SEISMIC EVALUATION AND RETROFITTING OF U.S. LONG-SPAN SUSPENSION BRIDGES

ASCE Subcommittee on Seismic Performance of Bridges

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ISSUES AND SOLUTIONS [1]

Subcommittee on Seismic Performance of Bridges [2]

Abstract

This paper is a work-in-progress. It is a first attempt to raise issues that require discussion on seismic evaluation and retrofitting of long-span suspension bridges in the United States. The objective is to initiate a dialogue on the issues, discuss the engineering problems and difficulties encountered in the evaluation process, and propose solutions to eliminate or reduce the vulnerabilities through retrofitting. The issues are both general and detail-specific. Most issues are applicable to short and medium-span bridges, also bridges of other types, including bridges in other countries. The issues deal with uncertainties, clarifications, and the need for additional information and research in such areas as: seismic hazards and risks; performance and design criteria; ground motions; geotechnical engineering, substructure mathematical modeling, and soil-structure interaction (SSI); actual conditions of structural components; superstructure mathematical modeling; ambient vibration testing; analysis of superstructure; suspension bridge component vulnerabilities; instrumentation and monitoring; laboratory testing; retrofitting; and the effects of limited funding and time constraints. One of the issues raised is whether seismic evaluations provide us with sufficient confidence that retrofitted long-span suspension bridges will perform as predicted? This paper is a compilation of issues submitted by subcommittee members. The issues will be updated after the scheduled panel discussion. Commentaries by the panelists are included. A report, which will include the updated issues and the proposed solutions, is planned.

1. Introduction

A long-span suspension bridge is defined as one where the length of the main span, the center to center distance of the towers, is $\geq$ 400 ft. (122 m). So far 57 long-span suspension bridges, varying in main span lengths from 400 ft. (126 m) to 4260 ft. (1299 m), and in age, as of 1995, from 7 to 141 years, have been identified as being in service in the U.S. (ASCE Subcommittee, 1995). Most of the 57 lifeline suspension bridges were designed and built in previous technological periods when our profession did not have the resources we have today. Many are aged and have seismic vulnerabilities that stem from -- deterioration, inherent weaknesses in components and details, and inadequate dynamic response characteristics. Seismic evaluation studies have been completed on a few of these bridges and are in progress on others. The retrofit on the Golden Gate Bridge is scheduled to start in late 1995.

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(1) Topic for a panel discussion at the 1996 Structures Congress in Chicago, IL. For information on panelists and commentaries, see sections 5.0 through 5.8.

(2) For list of participating subcommittee members, see section 3.

2. Issues

The issues are classified as follows:

2.1 Seismic Hazards and Risks
2.2 Seismic Performance Criteria
2.3 Design Criteria
2.4 Ground Motions
2.5 Geotechnical Engineering, Substructure Mathematical Modeling, and Soil-Structure Interaction
2.6 Actual Conditions of Structural Components
2.7 Superstructure Mathematical Modeling
2.8 Ambient Vibration Testing
2.9 Seismic Analysis of Superstructure
2.10 Suspension Bridge Component Vulnerabilities
2.11 Instrumentation and Monitoring of Full-Scale Bridges
2.12 Laboratory Testing
2.13 Retrofitting
2.14 Miscellaneous

2.1 Seismic Hazards and Risks

2.1.1 Seismic hazard for ground acceleration or shaking: the horizontal ground acceleration hazard is usually expressed as a percent probability of exceedance for the life-span of the bridge. The objective of the seismic ground motion hazard evaluation of a bridge is to define the characteristics of the earthquake shaking for different probabilities of exceedance or return periods for two different seismic hazard levels. For the low hazard level, typical return periods vary from 300-500 years for the functional evaluation earthquake (FEE) also known as the design earthquake, common moderate earthquake, or operating basis earthquake. For the high hazard level, typical return periods vary from 1000-2500 years for the maximum credible earthquake (MCE) also known as safety evaluation earthquake, safe-shutdown earthquake, structural safety earthquake, or rare large earthquake. The seismic hazard study starts with the identification of all the possible earthquake sources and the estimation of their probability of generating earthquakes of different magnitudes. Geological conditions that may affect the level and character of the ground motions, such as soft soil deposits or directivity effects, must be taken into consideration. The result of the seismic hazard evaluation is either a set of response spectra or time histories for longitudinal, transverse, and vertical components for different levels of damping and return periods. For long-span suspension bridges it is also important to include information about the spatial variability of the ground motion. The differential displacement as a function of distance is often used in design.

What are the probabilistic scenarios for peak "free field", zero period, ground accelerations? What are the probable rock ground motions at different response frequencies? Can best estimate curves for accelerations versus return period be derived for far and near sources at different epicentral depths? What parameters establish the selection of the design recurrence period? How is the importance of the bridge, the risk to safety and functioning of the bridge, and probable economic loss, factored in? Can an algorithm be established for the selection of a suitable recurrence period that relates importance of serviceability of the bridge as a transportation lifeline, vital highway artery, or the consequence of loss of service in terms of economic loss to the region?

2.1.2 Seismic hazard evaluation for the performance of members and major components of the bridge concerning their percent probability of exceedance for each of the following:

a. Limits of operating allowable elastic stress for the major components under the seismic provisions of the AASHTO code (dynamic elastic response).

b. Yield stress limits, inelastic buckling or compression limits, and permanent damage to major structural components. Under nonlinear dynamic response -- there may be limited, repairable, damage but the structure will remain open as a transportation lifeline.

c. Limits for ultimate stress, failure, or collapse limits of major structural components (failures will result in termination of service for an extended period).

d. Limit of the maximum bearing extension during travel.

Can procedures be established to decide what level of zero-period peak ground acceleration and the corresponding return period will satisfy the above limits and relate these to overall bridge performance?

2.1.3 Seismic lifeline risk: This risk must examine the importance of the bridge and its performance as a lifeline after the earthquake for power and water lines. Government agencies such as the Federal Emergency Management Agency (FEMA) should evaluate the bridge functionality for civil defense, police, fire, or medical emergencies. The agencies should also determine if the bridge carries a lifeline transportation route and whether it can remain functional after the MCE. Action should be taken to eliminate the risk of collapse, or provide alternate routes, if it is not feasible for the bridge to survive the shaking during the MCE.

2.1.4 Seismic risk to human life (injuries or fatalities): this risk must be evaluated by the owners. Highly traveled transportation routes over bridges in or adjacent to densely populated urban areas will have a high risk for the number of human injuries or fatalities that might occur if the earthquake strikes during peak hour traffic. The risk to human life for the MCE must be weighed against the everyday risk of probabilities of normal means of travel such as airplanes, automobiles, boats, and trains. The public should be informed through government, state, or public agency clarifications what risk is achievable or feasible within their budgeted programs. Our profession should lend assistance to the agencies in educating the public about the corresponding probable risks. Thresholds are needed to relate loss of human life with damage and deformation levels.

2.1.5 In the eastern and central U.S. what types of geological and seismological information are available for a reasonable estimate of seismic hazards? What degree of conservatism is built in for a specific seismic hazard evaluation?

2.2 <u>Seismic Performance Criteria</u>

2.2.1 The first step in a seismic retrofit project is the development of performance objectives for different levels of seismic hazard. This is a policy decision that the bridge owner must make considering the availability of funds, the importance of the bridge in the transportation network, the economic impact of the bridge closure, and the potential loss of life due to the bridge collapse. Existing codes such as AASHTO already provide the following performance criteria:

a. Small to moderate earthquakes should be resisted without significant damage.

b. Earthquakes with an 80 to 95% probability of non-exceedance in 50 years are used in the design process.

c. Large earthquakes should not cause collapse.

Long-span suspension bridges often provide a non-redundant transportation link and closure due to seismic damage can have a very serious economic impact. As a result, these bridges may need more stringent performance criteria such as serviceability after large earthquakes.

2.2.2 For important long-span bridges the fundamental policy issues incorporate the questions "for what level of earthquake ground motion shall the bridge remain functional?" and "what level of service is required after earthquakes as well as during repair construction?" An important ancillary question that must be addressed in setting policy is "what are the economic consequences to the region if the bridge were out of service after the MCE?" Modern engineering technology can provide retrofitting to enable the bridge to withstand a great earthquake with little loss of function; the primary limitations on the functional level maintained are the time and money required for analysis, plan development, and construction of the retrofit. Recognizing this, engineers can provide backup data and recommendations to assist bridge owners in making policy decisions.

2.2.3 In the case of the Golden Gate Bridge, the performance criteria for the retrofit of the bridge dictate that after the MCE of magnitude > 8, the bridge shall not be totally closed to the public for more than 24 hours after the earthquake and is expected to provide: access to emergency vehicles immediately after the earthquake, limited vehicular access within a few days after the earthquake, and full access within a month after the earthquake. Also, limited, repairable, damage to the bridge, consistent with the access requirements, is acceptable.

- **a.** How is the owner being assured that this performance criteria will be met considering that we have no knowledge of how long-span suspension bridges perform in MCE's?
- **b.** Which parts of the bridge are projected to be damaged?
- **c.** If similar criteria are specified for bridges on the east coast, how can owners be assured that access will be provided after the MCE when we are not sure of the reliability of the ground motions used in retrofitting?

2.2.4 In establishing performance criteria, should there be different philosophies for the eastern U.S. versus the west coast, and why?

2.3 Design Criteria

2.3.1 Design criteria must provide the technical guidelines to achieve the performance objectives. Project specific design criteria is needed for the seismic evaluation and retrofit of long-span suspension bridges because of the following coverage limitations in the current bridge codes:

- **a.** The codes do not apply to long-span bridges.
- **b.** The codes provide design methods for new structures that must be detailed to achieve ductile behavior. Existing long-span suspension bridges do not always meet detailing requirements such as transverse reinforcement in reinforced concrete or width/thickness (b/t) and slenderness (kl/r) ratios in steel members. Project specific design criteria must address the definition of strength and ductility of existing members and materials.
- **c.** The performance objectives for major transportation links may be more stringent than those envisioned in the codes.
- **d.** The codes do not have extensive provisions for retrofitting existing steel bridges.
- **e.** The seismic retrofit of long-span suspension bridges may include seismic devices, such as passive, active, or hybrid protective systems that are not specified in the current codes.
- **f.** The codes do not establish criteria for nonlinear dynamic analysis that is often used in long-span retrofit projects. In a nonlinear analysis, the seismic demand must be defined in terms of deformation, number of cycles, and residual strength.

2.3.2 Many long-span bridges are also exposed to critical non-seismic load conditions such as wind, ship collisions, ice flow, or scour. Sometimes, the design requirements for these extreme events are compatible, while in other cases they may be conflicting. Optimal design strategies should consider all hazards and not only earthquakes. Solutions to conflicting requirements that do not compromise the structure need to be found.

2.3.3 Is design for the MCE and the critical temperature a realistic condition?

2.3.4 The development of design criteria often needs support from research based on physical testing or detailed analysis of components. Coordination between bridge owners, design professionals, and universities in information sharing is very important.

2.3.5 Should the two level seismic design, (the elastic design level for the FEE and the inelastic design level for the MCE) be accepted as a standard evaluation procedure for all important suspension bridges?

2.3.6 If important bridges are expected to remain open after the MCE with limited, repairable, damage, is it necessary to evaluate these bridges for the FEE or is evaluation for the MCE adequate? Clarifications are needed as to what is meant by: a bridge remaining "open" after the MCE, and "limited, repairable, damage"?

2.3.7 Can the seismic ground motion accelerations at various periods (or frequencies) be established in terms of a percent probability of exceedance for the estimated life-span of the bridge? Some general resolution is needed about what percent probability of exceedance and what estimated life are appropriate? Now, FEE's have a 90% probability that they will not be exceeded in 50, 100, or 250 years. Should this probability be higher for important long-span bridges or is 90% adequate? Life-spans of bridges continue to increase. The oldest long-span suspension bridge in the U.S., the Wheeling Suspension Bridge (II) in West Virginia, was 141 years old in 1995. In determining ground motions, what useful life should be ascribed to such bridges? Should such bridges be assumed to last forever?

2.3.8 Deformations, ductilities, and displacements:

a. Deformation-based criteria are essential for a reliable evaluation of structural performance.
b. A bridge component can fail on its own (member failure), or it can fail with several adjacent members (local failure), or the whole structure can collapse (global failure). The consequences of these failure modes are quite different. Different ductilities can be used to represent these modes.
   Member failure mode ↔ Local failure mode ↔ Global failure mode.
   Ductility (member) ≥ Ductility (local) ≥ Ductility (global).
   The ductility of connections depends on load path, member ductilities, member strength, member stiffness, etc. How can the different ductilities be implemented in practice?
c. Should support displacements, similar to those experienced by one of the towers and the anchorages (3.3 ft. or 1 m longitudinal and 0.6 ft or 0.2 m lateral) of the Akashi-Kaikyo Bridge (main span = 6500 ft. or 1990 m) in the Hyogo-ken Nambu (Kobe) earthquake of January 17, 1995, be considered in seismic evaluation studies? Differential displacements and their associated stresses should be predicted in a time history analysis. What ground motion time histories and phasing for the orthogonal directions should be taken in the determination of the displacements?

