EVALUATING GROUND MOTION MODIFICATION PROCEDURES WITH THE CONDITIONAL SCENARIO SPECTRA

C.A. Arteta ¹, S. Mazzoni² and N.A. Abrahamson³

ABSTRACT

Since inelastic dynamic analyses have become routine practice in structural engineering offices, ground motion selection and modification procedures have grown in number and complexity in the last ten years. The primary reason for scaling and modifying time series is to attain specific intensity and/or frequency content characteristics that allow running a small suite of recordings over a structural model while still gathering meaningful values and ranges of engineering demand parameters. In this paper, we compare two ground-motion-modification procedures for structural analysis, which are based on matching a target Uniform Hazard Spectrum. One of the procedure matches the target spectra tightly at all periods of interest, while the other does it in a weak manner by matching the target on the average, while allowing for some variability in the spectra of the individual ground motions. Suites of ground motions selected and modified with these methods were used to perform nonlinear dynamic analyses of a code-compliant reinforced concrete multistory RC frame with special detailing. Both methods were evaluated based on the resulting risk of a set of structural engineering demand parameters. Results show that while the methods yield the same median response with known dispersion in the elastic domain, as represented by the response of elastic single-degree-of-freedom systems, the response of nonlinear inelastic systems subjected to both ground-motion suites vary in the median level (e.g. differ in risk) as well as in dispersion. It is shown that the ground motions of the tightly matched set produce median responses that are closer to their expected risk level.

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ABSTRACT

Since inelastic dynamic analyses have become routine practice in structural engineering offices, ground motion selection and modification procedures have grown in number and complexity in the last ten years. The primary reason for scaling and modifying time series is to attain specific intensity and/or frequency content characteristics that allow running a small suite of recordings over a structural model while still gathering meaningful values and ranges of engineering demand parameters. In this paper, we compare two ground-motion-modification procedures for structural analysis, which are based on matching a target Uniform Hazard Spectrum. One of the procedure matches the target spectra tightly at all periods of interest, while the other does it in a weak manner by matching the target on the average, while allowing for some variability in the spectra of the individual ground motions. Suites of ground motions selected and modified with these methods were used to perform nonlinear dynamic analyses of a code-compliant reinforced concrete multistory RC frame with special detailing. Both methods were evaluated based on the resulting risk of a set of structural engineering demand parameters. Results show that while the methods yield the same median response with known dispersion in the elastic domain, as represented by the response of elastic single-degree-of-freedom systems, the response of nonlinear inelastic systems subjected to both ground-motion suites vary in the median level (e.g. differ in risk) as wells as in dispersion. It is shown that the ground motions of the tightly matched set produce median responses that are closer to their expected risk level.

Introduction

Selection and modification of earthquake ground motion records for geotechnical and structural response history analyses (RHA) is a subject of ongoing research [1-8]. The primary reason for scaling and modifying these time series is to attain specific intensity and/or frequency content characteristics that allow running a small suite of recordings over a structural model while still gathering meaningful values and ranges of engineering demand parameters. For structural analyses one of the most renowned methods make use of a Conditional Mean Spectrum (CMS) [1] that serves as target to selected times series that follow a similar spectral shape. The target CMS matches a Uniform Hazard Spectrum (UHS) for a specific hazard level at one structural period, typically the fundamental one. One of the shortcomings of CMS as target at a single period is that structures undergoing seismic shaking of high intensity might experience fundamental period lengthening and depending on their configuration, higher mode participation might have an

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important effect on several structural responses [9]. To account for the latter, structural engineers sometimes face the problem of having to perform structural analyses runs with more than one set of ground motions corresponding to selections based on more than one CMS to account for the different relevant structural periods. Other methods for ground motion selection, which satisfy structural engineering desire to perform as few runs as possible, are focused on modifying time series in order to match an UHS. Theoretically, this is over conservative because no single ground motion response spectrum has as high spectral coordinates at all periods but with by using this method, the problem of having to run analyses with many ground motion sets is minimized.

In this paper, we present an application of the Conditional Scenario Spectra (CSS) methodology for the objective comparison between two ground-motion-modification procedures for structural analysis. The CSS are a set of realistic earthquake spectra with assigned rates of occurrence based on their spectral shape and intensity. It is based on the so-called Scenario Spectra coined by Abrahamson[10]. The modification methods presented herein are based on matching a target UHS with a selected return period (or hazard level). Of the two ground motion modification methods used, one matches the target spectra tightly at all periods of interest, while the other does it in a weak manner by matching the target on the average, while allowing for some variability in the spectra of the individual ground motions around the target. Suites of ground motions selected and modified with the methods in discussion were used to perform nonlinear dynamic analyses of a code-compliant reinforced concrete multistory frame with special detailing. The two methods were graded based on the resulting risk of the maximum roof displacement and the base shear force. This risk is calculated by comparing responses from the modification method sets to those from the CSS.

