Probabilistic Seismic Assessment of a Non-conforming Structure Using Scenario Spectra

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SUMMARY:
Probabilistic seismic assessments generally rely on Uniform Hazard Spectra (UHS) to describe the ground motion for a specific site. UHS, however, represent envelopes of the many different earthquakes that control the hazard at different spectral periods. Envelopes are unrealistic representations of the demand associated with any single event and can lead to conservative estimates. Moreover, seismic assessments based on UHS can produce little more than a pass-fail assessment of the structure of interest relative to seismic risk for the envelope. As an alternative for this assessment, UHS were broken down into a suite of Conditional Mean Spectra (CMS) representing realistic earthquake scenarios, and the CMS were used to generate multiple scenario spectra to reproduce the hazard of interest. Probabilistic seismic assessments employing scenario spectra can improve estimates of seismic risk by allowing an analysis of building response for individually selected earthquake scenarios whose unique frequency dependent energy information is preserved.

Keywords: Scenario Spectra, Non-linear Static Pushover

1. PURPOSE AND SIGNIFICANCE OF THE STUDY

For decades, seismic design practitioners have explicitly or implicitly relied on Uniform Hazard Spectra (UHS) to quantify the relevant seismic hazard, whether in the course of designing new structures or assessing existing ones. Throughout this time frame, UHS have defined the seismic demand used in the seismic evaluation and design of structures, regardless of the engineering methodologies employed, whether they were code-based or performance-based, site-specific or prescribed, static or dynamic, linear or nonlinear. UHS, however, are artificial envelopes of the earthquake hazard resulting from all the faults contributing to the hazard at the site. The simplification of the hazard into a smoothed spectrum combines demands that will not occur simultaneously, which results in spectra that overstate the actual hazard at a site. While this overstatement is not necessarily significant for all buildings in an area of high seismicity, it can be significant for some buildings --- especially for buildings whose earthquake response involves the engagement of multiple systems with different flexibilities, strengths and ductilities, and multiple significant modes of response. Moreover, the UHS approach to characterizing seismic demand can generate only a pass/fail result; no structural analysis can further parse the hazard and the available methods to adjust the UHS for different probabilities of recurrence such as those in ASCE 41-06 are simply scalar adjustments (ASCE, 2006). These UHS adjustments do not account for the individual influences of the various seismic sources that contribute to the shape of the UHS.

The scenario spectra method offers significant advantages for a building owner and engineer seeking information additional to that which can be provided by a UHS representation of the hazard. These advantages include being able to assess the seismic adequacy of a building versus individual earthquake scenarios and being able to easily identify the probability of an event capable of causing a pre-defined level of damage or a risk of collapse. Scenario spectra represent individual hypothetical earthquake events that each have a postulated source, rate of occurrence, and distance from the site, in stark contrast to UHS, which aggregate a large number of events into a single spectrum.

This paper describes the implementation of the scenario spectra method, in conjunction with nonlinear static pushover methods, in an assessment project for a 1960's non-conforming high-rise steel braced-frame structure in a region of high-seismicity. The paper provides discussion of the methodology used to generate the scenario spectra, focusing on how the method can be used to provide a fault-specific and rate-specific 'discretized' assessment of the seismic risk. In addition, the paper presents discussion of the methodology used to generate realistic estimates of structural response and structural capacity of the bolted steel braced frames and riveted gravity frames in the subject structure. Realistic estimates of structural response and capacity were critical to fully leveraging the advantages of the scenario spectra method.

In this study, risk was quantified by evaluating which and how many of the earthquake scenarios would result in damage great enough to exceed a pre-selected performance criterion. Specifically, the study sought to identify the risk of structural damage significant enough to jeopardize the safety of occupants of the building. At that performance point, damage to nonstructural systems may well be significant enough to introduce risk from those elements and may well be significant enough to render the facility non-operational for an extended period of time. However, quantifying of damage to, or risk from, nonstructural systems was not an explicit goal of this project. In the end, the scenario approach provided a more detailed understanding of the fault and magnitude specific seismic inputs predicted to result in unfavorable outcomes, as well as the probability of those unfavorable outcomes, than was possible using the UHS approach. The scenario spectra method achieves this by allowing for an analysis of building response for individually selected earthquakes and by preserving the frequency dependent energy content information of specific earthquake events.

