Seismic Performance Evaluations of Transportation Structures

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ABSTRACT

This paper discusses the steps that are necessary in assessing the seismic performance of existing and new transportation structures, including (1) characterizing expected ground motions, (2) setting performance criteria, (3) modelling and dynamic analysis, and (4) interpreting predicted dynamic response behavior. To establish a basis for assessing the performance of older existing bridges, the history of loading criteria over the past 50 years, as specified in the U.S. AASHO/AASHTO national bridge code, is reviewed.

INTRODUCTION

The importance of designing bridges to withstand the vibratory response produced during earthquakes was first revealed by the 1971 San Fernando, California earthquake during which many bridge structures collapsed. Similar bridge failures occurred during the 1989 Loma Prieta and 1994 Northridge, California earthquakes and the 1995 Kobe, Japan earthquake. As a result of these experiences, much has been done recently to improve provisions in seismic design codes, develop more effective detail designs, and advance modelling and analysis procedures for assessing seismic performance.

Unfortunately, many of the older existing bridges in the United States and other countries, which are located in regions of moderate to high seismic intensity, have serious deficiencies which threaten life safety during future earthquakes. Because of this threat, aggressive actions are being taken in California, New York, and elsewhere to retrofit such bridges to bring their expected performances during future earthquakes to acceptable levels. For new bridges, a rapid change is taking place toward "performance-based" design, which focuses on ensuring satisfactory performance under expected levels of seismic excitation.

SEISMIC LOADING CRITERIA

Revolutionary changes have taken place over the past 50 years in earthquake engineering as applied to transportation structures. This becomes apparent when one reviews the changes in seismic loading criteria specified by the American Association of State Highway Officials (AASHO) in its Standard Specifications for Highway Bridges, First (1931) through Eleventh (1973) Editions, and by the American Association of State Highway and Transportation Officials (AASHTO) in its 1973 Interim Specifications for Highway Bridges and the subsequent Standard Specifications for Highway Bridges, Twelfth (1975) through Sixteenth (1996) Editions, and in AASHTO's LRFD Bridge Design Specifications, First (1994) and Second (1999) Editions. All of the above-mentioned specifications apply to Ordinary Bridges having span lengths under 500 feet.

AASHO STANDARD SPECIFICATIONS, 1949-1975

Standard Specifications, 1949-1961

The first reference to considering earthquake effects on bridges came in the Fifth (1949) Edition of the Standard Specifications which stated that earthquake stresses should be considered; however, no guidelines for doing so were given. This same reference was stated again in the Sixth (1953) and Seventh (1957) Editions.

Standard Specifications, 1961-1975

The Eighth (1961) Edition of Standard Specifications was the first to specify an earthquake loading for design (EQ), namely

$$EQ = CD \tag{1}$$

which was to be applied statically in any horizontal direction as part of a Group VII load combination given by

$$\text{Group VII} = D + E + B + SF + EQ \tag{2}$$

in which D, E, B, and SF denote dead load, earth pressure, buoyancy, and stream flow, respectively. The numerical values of C were specified to be 0.02 for structures supported on spread footings where the soil bearing capacity was rated to be greater than 4t/ft^2 (383 kPa), 0.04 for structures supported on spread footings where the soil bearing capacity was rated to be less than 4 t/ft^2 (383 kPa), and 0.06 for structures founded on piles. The Group VII load combination was to be used in the working-stress design (WSD) with a 33-1/3 percentage increase in allowable stress because of the presence of the earthquake loading EQ. No seismic zone factors were provided in the specifications.

The above seismic loading provisions of the Eighth (1961) Edition of Standard Specifications were repeated, without modification, in the Ninth (1965), Tenth (1969), and Eleventh (1973) Editions. It should be noted that these seismic loading provisions were based mainly on the lateral force requirements for buildings developed prior to 1961 by the Structural Engineers Association of California (SEAOC).

