EVALUATION OF GROUND-MOTION SELECTION & MODIFICATION METHODS USING INELASTIC-RESPONSE HAZARD CURVES

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Abstract

Performance assessments of the built environment using numerical simulation uses suites of ground motions which are selected, scaled and modified such that their energy content matches, or exceeds, that of a target spectrum, which is computed from a probabilistic seismic hazard analysis. There are many methods of selecting, scaling and/or modifying ground motions in engineering practice today. However, there is no objective methodology to determine whether the suite of ground motion corresponds to the same level of hazard and/or risk as the target spectrum. This paper presents a through methodology where different ground-motion suites are evaluated using hazard curves (annual rate of exceedance) for inelastic response parameters instead of the elastic models used in the PSHA. Specifically, the paper evaluates three different methods of ground-motion modification: simple amplitude scaling, tight spectral matching (at the record level), and mean spectral matching (at the suite-average level). While tightly-matched records have no dispersion in the elastic spectral response, the records obtained from the other two methods do. Also, the mean of the spectra of the amplitude-scaled records are expected to be conservative in comparison to those obtained using the other two methods. These three different methods are, therefore, compared in the inelastic realm on the basis of the mean response of the suite as well as the dispersion of the results. The quantitative measure of comparison is based on probabilistic metrics rather than response measures.

Keywords: seismic risk; seismic hazard

1. Introduction

Nonlinear response history analysis is performed either as part of the design of a structure to size and detail members, or as part of the performance assessment of an existing structure or design. In the analysis a three-dimensional structural model, which characterizes the nonlinear inelastic behavior of all critical elements, is subjected to a suite of ground motions that represent a prescribed seismic hazard/risk level, in accordance with the governing building code. The response of the numerical model to the seismic excitation is compared to a set of prescribed limit-state criteria, such as deformation/damage limit states, to either design the members or assess whether the performance objectives are met. Modern codes set probabilistic performance objectives, such a 1% probability of collapse in 50 years.

Ideally, the analist would run a large suite of ground motions that represents the seismic activity at the site during the lifetime of the structure and check the limit states against the performance objectives. However, because the computational effort of running the hundreds of simulations of all amplitudes on a complex numerical model exceeds current capabilities and time constraints, the building code takes a simplified probabilistic approach to represent the seismicity of the site by prescribing a limited suite of ground motions, in the order of 10+, that envelope the hazard/risk level of interest. This simplification is a reasonable simplification when the objective of the analysis it to assess the response at a single hazard level or limit state.

This paper introduces a parallel procedure whose objective is to determine whether the limited suite of ground motions is consistent with the prescribed hazard level (demand) and limit states (response) corresponding to the the performance of objectives of the building code – limiting the probability of exceeding a specified limit state (risk of collapse). The procedure is based on first determining the large set of ground motions that is consistent with the hazard at the site and represents a possible seismicity scenario for the lifetime of the structure. The second step in the procedure is to run a simplified (2D coupled/uncoupled SDOF) nonlinear inelastic structural model to compute inelastic hazard/risk curves on key engineering demand/response parameters, such as ductilitiy. To evaluate the simplified method prescribed by the code, the same simplified structural model is also subjected to the code-compliant suite of ground motions being evaluated. The response of the structure to this suite is compared to that of the larger set to determine the rate of exceedance of the limit states reached by this this suite and determine whether it is consistent with the performance objectives prescribed by the building code.

To demonstrate the process, three different suites of ground-motions at three different sites were evaluated. The three suites of ground-motion correspond to three different methods of ground-motion modification – amplitude scaling, spectral matching, and mean spectrum modification. The three sites across the San Francisco Bay area were selected to span different, but still comparable, types of seismicity. At the time of publication, the findings in this paper are only meant to be qualitative to demonstrate the procedure and no conclusions on the actual ground-motions suites can be drawn.

2. Ground Motions for Nonlinear Response History Analysis

The intensity measure used by the building code to characterize the ground motions is the pseudo-acceleration (Sa) elastic 5%-damped design spectrum. The most recent building code is the upcoming ASCE 7-16, currently in its final stages of publication. This code sets the target spectrum to correspond to the MCE_R – risk targeted maximum-direction design spectrum – defined as the lower of the deterministic spectrum (enveloping the characteristic scenarios for the nearby faults) and the uniform-risk probabilistic spectrum. The probabilistic spectrum is defined as the spectral acceleration that corresponds to a 1% probability of collapse in 50 years, typically computed by modifying the 2%-probability of exceedance in 50 years uniform-hazard spectrum by the risk coefficients provided by USGS in its Design Maps tool [1]. These risk coefficients were determined by USGS by iterating on the integration of a generalized fragility curve and the elastic hazard curve (the annual rate of exceeding a ground motion) at each period. These hazard curves, which are also used to compute the uniform-hazard spectrum, are determined in the probabilistic seismic-hazard analysis using ground-motion models for elastic acceleration response spectra.

ASCE 7-16 requires that the suite of ground motions be selected and modified such that the records have similar seismological characteristics as the site of interest and that the average of the maximum-direction response spectra for the ground-motion pairs in the suite be of the same amplitude as the MCE_R target spectrum over a code-specified period range of interest. Specifically, if the records are amplitude scaled the average of the maximum component of the ground-motion pairs must generally match or exceed the MCE_R spectrum in the specified period range, with no point less than 90% of the target spectrum. If spectrally-matched, the average of the maximum direction spectra for the suite must match or exceed 110% of the target spectrum. The 10% reduction given to amplitude-scaled records accounts for the peaks and troughs in the spectra, which make it difficult to match the target spectrum closely. The 10% penalty on the spectrally-matched records is due to the concern that changes to the frequency content of the records and the loss of record-to-record variability in spectrally-matched records leads to a misrepresentation of the seismic risk (the combination of hazard and response). To maintain consistency in risk quantities, the target spectrum for the ground motions used in this project was the uniform-risk probabilstic spectrum, without consideration of the deterministic spectrum.