2. SCENARIO SPECTRA METHOD

The UHS is an envelope of earthquakes: de-aggregation of a UHS reveals which earthquakes control which part of the spectrum. For example, short period spectrum values may be controlled by low magnitude nearby earthquakes, whereas long period spectrum values may be dominated by large magnitude distant earthquakes. Even though the hazard values for individual periods have the same rates of occurrence, they will not be realized in a single earthquake. Hence, application of such a frequency rich spectrum results in exciting multiple modes of a structure simultaneously – more than what any single earthquake would be able to.

Through conditional mean spectra (CMS), as developed by Baker and Cornell (2006), the seismic hazard can be realistically depicted for any spectral periods of interest – most often the fundamental period of the structure and higher modes deemed significant. However, CMS do not readily allow for calculation of risk because, for example, a CMS derived from a UHS with a 2,475 year return period and conditioned at a spectral period of 0.2 seconds also contributes to hazard at a spectral period of 0.5 seconds for a 1,000 year return period. Hence, to accurately capture the rate of occurrence from a CMS, a set of "scenarios" need to be developed that collectively recreate the hazard at the spectral period ranges of interest.

The following steps outline the implementation of the scenario spectra method:

1. Develop hazard curves for spectral periods of interest.
   Hazard curves can be directly used from 2008 USGS seismic hazards data. Because the structure of interest on this project was located on a hard rock site, a complete probabilistic seismic hazard analysis using Next Generation Attenuation (NGA) relationships was conducted to generate the necessary hazard curves (Abrahamson and Silva, 2008). Figure 1a illustrates hazard curves for contributing faults for a period of 0.75 sec.

2. Develop a corresponding suite of UHS for all return periods of interest.
   For this project, nine UHS were created using hazard curves from Step 1 for a suite of return periods between 50 years and 10,000 years. See Figure 1b.

3. De-aggregate the hazard curves.
   De-aggregation of the hazard curves provides information about which events control the hazard at a given ground motion level by quantifying the percent contribution of specific earthquakes (magnitude-distance pairs) to the hazard at desired spectral period(s) for a given return period. For the project, hazard de-aggregation was conducted at the structure's fundamental period for a suite of return periods ranging from 100 years to 10,000 years. Figure 2 shows the hazard de-aggregation for the structure's fundamental period (0.75 sec.) at the 2,000 year return period.

4. Develop CMS for each return period of interest.
   From the hazard de-aggregation results for desired spectral period, the controlling magnitude and distance is used to calculate the median ground motion (for each return period). From each of these median spectra (anchored at the desired spectral period), the number of standard deviations (epsilons) that the UHS is above the respective mean spectrum is calculated for spectral periods other than the anchored period. Finally, the mean epsilon value is calculated accounting for correlations of variability between two spectral periods. A CMS is constructed using the median spectrum and the associated mean epsilons (Abrahamson and Yunatci, 2010). See Figure 3a for the CMS for a 2,000 year return period. In contrast to a UHS, the CMS accounts for the fact that different earthquakes contribute to the hazard at different spectral periods – the very definition of "conditional" – since the median ground motion is conditioned on spectral period of interest. The CMS also accounts for ground motion variability at different spectral periods – that is, it incorporates the correlations between the median ground motion level at the anchored period and the expected ground motion level at other spectral periods. Specifically, CMS use the expected (hence the term "mean") number of standard deviations above the median - the epsilon values – based on residuals from the attenuation relationships. For the project, twelve CMS were developed using the de-aggregation results and the residuals from NGA relations.

5. Develop scenario spectra from each CMS.
   Though the CMS accounts for ground motion variability at different spectral periods, it only captures the "mean" values of variability – that is, the CMS fails to represent the peaks and troughs in the spectra. See Figure 3a. A suite of scenario spectra can be used to capture this variability, however, care must be taken to account for correlations because the variability is not independent for each period. A scenario spectrum is essentially a conditional spectrum with an epsilon value other than the mean. For the project, a spherical semi-variogram model was used to generate the correlated epsilon values (Walling, 2009). The generated epsilon values from this model were then anchored to a reference epsilon value – one that represents the number of standard deviations above the median spectral value needed to reach the UHS at the desired period of interest. This procedure ensures that the hazard curve at the desired period is fully recaptured. Using the process described above, ten scenario spectra were developed for each of the twelve CMS for a total of 120 scenario spectra. Figure 3b illustrates scenario spectra derived from one CMS. For each scenario spectra, the rate of occurrence was calculated from the hazard curve at the desired period of interest distributed evenly over the number of realizations. For the project, the hazard curve at T=0.75 sec. was used in conjunction with the ground motion values [Sa(T=0.75 sec.)] associated with each suite of ten scenario spectra to calculate the cumulative rate of all scenarios. This cumulative rate was then divided evenly over all ten realizations.