AASHTO STANDARD SPECIFICATIONS, 1975-1992

As a result of the 1971 San Fernando, California earthquake during which many highway bridges were severely damaged, some of which even collapsed, the California Department of Transportation (Caltrans) issued new seismic design criteria for bridges in 1973, which formed the basis of the 1975 AASHTO Interim Specifications for Highway Bridges. The equivalent static lateral force loading specified in this document for bridges having supporting members of approximately equal stiffness was of the form

$$EQ = CFW \tag{3}$$

which was to be applied in any horizontal direction as part of the same Group VII load combination given by Eq. (2) in a working stress design with a 33 percent increase in allowable stress. In this equation, W represents dead load, F is a framing factor assigned the values 1.0 for single columns and 0.8 for continuous frames, and C is a combined response coefficient as expressed by

$$C = ARS/Z \tag{4}$$

in which A denotes maximum expected peak ground acceleration (PGA) as shown in a seismic risk map of the United States, R is a normalized (PGA = 1g) acceleration response spectral value for a rock site, S is a soil amplification factor, and Z is a force reduction factor depending upon structural-component type which accounts for the allowance of inelastic deformations. The numerical values specified for A were 0.09g, 0.22g, and 0.50g in seismic zones numbered I, II, and III, respectively. Numerical values for R, S, and Z were not provided in the 1975 Interim Specifications; rather, four plots of C as functions of period T were given for discrete values of A. Each of these plots represents a different depth range of alluvium to rock-like material, namely 0-10', 11-80', 81-150', or >150' (1' = 30.48 cm). Period T was to be evaluated using the single-degree-of-freedom (SDOF) relation

$$T = 0.32\sqrt{\frac{W}{P}} \tag{5}$$

in which P equals the total uniform static loading required to cause a 1-inch (2.54 cm) horizontal deflection of the whole structure at the center of gravity of the deck.

For complex or irregular structures, the 1975 Interim Specifications required use of the modal response-spectrum analysis method to generate design loads; and, in special cases of such structures having fundamental periods longer than 3 seconds, it required that they be designed using "current seismicity, soil response, and dynamic analysis techniques."

The same seismic loading criteria in the 1975 Interim Specifications were repeated in the Twelfth (1977), Thirteenth (1983), and Fourteenth (1989) Editions of AASHTO's Standard Specifications; however in these editions, the designer was given, for the first time, the choice of working-stress design (WSD) or load-factor design (LFD). When using the WSD, the same Group VII load combination given by Eq. (2) was specified to be used along with a 33 percent increase in allowable stress; however, when using the LFD, the Group VII load combination was changed to the form

$$\text{GroupVII} = \gamma\left[\beta_D D + \beta_E E + B + SF + EQ\right] \tag{6}$$

in which load factor γ was assigned the value 1.3, β_D was assigned the values 0.75, 1.0, and 1.0 when checking columns for minimum axial load and maximum moment or eccentricity, for maximum axial load and minimum moment, and for flexure and tension members, respectively, and β_E was assigned the value 1.3 for lateral earth pressure and 0.5 for checking positive moments in rigid frames.

AASHTO STANDARD SPECIFICATIONS, 1992-1999

After the 1989 Loma Prieta, California earthquake, the Applied Technology Council (ATC) issued its ATC-6 Seismic Design Guidelines for Bridges under the sponsorship of the Federal Highway Administration, Department of Transportation in 1981. These guidelines were reviewed and revised slightly by the National Center for Earthquake Engineering Research (NCEER) under sponsorship of the National Cooperative Highway Research Program (NCHRP) Project 20-7/45 to form the basis of AASHTO's Fifteenth (1992) and Sixteenth (1996) Editions of the Standard Specifications. In these editions, each bridge structure must first be classified as either "Essential" or "Other" in accordance with given definitions and then be assigned to one of four Seismic Performance Categories (SPC) A, B, C, or D as defined in Table 1 below

Table 1 - Seismic Performance Categories

Acceleration Coefficient	Bridge Classification	
	Essential	Other
$A \leq 0.09$	A	A
$0.009 \leq A \leq 0.19$	B	B
$0.019 \leq A \leq 0.29$	C	C
$0.29 \leq A$	D	C

in which the acceleration coefficient, A, for a given bridge site is taken from contour maps provided.