2.4 Ground Motions

2.4.1 Input ground motions in the form of acceleration, velocity, and displacement time histories are usually required to perform nonlinear time history analyses. Actual seismic records or synthetic ground motions can be used. The specification of ground motion input greatly affects the computed seismic response of a large bridge. The development of input motions for use in analysis must be coordinated with the geotechnical and seismological aspects of a project. Specific issues identified as important include:

a. Two-dimensional versus three-dimensional ground motion descriptions.
b. Combination of multiple components for time history analysis and modal combination for response spectrum analysis. Is there need for rules such as the 30% combination?
c. How should vertical ground motion spectra be specified? What are the effects of vertical motion on bridge response? Vertical ground motions are currently characterized by a ratio of V/H = 2/3. Is this characterization valid for all period ranges, site conditions, distances, and tectonic environments?
d. Specification of ground acceleration versus ground displacement.
e. Near field effects, particularly pulses, on structural response.
f. Specification of spatial variation of ground motion (due to wave propagation, source effects or geological conditions); quantification of effects on structural response of system and components.
g. How many time histories and which time histories should be used for nonlinear analysis of a structure?
h. Compatibility with the design spectra.

2.4.2 Should the effects of soil-structure interaction be considered in the determination of ground motions?

2.4.3 If sufficient seismological information is available for a bridge in the vicinity of a seismic fault, then input ground motions in the form of acceleration, velocity, and displacement time histories can be synthesized considering the generation of seismic waves from the rupture of an extended seismic source, the subsequent propagation of the waves through the layered earth, and their eventual arrival on the ground surface. For such a synthesis of ground motion time histories, detailed seismological information must be available to characterize the extended seismic fault (dimensions, location, orientation, rupture pattern) and the earth (number and thickness of layers, P- and S-wave velocities, density, attenuation). Such a description of ground motion accounts for near-field effects due to the rupture of an extended seismic fault in the vicinity of the bridge. All three components of ground motion (two horizontal and one vertical) should be computed at prescribed locations on the ground surface, along with the permanent ground deformation (if there is any considerable amount as with the Akashi-Kaikyo Bridge during the Kobe earthquake -- see section 2.3.8 c). Such models are usually restricted to a frequency content between 0 Hz and approximately 3 Hz. Although these models are capable of describing the very low frequency content, special care has to be taken to describe the frequency content above 3 Hz and to consider the local topography and soil conditions.

When only limited seismological information is available, or when a more design-based approach is desired, input acceleration, velocity, and displacement time histories can be generated using a probabilistic method, compatible with prescribed response spectra, to have a prescribed duration of strong ground motion, and to reflect a prescribed velocity of wave propagation and a prescribed coherency law. Such a methodology is much cheaper computationally than the seismologically based approach described above -- time histories can be generated at several locations on the ground surface in less than a minute on a personal computer.

2.4.4 Can the ground motion spectrum be deconvoluted into orthogonal simulated time histories that will cover the necessary frequency content ranges for the strong earthquakes that might occur?

2.4.5 Wave passage and spatial incoherencies:

a. Information on the modeling of the ground motion input to represent the primary, secondary, and surface wave propagations would be useful to practitioners. How good is our present technology in predicting the actual responses to these wave passages and their free field arrival times?
b. Guidance is needed for estimating the apparent velocities controlling the wave passage and site effects for different geologic site conditions, especially in the presence of low-velocity sediments.
c. Can the phasing of the different earthquake waves be modeled in the response analysis? Does random vibration theory analysis provide a better method for predicting the ground motion excitations?
d. Can probable wave passage including reflections and refractions be considered and characterized to predict their spatial variations at the multiple supports?
e. Can spatial incoherencies for the vibrational frequencies and the separation distances be mathematically expressed as complex variables and entered as input in the computer modeling? Some guidelines should be established for practitioners.

2.4.6 How should directionality questions pertaining to spatial variation of ground motions be addressed in the eastern U.S. without specific sources in mind?

2.4.7 There is an inconsistency between the AASHTO spectrum and the site-specific spectrum on the east coast based on a 500-year return period. Was the AASHTO spectrum developed based on the historical earthquake records of the west coast? Recent studies (scarce data) show higher response for short and lower for long period components on the east coast. Should AASHTO provide an appropriate baseline rock spectra for the east coast and the central U.S.?

2.4.8 Long-span bridges:

a. Ground motions are usually considered to be characterized by their amplitude, frequency components, and duration. For the analysis of most elastic structures, with periods between 0.3 and 2 seconds, the amplitude and frequency components are sufficient to represent an earthquake. However, for long-span bridges, since the fundamental vibration period (first mode) could be from 6 to 10 seconds, the earthquake duration becomes very important. This effect is especially significant for some regions where a large amount of the long period energy will be developed in a relatively short time in case of an earthquake. Does the ground motion spectrum adequately encompass the MCE's in the longer period ranges for longer duration earthquakes? Can we establish criteria that will provide our bridges with safeguards against failure or serious damage for the MCE's?

b. Long-span bridges are sensitive to long-period ground motions, but traditional ground motion recordings often poorly resolve long-period motions. There is a need to determine how to extract long-period ground motion information from traditional and alternate seismic ground motion recordings and how to fill missing observational information by simulated ground motions.

c. Now, structures are designed for the three translation components of acceleration. Should the design of long-span bridges also include the effect of the rotational components? When should rotational components be considered?

d. A unified ground motion study was done for all long-span bridges in the San Francisco Bay area. Should similar studies be done for areas with a cluster of long-span bridges such as the New York City metropolitan area, or will individual studies suffice? A unified study requires cooperation among all the bridge owners in the region.

2.4.9 What is the reliability of the data, obtained from seismographic stations, used in predicting ground motions?

2.5 Geotechnical Engineering, Substructure Mathematical Modeling, and Soil-Structure Interaction (SSI)

2.5.1 Liquefaction: The process of liquefaction involves the build up of pore pressures (and consequent reduction of effective pressure) in fine grain saturated soils in response to earthquake induced shear deformation. Since this is a progressive phenomenon, there must be a progressive decrease in the vertical capacity of friction piles as the effective pressure is reduced. Current liquefaction analyses are typically a "go/no-go" type of determination; i.e., either liquefaction is likely or it is not likely to occur. There arise the following questions:

a. Are prediction methods adequate or can they be refined? Can we refine for design-time histories?
b. Are there documented vertical capacity failures of friction piles in response to incipient liquefaction? What is the effect on friction capacity of piles?
c. What is the time relation between the onset of liquefaction and the peak dynamic response effects?
d. Do the liquefied soils exhibit a reliable residual lateral strength?
e. What is the effect of the liquefied layer on the lateral resistance of soil layers above the water table?
f. What are the dynamic lateral pressure distributions on piers and abutments?
g. Are there adequate design rules for mitigation techniques such as densification, stone columns, or displacement piles?
h. What is the state-of-the-art with regard to quantifying the liquefaction potential? What is the degree of confidence in preventing liquefaction failure during the credible earthquake? Is mitigation feasible for liquefaction potential?
i. Do different criteria, viz. standard penetration tests, relative density, and particle size distribution yield unified predictions for this potential hazard?
j. Are there dynamic computer programs that can calculate the soil pore pressures and relate these to the intergranular overburden pressures for a better prediction of this hazard?

2.5.2 Pile foundations:

a. The modeling of pile groups for stiffness and strength involves two major issues that are not clearly addressed in the codes. The first is the group effect for large pile groups. There is conflicting advice on actual group effects for dynamic stiffness and foundation strength and the advice is not supported by specific computational or empirical evaluations. In addition, there is virtually no information on the characterization of stiffness and capacity of large pile groups with closely spaced piles. The second issue is the characterization of nonlinear response of pile foundations under seismic load. This includes the nonlinear SSI and uplift response of pile groups after local "yielding" (or plunging) of the most heavily loaded piles, and the residual displacement of the pile group after multiple stress reversals. A related issue is the development of guidance on how nonlinear behavior of the pile group stiffness should be included in structural analysis.

b. For suspension bridges that have pile foundations, how can the interaction impedances and compliances be modeled? What parameters will determine the stiffness and damping properties of the piles and the surrounding soil structure for both normal and frictional dynamic forces? What connectivity modeling is appropriate to obtain a good prediction of the response for the MCE?

c. For bearing pile foundations that are driven to bedrock, should the pile-soil interaction be modeled using visco-elastic layers for the overlying soil and then a half space model for the bedrock boundary?

d. Can foundations with batter piles in existing suspension bridges be modeled to represent their higher lateral stiffnesses and predict the dynamic batter pile forces? If the ultimate capacities of the batter piles are exceeded, can the responses be determined?

2.5.3 Large foundations:

a. Approaches that differ substantially in methodology and degree of sophistication or complexity can potentially be used to model SSI for large caisson foundations. There is a lack of guidance to the practitioner regarding which approaches are appropriate for different situations. Highly nonlinear aspects of SSI are usually not modeled (e.g., soil yielding, gapping, and caisson uplift). In addition, it is not clear when kinematic interaction is significant. Guidance is needed on the importance of these effects and, if important, how to model them?

b. Are current foundation stiffness modeling techniques applicable to bridge foundations with large caissons/piers? If some of these techniques are applicable, which ones are recommended?

c. If a massive embedded foundation analysis approach is used, can an improved prediction of the response be achieved?

2.5.4 For anchorage and pier foundations where soil layers overlay the bedrock, soil attenuations may radically alter the responses. The degree of attenuation is a function of wave propagations through the rock and soil layers, and the resonances and frequency filtering that occurs. Can probable soil attenuations be integrated into the ground motion forcing functions to obtain a conservative prediction of the response?

2.5.5 To what degree can specific topography and geological conditions be considered for evaluating the bridge's response to seismic excitations?

2.5.6 Substructure mathematical modeling:

a. The dynamic response of the bridge at its foundations will depend on the SSI. Two key dynamic soil parameters are the shear modulus and damping. For strong motion earthquakes, the stress-strains will be nonlinear. Two fundamentally different types of damping phenomena occur in the soil or rock during strong motion excitations. These are material and radiation dampings. Material damping will be a measure of the energy loss from the soil hysteresis. For material damping, various laboratory and in-situ field tests should be used to establish the shear moduli. Can the field tests determine the effective modulus through shear wave velocity measurements? Radiation damping will be a measure of the energy dissipation of the waves propagating geometrically away from the foundations and out through the soil medium. In the past, half-space theory has been used to provide estimates of the magnitude of this radiation damping. However, a limitation to this theory is that it does not consider the reflections or refractions that can occur at harder soil layers or bedrock and with different angles of surface inclination at the layer interfaces. Radiation damping will be dependent on the dimensions of the foundations and on the equivalent radius of the soil contact area. Some past studies have shown that for horizontal and vertical translational motions, the radiation damping is in the order of 10%

or greater of critical damping. For rocking and twisting motions, past studies have shown that the radiation damping is quite small and in the order of 2% of critical damping. What information or tests can be used to establish a quantitative prediction for this radiation damping?

b. The effective soil springs for normal and frictional forces will be dependent on the frequency content of the earthquake spectrum. Can guidelines be established for the techniques of modeling the soil interface impedances at the foundations of the bridge's anchorages and piers?

c. How well can we consider the nonlinear response of the soil, and its amplifications, local foundation stability, and liquefaction potentials? What is our degree of confidence in predicting the upper bounds?