6. Check hazard capture at different spectral periods.
   New hazard curves from the developed scenario spectra are created and compared to original hazard curves at various spectral periods. To limit computational burden, only 120 scenario

spectra were used in the project. A larger number of scenario spectra would be able to capture the hazard at all spectral periods; however, with the limitation of 120 spectra, the fit at the desired period of interest was prioritized. Furthermore, because there was a secondary period of interest (short period range), the hazard recapture was checked carefully at both these periods. The process of generating 120 random spectra was iterated until hazard capture was acceptable at both the desired periods of interest. Figure 3c illustrates a check on this hazard capture at a period of 0.75 sec.

7. Modify the developed scenario spectra to suit the purpose of analyses.
   The scenario spectra were used to determine performance of the structure by visually comparing them to the pushover curve. For the project, each of the 120 spectra was modified with applicable damping values to account for effects of structural ductility (see discussion below). Figure 4 illustrates a comparison of two scenario spectra to two unidirectional pushover curves for the building.

8. Calculate the potential risk for severe structural damage or other criterion of interest.
   For each of the scenario spectra developed, a rate of occurrence was calculated that corresponds to the likelihood of the earthquake event. The summed rates of occurrence for the earthquake events that will cause severe structural damage was used to describe the potential risk for the structure.

The procedure above describes one method to apply the theoretical concept of scenario spectra in practice to precisely calculate the seismic risk for a structure. Limitations include selection of the small subset necessary to make the procedure manageable while simultaneously capturing the hazard at the desired periods of interest. However, the generation and visual depiction of scenario spectra can be automated which will allow for capturing hazard levels at multiple spectral periods instead of merely two as was done in the project.

3. DESCRIPTION OF SUBJECT STRUCTURE

The subject structure was designed in 1963 and is a steel-framed structure, with steel braced frames. Its 7-story tower is situated on a moderately sloping site and rises from a 1-story base structure whose plan is significantly larger than the footprint of the tower; the original plan of the base structure has been infilled over the years by various additions without seismic separations. A basement, larger in plan than the tower, is also present. The typical floors of the tower are constructed of ultra-lightweight concrete fill with a field-verified density of 4.4 kN/m^2 [91.5 psf] and steel decking, which is welded to and supported by wide flange steel beams and girders, and in turn by steel columns. At the base of the building, the steel columns are supported by a variety of reinforced concrete elements which, depending on its location on the sloping site and role of the column, are either spread footings, continuous strip footings/grade beams, or pilasters. The building foundation is supported on rock.

As originally conceived by the engineer-of-record, the seismic resistance for the building was provided by eight double-angle braced frame bays located at the corners of the tower, four aligned with each of the primary axes of the building. The bracing itself is configured as “x-bracing” and is connected via a variety of structural components including gusset plates, angles, WT sections, bolts and rivets. The steel bracing was designed to be connected to the beams and columns bounding the frames by “high-strength bolts”, a relatively contemporary construction innovation that was rapidly supplanting riveting as a connection technology at that time. The in-field measured length of the bolts indicated that the threaded portions of the bolts were excluded from both shear planes, thus providing greater seismic capacity than might otherwise have been assumed.

In actuality, however, the seismic resistance of the subject structure as a whole is supplemented by other building components, both structural and nonstructural as is the case for the majority of buildings, but is not always accounted for in design or assessment. The supplementary systems in the subject building include the framing of the primary steel gravity support, the interior partitions where they extend from floor to deck, and the exterior cladding. The gravity framing was likely not intended to participate in resisting earthquake forces by the original design engineer, however, it can be relied on to resist some of the lateral forces. As designed, the primary steel gravity support framing, i.e. the columns, beams and girders located on column lines, consisted of riveted steel framing. Instead of rivets connecting the girder and beam webs to the supporting angle as was shown on the structural drawings, one side of the web connection was field-verified to have been made with perimeter fillet welds which were estimated to be 6 to 8 mm. [1/4 to 5/16 in.] in size. The opposite half of the web connection was made with A-325 bolts to the supporting column.