No dynamic analysis is required in these editions for bridges having single spans, regardless of the value of the site acceleration coefficient A, and for all bridges in SPC A. All other bridges, regular or irregular, having two or more spans must be analyzed by at least one of two dynamic analysis procedures, namely, the single-mode spectral method (SMSM) or the multi-mode spectral method (MMSM). The SMSM is specified as minimum for regular bridges in SPC B, C, and D; while the MMSM is specified as minimum for irregular bridges in these same categories. An "irregular" bridge is defined as one having abrupt or unusual changes in mass, stiffness, and/or geometry from abutment to abutment; a "regular" bridge is one not meeting the definition of an "irregular" bridge.

The seismic input in any horizontal direction to be used in each of these minimum dynamic analysis procedures is specified in terms of an elastic seismic response coefficient, C_{sm}, as expressed by

$$C_{sm} = \frac{1.2AS}{T_m^{2/3}} \tag{7}$$

in which T_m is the period of vibration of the m^{th} mode, S is a site coefficient having the values 1.0, 1.2, 1.5, and 2.0, respectively, for soil profile Types S_1, S_2, S_3, and S_4 ranging from hard (S_1) to very soft (S_4), and A is an acceleration coefficient taken from the contour map prepared by the U.S. Geological Survey for the 1988 Edition of NEHRP "Recommended Provisions for the Development of Seismic Regulations for New Buildings". The values of A in this map represent peak ground accelerations having a mean return period of 475 years.

Since each analysis procedure generates internal force components in members caused by only a single horizontal component (x or y) of seismic input, the procedure selected must be repeated using the same response-spectrum seismic input applied in each of the two orthogonal horizontal directions. The corresponding pairs of internal force components $(Q_x \text{ and } Q_y)$ produced by both inputs must then be combined, using the "30-percent" rule, into two combined forms, $Q_x + 0.3\,Q_y$ and $Q_y + 0.3\,Q_x$, with the larger of these two used for design. It is more rational, however, to use the "40-percent" rule when the two orthogonal horizontal inputs are of the same intensity as specified in the AASHTO Standard Specifications. The square-root-of-the-sum-of-squares (SRSS) method, which is the basis for both the "30-percent" and "40-percent" rules, can be used directly to combine pairs of force components regardless of whether or not the inputs are of the same intensity.

Since inelastic deformations are allowed in ductile bridge elements, the combined elastic force components are then divided by appropriate response modification factors, R, as specified in Table 2 below to obtain modified earthquake response values, *EQM*.

Table 2 - Response Modification Factors (R)

Substructure	R	Connections	R
Wall-Type Pier	2	Superstructure to Abutment	0.8
Reinforced Concrete Pile Bents a. Vertical piles only b. One or more battered piles	 3 2	Expansion joints within a span of the superstructure	0.8
Single Columns	3	Columns, piers, or pile bents to cap beam or superstructure	1.0
Steel or Composite Steel & Concrete Pile Bents a. Vertical piles only b. One or more battered piles	 5 3	Columns or piers to foundations	1.0
Multiple-Column Bent	5		

These modified values, *EQM*, replace the values *EQ* in Eq. (2) for use in WSD of structures in Categories, B, C, and D, allowing a 50 percent increase in allowable stresses for structural steel and a 33-1/3 percent increase for reinforced concrete.

AASHTO LRFD SPECIFICATIONS, FIRST (1994) AND SECOND (1999) EDITIONS

The working-stress design (WSD) philosophy, which requires that calculated design stresses not exceed specified levels, underwent adjustment in the 1970's through the introduction of load factors reflecting the variable predictabilities of different load types, a philosophy referred to as load factor design (LFD). During the period 1988 to 1993, the AASHTO LRFD Bridge Design Specifications was developed using statistically based probability methods. The load and resistance factor design (LRFD) philosophy makes use of load and resistance factors developed through statistical analyses.