2.5.7 Soil-Structure Interaction (SSI):

a. SSI is such a fundamental problem that a study that does not include SSI provides meaningless results. Today, long-span bridges are analyzed by assuming that the ground motion is the one obtained from field records. Sophisticated models try to provide schemes of spatial and temporal variability of the ground motion. However, in the determination of such input motion, the presence of the heavy bridge superstructure is completely neglected. The ground motion with or without the bridge structures is considered, erroneously, identical. This is typical of studies that neglect the soil-structure interaction phenomenon. In addition, neglecting the interaction between the foundations and surrounding soil greatly alters the results. Results show that for the Vincent-Thomas Bridge in the Los Angeles basin, SSI greatly affects the bridge vibrations. When SSI is included, there are increments in the deck displacements about 250-300% with respect to the case of no SSI. What are the proper methods of analysis (linear versus nonlinear, finite element method versus boundary element method) to study this problem?

b. Sometimes, a soil-foundation model is used to determine the seismic motions at the base of the superstructure and another model is used to analyze the superstructure using the results from the first model. Is such an approach, which neglects the presence of the bridge in the first model or uses a simplified model of the bridge, reliable or should only one model incorporating the soil, foundations, and the superstructure be used? Proper methods on SSI should consider the entire soil-foundation-superstructure system. The entire system could be decomposed into two subsystems (soil-foundation and superstructure), but the analysis of each subsystem should be conducted by accounting for the presence of the other subsystem. For example, the equations of motion of the soil-foundation subsystem can be derived by considering both the forces on the foundation from the surrounding soil and the forces transmitted from the superstructure. These forces will depend on the motion of the foundation itself and on the forces transmitted to the superstructure. In this way it is possible to couple the responses of the two subsystems.

c. What are the practical means for obtaining reliable soil moduli and damping properties needed for SSI?

d. How are the modeling parameters for SSI arrived at? What are the parameters and assumptions used in establishing spring constants to represent the soil as nonlinear springs for normal and shear forces?

2.6 Actual Conditions of Structural Components

2.6.1 It is rare to find early bridges that actually resemble the original drawings. Problems associated with determination of the actual "as-built" conditions of the existing bridge include:

a. Locating the existing drawings.

b. Determining exactly what materials were actually used in construction (since old material descriptions rarely trace directly to existing standards).

c. Obtaining drawings for changes made to the bridge over time.

d. Verifying various "as-built" dimensions on the bridge to the drawings.

e. Finding specific details of the exact materials used to construct the bridge piers and assigning a strength value to them.

2.6.2 Before proceeding with the analysis, the "actual" condition of the bridge should be determined, otherwise the results may be meaningless. Problems associated with a proper definition of structural models for bridge structures are particularly important for existing bridges. In fact, while, for a new bridge, the assumption that the "real" structure is identical to the one obtained by the blueprints can be accepted, for an existing and aged structure such an assumption is not valid. In fact, damage caused over the life of structures by natural and man-made events, environmental loads (earthquakes, wind, and traffic) that have produced structural deterioration, and so on, cannot be detected by looking at the blueprints. The status of steel members, with corrosion and fatigue damage, cannot be determined by looking at drawings. Problems associated with corrosion are sometimes hidden behind paint or they are not in accessible locations (i.e., inside bridge cables and anchorages). Nondestructive testing techniques can be used to determine the real status of the bridge and to provide more realistic models of the bridge superstructure. Only after such an analysis has been performed and the real structure has been identified, will it make sense to study a retrofitting program for the structure. What are the nondestructive testing techniques available for existing structures and are they suitable for long-span bridges?

2.7 <u>Superstructure Mathematical Modeling</u>

2.7.1 Suspension bridges:

a. What key points should be considered in the modeling of 2-D and 3-D suspension bridges?

b. Are any contact type finite elements available that model the friction between the cables and the saddles on the towers of a suspension bridge?

c. How should the effective masses, mass moments of inertia, and damping of the anchorages and piers be modeled?

d. How well are the dead loads and mass distributions known? Are these within the generally accepted 5% variance? Many suspension bridges have undergone extensive changes in their superimposed dead loads due to reconstructions, or replacement of deck and/or wearing course. Have these bridges been recently surveyed to verify the dead load state and geometry against the original as-built plans or contract drawings?

e. Is the bridge totally symmetrical? Many suspension bridges are not symmetrical from anchorage to anchorage. Noted examples in the United States are the George Washington, the Tacoma Narrows, the San Francisco-Oakland Bay, and the Golden Gate bridges. Older existing suspension bridges may not be symmetrical due to settlements of the anchorages and piers, tower and superstructure erection tolerances, and asymmetries -- unsymmetrical pedestrian walkways and unsymmetrical roadway or track profiles. Many suspension bridges have very dissimilar approach span configurations. Unless these approach span framings are isolated from force transmission to their anchorages, then both the seismic loads and approach spans' stiffness compliances will constitute asymmetrical conditions for the dynamic response due to the earthquake excitations. Are these asymmetrical conditions reflected in the modeling? Is the retrofitted reconstruction modeled in the computer response analysis?

f. Laboratory tests show loss of ductility and reduced fatigue strength in existing wires of main cables. Examination of fractured surfaces shows the presence of corrosion assisted cracking and/or hydrogen embrittlement. How are these conditions, including the rate of wire deterioration, accounted for in the modeling?

g. If the suspender ropes have significant reductions in tension, or go slack in some locations under the action of the upward inertial forces, can this be considered in a nonlinear time history analyses by automatic adjustment of the stress stiffening (or incorporation of zero compression suspender elements)?

h. What assumptions are made with respect to the main towers and their dead load precompression? Are the precompression softening and P-delta effects considered in the tower stiffnesses? What are the assumptions on the tower cross bracing modeling? The existing dead load stresses will be dependent on the actual erection sequences that were used in the building of the bridge. A lack of availability of recorded information to track erection procedures and the construction history may pose limitations to the accuracy of analytical predictions of the response.

i. Is the deck modeled to participate for transverse and longitudinal deformations and shears due to transverse, torsional, and longitudinal inertial forces from the earthquake? What are the assumptions concerning the deck joints in the effectiveness of the deck system participations? Can the deck be effectively modeled using finite plate elements with orthotropic material properties and special connection elements at the expansion joints?

j. At the interfaces between anchorages, towers, and superstructure framing, suspension bridges are generally designed to allow free longitudinal and vertical translation and free rotation about the transverse and vertical axes. During a strong motion earthquake, sizeable transverse forces and torsional couples will occur due to the dynamic inertial forces and these will accumulate at the anchorage and tower supports. The percentage of these accumulations equilibrated at the superstructure level will depend on the relative stiffness and capacities of the framing and at the detailed boundary conditions at these supports. For many suspension bridges, the accumulated torques must be taken out solely by the vertical hangers or rockers at the support interfaces, and this will involve a transformation of the imposed forces from horizontal couples to vertical couples at the ends of the superstructure framing. The transverse forces will be reacted at the tongue and wind lock bearings at the centerline of the bridge, at the transverse tower faces, and at the anchorages. Can the analysis make a good determination whether there are adequate capacities in the end framings of the superstructure and at the support interfaces to transfer the maximum dynamic forces during the MCE? If elastic capacities or bearing limitations are exceededat these locations, can an inelastic damage analysis be used to evaluate the responses? Can theretrofitted bearings, bumpers, and dampers be effectively modeled in the computer analysis topredict the new improved response of the bridge?

k. Are the member end joint eccentricities and offsets at superstructure framing and at the supports modeled to reflect the actual joint equilibriums? These are particularly important at the laterals and where member depths are significant. Are connection lengths, depths, and stiffnesses modeled to reflect the actual joint deformations in the bridge? How are the shear, bending, and torsional properties of latticed members calculated?

2.7.2 The capacity of existing bridges, designed to resist wind loads, is sometimes very low to resist earthquakes. What logical steps are followed in establishing a model that incorporates needed retrofits where inherent seismic weaknesses exist? Can a trial and error process, which starts with an elastic analysis and initial engineering judgements for the modifications be first used and then followed up with subsequent inelastic analyses of an effectively retrofitted bridge? Such a process requires close cooperation between the analyst and the structural designer.

2.7.3 For the MCE when the plastic domains of the steel material are reached, how will local members or components whose elastic capacities are exceeded be handled? Are computer programs available, which can easily model the elasto-plastic joint behavior with nonlinear member properties in the response analysis? What are the acceleration levels and recurrence periods that will cause local member yields or failure?

2.7.4 Little capacity to resist rotation is usually built in for connections of most bridges in the eastern U.S. These connections will behave as semi-rigid connections during low to moderate earthquakes. How should such connections be modeled?

2.7.5 How can unreinforced masonry piers or underreinforced concrete bents be modeled and analyzed for the condition when their elastic tensile and shear capacities have been exceeded during the hysteresis that can occur?

2.7.6 Guidelines are needed for modeling isolators, shock absorbers, and damping devices.

2.8 Ambient Vibration Testing

2.8.1 Ambient vibration testing is used to validate a computer model at the service level when the materials are elastic. However, even with good agreement between the observed and computed mode shapes and frequencies at low strains, it has been difficult to extrapolate to higher strains. Can an ambient vibration survey of the suspension bridge be effectively scaled to predict actual responses, stiffnesses, and damping during a strong motion earthquake especially in the presence of brittle components such as unreinforced masonry/underreinforced concrete components such as piers, walls, and towers?

2.8.2 What should be the minimum percent agreement between results of computer analysis and ambient vibration testing before the computer model is regarded as acceptable?

2.9 Seismic Analysis of Superstructure

2.9.1 Seismic analysis is an important step in the evaluation and retrofit of long-span bridges. The seismic evaluations of bridges in the San Francisco Bay area, with extensive alignments and approaches supported on widely varying soil profiles, presented a considerable challenge in terms of analytic complexity. Issues that are central to these evaluations include:

i. variability of soil profile and ground motions along the alignments,
ii. large superstructure and number of piers,
iii. multiple-support seismic excitation,
iv. soil-structure interaction,
v. nonlinearities due to expansion hinges, plastic hinges, and P-delta effects,
vi. horizontal and vertical curvature, and
vii. analytic model size.

For such bridges, the evaluations showed that:

a. Structural response is sensitive to multi-support excitation and expansion joint nonlinearities.
b. Nonlinearities due to expansion hinges and joints, plastic hinges for the concrete elements, and P-delta effects were found to be significant.
c. Behavior cannot effectively be captured by the modal superposition method, since an unmanageably large number of modes are required.
d. There is significant ground motion variability (frequency content and intensity) along the alignment.
e. The overall effect of multiple-support time history analysis has been found to result in significant reductions in demand, compared to estimates obtained by linear uniform base excitation analysis.
f. Compared to the wave passage effect, incoherency of the ground motions appears to have little effect on structural response.

2.9.2 Some peculiarities of the dynamic analysis of long-span suspension bridges are:

i. tension stiffening effect -- the stiffness depends on the initial stresses due to dead load,
ii. multiple-support excitation due to long spans,
iii. dynamic characteristics of long period structures for which higher mode response is important, and
iv. nonlinear behavior due to yielding, buckling, uplift, rocking, or impact.

Nonlinear time history analysis is the only analysis method that can address all these issues but it must be supported by modal analysis to gain a good understanding of the structural behavior. The use of nonlinear time history analyses in the design process presents new problems to the bridge engineer such as:

a. Need for multiple analyses with different ground motions.
b. Evaluation and analytical modeling of damping. The results may be very sensitive to the assumed viscous damping and there is little information about actual values. Most computer programs use Rayleigh damping which provides a non-uniform damping ratio for different vibration modes.
c.. The nonlinear modeling of components requires sensitivity studies because there is uncertainty in their behavior.
d. The algorithms for numerical integration of the equations of motion require more parameters than response spectrum analyses, such as time step, amount of numerical damping, and convergence

tolerances.

e. The analysis process becomes time consuming and very large amounts of data are generated.

2.9.3 In the analysis of the Golden Gate Bridge, with the foundations supported on rock, effects of multiple-support excitations (including wave passage, extended source, and ray path coherency) were found to be small when compared with analysis using rigid base excitations.

a. Are comparisons available for other suspension bridges between the effects of multiple-support excitations and rigid base excitations?

b. When are multiple-support excitations more important than rigid base excitations?

2.9.4 Seismic analyses and evaluations greatly depend on developing and verifying computer models. A complete 3-D model of the bridge will be necessary to accurately predict the response of the bridge to seismic loading. While developing 3-D models it should be realized that the requirements of seismic modeling are different from those of static modeling. In static analysis, it is common practice to isolate the segment of the bridge due to the presence of expansion joints. A refined model for this segment needs to be developed for fatigue analysis and load rating. In seismic analysis it is not always possible to isolate the bridge at expansion joints because forces in transverse direction can be transferred across the joint. Furthermore, it is not necessary to have a very refined model of the bridge superstructure in seismic analysis. Thus, requirements for static analysis may be conflicting with those for seismic analysis and if possible, two separate models should be developed. However, developing separate models for static and seismic analysis could be expensive and in such cases concepts of dynamic degrees of freedom and mass lumping can be very useful for dynamic analysis. Judgements about what percentage of the live load should be used for the dynamic masses must be agreed on.

It is common practice to perform dead load analysis using 3-D models before performing seismic analysis. This serves as a good check of the model. Dead load analysis by itself is not sufficient for model verification. This is because seismic load path will be different than dead load path. It is desirable to apply static loads in lateral directions and study their load path from superstructure to substructure. Often, a structure may have rigid body modes due to the presence of expansion joints (suspended spans in case of truss bridges). Frequency analysis show that very small frequencies (high period) can excite the longitudinal vibrations of the suspended spans.