There are a number of types of interior partitions throughout the building, not all of which are capable of contributing substantially to the seismic resistance of the structure. Only some of the original partitions extend from floor to deck; notably those either along main longitudinal corridors, at elevator shafts, mechanical shafts and stairwells, or surrounding the vestibule located roughly in the middle of the building. The partition walls original to the building were sheathed with plaster on metal lath, and were supported by truss-like metal "studs." Originally, gypsum plaster was used in most interior locations, while cement plaster (stucco) was used in exterior locations. On remodeled floors, the partitions are constructed of gypsum board on cold-formed steel studs.

The exterior cladding, which in places spans floor-to-floor, is composed of stucco spandrel panels on the longitudinal perimeter walls of the tower and of mostly solid panels on the ends of the tower. These are also likely to participate in the resistance of earthquake forces to some degree. As designed, the cladding was constructed of stucco on metal lath on the exterior and gypsum plaster sheathing on the interior. The stucco is supported on steel studs, which frame to steel channel sections top and bottom. These channels are in turn connected to the perimeter steel columns.

Accurate characterization of the structure is of paramount importance in performance-based assessments since the performance is entirely contingent on in-situ conditions.

4. STRUCTURAL ANALYSES

Numerous component and global analyses were conducted in order to quantify the reliable seismic resistance of the subject structure. The premise of the seismic assessment necessitated a global analysis of the building that explicitly considered a variety of potential modes of nonlinear behaviors, including connection slip, foundation uplift, damage to structural components, and/or damage to nonstructural components, specifically the interior partitions and exterior cladding. In addition, accurate characterization of the relative rigidity of the various systems in the building was necessary to quantify the resistance that could be mobilized at increasing levels of drift. For example, under a purely elastic analysis, the braces would be predicted to buckle long before any substantial contribution of the flexible gravity frames could be achieved; however, by virtue of the potential for uplift and connection slip, the braced bays were found to be far more flexible than traditional analysis would allow. To accomplish the assessment, the behavior of components potentially subject to deformations that would exceed their elastic capacities was required to be modeled in a manner that enabled explicit consideration of post-elastic response. Therefore, SAP2000 Version 14 was chosen as the analysis environment. SAP2000 is well-suited for modeling the specific load-deformation relationships for the typical assemblies in the subject structure. The assemblies modeled to permit nonlinear behavior included the bolted braced frame assemblies in tension and compression, the welded/bolted beam-column connections, and the stucco and gypsum sheathed exterior cladding and interior partitions. Nonlinear behavior of the bracing assemblies was explicitly modeled using nonlinear links in series with beam elements representing the elastic portions of the assemblies. The characteristics of the nonlinear links employed in the bracing assemblies were developed via separate analyses in order to simulate the behaviors of bolt-slip and flexing of the gusset assemblies and flanges. All of the beam-column connections were initially modeled with nonlinear hinges to allow for post-elastic behavior. However, initial analyses showed that the majority of beam-column connections away from the braced bays would not yield except under very large displacements, and therefore the majority of the typical beam-column connections were modeled for subsequent analyses as fixed-end connections.

Nonlinear behavior of stucco and gypsum-sheathed assemblies was modeled using secant stiffness, which was adjusted manually to account for predicted variation in building drift over the building height. In addition, nonlinear behavior associated with foundation uplift was modeled using gap elements. Analyses in which the presence of all stucco and gypsum sheathing was eliminated were conducted to assist in understanding the contribution of the sheathing to strength and stiffness. In addition, analyses of the tower alone --- without the contribution of the surrounding ground story construction --- were also conducted.