The AASHTO LRFD Bridge Design Specifications, First (1994) and Second (1999) Editions, requires that each bridge component and connection satisfy all limit states in accordance with the relation

$$\eta \sum \gamma_i Q_i \leq \phi R_n \tag{8}$$

in which η is a factor related to a ductility factor η_D, a redundancy factor η_R, and an operational importance factor η_i in accordance with $h = \eta_D \eta_R \eta_i$, γ_i is a statistically-based load factor applied to force effect Q_i, and ϕ is a statistically-based resistance factor applied to the nominal resistance R_n. The numerical values to be used for these factors can be found in the LRFD Specifications (AASHTO LRFD, 1994 and 1999).

The value of Q_i for that value of *i* representing an extreme seismic event, designated as *EQ*, is found using the same procedure described above for Standard Specifications, Fifteenth (1992) and Sixteenth (1996) Editions.

An additional bridge classification, "Critical," has been added to the LRFD Specifications; and the number of substructure response modification factors *R*, have been increased to cover all three classifications, "Critical," "Essential," and "Other" as indicated in Table 3 below.

Table 3 - Response Modification Factors (*R*)

Substructure	Importance Category		
	Critical	Essential	Other
Wall-type piers-larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
♦ vertical piles only	1.5	2.0	3.0
♦ with batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
♦ vertical piles only	1.5	3.5	5.0
♦ with batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

DUAL STRATEGY OF SEISMIC DESIGN

The design of transportation structures to perform satisfactorily under expected seismic conditions requires that realistic earthquake loadings during their life times be specified and that the structural components be proportioned to resist these and other combined loadings within the limits of certain expected performance requirements. In regions of high seismicity, earthquake loading is often critical among the types of loading that must be considered because a great earthquake will usually cause greater stresses and deformations in the various critical components of a structure than will all other loadings combined; yet, the probability of such an earthquake occurring within the life of the structure is very low. On the other hand, a moderate earthquake is very likely to occur during the same period of time having the potential to produce damage unless controlled. Considering both types of earthquake, a dual-criteria strategy of two-level design is usually adopted for Ordinary Bridges as follows:

Functional Evaluation Earthquake (FEE) - A functional evaluation earthquake is defined as one, which has a relatively high probability of occurrence during the lifetime of a bridge structure. The structure should be proportioned to resist the intensity of ground motion produced by this event without significant damage to the basic system, thus allowing it to remain functional immediately following the FEE event.

Safety Evaluation Earthquake (SEE) - A safety evaluation earthquake is defined as the most severe event which can reasonably be expected to ever occur at the site. Because this earthquake has a very low probability of occurrence during the life of a bridge structure, significant structural damage is permitted; however, collapse and serious personal injury or loss of life should be avoided.

The challenge is to set seismic design criteria which will satisfy this dual-criteria strategy in a cost-effective manner.

Important bridges located on major, heavily traveled, routes, where no convenient alternative routes exist, are now being designated as LIFELINE BRIDGES. These bridges are expected to remain functional immediately following an SEE event; therefore, they must be proportioned to resist the intensity of this event without experiencing significant damage. Because of this specified high-level of performance during an SEE event, response under the FEE condition, as defined above, is relatively of minor concern.

CHARACTERIZATION OF SEISMIC GROUND MOTIONS

It is the authors' contention that at least one-half of the bridge earthquake engineer's overall problem, in either designing a new bridge or developing retrofit measures for an existing bridge, lies in establishing appropriate design ground motions which, along with other specified design criteria, will satisfy the dual-criteria design strategy described above. In the past, it has been common practice to represent the design ground motions using acceleration response spectra developed through statistical averaging of such spectra generated for families of recorded accelerograms representative of different site conditions. A deficiency of these spectra has been that they do not represent the same probability of exceedance, for a specified period of time, over the full spectral period (or frequency) range of interest. Further, the probability of exceedance of the spectral value at any specified period is not well known.

Because of these deficiencies, probabilistic risk assessment (PRA) methodologies have emerged having the objective of providing uniform hazard response spectra for a given site with each spectrum curve representing the same numerical probability of exceedance over the entire spectral period range of interest for a specified duration of time. Usually, these spectra are generated for the "rock-outcrop" condition at the site and then modified either through explicit site response analyses, using a computer program such as SHAKE, or by applying published site amplification factors.