One important reason for nonlinear behavior of a bridge during actual earthquakes is opening and closing of expansion joints. In long-span continuous truss or girder bridges, opening and closing of expansion joints can significantly change the forces acting on the substructure. Multi-mode spectral method as recommended by AASHTO cannot account for this kind of behavior. Nonlinear time history analysis is the best way to study this effect. However, time history can become a time consuming and expensive method of analysis. Another alternative is to perform multi-mode spectral analysis according to AASHTO specifications with different combinations of opening and closing of expansion joints. The joints should be closed to maximize the force on columns in a particular bent. Typically, three to four different cases of opening/closing of joints would be sufficient to get the maximum forces that can occur in the substructure. This study also provides insight on changes in load transfer with changes in joint status and to potential retrofit options.

2.9.5 2-D versus 3-D analysis, software packages, and validation:

a. To understand the behavior of long-span bridges, should one do only a 3-D nonlinear analysis, or should one do a series of analyses: 2-D linear, 2-D nonlinear, 3-D linear, and finally 3-D nonlinear?

b. Under what conditions would a 2-D analysis be acceptable for initial investigations?

c. Price/performance comparisons for linear and nonlinear analyses: Performing linear analyses can be efficient and inexpensive. However, the results from such analyses can be incorrect and misleading. Nonlinear analysis is still considered expensive and time consuming, but the continued advancement of computer technologies has significantly lowered the price differential between the two analysis types. Currently, some difficulty exists in choosing the best techniques from the many available programs. Experience and cost data from the different approaches taken by analysts would assist the earthquake engineering community in choosing the most cost efficient and reliable techniques. The National Information Service for Earthquake Engineering (NISEE) at the University of California, Berkeley, provides a first step in this direction with their Computer Software for Earthquake Engineering handbook.

i. Which of the commercial software packages have all the capabilities to do a 3-D nonlinear analysis using multiple support excitations?

ii. Which of the packages offered by NISEE have similar capabilities?

iii. Is data available on the price-performance ratios for the commercial packages?

d. When transient methods are used, a debate usually occurs with respect to the overall level of the transient (acceleration, velocity, displacement), in addition to how the transient is altered by soil conditions under the bridge. Murphy's law usually prevails, with one end of the bridge being located on dramatically different soil than the other end. This provides two different transients, which ideally should be simultaneously applied to their respective end of the bridge. Usually, the software program selected for the project cannot handle twin transients.

e. Computer models and analyses are becoming very complex. What techniques should be used for validation of the results besides ambient vibration testing? How reliable are hand calculations in verifying results of complex computer analyses?

f. Understanding the results of linear and nonlinear analysis: This key idea should continue to be emphasized when training new engineers. By its nature, linear analysis cannot account for material plasticity nor load redistribution. Earthquakes induce failure and plastic behavior. Analysts need to understand that linear analysis helps to verify design criteria rules that are behavioral simplifications and, sometimes, compromises reached by committee. However, linear analysis can assist in directing the efforts of more realistic nonlinear analysis for earthquake evaluation and thus help choose the appropriate solution method.

2.9.6 Do direct time step integration analytical solutions that have wave propagation forcing functions yield better reliable results for strong motion earthquake response, than obtaining this by a response spectrum analysis which will involve linear superpositions of the vibrational modal participations from the eigenvalue solution of the bridge model?

2.9.7 Fuzzy sets have been shown to provide useful relationships to bound uncertain response predictions particularly when the structural response is a highly nonlinear function with many uncertain model parameters. Can fuzzy set theory offer an alternative to random variable theory in representing the uncertainties of the response for strong motion earthquakes?

2.9.8 Vibration modes:

a. How many modes of vibration should be considered in the seismic analysis of a long-span suspension bridge?

b. How is a determination made about which modes will be important for the maximum overall and local responses? Can this determination be made by using output such as the maximum participation factors or generalized forces from the eigenvalue solution, and then using only those modes that are the major contributors to earthquake energies in the range of the bridge's excitation frequencies viz. 0.05 to 25 Hz?

c. What are the important higher mode excitations above 5 Hz? Are these uniquely excited local modes, (such as at the towers or cables) well understood? Modes should be generally classified as longitudinal, transverse, vertical, and torsional and have subclassifications according to symmetry in order that the responses can be categorized for overall behavior. The subclassifications can be either symmetrical or antisymmetrical about the longitudinal, vertical, and transverse axes of the bridge. How useful are generalizations for the modal behavior of suspension bridges with regard to understanding the unique response of a particular bridge?

d. How important is the coupling of the closely spaced modes, for the prediction of the response, for strong motion earthquakes of longer duration?

2.9.9 Damping:

a. Long-span bridges constructed from concrete, riveted steel, or welded steel, have inherent internal damping usually taken as 5% for computer modeling using Rayleigh damping. Selection of the damping value requires selecting two different periods of vibration. However, the damping value for different materials has never been measured for large displacements in real bridges.

b. How should damping be modeled in a seismic analysis? The need for nonlinear seismic analysis of suspension bridges is generally acknowledged. The results should be obtained by numerical integration in the time domain and this implies the necessity of defining a damping matrix. As pointed out in section 2.9.2 b, "The results may be ---------- different vibration modes." Rayleigh damping assumes that the damping matrix is linear with the stiffness matrix and the mass matrix and is given by: $[C] = \alpha [M] + \beta [K]$. Coefficients α and β should be adjusted to give the desired damping in two separate modes. But what are the "separate modes" and what is the "desired damping"? It is reasonable to think that the damping in an existing bridge can be experimentally determined for most vibration modes. But what confidence should be given to the experimental data if they are going to be applied to a large amplitude analysis? Also, the earthquake is supposed to mobilize high frequency modes whereas most of the published damping data are for the first few vibration modes. Although assuming Rayleigh damping can be a useful tool of analysis, there is no theoretical reason to include the geometrical stiffness matrix in this equation -- in this case damping is no longer classical damping.

c. Can the modal damping matrix be assumed to be predominantly diagonal? What damping ratios should be assumed for the different types of vibrational modes in the superstructure and towers? Have these modal damping values been verified under strong motion excitations? For suspension bridges, with the coupling of modes, when there are many closely spaced excited modes, the off-diagonal terms may be of the same order as the diagonal terms. What coefficients for these off-diagonal terms of the modal damping matrix can be used to obtain an accurate prediction of the response? Can reliable conservative predictions be made for the damping with coupled modes?

2.10 <u>Suspension Bridge Component Vulnerabilities</u>

2.10.1 The nature of long-span suspension bridges is such that the flexible cables can accommodate large motions between rigid multiple supports that do not undergo relative movements. The superstructures of the suspended spans act essentially as pendulums hanging from the main cables and their stability is achieved through the dead load prestress in the cables and suspenders that provide active restoring forces for load changes. In a strong motion earthquake, suspension bridges may be dynamically subjected, to both relative displacements between their multiple supports, and to relative motions of the supports themselves from the large inertial forces. These will cause large nonlinear displacements and geometries that are quite different from the normal design criteria. Therefore, in the dynamic response of a suspension bridge, the behavior of the more flexible primary members must be investigated very carefully. Specific questions that are unique to seismic response in suspension bridges are the following:

a. Can slippage of the main cable bands occur under the larger dynamic motions? Can the socketed connections withstand the dynamic out-of-plane oscillations for longer duration seismic events?

b. Will the suspender ropes have significant reductions in tension or go slack in some locations under the action of the upward inertial forces? If some suspenders go slack, what are the consequences in the dynamic response if this happens?

c. At the tower tops, can large longitudinal earthquake motions cause large additional main cable tensions that will result in high friction forces as these cables tend to be dragged over the saddle bearings? The dynamic response forces from the earthquake may result in complex states of stress in the cables at the tower and anchorage saddles. Complex states of stress may also occur at the cable bands where the effects from the band clamping forces, out-of-plane inertial forces, dynamic suspender pulls, local response rotations, and differentials in cable tensions all combine. Can criteria be set for the limit state to reduce the vulnerabilities at these critical locations?

d. Approach spans on many suspension bridges are particularly vulnerable for seismic forces due to weaknesses in their reserve for elastic stability or member ductility. Usually these spans have transverse bents and lateral bracings that were only designed for nominal wind forces, and have concrete or masonry piers and abutments that have minimal ductility and are particularly prone to shear failure when subjected to dynamic longitudinal and transverse earthquake forces. Can these approach spans be effectively retrofitted with isolation systems, new bearings, strengthened columns and bents to reduce the dynamic forces from a MCE to manageable levels, or is reconstruction necessary?

2.10.2 Cable-supported bridges that depend on the towers for the cable support have yet to be tested in a severe earthquake. Towers are non-redundant members and are designed primarily for compression loads. Towers are susceptible to overturning and buckling during earthquake loadings. Restraints or energy absorption devices may be used as a potential strategy for seismic resistance to control overturning or rocking effects. More work is needed to develop this strategy including both physical testing and analytical simulations. Considering the importance of these components as the primary non-redundant members in a cable-supported bridge and the earthquake vulnerability of a cable-supported bridge, research should be directed to investigate the ductility and limit states of these components.

2.10.3 Many suspension bridges are displacement limited in response, particularly in the transverse direction when the deck impacts the pier. Longitudinally, an option always exists to put in long travel expansion joints to allow more deflection, but the allowable transverse displacement is not easily altered.

2.10.4 Demands are computed from an analysis of a model and should reflect both static dead load, live load, temperature, and the dynamic inertia forces. Capacities are determined from detailed modeling or testing of specific components. The measures for demand and capacity must be consistent for a valid comparison. These measures depend on the limit state of interest. Specific issues identified as important include:

a. Are elastic force reduction factors applicable to long-span bridges when nonlinear behavior of components is expected?

b. Development of demand/capacity measures based on displacements and deformations instead of forces, and use of deformation in damage indices.

2.11 Instrumentation and Monitoring of Full-Scale Bridges

2.11.1 The complexity of the seismic analysis makes verification with ambient vibration testing or actual strong motion records very important. Therefore, instrumentation of existing bridges should be a priority.

2.11.2 A systematic inspection method using health monitoring and damage detection techniques should have more attention. The approach can provide a continuous nondestructive evaluation of the bridge integrity during traffic, wind, and earthquake events. The installed monitoring system can help make decisions to close bridges in case of damage or intolerable vibrations.

2.11.3 How is the instrumentation data obtained from an actual earthquake correlated with the analytical model predictions for the two level seismic design (FEE and the MCE)? Should the model be re-analyzed for the actual earthquake to verify the bridge performance?

2.11.4 If instrumentation of suspension bridges is necessary to learn about performance in earthquakes, what role are engineers, engaged in seismic evaluations, playing in convincing owners that such bridges should be instrumented? What is the status of instrumentation on suspension bridges that have been evaluated to date?

2.12 Laboratory Testing

2.12.1 Experimental testing and model development and verification of materials, members, and connections, form the basis for component and global modeling and analysis of long-span bridges. Specific issues identified as important include:

a. Material characterization: nominal, probable, and extreme ranges.

b. Characterization of in-situ materials in older structures, particularly materials subjected to corrosion and fatigue.
c. Cyclic behavior of connectors (bolts, rivets, and welds).
d. Cyclic behavior of connections; development of seismic resistant connections.
e. Local and overall stability.
f. Member capacity and ductility.
g. P-Delta effects.
h. Scale effects in testing.
i. Verification of mathematical models; reliability of mathematical models for predicting member and connection performance.
j. Development of damage indices.

2.12.2 Verification of material models through experimentation: Analysis results are only as good as the material models and analysis techniques. From recent experimental programs, valuable and competent material models have been developed. However, more awareness, cooperation, and programs are needed from the experimentalist and the analyst. Further, the organization of these efforts appears missing. Both parties appear to struggle for funding and neither appears to organize their collective efforts. There is a need to promote and coalesce material model verification programs with other elements of earthquake engineering.

2.12.3 Which steel bridge components should be tested in the laboratory to confirm predicted behavior? Long-span bridges are generally composed of steel or built up steel components that have not been designed to current day seismic design loadings and are vulnerable to damage in earthquakes. Component member ductilities for these extreme earthquake loadings are unknown and often these members are prone to failure in a nonductile manner. Although some data may be found in physical tests, the database is limited, particularly on tests into the post yield range needed to reduce the costs of physical testing many components. The finite element method may be correlated to available test results and extended to other component configurations. This approach of physical testing and correlation using detailed nonlinear finite element analyses could be applied to many steel components being used in steel bridges undergoing seismic upgrading. Research conducted on steel components used in long-span bridges, as described above, would be helpful.

