S_a > \bar{S}_a|r, m)$ corresponding to $p(S_a|r, m)$ are also known.

(5) Each lognormal distribution is truncated at some reasonable upper-bound S_a-level, usually the *mean*$+3\sigma$ level. After truncation, the distribution is scaled up so as to satisfy the unit area condition required of a probability density function.

(6) Finally, the annual probability of exceedance function (seismic hazard curve) for each discrete response spectral value $S_a(T, \xi)$, i.e. response spectral value for each specified pair of period (T) and damping (ξ) values, is obtained using the relation

$$P[S_a(T, \xi) > \bar{S}_a] = \sum_{i=1}^{n} N_i(m_o) \int_{m=m_o}^{m_u} \int_{r=0}^{\infty} p_i(m) \cdot p_i(r|m) \cdot P[S_a(T, \xi) > \bar{S}_a|r, m] dr\, dm \quad (1)$$

in which m_o is an arbitrary lower-bound value of m under which the corresponding seismic events contribute negligibly to the hazard. A value of 4.5 is commonly used for m_o. In evaluating the double integrals of Eq. (1), the probability functions in the integrand are discretized, converting the integrals to summations which are compatible with computer solution.

Having generated the sets of hazard curves representing rock (or hard-site) conditions using Eq. (1), uniform-hazard pseudo-acceleration response spectra for specified annual probabilities of exceedance (or mean return in years) over the entire period range of interest are obtained directly therefrom.

Figure 3 shows a set of such uniform-hazard response spectra plotted on a four-way log plot, which represent the expected ground motions at bed-rock level below the east end of the main span of the San Mateo-Hayward Bridge crossing San Francisco Bay. The four curves in this figure, representing mean return periods of 100, 500, 1,000, and 2,000 years, were generated by the firm Geomatrix Consultants of San Francisco for Caltrans (Geomatrix Consultants and Int. Civil Engr. Cons., Inc., 1992). Note that for each curve, drawing a straight line with a +45 degree slope through the spectral value at $T = 0.3$ sec, representing constant pseudo-acceleration spectral values, and a horizontal line through the spectral value at $T = 1$ sec, representing constant pseudo-velocity spectral values, produces a bilinear response spectrum of the Newmark/Hall form approximating conservatively the curved uniform-hazard spectrum over the period range $0.2 < T < 3.0$ sec. Presently, extensive seismic hazard analyses are being carried out by the United States Geological Survey (USGS) to develop equal-spectral-value contour maps for the entire United States with each map representing single discrete values of period T, damping ξ, and mean return period. These maps can serve the basis for generating uniform hazard response spectra for design purposes.

To obtain site-dependent uniform-hazard pseudo-acceleration response spectra for softer site conditions, site-response analyses must be carried out using time histories of input which are compatible with the uniform-hazard hard-site spectra discussed above. The results of such analyses should be qualitatively consistent with site-dependent spectra generated statistically from recorded accelerograms.

Coherency Functions

Let $a_{xi}(t)$ and $a_{xj}(t)$ represent the free-surface ground motions in the horizontal x-direction at bridge support locations i and j, respectively. These motions will differ depending upon the distance d_{ij} separating the two points. This difference is characterized by a complex coherency function

$$\gamma_{ij}(if) \equiv S_{ij}(if)/(S_{ii}(f) \cdot S_{jj}(f))^{1/2} \tag{2}$$

in which $S_{ij}(if)$ is the complex cross spectral density function for motions $a_{xi}(t)$ and $a_{xj}(t)$, $S_{ii}(f)$ and $S_{jj}(f)$ are the real power spectral density functions for motions $a_{xi}(t)$ and $a_{xj}(t)$, respectively, and f is cyclic frequency (Hao, Oliveira, and Penzien, 1989). This function is most often used in the approximate form

$$\gamma(if,\, d_{ij}) = \gamma(f,\, d_{ij})\; exp[i\; 2\pi f\, d_{ij}/v_a] \tag{3}$$

in which $\gamma(f, d_{ij})$ is a real function of frequency f and separation distance d_{ij} representing loss of coherency due to wave-scattering effects, and $exp[i\; 2\pi f\, d_{ij}/V_a]$ is a complex function of frequency f, separation distance d_{ij}, and the apparent wave-front velocity V_a (typically in the range $2 < V_a < 3$ km/sec) representing the wave-passage effect. For application, this velocity must be divided by $sin\theta$ to obtain the corresponding apparent wave velocity along the longitudinal axis of the bridge; θ being the angle between a line perpendicular to the bridge axis and a line connecting the closest point of the fault to the center of the bridge. To account for waves arriving at the bridge from other points along the fault, some minimum value of θ should be specified, say 30° (Tseng et al., 1994). Figure 4 shows a set of $\gamma(f, d_{ij})$ functions plotted against frequency f for discrete values of d_{ij}, which were developed by N. A. Abrahamson (Abrahamson, 1992). Studies have indicated that these functions are quite insensitive to local site conditions (Schneider, 1992); thus, the curves in Fig. 4 can be used in representing coherency at any site.