Figure 1 shows a set of uniform hazard curves generated recently by Geomatrix Consultants which represents a single horizontal component of "rock-outcrop" motion at the base of a marl layer resting on hard rock at the site of the planned Cooper River Bridges in Charleston, South Carolina (Geomatrix Consultants, 2000).

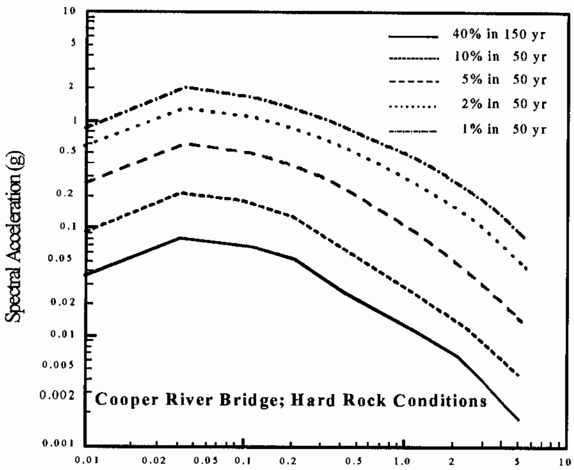

Fig. 1 Equal-hazard response spectra for hard rock (5% damped) (Source: Geomatrix Consultants, 1999)

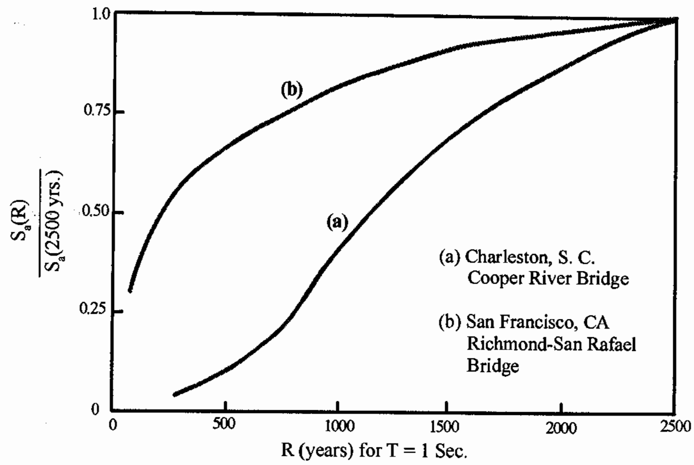

Fig. 2 Acceleration response spectral ratio $S_a(R)/S_a(2500 \text{ yrs.})$ vs. mean return period R

Once a set of uniform hazard response spectra have been generated for a specific site, a decision must be made as to which two probabilities of exceedance in a specified period of time (or mean return periods, T_R, in years) are the proper choices to represent the FEE and SEE events in satisfying the dual-criteria strategy of design previously discussed. The proper choices will, of course, depend upon the nature of the uniform hazard spectra generated. When considering a site in the eastern part of the U.S., one finds the uniform hazard spectral values for a mean return period equal to the life of a structure, say 150 years, will be very low in comparison with the corresponding values having a mean return period of say 2,500 years; see Fig. 1. On the other hand, when considering a site in the western part of the U.S., the 150-year mean-return-period spectral values will usually be high in comparison with and much closer to the corresponding 2,500-year values. To illustrate these differences, consider the acceleration response spectrum ratio $S_a(T_R)/S_a(2{,}500 \text{ years})$ for a fixed spectral period, say $T = 1.0$ sec where the response spectral ratios are plotted as a function of mean return period T_R in years for two specific sites: namely, the Charleston, S.C. Cooper River Bridge site and the Richmond-San Rafael Bridge site in the San Francisco Bay Area (Geomatrix Consultants, 1993; see Fig. 2).