The analyses performed in SAP2000 included both linear response spectrum analysis and the “Capacity Spectrum Method”, commonly identified as nonlinear static, or pushover, analysis. The Capacity Spectrum Method is an analytical/graphical means for comparing the force and deformation capacity of a building to the force and displacement demands imposed by an earthquake. The method involves developing a “pushover curve” of the structure that relates the lateral forces to the horizontal displacement, by considering its strength and displacement capacity. The pushover curve is then superimposed on a spectral plot of the earthquake demand under consideration, preferably in ADRS format (Mahaney et al., 1993). If the pushover extends beyond the earthquake demand spectrum, the building has the capacity to sustain that demand. Multiple iterations of analysis were conducted in order to better understand the significance of the contributions from each of the systems of interest and to vet the behavior of the model. For example, analysis runs were made with and without gap elements at the foundation levels, with and without partitions, with and without hinges at the beam-column joints, and with different assumptions about the contribution of these systems. These analyses employed “scenario spectra” as the primary basis for defining the intensity of earthquake ground shaking to which the building would be subjected, but also included a “code spectrum” to benchmark the results of the scenario spectra analyses.

Analyses that accounted for the various assumptions described above were conducted, as well as analyses of a model developed exclusively for a stability check in which no braces, partitions or cladding was modeled. That model relied only on the resistance of the three-dimensional beam-column framing for lateral stability. Pushover curves for two primary modes of vibration in each orthogonal building direction were developed and compared to the code UHS and each of the 120 scenario spectra. Analyses were also conducted in which the subject structure was loaded in two directions simultaneously using a 100% plus 30% load combination.

The 120 scenario spectra that were employed in the scenario spectra analyses represented discrete hypothetical events, each with a quantifiable risk. An illustration of the use of the Capacity Spectrum Method using scenario spectra as implemented in this study are provided in Figure 4, which depicts a the longitudinal and transverse unidirectional pushover curves for the subject structure versus two of the many scenario spectra used in the assessment. The two scenario spectra used in this figure are scenario number 33, a Magnitude 6.69 earthquake at a distance of 9 km from the subject building (red curve), and scenario number 84, a Magnitude 6.78 earthquake at a distance of 5 km from the building (green curve). The subject structure is considered to pass the spectrum for scenario number 33 because at a spectral displacement around two inches, its pushover curves cross and continue above (punch through) the scenario spectrum. However, the pushover curves do not “punch through” the spectrum for scenario number 84, which may mean that scenario number 84 is problematic for the building.

It is important to note that in developing the pushover analyses, we did not attempt to push the structure to an extreme displacement; rather, the decision as to where to terminate each pushover was governed by considering the computational time necessary to complete the analyses and by considering the reliability of the results, which diminishes as the nonlinearity of the behavior increases. Computation time also grows exponentially as the number of nonlinear elements in the model increases, and as the nonlinear elements are pushed out to the limits of their capacity. As such, it is believed that the building has displacement capacity beyond that which has been quantified using these analyses, and that the risk of structural damage significant enough to jeopardize the safety of occupants is likely less than has been identified. In short, the endpoints of our pushovers do not represent collapse. Moreover, we are reporting results at performance points that are less severe even than those at which the model remained mathematically stable. At the same time and as described above, relative to the endpoint of the pushover curves, it is important to keep in mind that although the structural analysis may indicate that the structural damage has not proceeded to the point that occupant safety might be endangered, damage to nonstructural systems may well be significant enough to introduce risk and may well be significant enough to render the facility non-operational for an extended period of time. Quantifying of damage to nonstructural systems, however, was not an explicit goal of this project.

5. CALCULATION OF RISK

Each of the 120 scenario spectra employed is associated with a particular ground shaking scenario, i.e. an earthquake magnitude, an epicentral distance from the property, and an assigned rate of occurrence per year. For each of the 120 scenarios, two random orthogonal horizontal orientations of motion as well as the geo-mean were produced. The spectra represent earthquakes on a variety of faults in the region of the subject structure, and were developed subsequent to analysis-based identification of the structural periods of vibration of primary interest. The rate of occurrence assigned for each scenario is a means of describing the likelihood of its occurrence in any given year. From initial examination of these spectra, it was observed that roughly two-thirds of the scenarios have the potential to cause structural damage of concern to the subject structure. From the rates associated with these scenarios, however, it is equally clear that these especially damaging scenarios are extraordinarily unlikely events that contribute relatively little to the site-specific seismic hazard.