Ground motions are amplitude scaled by applying a scalar factor to the acceleration trace in the time domain. This modification only affects the amplitude of the response without modifying the other seismological characteristics, nor the shape of the response spectrum. Scaled records maintain the record-to-record variability as well as the period-to-period variability, resulting in a spectral shape that has peaks and troughs as well as a "preferred" period range (more energy in a certain period range at less in others). The main disadvantages of scaling records are the loss of the relationship between amplitude and duration and running the risk of high spectral peaks over-exciting higher modes. Spectrally-matched records, on the other hand, are modified such that the shape of the response spectrum matches the target spectrum. Spectrally-matched records are consistent with the seismic-hazard analysis but do not preserve record-to-record variability in spectral accelerations of recorded motions, nor period-to-period variability. In addition, the spectral shape of records matched to a UHS/URS are not consistent with observed correlations between spectral accelerations at different spectral periods. This phenomenon is expected to lead to a bias in response. Nonstationary characteristics of records, such as velocity and displacements, may also be altered significantly by spectral-matching techniques.

Amplitude scaling results in an acceptable suite when the number of records is in the order of 20 or more records. Spectrally-matched records result in an acceptable suite when the assessment only considers mean response. A hybrid kind of spectrum modification was developed by the author [2] and is being used in practice because it yields an optimal suite when the number of records is less than 15. This Mean Spectrum Modification method modifies the frequency content of the records to ensure that only the average of the suite closely matches the target spectrum while maintaining the record-to record, and period-to-period variability.

Fig. 1 Location of Study Sites

3. Study Sites & Ground Motions

The three sites selected for the study are shown in Fig. 1. These sites were chosen because of their location relative to seismic sources with different seismicity characteristics, as shown by the deaggregation data shown in Fig. 2 and the uniform-risk spectra shown in Fig. 3. The geographic distribution of seismic sources and their contribution to the hazard for the GGB and SFO sites is very similar so only one site is shown. Even though the sites have different velocity profiles, the same shear-wave-velocity value of Vs_{30}=400m/sec was assigned to all of these sites. None of these sites is less than 5km from the fault that controls the hazard.

A site-specific probabilistic seismic-hazard analysis (PSHA) was performed for each site using the latest version of the hazard program Haz45, developed by Norm Abrahamson, to obtain the uniform-hazard spectrum and deaggregation data. The risk coefficients for these sites were obtained from the USGS Design Map Tool. To maintain consistency with risk calculations, only the probabilistic Uniform-Risk Spectrum (URS), with a 1% probabilisty of collapse in 50 years, was used in determining the MCE_R for the sites in this study. The design spectrum for many of these sites may actually controlled by the deterministic spectrum, leading to spectra values that are lower than what is shown in this paper, and thus higher risk values. The rotD50 spectrum obtained from the PSHA was rotated to the maximum-component spectrum by using the code-recommended amplification factors of 1.1 at T<=0.2 sec and 1.3 at T>=1 sec, with a linear interpolation in between. This spectrum was used as the target spectrum for the ground motions at each site. The target spectra for all the sites are shown in Fig. 3.

Using the USGS deaggregation data [3], shown in Figure 2, as well as the deaggregation data from the PSHA, individual suites of 10 ground-motion pairs were selected for each site. The ground motions were scaled and/or modified according to the prescriptions of ASCE 7-16 at the time of writing, with a period range spanning the entire period range of 0.01 to 10 seconds. The amplitude-scaled suite was scaled such that the average of maximum-component response spectra closely matched the target URS spectrum, without falling below 90% of the target spectrum. The modified records, both the spectrally matched ones and the mean-spectrum modified ones, were modified to a target spectrum of 110% of the URS for each site. An example of the ground-motion suites is shown inFig. 4. The Amplitude Scaling and Mean Spectrum Modification suites maintain variability in the elastic response, as is shown in the figure, while the Spectrally-Matched records have no variability.

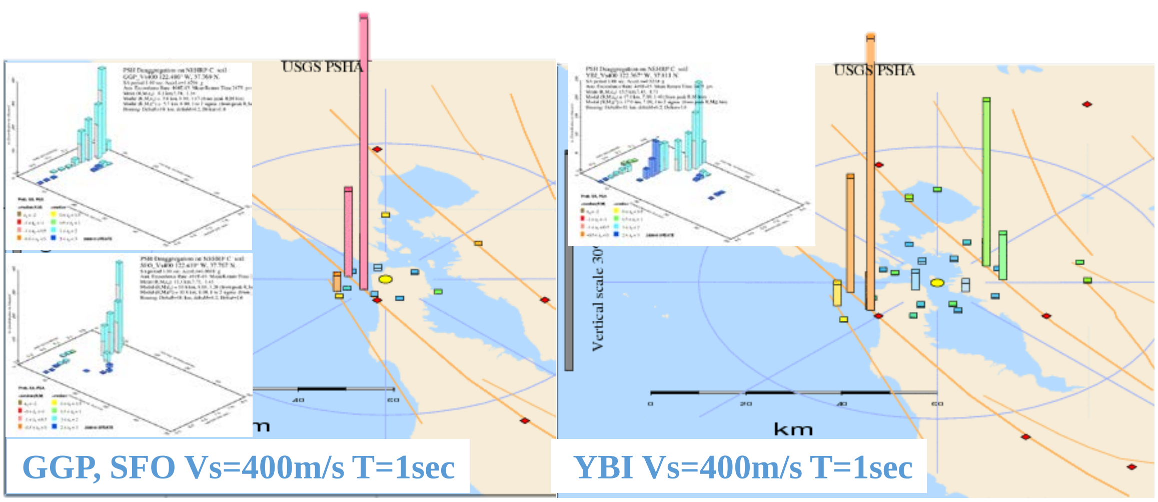


Fig. 2 USGS PSHA Deaggregation at Study Sites [3]