If a mean return period, T_R, equal to 2,500 years should be specified to represent the SEE event in designing a bridge for the Cooper River Bridge site using the AASHTO response modification factors shown in Table 3, the structural response due to this event will be extremely high in comparison with the response of an *FEE* event having an assigned mean return period T_R equal to say 300 years. Therefore, in this case, the SEE event will totally control the design. On the other hand, in designing a bridge for the Richmond-San Rafael site using the same mean return periods for the two events and the same response modification factors applied to the SEE event, the FEE event for which limited damage is specified would most likely control the design since the ratio $S_a(300 \text{ years})/S_a(2{,}500 \text{ years})$ is approximately 0.6. The above comparisons show the importance of assessing the performance of a final bridge design under both the FEE and SEE conditions to insure that the specified dual performance criteria have been met.

In addition to setting "rock-outcrop" response spectra representing the FEE and SEE events at a specific site, corresponding response-spectrum-compatible time histories of motion are usually generated for use in the design of bridges expected to experience inelastic deformations. These motions should be obtained by modifying "rock-outcrop" recorded time histories using the time-domain procedure of adjustment, rather than the frequency-domain procedure, since it results in modified motions closely resembling the initial recorded motions (Lilhanand and Tseng, 1988). The recorded motions selected for modification should initially possess durations and peak ground accelerations, velocities, and displacements similar to those of the target design seismic event. For a near-field seismic event, e.g., within about 10 km of the site, the recorded motions selected should contain a definite velocity pulse or so-called "fling."

When generating components of motion to be used as seismic inputs at multiple-pier locations, they should reflect realistic spatial variations produced by wave-passage and wave-scattering effects and be response-spectrum-compatible. Thus, an appropriate wave-passage velocity (usually shown to have a speed in the range 2,000 to 3,000 m/sec by instrument-array recording data), an established set of coherency functions (Abrahamson et al, 1991) characterizing wave-scattering, and the initial response-spectrum-compatible time histories of motion should be used to generate the desired "coherency-compatible and response-spectrum-compatible" time histories of acceleration to be used as inputs at multiple-pier locations (Tseng and Penzien, 1999). Corresponding time histories of velocity and displacement should also be generated. The frequency-domain procedure should be used when interpolating motions at intermediate pier locations.

Standard site-response analyses using the above-described "rock-outcrop" motions as inputs to soil columns representing conditions at multiple-pier locations should be carried out to obtain corresponding soil motions at discrete elevations over the foundation depths. These motions are needed in order to allow an assessment of soil-foundation-structure interaction (SFSI) effects (Tseng and Penzien, 1999).

MODELLING AND DYNAMIC ANALYSIS

Over the years, finite element modelling and analysis, along with associated computer programs, have advanced greatly. Modelling can now include nonlinear elements such as nonlinear force-velocity and force-displacement hysteretic elements, and can account for nonlinear geometric effects. Much remains to be done, however, in defining elements which can realistically represent in three-dimensional forms the variety and mutual interaction of nonlinearities present in bridge structures, including those involved in soil-foundation interaction.

For determining low-level seismic response of complex or irregular bridge structures, the governing equations of motion used in evaluating seismically induced internal forces can be expressed in the linear time-domain matrix form

$$\begin{bmatrix} M_{ss} & M_{sf} \\ M_{sf}^T & \left[M_{ff}^s + \overline{M}_{ff}\right] \end{bmatrix} \begin{Bmatrix} \ddot{u}_s(t) \\ \ddot{u}_f(t) \end{Bmatrix} + \begin{bmatrix} C_{ss} & C_{sf} \\ C_{sf}^T & \left[C_{ff}^s + \overline{C}_{ff}\right] \end{bmatrix} \begin{Bmatrix} \dot{u}_s(t) \\ \dot{u}_f(t) \end{Bmatrix} + \begin{bmatrix} K_{ss} & K_{sf} \\ K_{sf}^T & \left[K_{ff}^s + \overline{K}_{ff}\right] \end{bmatrix} \begin{Bmatrix} u_s(t) \\ u_f(t) \end{Bmatrix} = \begin{Bmatrix} 0 \\ \left\{\overline{K}_{ff}\overline{u}_f(t) + \overline{C}_{ff}\dot{\overline{u}}_f(t) + \overline{M}_{ff}\ddot{\overline{u}}_f(t)\right\} \end{Bmatrix} \tag{9}$$