By comparing the pushover curves to each of the scenario spectra, a suite of scenarios thought to have potential to cause severe structural damage that is likely to jeopardize the safety of occupants was identified. Two methods were used to compare the pushover curves to the scenario spectra. In the first method, the pushover curves for the models that were loaded uni-directionally were compared to the larger component of each scenario spectrum and to the geo-mean spectra multiplied by a factor of 1.3, which accounts for bi-directional loading effects. In the second method, the pushover curves for the model that was loaded bi-directionally were compared with the geo-mean scenario spectra. In general, a conservative estimate of 3.6 cm. [1.4 in.] of interstory drift for the critical direction was determined to be attainable prior to the onset of structural damage of potential concern to occupant safety. The 3.6 cm. value (which corresponds to an interstory drift ratio of approximately 1%) comes from the nonlinear analyses as the response of each structural element and connection was tracked; 3.6 cm. of interstory drift coincides with the point at which a number of elements and connections reached a stress or deformation level that was believed to represent the maximum stress or deformation that could be tolerated. At this level of drift, the plaster and gypsum sheathing in the full-height partitions and in the spandrel panels throughout the building would experience significant cracking; it is believed that they would contribute a considerable amount of energy dissipation to the building response, in addition to that provided by other portions of the yielding structure. Therefore, as a final step prior to making the comparisons between the scenario spectra and the pushover curves, we adjusted the scenario spectra to account for 15 percent damping. Whereas elastic analyses traditionally employ a 5 percent damped spectrum, in buildings that experience modest amounts of damage, the effective damping in the building increases. The choice to use 15 percent damping (or a ductility of approximately 1.5) is an engineering judgment, but is based on published literature and testing. Using the above methodology, the combined risk of severe structural damage that might reasonably be judged to endanger the safety of building occupants was computed as the sum of the rates associated with the suite of events identified.

After comparing the calculated pushover curves to each of the 120 scenario spectra, we concluded that the potential risk of severe structural damage that might reasonably be judged to endanger the safety of building occupants --- the summed rates --- is about 1:2900 in a one year period for method one and

1:2700 in a one year period for method two. It should be noted that by making detailed efforts to calculate the consequences of forcing the structure further out along its pushover curve, this computed risk could likely have been reduced even further. However, it was elected to terminate the analyses at a level at which the building response could be reasonably well captured by currently available analysis methods. It should be understood that at this level of response, damage to structural elements has occurred, including but not limited to some of the steel framing and connections at the base of the structure. This damage would not present a specific safety hazard to occupants; rather, at this level of response, there is probably far greater risk to occupants due to the performance of nonstructural systems than from the behavior of the structural elements themselves.

To further clarify the computed risk, we compared the above-described pushover curves to a code spectrum in accordance with the methods described above, and evaluated the results of the stability analysis, performed without braces, partitions and cladding. The point of the analysis was to check stability assuming that the gravity frame alone is available to resist P-delta effects. The analyses indicate that the building is adequate to withstand the code design spectrum, conservatively considered at 5 percent damping. The code spectrum imposes a displacement demand equivalent to 15.2 cm. [6 in.] of roof displacement in the longitudinal direction and 13.2 cm. [5.2 in.] in the transverse direction. From the stability analysis, we estimated that the beam-column frames are adequate to provide stability out to a roof drift of about 30.5 cm. [12 in.] under the conditions assumed, well in excess of the estimated 15.2 cm. [6 in.] maximum displacement demand.

6. CONCLUSIONS

The scenario spectra method provides a valuable tool for identifying the source and extent of seismic hazard for a particular structure with much more specificity than is permitted by building code uniform hazard spectra. For the project discussed, this methodology provided the client with key information to assist in decision-making on a particular existing building.