Using the appropriate design response spectrum for a particular site and the coherency functions in Fig. 4, the x-components and y-components of horizontal free-field motion at the multiple support locations of a bridge structure can be generated which are both response-spectrum-compatible and coherency-compatible (Geomatrix Consultants and Int. Civil Engr. Cons., Inc., 1992).

Near-Field Motions

When generating response-spectrum-compatible and coherency-compatible near-field (source-to-site distances less than 10 km) time histories of ground motion to serve as the horizontal multiple-support inputs to an *important* bridge structure, recognition should be given to the observation that when the fault rupture is of the vertical-plane strike-slip type the fault-normal components of motion are usually somewhat higher in intensity than the corresponding fault-parallel components. For certain San Francisco Bay bridge sites, Geomatrix Consultants has prescribed response spectral values for the fault-normal components in the period range 2 to 5 sec to be 20 percent larger than the corresponding average values for the fault-normal and fault-parallel components (Geomatrix Consultants and International Civil Engineering Consultants, Inc., 1993). This 20 percent increase is gradually decreased below the 2-sec period value. Having generated time-histories of motion in the fault-normal and fault-parallel directions at each bridge support location, they are then transformed into components of motion in the principal directions (longitudinal and transverse) of the bridge structure.

Recognition should also be given to the presence of the so-called *velocity-pulse* motion in the near field caused by the permanent fault offset. This transient-type motion should be added to the response-spectrum-compatible and coherency-compatible time histories of motion. Because of the relatively long duration of such velocity pulses, they can be very damaging to yielding bridge systems.

MODELLING AND ANALYSIS OF BRIDGES

The modelling of bridges for seismic performance evaluations ranges from simple single-degree-of-freedom (SDOF) models to complex multiple-degree-of-freedom (MDOF) models. The SDOF models may be of the single lumped mass type or of the distributed-mass generalized type as described in the AASHTO Standard Specifications. The MDOF models are usually of the lumped-mass finite-element type. When the number of bridge members becomes excessively large, super elements, each consisting of multiple finite elements, can often be used effectively to reduce the total number of DOF in a particular bridge model. Also, a limited number of discrete foundation impedance elements representing soil-structure interaction effects can be used to further reduce the number of DOF. Separate rigorous modelling of the foundation system is usually required to obtain these impedance elements.

Even though the design criteria for a particular bridge may allow considerable inelastic deformation to occur during a maximum design seismic event, say during a Safety Evaluation event as defined by Caltrans, the initial seismic response analysis should always be performed using a linear elastic model. Dividing the component forces so obtained by their corresponding yield forces, to obtain so-called force demand/capacity ratios, will provide a good indication of what components are most vulnerable in the total system. For most uniform bridges (fundamental periods $T > 0.5$ sec) which can be modelled reasonably well as SDOF systems, the maximum global displacements obtained from elastic analyses will be approximately the same as those for the real yielding systems; thus, global ductility demands (maximum global displacements divided by corresponding yield displacements) can be estimated accordingly. Local ductility demands can be much higher, however, so care must be taken in assessing their values.

For complex bridges, nonlinear modelling is required to obtain realistic predictions of behavior during a maximum design event. This nonlinear modelling should start with those components showing force demand/capacity ratios considerably greater than unity by the linear elastic analyses. It may be necessary, in the interest of keeping the overall bridge-system model simple, to remove multiple-component portions of the system where local yielding has been indicated by the elastic analysis and to separately evaluate their own global nonlinear hysteretic force-deformation relations. Inserting each portion back into the overall model as a nonlinear super element having limited DOF, the largely linear elastic model, but with a limited number of nonlinear super elements, can be analyzed using nonlinear analysis techniques. Should this analysis show substantial changes in the inelastic deformation patterns, some further adjustments of the nonlinear super elements may be necessary.

This author cautions against *full* nonlinear modelling of a large complex bridge system, i.e. modelling each element in the total system in nonlinear hysteretic form, when conducting a seismic performance evaluation. Such an approach, which could involve hundreds of thousands of DOF, is likely to be unsuccessful. Nonlinear modelling should be introduced slowly starting with a fully elastic model so as to always maintain confidence in the validity of the results obtained.