**Probabilistic Seismic Hazard Analysis (PSHA) and Code-Based Seismic Demand Estimation**

Treasure Island in California, United States was selected as the site of interest for estimating the seismic hazard and code-based seismic demand for the design of a reinforced concrete structure. The site is in the San Francisco Bay Area with coordinates (37.82121°N, 122.37163°W). This area is known to be an active crustal region of high seismicity. The seismic hazard at a site, influenced by $k$ sources (e.g. faults), can be estimated by the integral in Equation 1:

$$\nu(S_a > z) = \sum_{k=1}^{^\text{Faults}} N_k(M_{\text{min}}) \int \int f_{M,k}(m)f_R|M,k(r|m)P(S_a > z|M,R)drdm$$

(1)

where $\nu(S_a > z)$ is the annual frequency with which spectral acceleration (at a given period) $z$ is exceeded; $M$ is the earthquake magnitude, and $R$ is the distance from the source to the site; $N(M_{\text{min}})$ is the annual rate of earthquakes with magnitude greater than or equal to $M_{\text{min}}$; $f(m)$ and $f(r|m)$ are probability density functions expressing the relative change of occurrence of different earthquake scenarios; $P(S_a(T) > z|M,R)$ is the conditional probability of observing a ground motion parameter, such as spectral acceleration ($S_a$) (at a given period) greater than $z$, for a given earthquake magnitude and distance. Typically, this conditional probability is estimated using available empirical Ground Motion Prediction Equations (GMPEs) [11-15].

Hazard curves from a PSHA for the site are depicted in Figure 2a for different periods. The hazard deaggregation gives the fractional contribution of different scenario pairs (e.g. earthquake
magnitude and distance) to the total hazard. For the Treasure Island site, contribution to a selected hazard level of $2.5 \times 10^{-4}$, corresponding to a return period of $TR = 4,000$ years is commanded by earthquake scenarios with distance to seismic source, $R = 5$ to 30 km and Moment Magnitude, $M_w = 6.0$ to 8.5; the selected NEHRP soil class was C with $Vs30$ in the range 360 to 760 m/s. Figure 1 describes the parameters contributing to the seismic hazard at different spectral periods for the selected hazard level which approximately has a probability of exceedance of 2.5% in 100 years.

Following conventional practice using ASCE 7-10 [16], Maximum Considered Earthquake (MCE) seismic demands are obtained at the short period and at 1s period. These are adjusted for site effects, and then further adjusted to the design level by factor 2/3. The resulting design spectral acceleration parameters are $SDS = 1.00$ and $SD1 = 0.52$. Because the case study building is categorized as Risk Category I and has $(0.50 \leq SDS)$ and $(0.20 \leq SD1)$, it is assigned to Seismic Design Category D. For design, this elastic design response spectrum was reduced by response modification factor $R$ to account for the expected inelastic behavior. Figure 2b shows three code-based response spectra: MCE level, design level ($Sa$) and design level affected by $R=8$ and a calculated over-strength factor from pushover analyses ($\Omega \approx 1.23$). Also shown is an estimated target UHS for a return period of 4,000 years.

**Structural Design and Inelastic Modeling**

An 8-story reinforced concrete building was selected as the object of study to analyze the impact of two different ground motion selection methods in the response of the structures. Figure 3 depicts the general geometric properties of the structure of analysis. Plan dimensions are 117 ft x 97 ft and total height is 96 ft. The structural system comprises circular gravity load columns and external special moment resistant frames (SMF) to resist lateral loads due to seismic demand. Analyses for combined gravity and lateral seismic loads followed ASCE-7. A three-dimensional, linear structural model of the buildings was implemented in the computer software ETABS [17]. The model accounted for degraded stiffness of the structural elements due to seismic loading. The effective inertia of the beams was set to 35% of the gross inertia and those of the columns were set in the range 50% to 70% according to their gravitational axial loading. A response modification factor $R = 8$ was selected. Concrete was assumed normal weight ($\gamma = 150$ lb/ft$^3$) with nominal strength $f'c = 6$ ksi for beams and columns of the SMFs. The elastic modulus of the reinforced
concrete structural elements was computed as $57000\sqrt{f'c}$ [ksi]. Reinforcing steel was assumed ASTM A706 with nominal yielding strength of $f_y = 60$ ksi.