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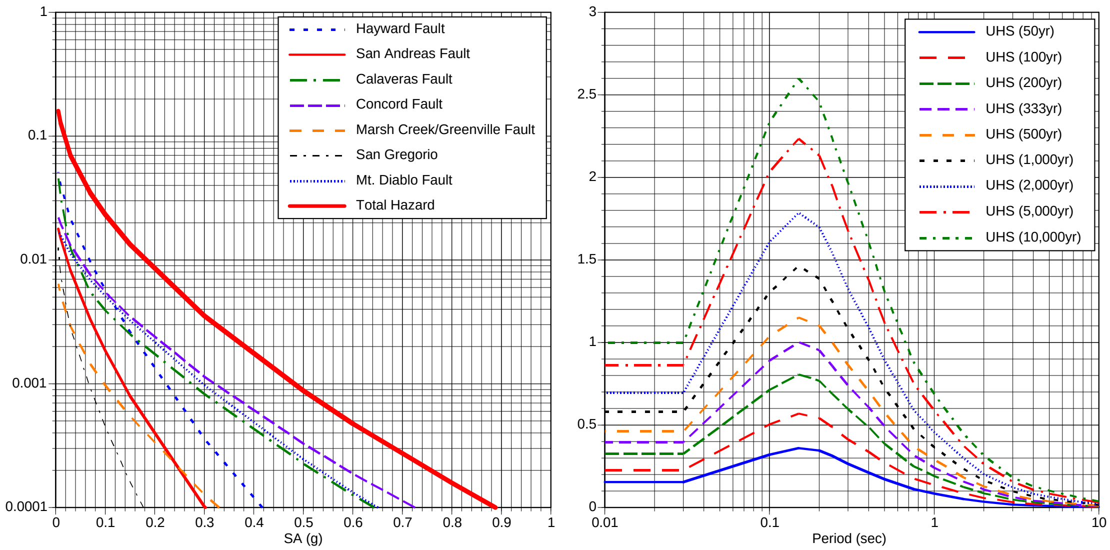


(a) **(b)**

Figure 1. (a) Hazard curve for contributing faults at T=0.75 sec., plotted as spectral acceleration versus probability of exceedance in one year. (b) Spectra for various hazard levels, plotted as period versus S_a.

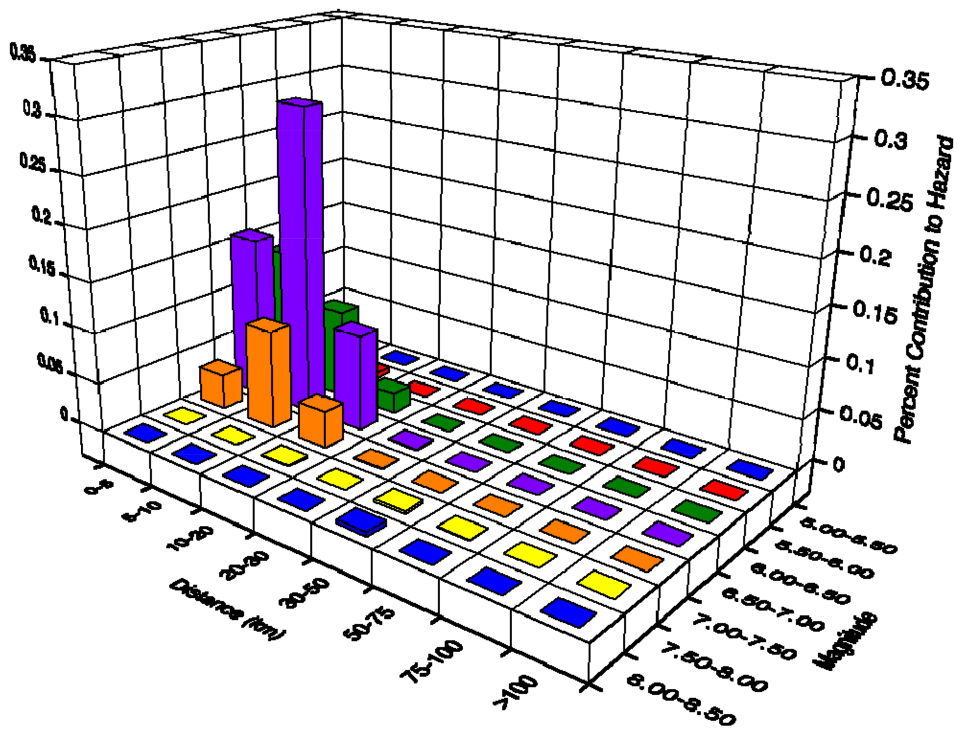
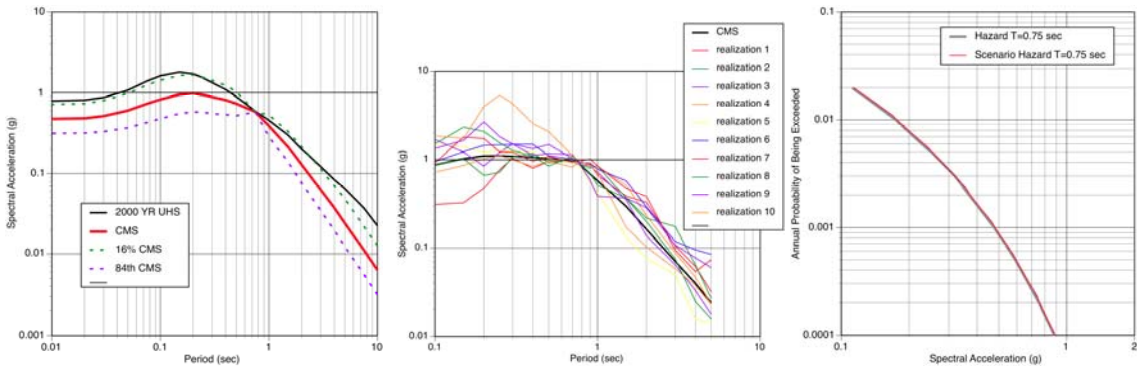