LOAD REDUCTION FACTORS

Having established the design response spectra and performance requirements for a particular bridge, it remains to specify load reduction factors for design which are consistent with the established inputs and corresponding inelastic deformations (or damage) permitted. Since the SDOF model is often used in assessing seismic performance, let us examine the relationship between load reduction factor and amount of inelastic deformation produced. Figure 5 shows the SDOF model and two different global force-displacement relations: one representing linear elastic response, the other representing elastic-plastic response. The relative displacements, v_e, v_y, and v_{ep} are the maximum absolute value of elastic response, yield-level response, and maximum absolute value of elastic-plastic response, respectively; and the associated forces f_e and f_y are the maximum absolute value of elastic force and the yield force, respectively. The equation of motion controlling both the elastic and elastic-plastic responses is given by

$$\ddot{v}(t) + (4\pi\xi/T)\dot{v}(t) + f(v)/m = -\ddot{v}_g(t) \tag{4}$$

If $f(v)$ follows the elastic force-displacement relation in Fig. 5, the maximum absolute value of relative displacement is given by the specified design acceleration response spectrum using

$$v_e = S_a T^2/4\pi^2 \tag{5}$$

When $f(v)$ follows the elastic-plastic force-deformation relation, the maximum absolute value of relative displacement v_{ep} is given by solving the nonlinear equation of motion (Eq. 4) using a time-history of ground motion $\ddot{v}_g(t)$ which is compatible with the specified design response spectrum. Dividing this maximum relative displacement by the yield displacement gives the global ductility demand

$$\mu \equiv v_{ep}/v_y \tag{6}$$

To control the global ductility demand to a specified performance limit, the maximum elastic force f_e corresponding to the specified design response spectrum is divided by a load reduction factor R to obtain the design force. Assuming ultimate strength design, as used for reinforced concrete bridges, the load reduction factor can be expressed in the form

$$R \equiv f_e/f_y = v_e/v_y \tag{7}$$

in which f_y and the corresponding displacement v_y represent initial gross yielding in a ductile mode of response, usually a flexure mode. Combining Eqs. (6) and (7) gives

$$R = \mu \, v_e / v_{ep} \tag{8}$$

It has been shown (Newmark and Hall, 1982) that the ratio v_e/v_{ep} in Eq. (8) depends on the period T of the SDOF system. For ground motions characteristic of firm soil conditions, this ratio is approximately equal to unity for T greater than about 1/2 sec; however, for T equal to about 1/4 of a second, this ratio is approximately given by

$$v_e/v_{ep} = (2\mu - 1)^{1/2}/\mu \tag{9}$$

Regardless of the type of ground motion input, this ratio approaches $1/\mu$ as $T \rightarrow 0$. The transitions going from unity at $T = 1/2$ sec to the value given by Eq. (9) at $T = 1/4$ sec and then on to the value $1/\mu$ at $T = 0$ should on a statistical-average basis be reasonably smooth; thus, the R versus T relations for discrete values of μ will have the general appearance shown by the set of curves in Fig. 6 (Penzien, 1993). Note that for T greater than 0.5 sec, the global ductility demand μ is equal to the load reduction factor R.

Recognizing the type of relationships shown in Fig. 6, the Applied Technology Council has recommended to Caltrans that the component forces obtained from elastic modelling and analysis be divided by *force reduction divisors* Q to obtain the corresponding design forces (ATC, 1995). These force reduction divisors are to be interpolated from the curves shown in Fig. 7 which express Q as a function of the period ratio T/T_g for discrete values of an *adjustment factor* D. The characteristic ground-motion period T_g corresponds to that period at the peak of the input energy spectrum, and is to be taken as that value at the intersection of the nearly constant pseudo-acceleration and pseudo-velocity segments of the design response spectrum. Typical values for T_g would be 0.5, 0.75, and 1.0 sec for hard, medium, and soft sites, respectively. The recommended values of D to be used in selecting values of Q from Fig. 7 are shown in Figs. 8(a) and 8(b) for the *minimum-performance* and *important* bridge categories, respectively. Note that these adjustment factors are expressed as functions of member aspect ratio L/H and that they differ from one type of component to another depending upon component ductility capacities. In assessing the rationale of this procedure, one should recognize that the global ductility demand is usually controlled by local yielding in elements of a single type, e.g. the columns; in which case, the maximum forces developed in the other elements are limited to those levels which are consistent, in an equilibrium sense, with the maximum strengths of the yielding elements.