2.12.4 What are the large strain nonlinear properties of latticed, riveted, and bolted members of the web and chords in the stiffening trusses of the main bridge spans, and in the trusses of the approach spans; in the columns and bents of the approaches, and in the lateral systems of the bridge? What tests can be used to establish these?

2.12.5 Can the stress concentration factors for certain types of critical member connections, viz. the ends of knee braces and diaphragms, and the ends of the transverse bracing or lateral system members be established through strain gaging and testing? Testing should be developed to locally predict large strain performance and damage.

2.12.6 What assumptions are made in determining the properties of elastomeric type isolators under large strain for long earthquake durations? Are only translational flexibilities considered? Can performance specifications and tests be used to calibrate these properties?

2.12.7 Can the coefficients for predictions of a modal structural damping under strong motion excitations be verified from monitoring of existing suspension bridges or from nonlinear laboratory testing?

2.13 Retrofitting

2.13.1 There are two main strategies for seismic retrofit, strengthening to resist the seismic demand or the use of isolation or energy dissipation devices to reduce the seismic demand. Both approaches can also be combined to achieve an optimal solution. Uncertainty due to modeling and variability of the ground motion must be kept in mind in the design process. Ductile load paths must be created and brittle modes of failure must be avoided. The preparation of contract documents for a seismic retrofit project presents the problems of any rehabilitation project, such as:

a. Verification and availability of as-built drawings.
b. Lack of information about the original design.
c. Need to check the construction sequence to assure structural stability when replacing members.
d. Need to minimize interference with traffic.

2.13.2 Selecting a seismic retrofit scheme: Often, conventional retrofit techniques address seismic conditions, but at the same time, will increase stress on the bridge under wind and/or heavy traffic conditions. An owner has suggested that the retrofit method must "make the bridge rigid under wind load, flexible during an earthquake, and not alter the traffic vibration signature of the bridge deck." Short of active control, no conventional technique provides these characteristics.

2.13.3 Many older steel bridges require seismic retrofitting of braced towers and trusses. These units are usually made from steel members with angles or channels connected with lacing bars or stay plates. Connections are usually rivets with gusset plates. What are the capacities of these members and what retrofit measures are effective?

2.13.4 Connections do not in general have the capacity to develop the full strength of the primary members and in some cases are not capable of withstanding unloading (i.e., cable anchorage's) and cyclic loading. The added lateral stiffness of deck systems in cable-supported bridges are dependent on the connectors used within the deck systems and the connection of the deck system to the towers. The advantage of stiffened deck systems is not realized when these connections fail. Retrofit schemes to strengthen these connections will provide additional resistance to seismic loading and, in some cases, allow members into the yield range to develop some ductility in connecting components.

2.13.5 Articulations used at the expansion joints in existing bridges in most cases are designed to service load conditions to provide for temperature and shrinkage movements. These movements are generally not of the same magnitude needed for anticipated maximum earthquake loadings. Restrainer designs are used to reduce the earthquake movements and in some cases where their use is not enough to resist unseating leading to potential collapse, catcher blocks are used to avoid collapse. Additionally, sacrificial expansion joints that are used to provide additional movements during earthquakes are only being used for some very limited cases and may potentially be used for a broader range of bridges if design details and guidelines are developed for their use. Expansion joints that provide for relative movements between bridge frames are an ideal location to install energy absorption devices. Bridge designers are, in general, not familiar with the technology, available devices, and their applications. Although there has been some testing of these devices by the manufacturers there are no available guidelines for the bridge designers to assist them in the potential use of these devices, the design of these devices, and the overall evaluation of the benefits of the installed device in reducing the seismic response of a bridge. Design guidelines that are focused on the expansion joints that consider all possible strategies (e.g., restraint, energy absorption, damping, movements for earthquake response, etc.) are urgently needed and could be developed with a recommended list of alternative or acceptable devices.

2.13.6 Many older long-span bridges are supported on large block masonry or lightly reinforced piers. Practical and efficient retrofit measures need to be developed to economically reuse these piers.

2.13.7 Energy dissipation devices (dampers) have great potential for retrofit measures by absorbing energy from the structural system, and then dissipating the energy, usually as heat. For long-span bridges, these devices need to be large, able to perform in extreme environments, reliable, and durable with low maintenance. Performance, constitutive laws of behavior, and test methods, particularly at full scale, are not well known or developed at this time. Performance and durability under traffic and wind cyclic loadings also need to be evaluated. In addition, extensive sensitivity studies of the seismic performance of long-span bridges with passive control devices are needed for guidelines about the location, distribution, and optimum design parameters.

2.13.8 Many long-span bridges are supported on steel columns or braced towers connected to concrete piers or caissons via anchor bolts. Under seismic forces, the anchor bolts elongate, but then do not participate in structural action. Semi-rigid anchoring devices act as anchor bolts but can elongate through many cycles of motion, absorbing energy with each cycle. This is similar to adding yielding steel hysteretic devices. However, these yielding hysteretic devices are not limited to anchorage installations but can be used wherever large relative displacements occur in the bridge such as at expansion joints and abutments. Full scale load and low cycle fatigue behavior is not well known. Optimization of the configuration of the semi-rigid device to improve energy dissipation and to ensure ease of application to a variety of structures needs to be done.

2.13.9 A shock transmission unit is a simple device that provides the engineer a method of temporarily creating a fixed connection, when desirable, which would during normal operations remain as a moveable connection. The device is sometimes referred to as a lock-up device. The unit is connected between adjoining separate structures, or between elements of structures, and has a benign effect on the bridge during normal periods of time. Upon receipt of a sudden short duration shock (dynamic) load the device locks up and transmits the load through the structure. In effect the device creates a rigid link within a fraction of a second when the sudden load is applied, affording the possibility of sharing the load throughout the structure. However, once the shock load is removed the device again reverts to its benign influence and the structure behaves in a normal manner. The selection of damping devices or shock transmission units, to avoid pounding problems between stiffening trusses and towers of suspension bridges, depends on the end conditions. In the case of the Golden Gate Bridge, because the stiffening trusses are connected to flexible towers, damping devices were recommended to dissipate the energy. However, in the case of the suspended spans of the San Francisco-Oakland Bay Bridge, shock transmission units (locking devices) were recommended between the stiffening trusses and the towers because there are no tall piers and both abutments are on rock. The pounding forces are transferred directly to rock. What guidelines are available for the design of shock transmission units?

2.13.10 The following questions pertain to the retrofit of the Golden Gate Bridge, based on a review of literature in the public domain:

- **a.** Tower hangers, end framing of laterals, and stiffening truss: What proportion of the end torque reactions do the hangers take? How are the torques equilibrated in the ends of the stiffening truss?
- **b.** Towers: If the tower end uplifts, what are the dynamic forces and impact on the top of the concrete piers. Can we have a brittle or crushing failure in the concrete? What are the inelastic triaxial principal stresses?
- **c.** Pylons: Torsional loadings are a function of the direction of the shock waves. What assumptions were made with respect to the shock directions and forces? What is the effect of the allowed pylon "uplift" on the base and to the suspender tie-downs?
- **d.** Stud connection of steel plates to concrete pylon walls: What are the proportions of the shear flow forces to the studs and in interface friction? The studs will have high bearing stresses beneath the head and washer?
- **e.** It is not clear what was the reasoning behind neglecting the interaction, in the analysis, between the Fort Point Arch and the pylons?

2.14 Miscellaneous

2.14.1 Limited funding and time constraints:

- **a.** Availability of funding does impact retrofitting. But should it also affect decisions or methodologies used in the seismic evaluation such as the choice of the return periods to be considered for the two level seismic design?
- **b.** If limited funding precludes: a site-specific study for determining ground motions, soil-structure interaction, and 3-D nonlinear analysis using multiple-support excitations -- what procedure should be followed for determining ground motions and soil-structure interaction, and what type of analysis should be used? If a thorough seismic evaluation cannot be performed because of limited funding and time constraints, are we risking substantial damage or failure? Is this acceptable?

2.14.2 Do seismic evaluations provide us with sufficient confidence that retrofitted long-span suspension bridges will perform as predicted?

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4. Disclaimer and Acknowledgement

Views expressed in the paper are those of the above-listed subcommittee members and not those of the ASCE or their employers.

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5. COMMENTARIES

The commentaries are by the eight panelists: Ahmed M. Abdel-Ghaffar, Serafim G. Arzoumanidis, Roy A. Imbsen, Klaus H. Jacob, Mark A. Ketchum, Herbert Rothman, Charles Seim, and J. P. Singh.

5.1 COMMENTARY BY AHMED M. ABDEL-GHAFFAR [4]

5.1.1 The Importance of Structural Health Monitoring

For nearly a century U.S. bridge engineers had predominated the design and construction of long-span suspension bridges; this era ended with the record-breaking Verrazano Narrows Bridge in 1964. These important lifeline structures are expensive to maintain and they frequently pose complex technical problems. Authorities of suspension bridges throughout the world have recognized the importance of bridge "health" monitoring and management in securing the operation of their bridges and in protecting the vast investments made in these road and rail transportation systems.

The process of elaborate instrumentation, measurements and analysis of dynamic response data, including the seismic performance evaluation, supplemented by immediate and/or long term maintenance, rehabilitation, strengthening and retrofitting programs -- can be considered as essential "health" monitoring and structural diagnostic operations. Dynamic response data, recovered from comprehensive instrumented bridges subjected to dynamic service and environmental loadings, can be valuable in improving the state-of-the-art of engineering capabilities to provide efficient maintenance, serviceability, disaster recovery, strengthening, and recovery programs. It is also essential for damage detection usually induced by wind and seismic loadings. Damage consisting of single or multiple defects can be defined as the reduction of either the strength, the stiffness, or both. Finally, it is important to point out that further, or new, implementation of structural control schemes or methodologies will require precise motion monitoring.

5.1.2 Lessons Learned from Seismic Input-Response Records

Records recovered from instrumented suspension bridges were and will be very important in the identification of seismic vulnerabilities that will lead to more realistic conceptual retrofitting and strengthening schemes and cost estimates. Moreover, there is an urgent need for:

i. studying the effectiveness and reliability of both tuning or damping augmentation devices and strengthening measures,
ii. evaluating current conditions or structural capacity and the seismic safety demands, and
iii. reengineering of critical joints and members such as: tower-deck and deck-cable bent connections, tower-base and pier connections, tower struts above deck level, cable-tower-anchorage saddles, cable-deck connections, and side-span articulation.

In summary, to improve the overall seismic performance of suspension bridges, the following guidelines should be considered (to achieve a successful seismic retrofit design):

a. Realistic evaluation of structural conditions as well as, as-built capacity.
b. Site seismic risk evaluation including functional and safety earthquake-type motion scenarios.
c. Structural and computational modeling of these bridges to determine their dynamic characteristics, seismic forces, deformations, and demands.
d. Installment of comprehensive instrumentation with wide dynamic range including utilization of the state-of-the-art real-time monitoring systems. Different vibration environments exist for the various excitation sources which include ambient vibration, high wind, and strong earthquakes. These environments have characteristic ranges of amplitude, frequency, and duration. Monitoring

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requirements are, therefore, different with distinct system configurations required for each environment. Developments of real-time system components such as sensors, data transmission, signal conditions, recording, and analysis software must be used for the needs of a particular bridge.

e. Calibration and validation of the computer model using response data.

f. Reengineering of the critical joint, bearing, member, and assembly in such a way that acceptable damage and limited displacement and impact can occur in a controlled fashion.

g. Identification of critical joints or structural components that can be augmented by energy-absorption or dissipative devices (the wind-lock between the stiffening truss and the tower is an example in which these multi-defence devices can be installed).

h. Despite the universal problems of budgetary constraints and limitations of funding for operation, maintenance, rehabilitation, replacement, strengthening, and retrofitting of old existing bridges -- the allocation of resources from federal, states, counties, cities, etc., should be secured and closely analyzed for these transportation and landmark infrastructures.

i. Integration of measurements, computer models, response-calculation software, data analysis and appropriate system identification techniques to provide essential information for the serviceability and the disaster recovery of the bridge in the aftermath of strong earthquake shaking.

j. Data gathering of international bridges in a unified format that will be beneficial to the bridge engineering community such as the results and analysis of full-scale dynamic tests, seismic counter-measure techniques, seismic behavior evaluation, and experimental studies of supplementary damping.

5.2 COMMENTARY BY SERAFIM G. ARZOUMANIDIS [5]

Long-span suspension bridges are usually critical links in the transportation network they serve and, therefore, are retrofitted to strict performance criteria. Most often such major structures include extensive approaches. The approaches consist of smaller spans that do not classify as long-span structures. However, as part of major crossings, the approaches must be retrofitted to the strict performance criteria as the main bridges. A realistic seismic evaluation and retrofit program must consider all types of bridges in a crossing (including short-span bridges) and not just the suspension bridge. In fact, experience shows that most of the vulnerabilities in suspension bridge crossings are found in the approaches and not the suspended spans.