in which all of the M, K, and C letters denote mass, stiffness, and damping matrices, respectively, all u quantities denote time-dependent total-displacement vectors, subscript s denotes the number of DOF in the structure, excluding its f DOF located at the structure/foundation interface, and a bar placed above a letter indicates that the quantity applies to the f DOF of the foundations when isolated from the structure and subjected to the seismic free-field soil environment. For a bridge having multiple pile foundations, the structure/foundation interface is normally specified to be at the lower surface of each footing having six degrees of freedom (three translations and three rotations). Thus, the footing masses are included in the structural system; however, to satisfy pile-head boundary conditions, rigid massless footings are included in the isolated foundation system. The total displacement vector $\bar{u}_f(t)$ in Eq. (9), and corresponding velocity and acceleration vectors, $\bar{\dot{u}}_f(t)$ and $\bar{\ddot{u}}_f(t)$, respectively, represent motions in the f DOF of the isolated foundations. These motions have been referred to in the literature as the "scattered" foundation motions (Tseng and Penzien, 1999).

The foundation stiffness, $\bar{K}_{ff}$, damping, $\bar{C}_{ff}$, and mass, $\bar{M}_{ff}$, matrices in Eq. (9) which have constant coefficients represent, collectively, approximations of the complex frequency-dependent foundation impedance matrices. These approximations have been made to remove frequency-dependent parameters in the equations of motion, thus allowing a time-domain solution of the equations of motion. If the same equations of motion were expressed in the frequency domain, then such approximations would not be necessary as the complex frequency-dependent impedance functions are fully compatible with a frequency-domain solution. The damping and mass terms on the right-hand side of Eq. (9) usually have small effects on the solution; however, their importance should be checked.

The full viscous damping matrix on the left hand side of Eq. (9) is usually expressed in Rayleigh (mass and stiffness proportional) form with two proportionality constants being assigned numerical values to limit the modal damping ratios to levels within acceptable bounds over the range of frequencies dominating seismic response. It is authors' opinion that this Rayleigh form of damping is too simplistic and unrealistic, and changes are needed to improve such a Rayleigh form of damping used in dynamic modelling, and that structure-analysis computer programs should be changed accordingly.

The above equations of motion have been expressed in the time domain, as it is necessary to do so when nonlinearities in the super-structure system, i.e., structural system above the foundations, are represented. These nonlinearities usually occur in the form of hysteretic force-displacement relations of individual components, thus requiring that the linear forms represented in the third term on the left-hand side of Eq. (9) be changed to the appropriate nonlinear hysteretic forms. Special damping devices having nonlinear viscous properties will require modifications to the second term in this equation. Having established all nonlinear forms, the corresponding coupled equations of motion can be solved for total displacements $u_s(t)$ and $u_f(t)$ using step-by-step numerical integration procedures. The use of total, rather than relative, displacements is required to avoid superposition of solutions, which is invalid when treating nonlinear systems. To complete the dynamic analysis of the overall bridge system, the time histories in vector $u_f(t)$ must be applied as inputs to each isolated soil/foundation model in a separate "feed-back" analysis (Tseng and Penzien, 1999).

Since the low-level seismic response produced by the FEE event remains essentially elastic, the linear equations of motion, Eq. (9), can be used directly yielding reliable results, even for structures having very large numbers of DOF; however, when these equations are modified to represent the variety of nonlinear component behaviors occurring under SEE conditions, the predicted response results are much less reliable. These less reliable results are due primarily to the lack of realistic modelling of the nonlinear components under their three-dimensional time-dependent deformation conditions.

It should be realized that increasing the number of degrees-of-freedom in modelling a particular structure does not necessarily improve the accuracy and reliability of global response results obtained therefrom, especially when nonlinearities develop in the system. Often better predictions of global response can be obtained using wisely chosen generalized "super elements" resulting in fewer degrees-of-freedom.