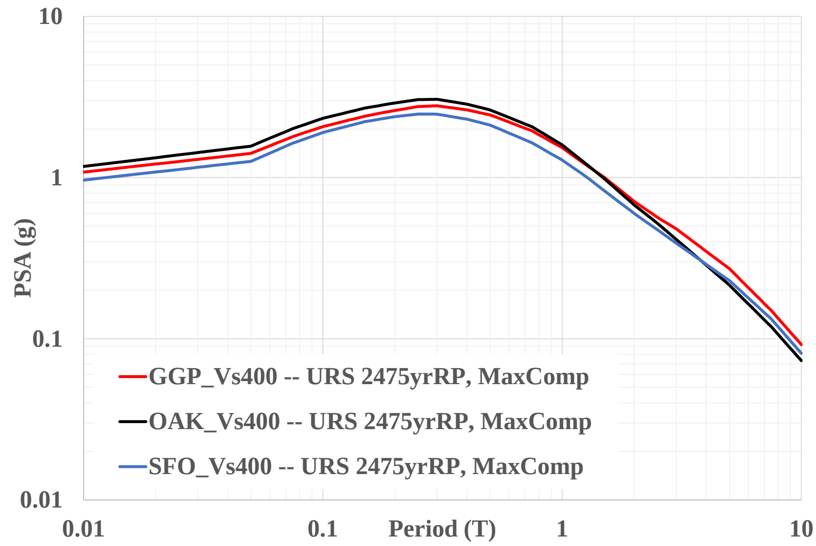


Fig. 3. Site-Specific Uniform-Risk Spectra

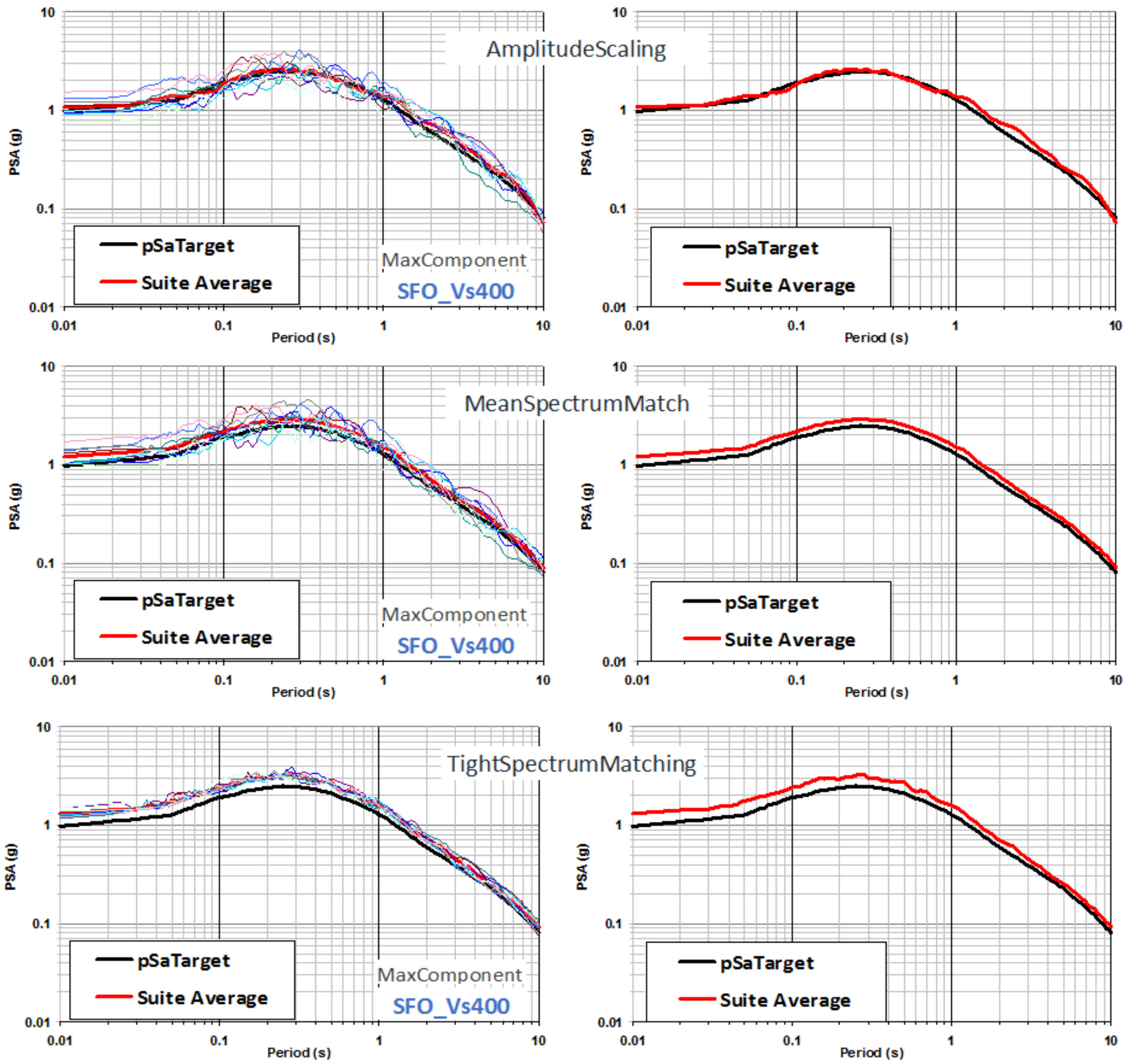


Fig. 4 Ground-Motion Suites for SFO site

4. Nonlinear Inelastic Structural Model

The structural model used in the design or performance assessment is a highly-detailed 3D MDOF model which includes all elements that are part of the gravity and lateral-load resisting systems. Each element that is expected to undergo nonlinear inelastic deformation is properly represented in the model, including the hysteretic behavior and strength and stiffness degradation. This high level of detail in the model is necessary to determine how the system response affects each structural element, and vice-versa. The amount of time required to build, calibrate and validate this model is significant, as is the amount of time needed to run the analyses and post-process and interpret the results. In a design process, this time is multiplied by the number of design iterations and load combinations.