Seismic load effects on the structural members were calculated by means of the Equivalent Lateral Force Analysis described in ASCE 7. The design spectrum depicted in Figure 2b was used as the seismic demand after reducing it by R. A modal analysis was performed over the structure to calculate its dynamic properties. Since the nonlinear modeling was performed over a 2D-model representative of half the structure in the EW direction, in the following, the mentioned values will only refer to that direction of analysis. The fundamental period was $T_{1EW-Dir} = 1.75$ s and second and third translational modal periods were 0.53s and 0.27s respectively. Design base shears including response modification factor $R=8$ were $V_{bEW} = 721.5$ kN ($V_{bEW} / W = 4.8\%$). Maximum inter-story drift ratio was 1.61% which is below the maximum allowed of 2%. Design of the reinforced concrete structural elements (beams and columns) was performed in accordance with provision for special moment frames in ACI-318-11 [18]. Columns were 32 x 32 in. with minimum longitudinal steel ratios (total area of steel to gross area ratio) $\rho_{min} = 1.00\%$ and transverse steel ratio with $Av / bs > 0.87\%$, where $Av$ is the total area of transverse reinforcement within distance $s$ of two adjacent layers and $b$ is the core dimension of column. Beams were 22 x 32 in. with longitudinal steel ratios (area of tension reinforcement divided by web width and effective depth) ranging from $\rho = 0.39$ to 0.54% and transverse steel ratios $Av/bs > 0.63\%$. The slab was 8 in. thick at all floors.

To account for epistemic uncertainty in modeling, two planar inelastic mathematical models, representative of half the structure in the EW-direction, were constructed: (i) LC Model and (ii) NLC Model. The LC Model (leaning column model) accounts for P-Delta effects due to high axial load in the gravity load resisting system through a so-called leaning column which is a mathematical artifact that effectively reduces the tangent stiffness of the system at large displacements. NLC Model only accounts for P-Delta effect due to axial loading of the SMF. The software package OpenSees [19] was selected as the modeling tool because its nonlinear analysis capabilities allow conducting a large number of simulations and its efficacy has been validated by
the research community for many years. Static nonlinear analyses ("pushover") and dynamic nonlinear analyses were performed on the structures.

Force-based nonlinear beam-column elements with concentrated plasticity at the ends were used to model all structural elements [20]. Fiber sections assigned to the plastic hinge regions simulate material nonlinearities while accounting for moment-axial load interaction. For nonlinear dynamic analyses, mass and stiffness-proportional Rayleigh damping was used to simulate the energy dissipation characteristics of the building that is not accounted for by the nonlinear behavior of the structural elements. The Rayleigh damping coefficients were established to achieve a damping ratio of \( \zeta = 2.5\% \) at periods corresponding to the first and third translational vibration modes of the linear model. Calculated periods for the nonlinear models were obtained after applying the vertical load, therefore some initial service level cracking is accounted for. The first, second and third periods of the nonlinear model are: \( T_{1EW-NL} = 1.73s \), \( T_{2EW-NL} = 0.54s \) and \( T_{3EW-NL} = 0.29s \), which closely match those of the elastic model with cracked sections. Figure 4 shows pushover curves for the two OpenSees models. The influence of P-Delta effects is apparent on the LC Model. The design base shear and yielding base shear, from displacement based analysis ("by hand") of the designed structure, are also presented. The minimum overstrength factor, defined as the ratio between the maximum base shear of LC Model and the design base shear is, approximately 1.12; the maximum overstrength factor, defined as the ratio between the maximum base shear of the NLC Model and the design base shear is, approximately 1.57.

Structural systems subjected to high intensity shaking undergo relatively large displacements and their period of vibration shifts toward larger values, product of stiffness degradation of the structural elements. For the system shown in Figure 3, the expected lengthening is, on average, approximately 1.5 times the elastic period of vibration. This suggests that to estimate the response of this inelastic multi-degree-of-freedom system (MDOFS) properly, the frequency content of the selected ground motions must be rich enough to properly excite the structural vibration modes that most contribute to the response. For the building in discussion, at least the first and second elastic modes should be excited with enough energy and the expected degraded period should also be covered by the frequency content range of the times series used as the uniform excitation at the base. The building code achieves this by requiring that the average spectrum of the ground motions

Figure 3. General layout of structural elements: (a) plan; (b) elevation of Frame 1.
does not fall below the target design spectrum for the site in a period range between 0.2 and 1.5 times the fundamental elastic period of the structure being analyzed.