Suspension bridges in the U.S. are exclusively steel structures, usually many years old. During strong seismic events, bridge components are subjected to extreme loading conditions. Recent seismic evaluations of such structures have found the problem of how to accurately predict the capacity of riveted latticed members and connections of complicated configuration, the problem of stiffness contribution of secondary systems such as the roadway deck, the problem of accounting for the effect of corrosion, etc.

Unlike the joints of concrete members, many steel frame connections are semi-rigid. The exact load-deformation response of such connections is not easy or practical to determine analytically. However, proper consideration of the stiffness and ductility of these connections is important for accurate response analysis. Strengthening of such frame connections increases the overall stability, but also stiffens the structure attracting higher seismic loads. On the other hand, the ductility of such connections is not known without an experimental investigation.

In the eastern U.S., there is considerable uncertainty regarding the prediction of ground motions, especially for strong seismic events. The geological structure of faults is not known sufficiently to associate the generation of seismic events to specific faults, as in the western U.S. Thus ground motions developed for a specific site can be assumed arriving from any direction. Among other problems, this uncertainty does not permit a rigorous treatment of the effect of out-of phase motion of the foundations.

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The accurate prediction of ground motions, along with the required performance of a bridge for specific levels of seismic hazard, have a profound effect on the cost of retrofitting. In particular, the appropriate level of seismic hazard for design is an important question. On the west coast, earthquakes of nearly the maximum anticipated level are known to occur at relatively frequent intervals. The available seismic records can provide a basis for establishment of various hazard levels. On the east coast, on the contrary, strong earthquakes occur rarely and the scarce available data are inadequate for such determination with confidence. It appears, therefore, reasonable to ask whether an adjustment to the performance criteria for the east coast bridges is warranted to account for this effect.

Computer models prepared for the determination of the seismic response of bridges are intended to represent the behavior of bridges during an extreme loading event. The importance of ductility which is known to increase with the amplitude of vibrations, is considerable during an event of this type. Validation of the computer models through ambient vibration tests is advantageous. However, there are serious limitations to the usefulness of such tests. Ambient vibrations are of small amplitude and composed of modes, which are sometimes different from the modes due to ground motion excitation. These features prevent the direct verification of computer models prepared for the consideration of seismic events, that induce large vibrations and include certain modes, e.g., some longitudinal modes, not excited by traffic and wind.

Energy dissipation devices are very useful for the retrofitting of suspension bridges. Currently, there are numerous devices with energy dissipation capabilities. These devices have a generally limited record of past applications and, therefore, require a careful program of testing for verification of their performance. The application of such devices, many with complicated load deformation response, require sophisticated nonlinear computer programs for analysis.

Seismic evaluation and retrofitting of major structures occur in a two-step process. The first step is the phase of seismic condition assessment and recommendation of retrofit measures. The second phase is the detailed design of the retrofit measures and verification of the performance of the structure after the application of the retrofit measures. In this process, it appears reasonable that the most sophisticated (and expensive) analytical procedures are reserved only for the second phase of work.

5.3 COMMENTARY BY ROY A. IMBSEN [(6)]

The uncertainties associated with seismic design of major bridges warrants input from experts in the earth sciences, geotechnical engineering, material's behavior, and structural analysis and design. Although the use of decision trees has been incorporated in determining the seismic hazard by the earth scientists, there still are many other uncertainties that have not been quantified explicitly and treated systematically. Until this has been done and accepted by the profession, an intermediate step of using experts to serve as a peer review panel or advisory group is being taken to at least qualitatively reduce the uncertainty of the overall seismic design or retrofitting. Having expert opinions from various disciplines working collectively on a single bridge is being done on most of the large bridges currently being designed or upgraded.

A peer review panel may be formed by the owner or consultant to provide oversight and design criteria review of the consultant's work. The panel is intended to assist the owner and consultant in monitoring compliance with the criteria set forth by the consultant and the owner as well as basic design criteria required by the applicable codes and regulations. The panel is also intended to facilitate coordination between the consultant and the owner. The panel usually meets several times throughout the duration of the project with the design teams individually, and also meets periodically with all the project

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[(6)] President, Imbsen and Associates, 9912 Business Park Drive, Suite 130, Sacramento, CA 95827 managers of the consultant teams. For example, in the Golden Gate Bridge Seismic Retrofit Project, the panel is composed of five design professionals: one furnished by each of the three consultant teams, and the remaining two members appointed by the District Engineer. The panel has appointed its own chairperson and will report to the District Engineer. The panel will review the seismic retrofit strategy prepared by the consultant team, and coordinate the review of the strategy with the owner.

Panels are usually asked to submit brief interim reports to the owner and may submit a final report of its opinion on the consultant team's work once the design work has been substantially completed. These final reports usually become a permanent public record of the panel's evaluation of the quality of each design team's work.

5.4 COMMENTARY BY KLAUS H. JACOB [7]

5.4.1 Bridge as Link in Regional System

When a seismic retrofit of an existing long-span bridge is undertaken, the importance of the bridge in the regional transportation network needs to be evaluated, i.e., the entire system needs to be considered rather than each bridge individually. Bridges of a regional system are often owned by different authorities. In that case it is important that these different administrative authorities cooperate in a regional master plan for retrofitting of the system as a whole to ensure that some of its main arteries remain functional after a severe earthquake. If elements within the system are not retrofitted according to their function and importance in the system, then the system may not be able to perform satisfactorily during an earthquake.

5.4.2 Performance Criteria, Two-Level Designs, Recurrence Periods, and Iterative Approach

Once the importance of a bridge in a system is known, then proper bridge performance criteria can be developed. For long-span bridges seismic performance criteria, with regard to service and damage levels, are typically established for two types of seismic events: ordinary and severe earthquakes. In several cases in the eastern U.S., the two levels have been associated with two different recurrence periods, most commonly 500 and 2,500 years, although no formal consensus has yet developed. Different bridge owners have contemplated different hazard levels (recurrence periods, or annual probabilities of exceedance), sometimes dictated by cost/benefit considerations rather than by applying uniform criteria for hazard or risk. In many instances it may be beneficial to revisit the design levels (recurrence periods, and perhaps even the performance criteria) after the vulnerability of the bridge in its present state has been assessed, and the engineering options and costs for retrofit have been established preliminarily in a first-order approach. In some instances, raising the design levels (and hence additional safety) can be achieved at low additional costs. In other cases, it may not be cost-effective to retrofit the bridge to higher design motions. For instance, if alternative bridge routes in a regional traffic system can be more cost-effectively retrofitted than is the case for a given bridge (see section 5.4.1 above), then such alternatives may be pursued. An iterative system-wide approach may yield the most cost-effective solutions.

5.4.3 Validation of Bridge Models by Ambient Vibration Measurements and Modal Analysis

Modern microprocessor-controlled digital accelerometers can be readily used, and have been used on several east coast long-span bridges, to measure ambient vibrations and modal behavior of bridges, including the boundary conditions across joints, bearings, and revealing soil-structure interaction. With these new technologies no wires have to be strung and consequently interference with traffic is virtually nonexistent. Cross-spectral analysis of the measured vibrations yields modal periods, damping, mode types and shapes. The ambient vibrations do not simulate the nonlinear deformation under seismic

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loads. But results from them are essential to validate the computer models of large bridges in their linear deformation range and for boundary conditions at low excitation levels. In several cases the ambient vibration measurements have revealed bridge behavior different from that assumed in simple analytic models that tended to decouple motions between different portions of the bridge (i.e., between foundations, piers, and superstructure; or between approaches, anchor and, main spans). The measurements can be repeated once the retrofit is completed in order to check whether the modified bridge behaves as intended by the retrofit measures.

While some experts still question the utility of ambient vibration measurements because they only reflect low-level excitation, it is obvious that if there are difficulties in making accurate computer models of the bridge behavior in the linear range of deformation, there is little hope that the nonlinear response models for larger excitations will be valid. Hence, low-level validation of computer models by ambient vibration testing is recommended as an essential benchmark test of the procedures used for evaluating existing long-span bridges. They are a useful step along the path, and not the end.

5.4.4 Design Ground Motions: How Much Information Does the Engineer Actually Need and Use?

Probabilistic seismic hazard analysis is the most common (but not the only) method to derive uniform hazard spectra (i.e., damped response spectra), especially for rock sites. Such spectra can be used initially for modal analysis under linear assumptions. Since for large bridges, and especially on soft soils nonlinear response analysis is crucial, it is important to develop ground motions in the time domain. To choose realistic ground motion time series it is necessary to "de-aggregate" the uniform hazard spectra into contributions from constituent events expressed as magnitude-distance (M-d) combinations. In the eastern U.S. where the causative faults are rarely known, the de-aggregation typically yields a suite of events ranging from small earthquakes (M=5) at short distances to the largest possible magnitudes (say M=6, 7, or even larger, depending on the seismic environment) at much larger distances. Once these M-d combinations are determined, it is then possible to select appropriate regional strong-motion records from existing databases; however, often such records do not exist. Fortunately, seismologists are now in the position to produce highly realistic ground motions by computer simulations. These simulated motions are constrained by geophysical and geotechnical information available for a given region and site such as: crustal velocity, density and attenuation (Q) structure; crustal scattering properties controlling the shaking duration as a function of distance (it mimics the 3-D lateral heterogeneities in the crust); range of seismic source depths; stress drop; type of faulting (i.e., thrust, normal, or strike-slip motion); extended rupture processes including their directivity effects; the nonlinear soil properties where applicable; and the natural variability of, and any uncertainty about all of the above.

The structural engineer jointly with the seismologist and geotechnical engineer must early on in the project make tough decisions regarding the number and type of ground motions the seismologist shall provide. Generally, the seismologist can provide much more detailed ground motion information than is often used by the engineer for the actual analysis. The ability to provide detailed ground motion information should not distract from the fact that the information is inherently uncertain, often by a factor of two or more, as is further discussed below.

Here are some of the questions that need to be resolved between the engineer and seismologist before the design ground motions are produced:

a. How many events (M-d combinations) per recurrence period should be selected to reflect the range of hazard-consistent ground motions over the spectral frequency range of interest to the site and the bridge? *(Suggested answer: in the eastern U.S. at least three, e.g., M= 5, 6, and 7 events at their hazard-consistent distances).*
b. Shall all three components of ground motions be provided, i.e., two horizontal components and one vertical component? *(yes).*
c. Shall the ground motions be provided as ground accelerations, velocities, displacements or all of the above? *(all of the above).*

d. At how many support points shall the motions be provided? *(this depends on how variable the soil and rock conditions are along the bridge and its approaches).*
e. At what depths below grade shall the ground motions be provided? The answer is especially important if and when foundations (caissons, piles) are present that may or may not reach to rock but are also imbedded in, or surrounded by, soils of variable stiffness and degradeability with strain, perhaps even showing potential for liquefaction.
f. Is it sufficient to describe ground motions at given points, or is knowledge of the entire propagating wave field required, including azimuthal effects and wave-field incoherence? *(the latter).*
g. How many (stochastically variable) ground motion realizations of the "same" event (i.e., same M-d combination) shall the seismologist provide so that the variability of the nonlinear soil response, nonlinear soil-structure interaction, and nonlinear structural response can be explored? *(at least five realizations per M-d event).*
h. In case of the presence of soils, particularly soft soils, shall the seismologist provide the three-component ground motion wave-field simply at a given reference level in the rock below the rock-soil interface at a profile along the bridge, so that the geotechnical/foundation engineer can use that wave field as input into a finite-element or finite-difference model that may contain a three-dimensional undulating rock/soil interface with overlying nonlinear soils and foundations imbedded? Or alternatively, should the ground motions be migrated from the rock-soil interface through the soils by SHAKE-like one-dimensional equivalent linear soil analysis? Should this be done for each foundation support point separately, while retaining the accurate phase relationship due to wave propagation for a certain azimuth relative to the bridge axis? *(either way).*
i. How many directions of approach (azimuths) of the propagating wave field with respect to the bridge axis are required?
j. Will the computer analysis of dynamic bridge response properly handle the question of finite deformations? For instance, will it properly combine the *displacements* made up of *quasistatic deformations* in the free-field between piers, and *dynamic displacements* in the structure, across joints and bearings (for seat width), pounding at abutments or between deck and towers etc.? And how should the input ground motions be provided properly phased (according to wave propagation and other spatial coherence factors) in one or more of the following time-domain spaces: acceleration, velocity, and/or displacement? *(depends on what program and input options are used).*

As one can see, if all these options were pursued, dozens, if not hundreds of nonlinear computer runs of the (3-D?) soil, foundation and bridge response in the time domain would be required. Rarely are the financial and computing resources available to an engineer to follow through on all these options. *It is, therefore, of the utmost importance that the structural and geotechnical (foundation) engineers define early in the project together with the seismologist what their approach exactly will be; and how to minimize the number of ground motions to be provided for actual usage without losing important options to explore the possible response range of the bridge under the range of realistically possible ground motions.*

The last issue begins to illuminate an apparent but often denied dichotomy between engineering and seismology approaches. In these days, seismic hazard assessments tend to be carried out probabilistically, but structural and soil-structure interaction analyses tend to be carried out deterministically. This leads often to problems where the two disciplines abut, but must intersect. Seismologists repeatedly state that ground motions, however parameterized, may differ readily by a factor of two or more for a given exposure level or design event. Some engineers state that such high uncertainties in input motions are unacceptable. Is this really true? They should be unacceptable only if a structure did not have reserve strengths, or rather lacks ductility and/or redundancy -- in short, lacks toughness. If a structure fails because of a factor two in input motions, it can hardly be a good structural design or is poorly founded (if liquefaction or soil/structure response is the culprit).