The large number of scenario analyses used to compute the inelastic hazard curves precludes the use of a detailed structural model to evaluate the ground-motion suite. In addition, the structural model of the building may not yet be available at the time of the ground-motion selection. The scenario analyses, therefore, need to be performed on a simplified nonlinear inelastic 2D SDOF model that closely represents the global response of the structure of interest. The response of the two directions of response may be coupled (circular yield surface) or uncoupled (square yield surface). A coupled model was used in the work presented in this paper. To ensure that the structure is represented in the evaluation of the ground-motion suite, a range of SDOF models needs to be used, with variations on both the fundamental period and the strength. The range of periods and strengths presented in this paper is more extensive than needed for a single site.

The generalized nonlinear structural model used in this demonstration is shown in Fig. 5, with the analyses performed using OpenSees [4]. This generalized model is consistent with what is assumed in ASCE 7-16. This model was chosen because it represents the critical characteristics of inelastic response – nonlinearity, hysteresis, strength and stiffness degradation due to cycling and ductility, shown in the figure insert. The main variables in the model are as follows:

- T: Fundamental elastic period (range: 0.01sec-10sec)
- ξ: Damping Ratio (% of critical). Elastic Model: 5% Inelastic Model: 2% (hysteresis is modeled directly)
- MCE_R: Risk-Targeted MCE spectrum, taken equal to the Probabilistic Uniform-Risk Spectrum in this study, with 1% probability of exceedance in 50yr
- Elastic Design Spectrum: Defined as 2/3 of MCE_R, as is done by ASCE 7-16.
- V_E: Elastic Lateral-force demand: spectra spectral acceleration from elastic design spectrum (the 2/3 factor was kept for consistency with ASCE 7-16 definitions) (This value is site-specific and period-dependent)
- V_y: Yield strength of model.
- Δ_y: Yield displacement of model
- R_d: Estimated-Strength Reduction Factor: the ratio between the yield strength and the elastic lateral-force demand V_y/V_E. This value is a combination of the strength-reduction factor R and the strength-amplification factor Ω. (Range: 0.5-10) Values of Rd<1 correspond to a structure designed to yield above $2/3MCE_R$)
- C_y: Strength Factor: the ratio between the yield strength and the MCE demand. $R_d = C_y/(2/3)$. E.g. when Cy=1, Rd=1.5.
- μ: Displacement ductility capacity: Lateral displacement at collapse initiation (strength loss) divided by the yield displacement (A value was set at 10 so that lower values can be monitored in post-processing
- β: Ratio of post-yield to elastic stiffness (A value of 1% was used to avoid numerical instabilities)
- γ: Residual-strength, ratio of residual strength to yield strength (A value of 0.3)
- ρ: Residual-strength deformation, deformation at onset of residual strength (A value of 20 was used)

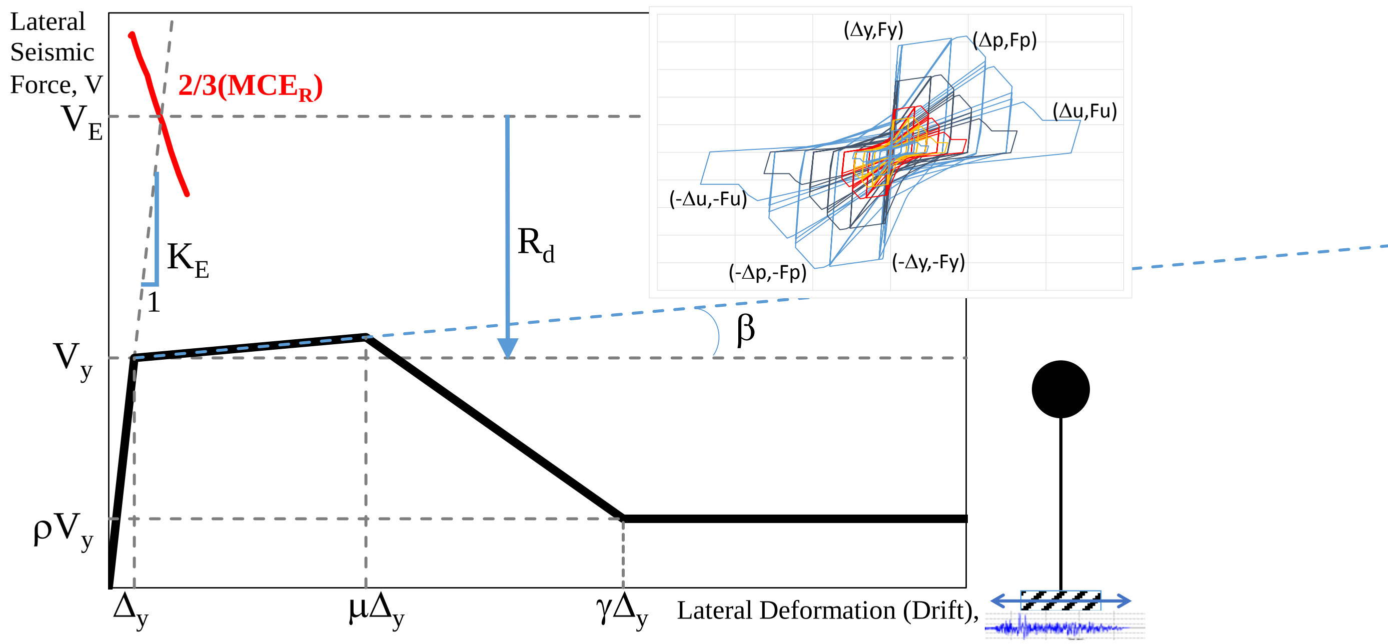


Fig. 5 Simplified SDOF Nonlinear Inelastic Structural Model in OpenSees

5. Scenario Ground Motions & Inelastic Hazard Curves

Using the hazard-analysis output, such as hazard curves, deaggregation, and conditional spectra, a suite of scenario ground motions that reproduce the hazard can be assembled through a process developed by one of the authors [5]. Through an iterative procedure, a set of 300-500 records were selected from the NGA-West2 ground-motion database where each record is scaled and assigned a rate of occurrence. Once the response spectra for these records are computed, a hazard calculation can be performed to compute the hazard curves by integrating over all of these scenarios.