RETROFITTING HAZARDOUS BRIDGES

Many hazardous bridges now (1995) exist in countries (or regions) of moderate to high seismicity posing a threat to public safety and the local economies. For example, approximately 2,000 state-highway bridges in California have been identified as requiring detailed evaluation and retrofit. Most of these bridges are of pre-1971 design which are known to have structural deficiencies of the type previously described.

Retrofit Actions

Immediately following the 1971 San Fernando earthquake, the California Bridge Department inspected the bridge failures in the region of the earthquake and concluded that the unseating of bridge decks at abutments and expansion joints was the principal cause of bridge collapse (Fung et. al., 1971). The decision was then made that all state-highway bridges in areas of high seismicity be retrofitted with hinge restrainers, a program that later became known as the Phase I Bridge Seismic Retrofit Program. Because the effectiveness of the first-generation hinge restrainers in preventing collapse was uncertain, Caltrans initiated a series of laboratory tests at the University of California at Los Angeles (UCLA) in 1985 aimed at improving restrainer designs (Selna, 1989). Based on the performance of bridges during the 1989 Loma Prieta and 1994 Northridge, California earthquakes, the hinge-restrainer retrofit program was shown to have been effective.

In the spring of 1987, as the Phase I hinge-restrainer retrofit program was progressing, Caltrans initiated a single-column retrofit research program at the University of California, San Diego (Priestley and Seible, 1991). Results of this research showed that enclosing columns in steel jackets could significantly increase their shear strength and ductility by providing confinement of the concrete. In 1988, while this research program was underway, Caltrans initiated a Phase II Bridge Seismic Retrofit Program that consisted of retrofitting single-column piers and, when needed, strengthening their associated footings and foundations. This retrofit program started well ahead of completion of the research program at UC San Diego; however, it took advantage of the early test results of that program by making use of test data as soon as made available.

Following the 1989 Loma Prieta earthquake, which caused major damage to double-deck viaducts in the San Francisco Bay Area, including collapse of an approximately one-mile segment of the double-deck Cypress structure in Oakland, the California Senate enacted a bill which called for complete retrofit or replacement of all "publicly owned" bridges in the State, so that they would meet the then-current seismic safety criteria. The funding authorized by this bill greatly accelerated the Caltrans' retrofit programs then in place. It also allowed implementation of multicolumn and complex-structure seismic retrofit research and implementation of the Phase III Bridge Seismic Retrofit Program of retrofitting bridges supported on multicolumn bents.

Selection of Bridges for Retrofit

Caltrans' selection and ranking of bridges for retrofit has been based on a three-step process as follows:

(1) A ranking of the approximately 24,000 state-highway and local bridges in the State in terms of need of retrofit was established using a computerized prioritization algorithm that evaluates various attributes of each bridge using information available in Caltrans' computer data file. This evaluation takes into consideration each bridge's vulnerability (V_f) to strong-motion earthquakes, the seismic hazard (H_f) at its site, and the impact (I_f) of a possible failure on the local community. Each of these principal factors, V_f, H_f and I_f are measured in terms of a sub-set of factors. Namely, V_f is measured in terms of year designed (0.25), outriggers or shared columns (0.22), abutment type (0.8), skewness (0.12), drop-type failure (0.165), and bent redundancy (0.165); H_f is measured in terms of site conditions (0.33), peak rock acceleration (0.38), and duration (0.29); and, I_f is measured in terms of average daily traffic on the bridge (0.28), average daily traffic under/over the bridge (0.12), leased air space under the bridge -- residential, office (0.15), leased air space under the bridge -- parking, storage facility (0.07), land features -- route or stream type, state or federal ownership, etc. (0.07), route type on bridge (0.07), detour length (0.14), and critical utility (0.10). The numbers shown in parentheses are maximum weighting factors

which add up to unity for each of the subgroups V_f, H_f and I_f. Each maximum weighting factor is further scaled from zero to one for each bridge in a specified manner to reflect the severity of its contribution to the need for retrofit of that particular bridge. Adding up these scaled factors for each of the three categories, yields numerical values for V_f, H_f and I_f, each of which falls in the range zero to one. These numerical values for each of the approximately 24,000 bridges were introduced into the algorithm

$$\text{Priority Rating Index (PRI)} = (A_f H_f)(0.60 I_f + 0.40 V_f),$$

in which A_f is a seismicity frequency-of-occurrence parameter assigned a value in the range 0.25 to 1.0 for each bridge site, to obtain its PRI number. Using the PRI numbers, a ranking of all bridges for possible retrofit was obtained. From this ranking Caltrans judged that 4,612 of the 11,895 state-highway bridges, and 6,807 of the approximately 12,000 local (county and city) bridges were not candidates for retrofit, leaving 7,283 and 5,193 state-highway and local bridges, respectively, as potentially hazardous structures.