Personally, I strongly recommend that in addition to using probabilistically derived ground motions, a common-sense sanity check be performed by asking the question: what is a set of geologically and seismotectonically *plausible upper-bound scenario earthquakes* (PUBSE's)? They may be equal to, or less than, what is often termed maximum credible earthquakes (MCE's). When ground motions for such plausible events are given, the seismologist must, however, state his best estimate of the annual probabilities that are associated with such events.

5.4.5. Liquefaction

Liquefaction analysis often tends to be based on particular empirical relations (for instance those given by the much used Seed-Idriss procedure). However it must be kept in mind that these and similar relations were typically derived using a global database that is more applicable to California than eastern U.S. ground motion conditions. Analytical procedures now exist that allow the usage of actual ground motion time histories as input appropriate for the region and site, in order to compute pore pressure built-up and at least the onset of liquefaction. However, these research-level methods are not yet widely used in routine engineering applications.

5.4.6 Comparison with AASHTO Spectra and Issues Concerning Time Series Matched to Target Spectra

Engineering firms or bridge owners often specify that the site-specific ground motions, or rather the damped elastic response spectra be compared to AASHTO design spectra anchored to seismic zone factors taken from the maps depicting exposure levels of 10% exceedance in 50 and/or 250 years. When such comparisons are done in the eastern U.S. and on stiff soil or rock sites, it generally turns out that at short periods (≤ 0.3 sec) the site-specific ground motion spectra exceed the AASHTO spectra by a considerable margin, while at long periods (≥ 2 sec) they generally fall below the AASHTO spectra, and often by a considerable margin.The capping at short periods of the AASHTO spectra is more compatible with California ground motions and poorly matches eastern ground motions. This is particularly important for some of the older long-span bridges in the eastern U.S. which, in some cases, have rather stiff but potentially brittle (unreinforced) masonry piers that attract high-frequency forces but do not respond to them very well.

The design accelerations in the AASHTO spectra fall off at long periods essentially with $T^{-2/3}$. In contrast, long period ground motions tend to fall off more rapidly proportional to T^{-1}, and eventually -- for periods longer than the inverse of the so called "Brune corner frequency" -- with T^{-2} (!!). In addition, since the hazard in the eastern U.S. has small contributions from large earthquakes compared to California, the AASHTO design spectra tend to be highly conservative. When comparing them to actual uniform hazard spectra for the eastern U.S. they appear quite unrealistic. Of course the conservatism was built in intentionally when the Applied Technology Council (ATC) developed these curves some 20 years ago. It was felt at that time that little was known about long-period motions and moreover, large structures tend to be less redundant and, hence, some additional conservatism appeared warranted. If this conservatism is removed when using site-specific ground motion spectra for design, then this should be made with the full awareness of the possible engineering consequences and implications. On soft soil sites, the site-specific (elastic) design ground motion spectra can exceed the AASHTO spectra by large multipliers of three or four, or sometimes even more.

It has been a common practice to produce a single set of ground motion time series that match given target spectra (whether of the AASHTO type or of site-specific vintage); they can be made to match within a prescribed error limit. This practice, while attractive to the engineer in providing a single matched record instead of a suite of records from corresponding multiple M-d combinations, has severe handicaps from a seismological viewpoint and has corresponding engineering implications. The time series that conforms to a uniform hazard target spectrum or to an AASHTO design spectrum is a time series that never will occur in any single earthquake. It is a time series that attempts to mimic a mixture of similarly likely events that contribute to the design level. It is an event that approximately embodies all possible design earthquakes in a single time history. Thereby the spectrally matched time series tends to be more conservative (albeit less informative about the likely behavior of the bridge in any single real earthquake). This is not to say that spectrally matched time series should not be used. It is only to say that any individual real earthquake will hardly ever produce ground motions like those embodied in a time series whose response spectrum is matched against a uniform hazard spectrum or an AASHTO design spectrum.

5.4.7 Conclusions

While modern seismological tools are available to provide the engineer with a sometimes dazzling complexity of rather realistic ground motions, or even entire propagating wave fields, it must be kept in mind that ground motions are by nature (and not because of our lack of knowledge or data) inherently variable. In addition, our imperfect knowledge adds uncertainty and sometimes bias (error) to this quite natural variability. Therefore the engineer must find design methods that make the response of structures such as long-span bridges as immune as possible to deviations from the adopted design motions. The fact that computers can crunch a lot of numbers is not, and never will be, a substitute for common sense and prudent engineering. It is only a tool in the hands of smart engineering that can show quantitatively and rapidly the "what if" scenarios. Perhaps the greatest challenge for sound engineering of large-scale bridges lies in the requirement for most attentive cross-disciplinary interaction between the structural engineer, geotechnical/foundation engineer, and the seismologist. It is in this interdisciplinary arena where progress is most needed and still can be made.

5.5 COMMENTARY BY MARK A. KETCHUM [8]

5.5.1 Introduction

The scientific seismic assessment of long-span bridges has been given serious attention by the bridge engineering profession for only about five or six years. Before that date, and even to a great extent today, many engineers involved in the engineering of such bridges apparently believed that long-span bridges in general, and suspension bridges in particular, were for some reason immune to the effects of earthquakes. An important exception to this belief of immunity was avowed by the late Professor Frank Baron (Baron 1979), whose paper proposed procedures for performing seismic investigations of suspension bridges. His recommendations went largely unheeded for ten years until the 1989 Loma Prieta earthquake damaged many bridges in northern California. Since 1989, seismic investigations of many major west-coast suspension bridges have been completed, with scopes of work remarkably similar to Baron's proposal. Numerous technical issues remain unsolved, however, some of which are discussed below. Purely policy issues are not discussed.

5.5.2 Ground Motion Issues

Ground motion used in many suspension bridge studies to date have been developed by performing probabilistic and deterministic seismic hazard assessments to determine "target" spectra for rock motions, synthesizing spectrum compatible "control" histories of rock motions representative of the site, adjusting these motions for wave passage and incoherency effects using either analytically derived transfer functions or semi-empirical coherency functions to determine rock motions at each bridge support, readjusting these motions for spectrum compatibility, and then using these motions to drive the bridge model (if the bridge is founded directly on rock) or SSI models of the foundations (if required). At most only a few fault-rupture scenarios have been explicitly considered on any one bridge. The tacit assumption has been made that spectrum compatible motions are adequate to determine damage.

An issue that remains unresolved in this approach is whether the bridge will perform significantly differently under "true" earthquake ground motions, which are generally not as compatible with the target spectra as are the synthetic motions. The spectrum compatible motions are known to provide envelopes of response maxima when the structural response remains linear, i.e., when it is undamaged, but not when significant nonlinearities are expected. This issue is most important when significant response nonlinearities are expected in an earthquake for which

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the bridge is expected to remain functional. Resolution of this issue would, probably, require the analysis of a particular bridge, under several different sets of modestly spectrum compatible ground motions, representative of actual motions at the site, using bridge models that explicitly account for all important nonlinear behavior modes.

5.5.3 Component Performance Issues

The material and structural forms of most structural components of U.S. suspension bridges are from a previous era of structural engineering. This is true both for superstructures and foundations. When many of these bridges were built, the profession was still strongly attached to an allowable stress method of design, so the inelastic performance of components and connections was not a primary design consideration. Member compactness and detailing criteria were not uniform even on bridges built at about the same time. Most of the recent experimental research on inelastic member performance has concentrated on modern members and the development of improved details. Therefore, the profession has a remarkably small database of experience to correlate demands on these components to damage, and the demand limits in recent seismic work have been set largely on the basis of judgment.

An improved understanding of the correlation of member demand to member damage could be provided by a research program based on a combined analytical and experimental approach. An experimental approach based on cyclic inelastic testing of large-scale models of members or subassemblies, would provide the strongest basis for limits on cyclic inelastic demands. Such an experimental approach, however, would be very costly. Therefore, to optimize the knowledge gained from the investment in research, an analytical approach could be used prior to implementing the experimental program. The analyses, making use of now-available inelastic finite element modeling technologies, can be used to avoid some of the pitfalls and expense of physical experiments. It is important that all of the research consider the very important three-dimensional nature of the structures and demands.

5.5.4 System Performance Issues

The engineering profession has gained significant experience in the recent past in the global seismic response analysis of extended structures. There are still unresolved questions, however, about the correlation of damage to vulnerability. For example, at issue in a recent suspension bridge evaluation was what risk to safety and functionality is represented by, for example, buckling of a truss chord or fracture of a floor beam. The question can be posed as: what damage level to the component represents a limit state with respect to safety, functionality, and repairability of the system? Resolution of this issue requires addressing a wide range of considerations -- the impact of member damage on system vulnerability is related not only to the form and behavior of the member, but of its function in the system.

The correlation of damage to vulnerability can be assessed using an empirical approach only by waiting for a big earthquake. An analytical approach is required if predictions are expected. Such analyses would probably involve evaluating global nonlinear bridge models under the multiple ground motion scenarios as discussed previously. The global bridge models would by necessity incorporate rational models of all significant response nonlinearities, supported by local subsystem models. Since the global models would need to provide predictions of damage rather than of vulnerability, and since damage is generally concentrated even when vulnerability is distributed, the results of the studies would be of most value if "random" variations of capacities and initial imperfections were modeled.

5.5.5 Miscellaneous Issues

Many related issues remain unresolved in addition to the major ones discussed above. These include:

a. constructibility: what construction technologies are applicable to aging structures and materials,
b. component interaction: how much damage is caused by and in decks and other components that were designed without consideration of interactive behavior,
c. damping: what level of energy dissipation (modeled as damping) is provided by the various types of bridge components at various levels of damage,
d. seismic-resisting "devices": are isolation/damping/lock-up devices a reliable method of providing improved seismic resistance,
e. interaction with live load: what influence does the significant live load demand on truss components have on seismic vulnerability,
f. risk: what is the cost of risk versus the cost of retrofit versus the cost of repair for a critical transportation link, and others.

If policy issues are added to the discussion, the list becomes even longer.

5.5.6 Summary

The rational and scientific seismic evaluation of suspension bridges is a fairly new field and, as such, there are numerous issues that have yet to be resolved. Major issues include the influence of various aspects of ground motions on inelastic structural behavior, the correlation of demands to damage, and the correlation of damage to vulnerability. Many other issues are also deserving of recognition. Some of these issues are common to many different bridge types; for some of these, however, the resolution may be possible only on a bridge-by-bridge basis. Since no major suspension bridge has been "tested" in a major earthquake, and since no retrofit construction to recent standards has been initiated on a suspension bridge, the responsibility of the engineering profession to the public can perhaps be best served by continuing to address these issues and build the knowledge-base on every bridge that we encounter.

5.6 COMMENTARY BY HERBERT ROTHMAN [9]

5.6.1 Transverse Seismic Forces

Transverse seismic forces are far greater than the design static wind forces for all but the longest span bridges. As a result, the lateral systems are generally under capacity. It can be expected that in addition to strengthening the laterals and their connections, the wind connections at the ends of the spans will need alteration. The "wind tongue" details are generally very congested and access for reconstruction is limited. If the overstress exists for "safety" (damage accepted, but not collapse) only, economical retrofits can be made that bypass the wind tongues. They are generally in the form of tension members that span from the channelward side of the lateral system gusset to the tower legs, pylon, or anchorage. Because these ties are flexible, and allow motions of one to two inches (25 to 50 mm), damage to finger joints can be expected. This may make the joint uncrossable, and the retrofit unacceptable for an operating level event. If such ties are otherwise desirable for operating earthquakes as well, the finger joints can be replaced with modular joints that have adequate lateral motion capacity.