The ASCE 7-16 criteria apply only to three-dimensional models with bidirectional lateral loads, but the ground-motion selection and modification is based on a scalar resultant, such as the maximum component. However, building structures have two primary orthogonal direction of response and the limit states, such as story drifts and ductility levels are evaluated for each directions independently, not at the resultant. Consequently, the hazard curves computed in this paper are computed for the rotD50 component (the median value of the projection of the response in all directions).

When elastic response spectra are computed, the same elastic hazard curves as those computed by the PSHA were obtained, as shown in Fig. 6. Similarly, inelastic hazard curves can be computed when the inelastic model is used. The intensity measure for inelastic response of softening systems needs to be deformation based, such as displacement, drift, or ductility. Because ductility is a normalized measure with relevant information about the level of inelastic deformation and collapse limit states, it is considered a convenient measure for comparison and evaluation. The elastic and inelastic hazard/risk curves for ductility are shown in Fig 7. The main feature to notice in the inelastic hazard curves for ductility is the change in slope due to the softening of the system post-yield. The pre-yield portion of the hazard curves is also different because of the different levels of damping used by the two models – 5% for elastic and 2% for inelastic. There are three graphs in the figure, the only difference between these graphs is the limit state being considered, defined by the ductility level. The annual rate, return period (1/annual rate) and probabilities of exceedance in 50 years (assuming a Poisson distribution) are quantified in the table in the figure.

The inelastic hazard curves represent a risk probability, depending on the definition of the limit state. For example, the collapse capacity of a non-ductile building may be 2, while that of a ductile building may be between 4 and 8. Collapse may also be defined in terms of lateral drift, in which case the hazard curves would be computed in terms of Annual Rate versus Drift.

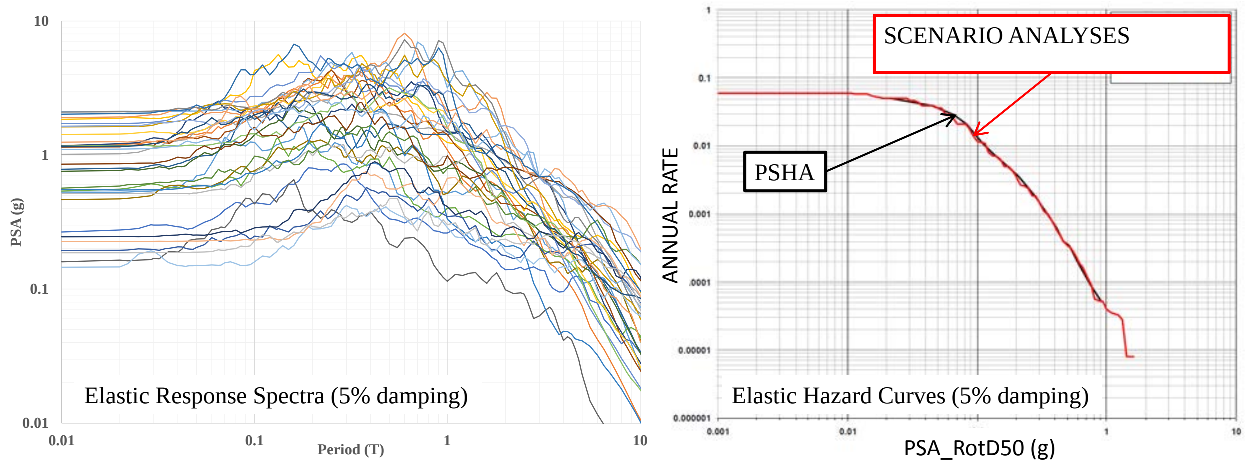
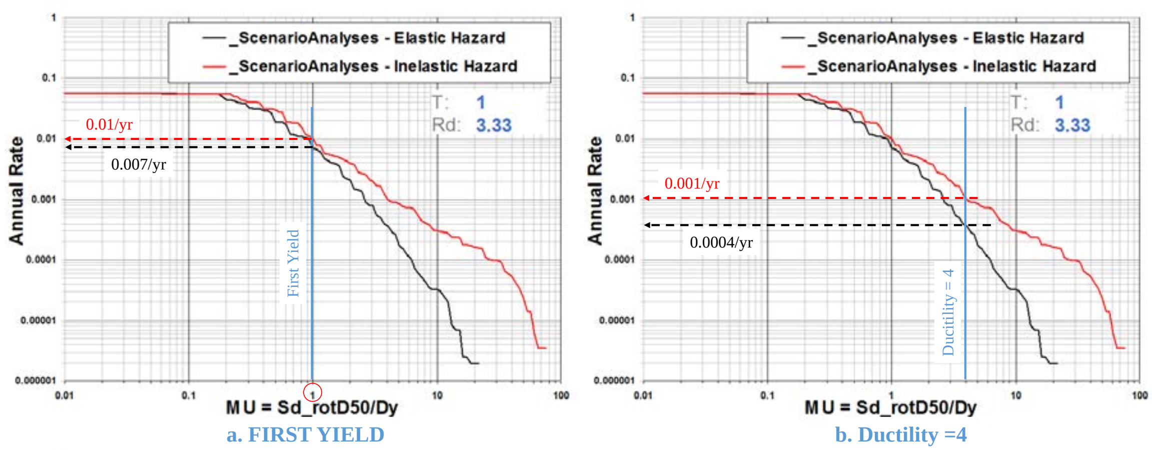
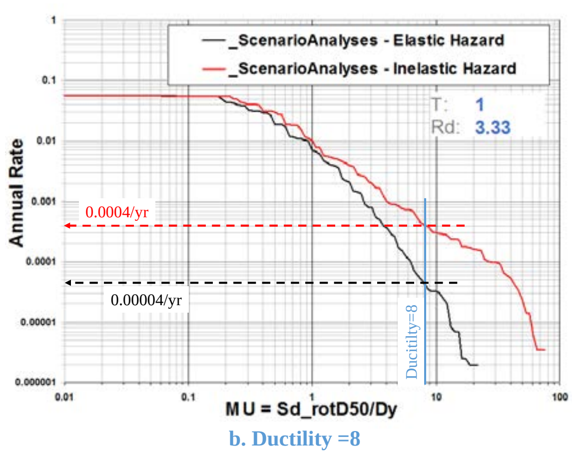