(2) Caltrans' more-experienced design engineers carried out a detailed plan review of the 12,476 (7,283 + 5,193) state-highway and local bridges remaining as potentially hazardous at the end of Step (1) described above. This review screened out 3,595 of the state-highway bridges and 4,152 of the local bridges, leaving 3,688 state-highway and 1,041 local bridges as potentially hazardous structures.

(3) A detailed seismic evaluation of the 4,729 (3,688 + 1,041) state-highway and local bridges remaining at the end of Step (2) is being carried out in order of priority to identify structural deficiencies, to develop retrofit-cost estimates, and to reprioritize them for retrofit action. As of June 1994, 1,151 of the 3,688 state-highway bridges remaining at the end of Step (2) had been screened out as not needing retrofit, leaving 2,537 as potential candidates; as of June 1994, it had been estimated that about 2,000 out of the total inventory of 12,176 state-highway bridges would require detailed evaluation and retrofit. Of the 1,041 local bridges which remained as potential candidates for retrofit at the end of Step (2), 393 have been processed through the Step (3) seismic evaluation, leaving 648 to be processed. Of the 393, 219 have been found to need retrofitting.

The three-step process described above appears to be quite effective overall in ranking bridges for retrofit; however, further improvements can undoubtedly be made, particularly with regard to the PRI algorithm presented in Step (1). To bring about significant improvements in its analytical form and the weighting factors involved, extensive correlation studies of observed seismically-induced and analytically-predicted damages for a variety of bridge types and site conditions will, most likely, be required.

Effectiveness of Retrofit Actions

While the first-generation hinge restrainers installed at expansion joints following the 1971 San Fernando earthquake were later found to be deficient, those of current design have proven to be very effective in preventing the unseating of girders at their hinge seats. Effective hinge restrainers, however, often lead to higher deck-level seismic forces, which can cause shear and/or flexure failures in the piers and failures in their foundations, thus, requiring that effective single-column and multicolumn retrofits, along with their associated foundation retrofits, be carried out.

The current retrofitting of single-column-pier bridges in California appears to be effective based on the fact that during the Northridge earthquake, 24 retrofitted bridges in the region experiencing peak ground accelerations greater than 0.50g and 60 retrofitted bridges in the region having peak ground accelerations greater than 0.25g performed satisfactorily. Although, only a small percentage of the multi-column-pier bridges selected for retrofit have been so treated, it is expected that after retrofitting is complete, satisfactory performance will be shown during strong seismic events. Similar satisfactory performance of the long-span toll bridges, 12 of which exist in the State, can be expected after retrofitting is complete.

Deck isolation can be used effectively to reduce seismically-induced forces in a bridge structure provided (1) the isolation system has suitable force-displacement and damping properties which will be maintained over the life of the structure, (2) the system will remain stable under the combined static and seismic loadings during a maximum design event, and (3) the overall system can safely tolerate the associated large shear displacements produced in the isolation system. Since the use of deck isolation increases the fundamental period of the overall system, peak free-field ground acceleration is no longer a critical parameter to the seismic response. In this case, peak free-field ground velocity becomes more critical and, with sufficient increase in period, the peak free-field ground displacement can become the most critical parameter. Because of the sensitivity of deck-isolated bridges to the longer periods of free-field ground motion, such isolation should be avoided at very soft sites. Also, at sites located adjacent to active faults where the free-field ground motions are likely to contain strong *velocity-pulse* type motions resulting in large permanent fault offsets, deck isolation should be avoided.

PEER REVIEWS

In response to the report "Competing Against Time," California's Governor George Deukmejian issued an executive order in June, 1990 directing Caltrans to establish a formal review process (Housner, 1990). Caltrans then established a Seismic Advisory Board, charging it with review and evaluation of Caltrans seismic policy and technical procedures. Since its first meeting on August 1, 1990, the Board has met on a regular basis. Caltrans has also established a seismic-safety peer review process to consult and advise on various technical issues, including seismic design criteria, design details, and complete designs. The purpose of such reviews is to integrate the latest technology into Caltrans' overall program of retrofitting hazardous bridges and into its normal activity of designing new bridges.