5.6.2 Longitudinal Seismic Forces

Longitudinal seismic forces are especially severe in bridges with pylons (vertical piers with a saddle from which a straight backstay goes to an anchorage). The longitudinal force, due to a side span fixed to the pylon, and the pylon weight itself, can slide the cable through the saddle, or break the bolts connecting the saddle to the pylon.

a. Saddle covers: Cable sliding can be resisted by bolting large covers over the cable to the top of the saddle to increase cable/saddle friction. This does not help pylon/saddle sliding, and makes for

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difficult inspection of the cable at a vulnerable corrosion point. Further, these saddles are usually furnished with bolted covers to overcome the dead and live load sliding. There may not be space for additional bolts.

b. Freeing side spans: The side span force can be relieved by freeing the side span from the pylon. The span will then swing like a pendulum during an earthquake as does the main span. If the link connecting side span to pylon is not removed to allow this motion but is, instead, weakened so as to be a fuse that breaks at a predetermined force, existing expansion joint and existing wind tongue provisions designed for temperature and wind need not be modified.

c. Swinging: This will however require installation of protective springs and dampers to prevent impact with the pylon. These devices may be placed at the tower or pylon, whichever is more convenient. When the side span is allowed to swing freely, cable bands adjacent to the pylons which support short (and therefor stiff) suspender ropes may have to be strengthened to resist the disproportionate share of longitudinal force that will be imposed upon them.

d. Wire rope ties: The longitudinal forces can also be resisted by the installation of wire rope ties from the saddle or top of pylon to the cable. The connections to the cable will be similar to cable bands and will depend upon bolts for frictional resistance. The ropes will be nearly parallel to the cables and exist over a short length at the pylons. They will not have a significant affect on the bridge's appearance but, as with all cable bands, the bolts must be retightened periodically.

5.6.3 Cable to Truss Ties at Center of Main Span

Connections of the cable at the center of the main span to the stiffening truss have become standard retrofits on wind susceptible bridges because they fully suppress the second torsional vibration mode. This mode takes place when the two cables swing in opposite longitudinal directions. There is very little stress induced in the cable and therefore little strain energy, long periods, and little damping. The torsional mode can be driven by a transverse component arriving at different times at each tower. There are unlikely to be high seismic stresses due to this condition, but whatever stress is generated can be easily eliminated. Even this small advantage should be adopted if there is concern about aerodynamic stability. The ties somewhat reduce the longitudinal swinging (described above) by reducing overswing of the suspended structure relative to the cable. They may, however, serve a much more important function if they are converted to members with viscous dampers inserted into them. This damping will reduce longitudinal motion for all other antisymmetrical longitudinal modes, reducing the need for retrofits to anchorages and towers.

5.6.4 Damping

There is uncertainty concerning damping in these bridges which can be, in part, resolved by ambient vibration surveys. However, extrapolation to strong motion damping is not obvious, particularly if damage is accepted. It is for this reason that relatively arbitrary damping percents are used for seismic analysis. Some improvement in estimating damping can be arrived at if bridges can be excited by other forces such as wind. Two such bridges, where records presently are believed to exist, are the Bronx-Whitestone and the Golden Gate Bridges. Old paper records taken during winds up to 50 mph at the Bronx-Whitestone Bridge have been digitized and analyzed for frequency shift and damping. They showed that, at the frequencies of greatest interest, damping increased very slightly for both vertical and torsional modes. The increased damping was well below the approximately 5% value generally assumed for seismic evaluation. Securing damping values due to wind or earthquakes from strong motion records is feasible. Instrumentation that turns itself on at preset accelerations should be installed wherever feasible for this purpose.

5.7 COMMENTARY BY CHARLES SEIM [10]

5.7.1 Introduction

The seismic vulnerabilities of highway bridges have only recently been revealed to the bridge engineering profession by tragic failures in recent earthquakes in California and Japan. But the traveling public quite rightfully demands safe passage over a bridge regardless of what natural forces the structure may be subject to. However, the bridge engineering profession has begun to develop seismic spectra and ground motions to apply seismic evaluation techniques and retrofit measures. The construction industry has also responded by developing seismic devices such as isolators, lock-ups, and energy absorbing devices, and academia is embarked on research and testing programs. Bridge engineers now have developed a repertory of technology from which to draw for evaluating and retrofitting bridges.

Admittedly, there is much we don't know but, if we carefully and diligently apply what we do know to successfully retrofit common bridges, then we can also carefully and diligently apply what we do know to evaluate and retrofit long-span suspension bridges. This paper is the first effort of the profession to critically evaluate what we do know and apply that knowledge to the consistently more complex problems inherent in long-span suspension bridges for use by the engineering profession.

If we think of this paper in a positive manner as a collection of knowledge presently known at this time by the bridge engineering profession, then this will allow practitioners to evaluate and retrofit critical bridges such as the Golden Gate Bridge with some degree of confidence for a successful application. Conversely, as the paper points out, there are extensive voids in the profession's knowledge. The paper will hopefully stimulate research efforts to fill those voids. Any well thought out retrofit of a long-span suspension bridge will be immensely better than doing nothing.

Let us start by looking at some critical issues that the paper discusses for the seismic evaluation and retrofitting of suspension bridges in the United States.

5.7.2 Seismicity of the Bridge Site

Certainly in California and along the Pacific Coast, and perhaps the Midwest, seismicity is well enough known to determine fault locations and earthquake magnitudes as a function of return periods and distance. The remaining area of the U.S. may not be as well known because of lack of records or systematic study. But I think enough is known to determine in the U.S. the degree of seismic risk at most bridge sites sufficient to warrant an investment of time and money to retrofit important bridges.

5.7.3 Performance Criteria

This is a quasi-engineering issue as the performance of a suspension bridge during, and the condition of the bridge after, an expected earthquake is basically the owners responsibility to determine in conjunction with the design engineer. The seismicity of the site may also influence the decision of the level of performance that can be expected. In addition, non-redundant bridges and the needs of the local areas for lifeline corridors also need to be given special weight.

The bridge can be designed for several levels of seismic activities and several conditions of structural damage. Non-redundant bridges can cause extreme economic hardship to communities as noalternative route may be available for months after the event. Lifeline corridor structures must provide passage for emergency vehicles after the event. In each of these cases, a higher level of seismic retrofit must be achieved which of course increases the cost. The design engineer can provide cost data and advice to the owners to help them make intelligent tradeoffs.

Certainly the minimum performance level must be safety against collapse. The public expects and demands no less.

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5.7.4 Soil Structure Interaction (SSI)

SSI is going through a technology transfer from the nuclear industry to bridge engineering. I think much work is required before the bridge engineering profession shakes out the most useful and effective methods to use. Until this happens, for most suspension bridges and most bridge sites, employing any form of SSI most likely will change the degree of but not the retrofit measure strategies. Applying SSI usually reduces the bridge's response. But until the profession can rely on a standard SSI analysis, the bridge designer should exercise caution and not allow too much reduction for now.

5.7.5 Analysis

More has been written about analysis because computer modeling and analysis is, perhaps, more highly developed than any of the other seismic retrofit issues. Bridge designers today have a great variety of highly developed programs to choose from and a tendency exists to run the most sophisticated program is always present. Modal superposition methods may not capture the bridge behavior without a large number of modes and usually suspension bridges contain nonlinearity. This method may be a good simple exercise to learn how the bridge behaves dynamically if followed by additional analysis. As a minimum, suspension bridge dynamic programs should be geometrically nonlinear, three-dimensional, and being capable of upgrading to material nonlinearity and multiple-support excitations.

When material nonlinear elements are used in the analysis, the post-yielding behavior needs to be described. But perhaps the greatest unknown facing the bridge designer today is the dynamic and post-yielding cyclic behavior of the old styled latticed members installed on most suspension bridges built in the U.S. Undoubtedly some research on the cyclic nature of these members would pay off with better understanding and improved modeling capabilities and economic retrofitting.

5.7.6 Design

The most sophisticated analysis of the dynamic vulnerabilities of a long-span suspension bridge is for naught unless the concepts can be followed with a proper design which a qualified contractor can understand, bid with some assurance, and construct without major claims and delays to the owner. Analyzing on a computer can be fun but designing is hard pick and shovel work.

The constraint is the structure itself. The retrofit must fit into the structure and have some architectural harmony with the bridge. The project must undergo the environmental assessment process. People have been viewing, and in some cases like the Golden Gate Bridge, admiring the structure for years. Gloving on hunks of stiffening or bracing can have a jarring impact on the viewer. Rather, added materials and seismic devices must fit into the structure and be capable of being built at a reasonable cost. Designers must work hard to achieve this.

There are of course many other issues that others have addressed in this paper. The bridge engineering community is pursuing its never ending quest of solutions to the problems of seismically evaluating and retrofitting suspension bridges.

5.8 COMMENTARY BY J. P. SINGH [11]

5.8.1 Issues Concerning Multiple-Support Motions

Seismic analysis of long-span bridges for new or retrofit design requires synthetic ground motions as input. Synthetic ground motions for this purpose that account for the spatial variations consist of spectra and coherency-compatible multi-directional multiple-support motions. Some important issues concerning multiple-support motions for long-span bridges are discussed below.

5.8.2 Wave fields

Seismic wave fields describe the proportion of spectral energy and characteristics of the displacement wave forms for different components of the ground motion. Although multi-directional response spectra can be obtained using empirical methods, use of analytical-empirical methods allows better estimation of spectral energy in different directions.

5.8.3 Wave passage

The wave passage effect, due to non-vertical wave propagation, induces phase shift producing out-of-phase motions at different supports. Most empirical solutions of this problem introduce time shift using apparent wave velocity along the bridge considering properties of S-waves. A better solution of wave passage should consider multiple wave types (such as P, SV, SH, L and R) using analytical-empirical methods.

5.8.4 Spatial Coherency

Spatial coherency or incoherency accounts for local wave scattering and complex 3-D wave propagation. The empirical target coherencies are based on statistical analysis of the recorded ground motion data obtained from 2-D strong motion arrays. Analytical-empirical solutions show that the coherencies at long periods (periods greater than 1/2 sec) are more deterministic in nature and reflect the wave passage and site effects. Where wave passage and site effects are significant, it is important that deterministic coherencies, obtained at longer periods from analytical-empirical studies, be preserved to maintain the wave passage and site effects and the coherencies at higher frequencies (frequencies greater than 2 Hz) could be replaced with statistical models to fit the empirical target coherencies.

5.8.5 Wave forms

The strength and character of the displacement wave forms on the two orthogonal components can be considerably different with time periods of ground motions that can vary from complete correlation to time periods of little to no correlation. The wave forms are reflective of the wave fields and their character should be preserved during development of synthetic ground motions.

5.8.6 Local Site Effects

Sites overlain by soil cover over rock half space are generally analyzed for site response using one-dimensional wave propagation methods. However, variations in local geology for rock sites are generally not considered in estimation of variations in site response. The effects of rock sites with complex geology containing rock formations with different wave speeds and varying bedding orientations (strike and dip) can be significant for determination of the site response. Recent studies performed for the Golden Gate Bridge show that geologic complexity and variations in the shear wave velocities of the rock produce significant variations in ground motions over short distances. Estimates of site response of such materials requires the use of two- and/or three dimensional models.

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5.8.7 Spectra Compatibility/Incompatibility

Current procedures require spectra compatible ground motions for use in the seismic analysis of long-span bridges. Selected time histories of ground motion are fitted to the target spectra using frequency domain and/or time domain procedures. This allows the structure to be shaken to the spectral energy fitted to the target level in all frequency bands.

Tight spectral fitting, using time domain procedures, alters the phase spectra and, thus, the shape of original wave forms of individual components and the correlation between two orthogonal components. The frequency domain fitting procedures, on the other hand, if not carefully performed can distort the displacement wave forms as well as over energize the time histories in certain frequency band well beyond the energies computed from real recordings. Careful fitting of the response spectra in frequency band domain with some acceptable tolerances, however, allows preservation of the phase spectra wave forms and realistic energy in the time history.

Spectra compatible motions may not result in maximum response of the structure in terms of differential motions (relative displacements and differential shaking between the supports). It is possible that spectra incompatible time histories, reflective of more realistic wave fields, wave passage and local site effects may give higher response of the structure in terms of differential motions.

5.8.8 Research Needs

a. Study the effects of complex rock geology on variation of ground motions. Compare the variability in site specific motions with spectra compatible ground motions to evaluate the differences in differential motions (relative displacements and shaking) and their effects on the structural response.

b. Study the coherency data to separate the deterministic and statistical coherency in different frequency bands and evaluate its effects on the structural response.

6.0 References

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