Fig. 6 Representative Elastic Response Spectra and Elastic Hazard Curves of Scenario Ground Motions





		Ductility Level Limit State		
	Model	1	4	8
Annual Rate	Elastic	0.01	0.001	0.0004
	Inelastic	0.007	0.0004	0.00004
Return period	Elastic	100	1000	2500
	Inelastic	143	2500	25000
Probability of Exceedance in 50yr	Elastic	39%	5%	2%
	Inelastic	30%	2%	0.2%

Fig 7. Comparison of Elastic Inelastic Hazard Curves for Typical Site, T=1, Rd=3.33

For the case of an elastic system, only two parameters are needed to compute the hazard – the period and the hazard level. For an inelastic model, the additional parameter of strength is introduced. As described in a previous section, the strength parameter is defined in terms of the ratio between the lateral capacity of the system and the elastic demands (defined as 2/3 MCE spectra, consistent with ASCE 7). For each period, thus, there exists a set of inelastic ductility hazard curves, as shown in Fig. 8 and Fig. 9. These hazard curves may be read in different ways, depending on the purpose. For example, for the site and period shown in Fig. 8, a structure with a lateral strength of ½ of the Elastic demand (Cy=0.5 and Rd=1.33) has a 1% probability of exceeding a ductility of 4 in 50 years. The same structure at the site shown in Fig. 9 has a probability lower than 1% of exceeding a ductility of 4 in 50 years. To achieve the same probabilistic limit state, a structure at the OAK site only needs to have a strength ratio of 0.3 (Rd=2).