The use of outside specialists in the retrofit program has proven to be very effective in improving the seismic safety of California's bridges. It is clear that such specialists should be involved throughout the process, starting with selection of bridges for retrofit and continuing on through the conceptual and detail-design phases of the program.

RESEARCH NEEDS

No attempt will be made here to provide a comprehensive list of research needs related to advancing the state-of-the-art of bridge engineering in regions of moderate to high seismicity; rather, only certain general areas in need of investigation will be mentioned. Recognizing the trend towards specifying different levels of performance for prescribed mean return periods, more realistic probabilistic characterizations of expected free-field ground motions and improved nonlinear modelling and analysis procedures are needed.

Clearly, all of the steps involved in generating site-specific uniform-hazard response spectra need further attention, including the methodologies involved and their associated uncertainties. Also, further attention should be given to time-history characterization of ground motions, especially in the near-field where *velocity-pulse* type motions are expected. In support of these activities, every effort should be made to strengthen field investigations and strong-motion instrumentation programs.

Linear modelling of bridge structural systems for low-level seismic response evaluations can be accomplished quite satisfactorily; except that, in many cases, the foundation modelling becomes quite uncertain. For high-level seismic response evaluations which require nonlinear modelling, large uncertainties can be present in both the foundation and bridge structure itself. Obviously, the behavior of pile, cast-in-drilled-hole shaft, and caisson foundations under maximum design seismic conditions need to be better understood, which will require rigorous analytical investigations correlated with laboratory and field-test results. The modelling of structural components under large-deformation cyclic conditions can also have large uncertainties, particularly for old construction which does not meet modern code requirements, e.g. the laced members, riveted connections, and built-up sections of old steel bridges which will experience global and local buckling.

While many computer programs have been developed to carry out nonlinear seismic performance evaluations of structures, further improvements should be made to those programs most applicable to bridges. They should be sufficiently versatile to accommodate a large variety of linear and nonlinear elements under large-deformation cyclic conditions. Validation of such programs, starting with full elastic models and progressing through nonlinear hysteretic models of increasing complexity, is essential. To provide a basis for validating modelling and analysis procedures, more of the larger important bridges located in regions of high seismicity should be instrumented to record their responses during seismic events.

More effort should be given to improving the algorithm for ranking bridges for possible seismic retrofit; and, the development of retrofit concepts along with laboratory testing of their effectiveness should continue.

ROLE OF GOVERNMENTS

Since bridges are primarily owned by the public, governments having jurisdiction over them must accept full responsibility for their safe operation during maximum credible seismic events. It is essential that the engineers involved produce designs that will, as a minimum, insure life safety. Designing for levels of performance higher than that required for life safety, i.e. to levels that will significantly limit damage, raises economic, social, and political issues which require the participation of planners and decision makers other than engineers for their resolution. Engineers must, however, provide the technical basis for others to make sound decisions on this matter.

Governments, in their role of protecting the public, have a responsibility to support the engineering community in its efforts to improve the state-of-practice of bridge engineering; and likewise, they have a responsibility to support the research community in its efforts to improve the state-of-the-art. Engineers have the responsibility of working cooperatively with government officials to bring about this support.

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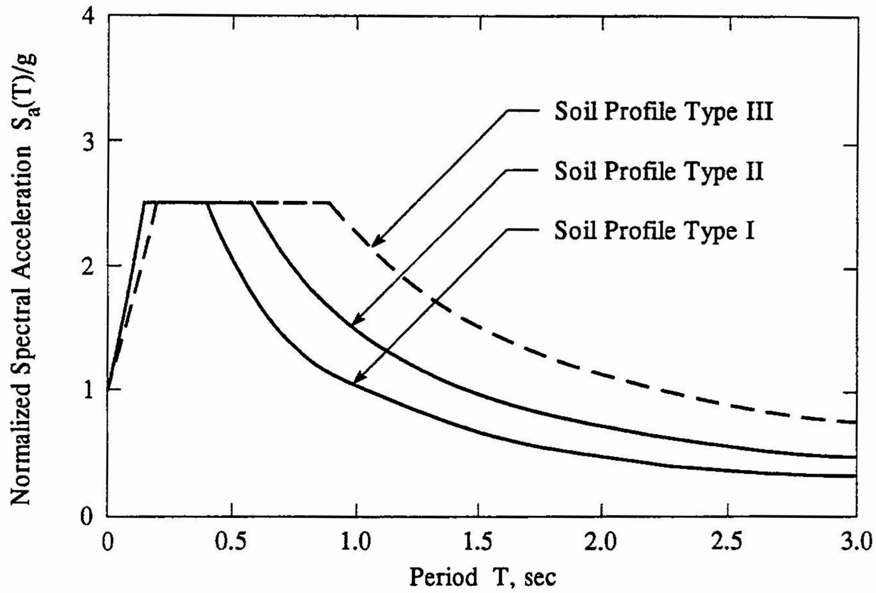


Figure 1 AASHTO, 1992, Normalized acceleration response spectra.