ASSESSMENT OF SEISMIC PERFORMANCE

The procedure one should use in assessing seismic performance of a transportation structure depends upon (1) type of structure, regular or irregular, (2) level of seismic excitation, FEE or SEE, and (3) stage of the design process, preliminary or final. Because of lack of space in this paper, only the irregular structure will be discussed herein. The performance of a regular structure can be treated similarly; however, due to its simplicity, a less rigorous assessment of performance is usually adopted.

When assessing the SEE performance of an irregular structure, i.e., one having abrupt changes in mass, stiffness, and/or geometry, a separate linear response-spectrum modal analysis of a multi-degree-of-freedom (MDOF) finite element model is usually carried out first for each of three (x, y, and z) rigid-boundary inputs as defined by their corresponding acceleration response spectra. Maximum values of internal force components are evaluated and divided by their corresponding strength capacity values to establish force demand/capacity ratios for members of the initial design. Demand/capacity ratios greater than unity are, of course, fictitious since they cannot occur; however, they do provide an indication of where inelastic deformations are likely to occur first and, to a limited extent, some measure of the magnitudes of these inelastic deformations. The accuracy of this information depends very much on the amount of redundancy in the structural system. If the system is highly redundant, the distribution of internal forces will change each time an individual component undergoes inelastic deformation, which will continue until a collapse mechanism is reached. Nevertheless, the results of the linear response spectrum analysis will provide guidance toward making effective modifications to the initial design, leading to an improved (preliminary) design in terms of meeting the SEE performance criteria.

A seismic performance assessment of the preliminary design of an irregular structure under the SEE condition should focus primarily on evaluating global displacements and deformations in those individual components which experience inelastic deformations. A response-spectrum modal analysis, along with response modification factors, is invalid and should not be used at this stage of the design process. Rather, nonlinear modelling of the overall system, including foundations, should be established and nonlinear time-history analyses should be carried out to determine maximum values of component deformations, which can be compared with their corresponding deformation capacities. Deformation capacity of a component is defined as that deformation level at which the component's intended performance starts to exceed its acceptable level with increasing deformation.

In carrying out these nonlinear time-history analyses, simultaneous three-dimensional (x, y and z) response-spectrum-compatible time-histories of seismic input should be used, since superposition of separate solutions is no longer valid due to the nonlinear character of response. Further, for long, strongly coupled structures along its alignment, multiple-span segments of the total structure should be modelled; and, simultaneous three-component time histories of seismic input should be applied at each pier location. From pier-to-pier, these inputs should possess appropriate spatial characteristics reflecting realistic wave-passage, wave-scattering, and local site-response effects; and, as mentioned previously, if located in the near field to a controlling seismic source, each input should possess an appropriate velocity pulse (or fling). The critical nonlinear response of a transportation structure in such a location will most likely be dominated by its response to such velocity pulses.

In assessing the performance of a final design under the SEE condition, it is recommended that a minimum of three independent sets of three-component seismic inputs be applied to the nonlinear model separately and that the largest of the resulting maximum values of any critical response be used in assessing performance. This recommendation is made because of the large variations, which usually occur due to nonlinear effects, in critical response values.

When conducting a nonlinear time-history analysis as described above, approximations in the modelling of the structural elements are usually made, e.g., assuming idealized hysteretic force-displacement relations or, even simpler, using equivalent linear relations along with increased hysteretic damping. While these approximations are usually acceptable in conducting a demand analysis of the global system; they are not suitable for an assessment of member capacities. To evaluate such capacities, it is now standard practice to conduct inelastic static (pushover) analyses under controlled monotonic displacement and/or force conditions, noting the formation of inelastic deformations as they take place up to the point of maximum allowable performance. If the bridge structural system can be modelled adequately with only one independent degree-of-freedom, e.g., a transverse frame supporting a single deck, then the pushover analysis is straightforward. However, if the structure is irregular requiring more than one degree-of-freedom, it is difficult to perform a meaningful pushover analysis for the complete structure. The more independent degrees-of-freedom contributing to seismic response of the structure, the more difficult it is to perform a meaningful pushover analysis for the structure system. In such cases, pushover analyses for local elements and/or subsystems will be more meaningful. Special focus on improving capacity analyses is needed in order to improve one's ability to assess seismic performance.

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