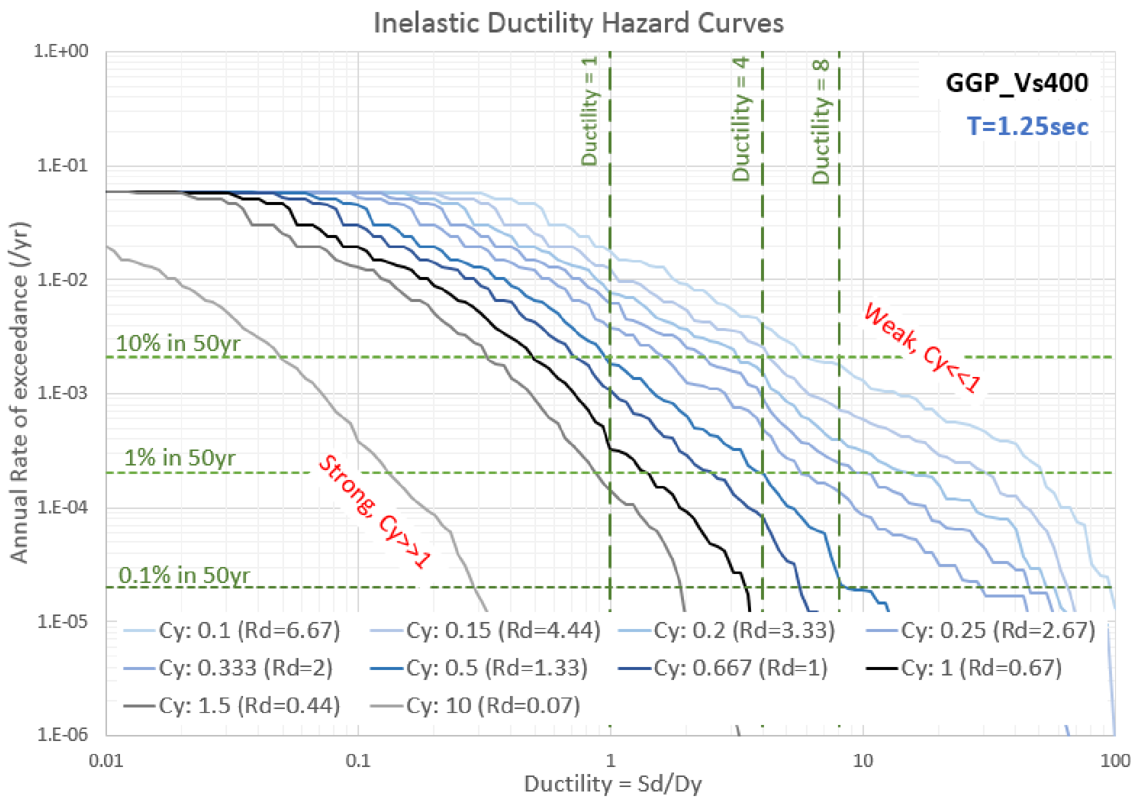


Fig. 8 Inelastic Ductility-Hazard Curves for GGP Site, T=1.25s

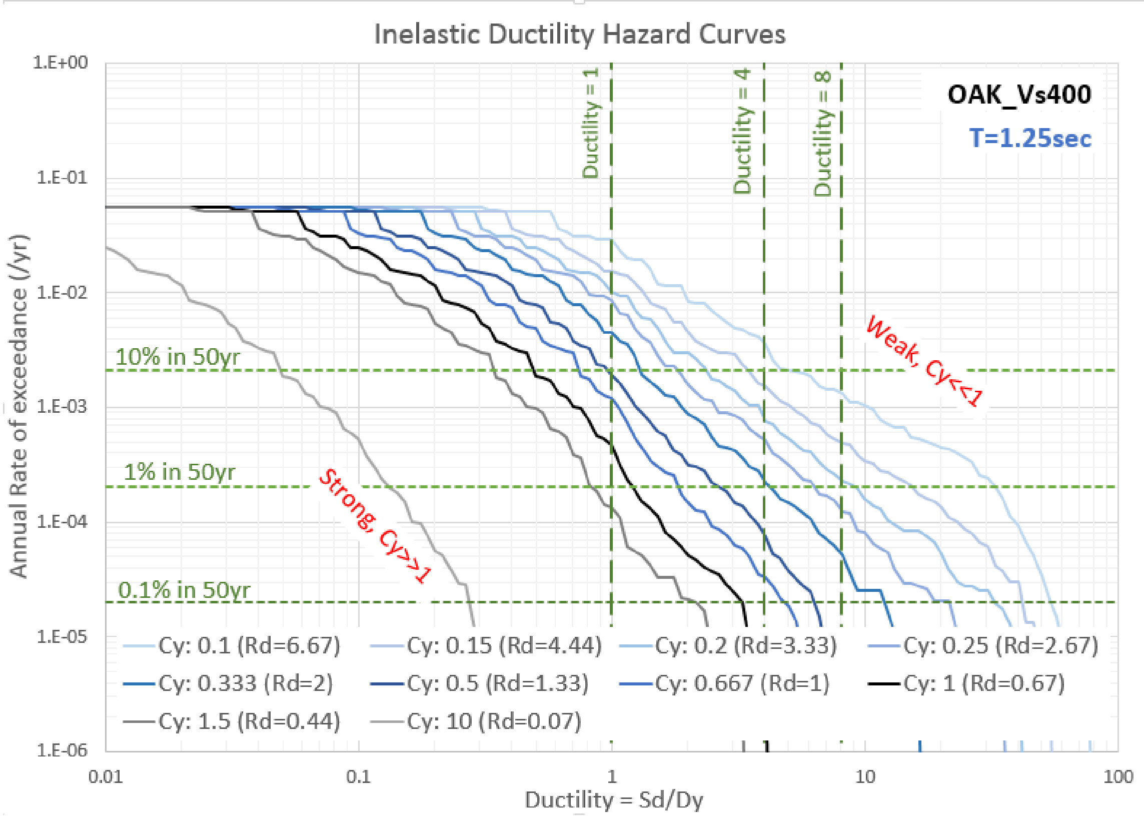
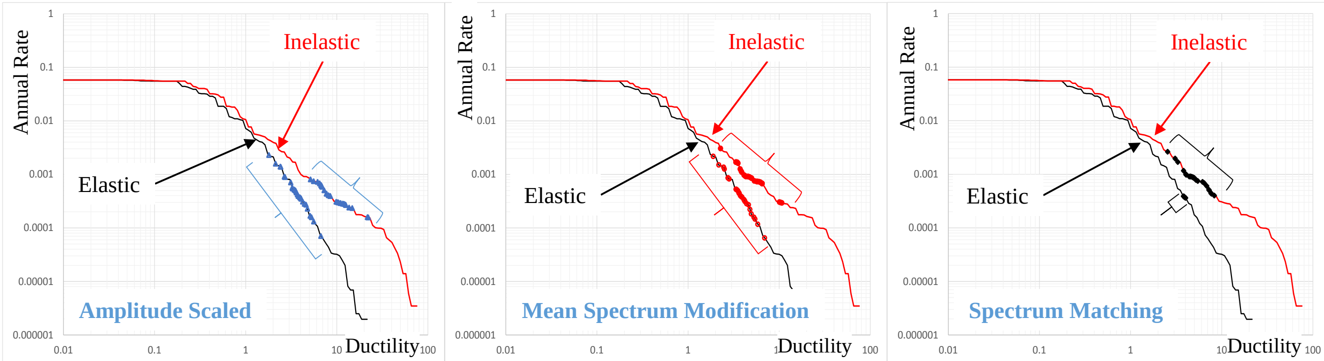


Fig. 9 Inelastic Ductility-Hazard Curves for OAK Site, T=1.25s

6. Evaluating Ground-Motion Suites Using Inelastic Hazard Curves

The inelastic model presented in Section 4, with the same range of input parameters that were used for the inelastic hazard curves, was subjected to the different suites of ground motions selected and modified for the three different sites. The key parameters in the evaluation were the fundamental elastic period, T, and the Estimated-Strength Reduction Factor, R_d. The response of the models was quantified in terms of ductility demand – the ratio of the maximum lateral displacement to the yield displacement – as was done for the hazard curves. The response values were computed for each primary direction of lateral loading and averaged averaged.

To plot the ground-motion analysis results on the hazard curves, the ductility hazard curves were interpolated to the determine the annual probability of exceedance for each ground-motion ductility demand (for each period and strength factor), as shown in Fig. 10 for a period of 1sec and an effective-strength factor R_d equal to 3.33. The figure shows the difference in the deformation amplitudes and spread between elastic and inelastic response. Comparing the hazard values rather than ductility values indicates that, even though deformations may increase from an elastic analysis to an inelastic analysis, annual rates of exceedance may actually increase, corresponding to a higher probability level, or decrease, corresponding to a lower probability level, which is more “conservative.”



B. Period = 1sec, Rd = 3.33

Fig. 10. Interpolation of Ground-Motion Response on Hazard Curves

Both the individual-record response and the suite-average response values for all the strength levels for a specified site and period are then plotted on the same graph as the ductility hazard curves, as shown in Fig. 11. The data shown in the figure can assist the designer in the iteration process of design in selecting the appropriate lateral-load strength that is consistent with the desired performance objective. Ductility=4 and the 1% probability of exceedance in 50 years are added to the graph as an example of a performance objective. The three different color/symbols in the graphs correspond to the three different ground-motion modification procedures.