Figure 4. Pushover curves.

**Ground Motion Selection and Modification Procedures**

*Loose Mean-Spectrum match (LM) set*

The LM set comprises twenty ground motions that were selected, scaled and their frequency content was modified such that their average spectrum matches the 4,000-year return period UHS for the site (Figure 5a). This method of ground-motion modification allows for the variability around the mean which tight spectral matching removes [21]. In addition, the frequency-content modification removes the higher-mode amplification effects that results when records are simply scaled and not modified.

*Tight Component-Spectrum match (TM) set*

The TM set also comprises twenty ground motions as shown in (Figure 5b). The frequency content of these records, however, was modified such that the response spectrum of each component matched the target UHS for the site. This method is used in many structural-engineering project because of it meets the design code requirements and is easy to implement and use.

Figure 5. Selected ground motion set and their variability: (a) LM set, with variability around the median; (b) TM set, without much variability; (c) variability (in log units) of the LM and TM sets for different periods.
**Structural Responses**

The inelastic mathematical models of the building were subjected to simulated seismic shaking using the two sets of ground motions described above. Excluding collapses, Figure 6 show two engineering demand parameters (EDP) for each set of runs: (a) maximum roof drift ratio (RDR), defined as the lateral displacement of the roof divided by the total height of the building, versus the elastic spectral displacement at the elastic fundamental period \((Sd(T1))\); and (b) maximum base shear normalized by the weight of the building versus spectral acceleration at the fundamental period \((Sa(T1))\). Under each set of ground motions, the response of both mathematical models of the building (i.e. NLC and LC model) is included. It is apparent how both ground motion sets differ in dispersion of the elastic ground motion parameters (i.e. spectral coordinates of pseudo acceleration or displacement are concentrated around a single value for the TM set), while the structural response dispersion difference is not as obvious.

Median values and dispersion estimates of the EDPs of interest are depicted in the boxplots of Figure 7. Median values, product of the TM set runs, are 3% to 13% larger than those from the LM set. The structural responses reported here follow a logNormal distribution with unbounded values on the upper tail of the distribution due to structural collapses. To overcome the latter, the dispersion of the EDPs is calculated with a standard deviation estimate according to Equation 2:

\[
\sigma_{\text{log}} \approx \frac{\ln Ri_{p84} - \ln Ri_{p16}}{2}
\]

where \(\ln Ri_{p16}\) and \(\ln Ri_{p84}\) are, respectively, the 16\(^{th}\) and 84\(^{th}\) percentile of the natural logarithm of the response of interest. The dispersion from the runs corresponding to the LM set is 2.3 to 2.9 times larger than those from the TM set. For analyses under the TM set, it is important to notice how the response of the inelastic MDOFS dispersion is not as low as the dispersion of the input ground motions, represented by their elastic response spectra. This result supports the hypothesis that spectral shape is not the only relevant parameter of the input ground motions that drives the response of inelastic MDOFS.

Figure 6. Structural response under each set of ground motions: (a) maximum roof drift ratio versus elastic spectral displacement at the fundamental period; (b) normalized maximum base shear versus spectral acceleration at the fundamental period.
It is observed that the structural responses of the inelastic MDOFS under both sets of modified ground motions differ in their median value as well as in their dispersion. Based on the latter, an open question that still needs to be answered is: which method produces a more adequate answer. To respond to this, the following focuses on defining a methodology to grade the median values of the EDPs of interest, based on their associated hazard. It is expected that the better estimate of the median EDPs will have a hazard closer to the hazard level of $2.5 \times 10^{-4}$, since both ground motion modification methods were based on matching the UHS with 4,000-year return period.

The conditional scenario spectra (CSS) are a set of realistic earthquake spectra with assigned rates of occurrence based on their spectral shape and intensity. To ensure that each spectrum has the correct shape, the CSS ground motion selection procedure makes use of estimated CMSs, anchored at the fundamental period of the structure, at different hazard levels. Ground motions time series are selected based on the hazard deaggregation at the site and are scaled to capture the peak and trough variability around a CMS at various hazard levels. The initial assigned rate of occurrence for each time series is based on the hazard level of the UHS at the conditioning period of the aforementioned CMSs. The assigned rates to each time series are then numerically optimized such that their calculated hazard matches the target hazard curves for a range of hazard levels and frequencies of interest. Figure 8a presents the set of 402 ground motion spectra used for this study along with their final assigned rates of occurrence (Figure 8b). As mentioned before, the main feature of the CS is that the suite of records used and their assigned rates, allow recovering the hazard at a site over a range of periods. The hazard recovered from the CS is estimated by means of Equation 3:

$$\nu_{CS}(S_a > z) = \sum_{i=1}^{\#CS} \theta_i P(S_{a,i} > z)$$

where $\nu_{CS}(S_a > z)$ is the annual frequency with which spectral acceleration $z$ is exceeded for a given period; $\theta_i$ is the assigned rate of each time series; and $P(S_{a,i} > z)$ is the probability (e.g. either 1 or 0) that the spectral acceleration of time series $i$ exceeds level $z$ at the period of interest. Figure 9 shows the target and recovered hazard curves for hazard levels from $10^{-2}$ to $10^{-5}$ and three periods of interest: the first ($T_1$) and second ($T_2$) elastic periods of the structure of this study as
well as the degraded period ($T_{f} \approx 1.5T$). The recovered hazard curves are in good agreement with the target seismic hazard curves for the Treasure Island site, being closer for the conditioning fundamental.

**EDP Hazard Curves from CSS**

Figure 10 shows EDPs estimated using the inelastic NLC model and the ground motion set associated with the CS described above. The same type of data was estimated with the LC model and the results were averaged with equal weights for each model. It is interesting to notice that one of the depicted EDPs is bounded by the strength of the building while the other is practically unbounded and is related to the displacement of the framing system. It is also worth mentioning, that the CSS ground motion set covers low and very high levels of intensity that may drive the inelastic model into the collapse range.

To construct an objective ground for comparison of the EDPs estimated with the LM and TM ground motion sets, product of the modification methods in study, the EDPs from the CS set, along with the assigned rates of each time series, can be used to estimate a hazard curve for EDPs using Equation 4:

$$\nu_{EDP}(EDP > d) = \sum_{i=1}^{\#CS} \phi_{i} P(EDP_{i} > d)$$

where $\nu_{EDP}(EDP > d)$ is the annual frequency with which demand level $d$ is exceeded; and as before, $P(EDP_{i} > d)$ is the probability (e.g. either 1 or 0) that the EDP from time series $i$ exceeds level $d$.

Figure 8. Scenario spectra: (a) 402 scenario spectra (5% damped); (b) assigned rate of occurrence for each spectrum of the CS.
Figure 9. Hazard curves at different periods recovered from scenario spectra.

Boxes (a) and (b) of Figure 11 and Figure 12 show hazard curves for the maximum RDR and the normalized maximum base shear. Also shown are the responses from the TM and LM set runs, which are placed at the expected abscissa of 4,000 years return period (rate of 2.5 \times 10^{-4}). To compare the adequacy of the response estimates from both sets of ground motions, Figure 11c and Figure 12c depict the median values of the EDPs of interest and their interpolated rate of being exceeded. It is observed that the interpolated rates for the TM set medians are closer to the expected hazard of 1/4,000 years. Furthermore, the median value of the normalized maximum base shear estimated with the TM set is almost on top the hazard curve for return period 4,000 years. On the other hand, due to the steep behavior of the aforementioned EDP, the rate of the median response from the LM set is deemed unconservative at 1/850 years approximately.

Figure 10. Engineering demand parameter response from CS runs on the NLC model: (a) maximum roof drift ratio versus elastic spectral displacement at the fundamental period; (b) normalized maximum base shear versus spectral acceleration at the fundamental period.
Figure 11. Hazard curves for maximum roof drift ratio: (a) with responses from the TM set; (b) with responses from the LM set; (c) interpolated rates for median values from runs with the TM and LM set.

Conclusions
A methodology was presented in which Conditional Scenario Spectra (CSS) are utilized to construct hazard curves for engineering demand parameters (EDPs). The rates of these demand parameters serve as basis to grade the adequacy of two sets of ground motions, product of two different modification procedures. The two sets are similar in that their mean response spectrum approximately matches a target UHS with return period 4,000 years. The two sets differ in that one had variability around the mean spectrum while the other is a set comprised of spectrally matched time series with low variability.

The estimated median EDPs of each procedure are graded by interpolating their rate of being exceeded within their hazard curves from the CS. It was observed that median inelastic responses from the low variability ground motion set have rates closer to their matched target UHS.

In addition to the value of the results of this comparison, the methodology used in this paper is a valuable tool in evaluating any set of ground motions.

Figure 12. Hazard curves for normalized base shear: (a) with responses from the TM set; (b) with responses from the LM set; (c) interpolated rates for median values from runs with the TM and LM set.
References