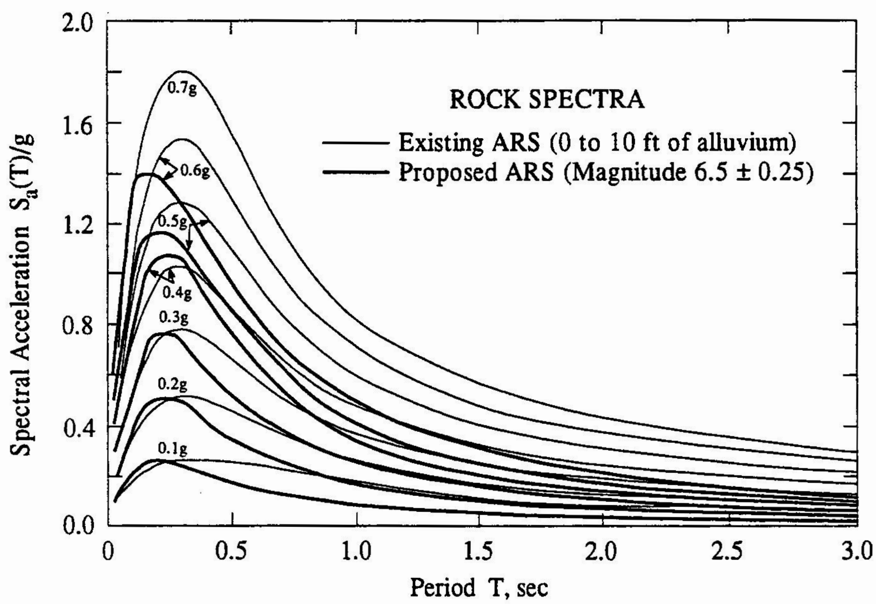


Figure 2 Caltrans' current and proposed ARS curves.

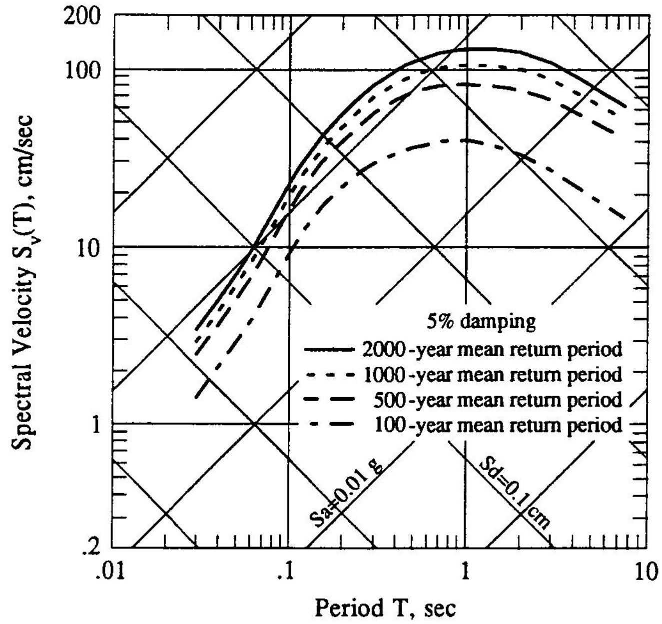


Figure 3 Uniform-hazard response spectra for sites at the east end of the San Mateo-Hayward bridge.