Figure 2. Hazard de-aggregation for T=0.75 sec. spectral period at the 2,000 year return period.



(a) (b) (c)

Figure 3. (a) CMS constructed from the hazard de-aggregation for T=0.75 sec. and a 2,000 year return period. (b) Scenario spectra constructed for one of the CMS. (c) Hazard curve recreated from the 120 scenarios as a check for T=0.75 sec.

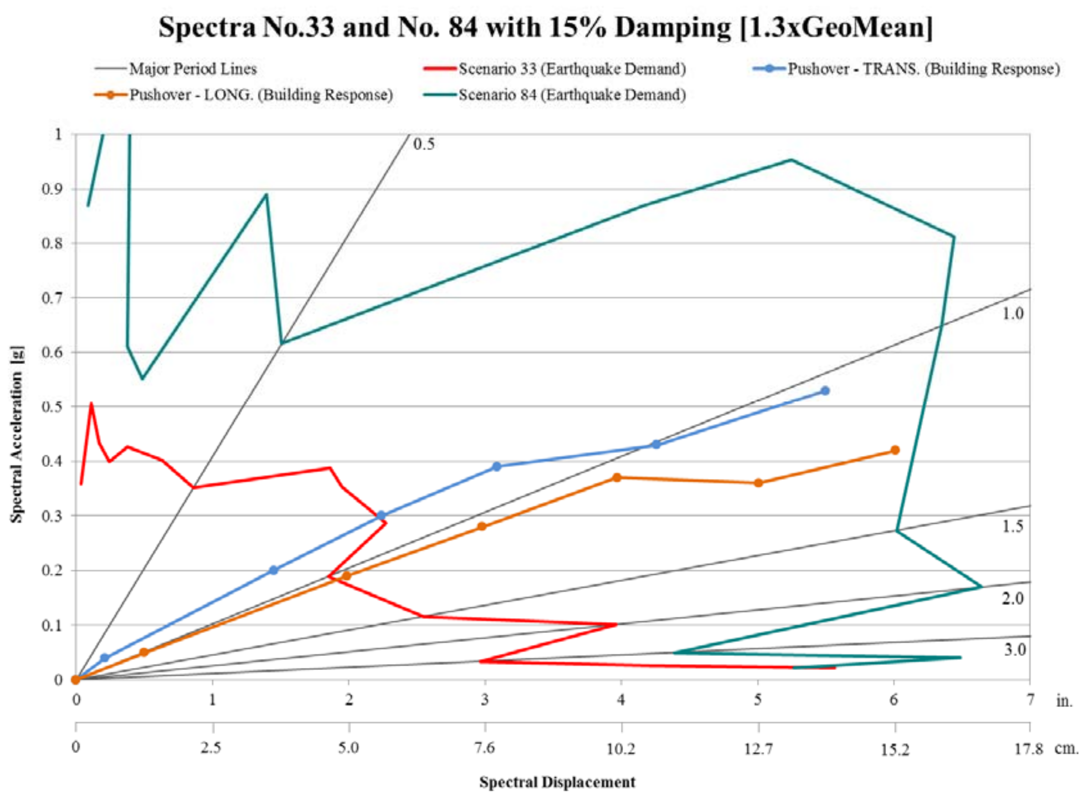


Figure 4. ADRS plot of unidirectional pushover curves and scenario earthquake spectra.