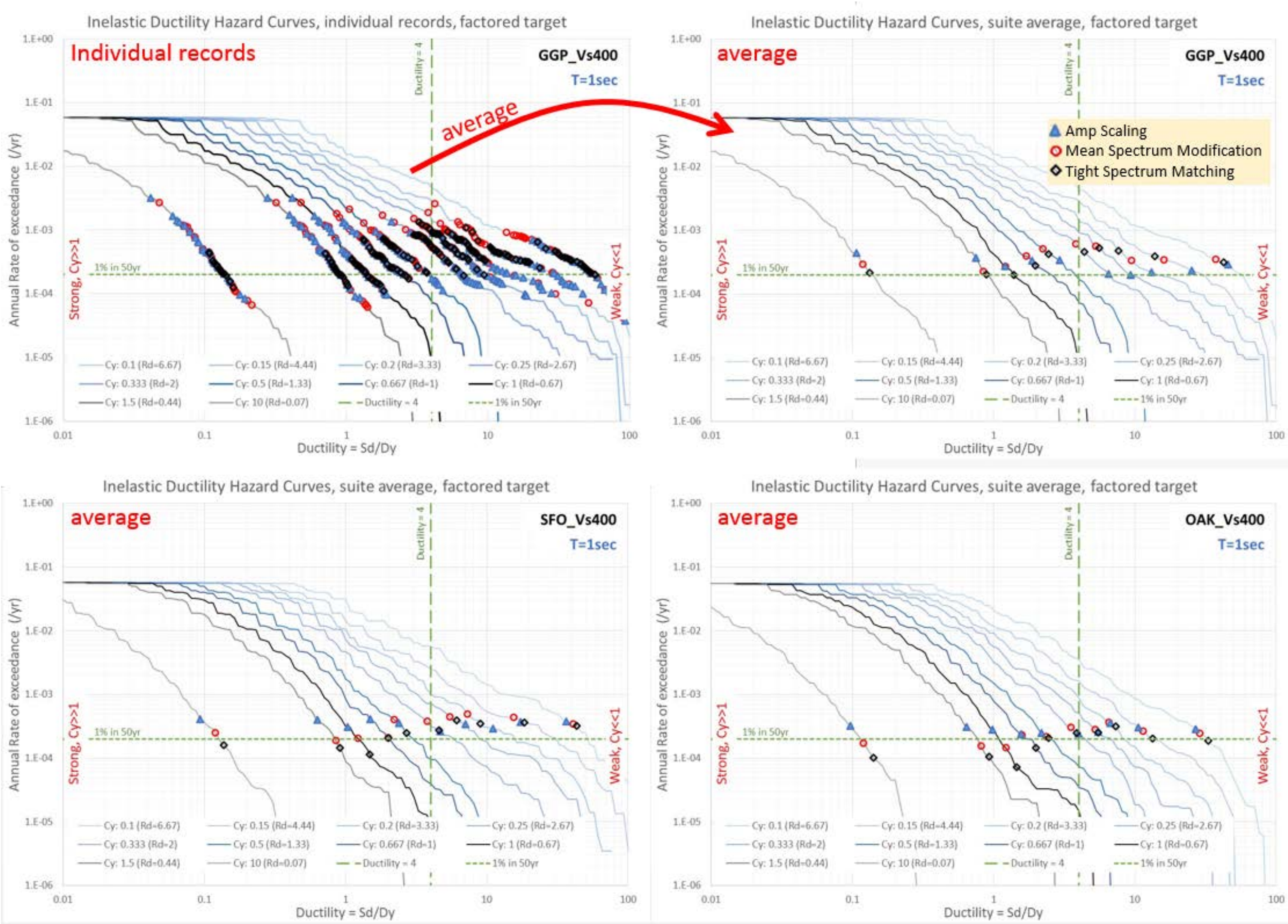


Fig. 11. Ground-Motion Response & Inelastic Ductility Hazard Curves (Blue Triangle=Amplitude scaling, Red Circle=Mean Spectrum Modification, black Diamond=Tight Spectrum Match)

As is shown in the figure, there is not consistent difference between the three different ground motion suites (different modification methods) at the three sites. The use of the ductility hazard curves along with the ground-motion analysis results, however, help determine whether the design process has converged to the desired performance objective. The use of a complete set of ductility hazard curves allows for the assessment of the design or structure at different performance objectives. For example, the designer can assess the probability of reaching first yield to determine the probability of maintaining an elastic response.

7. Conclusions

The quantitative objective of the procedure described in this paper is to determine the annual rate of exceedance associated with the nonlinear response of a structure to a suite of ground motions selected and modified in accordance with the building code. This performance measure has many applications. One important application is the evaluation of a ground-motion suite to determine whether it is appropriate for the performance objective of a design or structural assessment, or to compare different suites of ground motion, as well as different ground-motion modification methods, as was demonstrated in this paper.

The procedure described in this paper can also be used to determine the expected performance of a structure designed using the building-code requirements, in probabilistic terms. As was mentioned in the paper, in regions of high seismicity, the design spectrum used for ground-motion selection and scaling may be controlled by the deterministic spectrum. In this case, the structure is designed to have a higher probability of collapse than the 1% in 50 years prescribed by the code, but is not quantified. The procedure described in this paper can be used for the quantification of risk in this case.

This procedure raises many questions on the implementation of the building-code requirements: how do we interpret the 1% probability of collapse in specific terms of building performance? The computations that are typically performed to compute the risk-targeted spectrum are based on generalized 1D hazard and response calculations, while the building response is performed in a 3D model with two principal inertial DOFs, as well as higher modes.

One important additional consideration that has not been presented in this paper, but can be included in the procedure, is the consideration of higher-mode effects, such as tall or irregular buildings. The simplified model can be extended to more degrees of freedoms by implementing a simplified stick model that has a mass associated with each building floor.

8. References

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