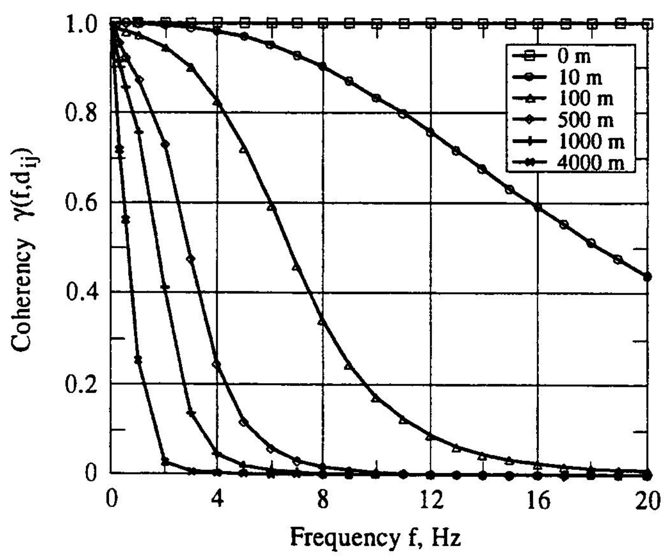
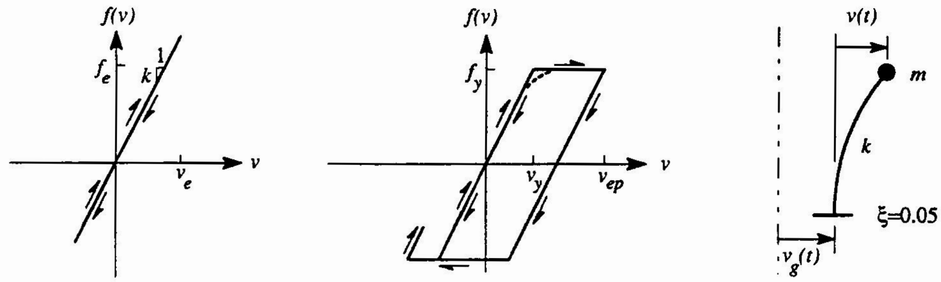


Figure 4 Coherency functions for horizontal motions.



Elastic Response | Elastic-Plastic Response | $T = 2\pi\sqrt{m/k}$

Figure 5 Single degree of freedom system.

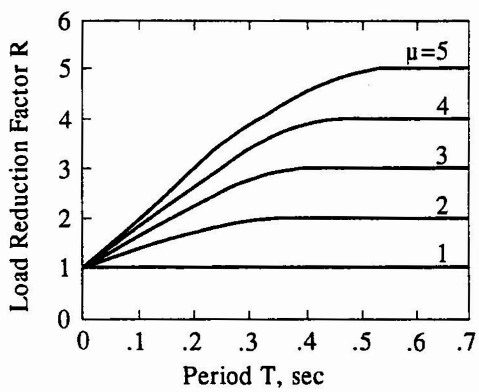


Figure 6 Load reduction factor R vs. period T relations for discrete values of ductility demand μ.

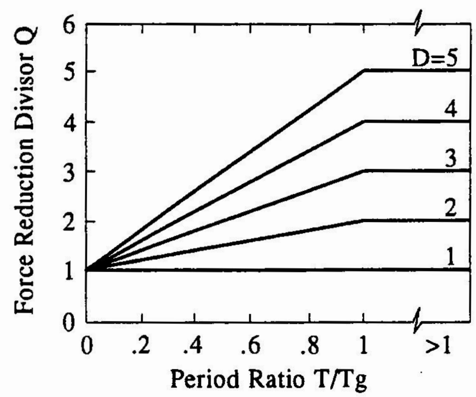


Figure 7 Force reduction divisor Q vs. period ratio T/Tg for discrete values of adjustment factor D.

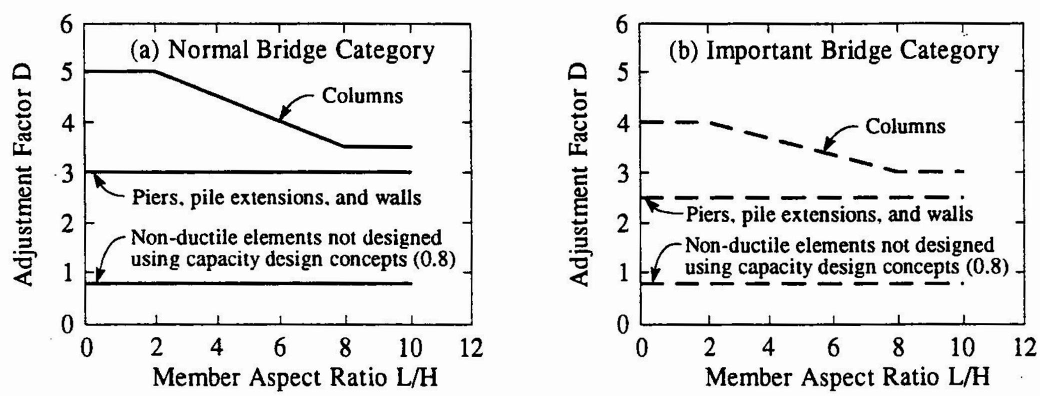


Figure 8 Allowable values of adjustment factor D vs. member aspect ratio L/H.