Ground motion selection and acceptance criteria when multiple seismic sources contribute to MCE ground motions

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ABSTRACT
Performance-Based Earthquake Engineering (PBEE) has become more common in the analysis and design of tall buildings that are being planned and constructed using alternative means of compliance because of the implementation of structural systems that provide satisfactory performance but are not allowed by the building codes. Design and analysis using PBEE in the United States has been advanced by documents from the Tall Buildings Initiative of the Pacific Earthquake Engineering Center and the Los Angeles Tall Buildings Structural Design Council. The procedures in these documents rely upon three-dimensional nonlinear response history analyses to demonstrate a low probability of collapse when subjected to risk targeted maximum considered earthquake ground motions as defined by ASCE 7 and adopted in the International Building Code. ASCE 7 provides a framework to establish the ground motions and the newest edition (ASCE 7-16, 2016) provides more guidance in the ways ground motions are to be specified in terms of the acceptable hazard and risk levels as well as criteria for appropriate ground motions to be used in the response history procedures. ASCE 7-16 now permits the use of ground motions scaled to scenario spectra (conditional mean spectra) as an alternative to the risk targeted uniform hazard spectrum. Despite this guidance from ASCE and the PBEE guideline documents, there are situations that are not yet addressed that could affect the generation of the scenario spectra and the selection and scaling of appropriate time histories. One of these situations occurs when the hazard disaggregation from the probabilistic seismic hazard analysis indicates that there is not a single dominant seismic source, but rather there may be multiple sources with different predominant magnitudes and distances that significantly contribute to the ground motion hazard at a site. Approaches to account for situations such as this are discussed in this paper to properly account for the different sources and selection and scaling of appropriate time histories.

1 INTRODUCTION

Performance-Based Earthquake Engineering (PBEE) is becoming the preferred analysis and design methodology for tall buildings in the western United States, because it provides an alternative means of compliance to prescriptive building code requirements that may impose limitations on height and structural systems. Design and analysis based on PBEE principles provide a means that structural systems not allowed by the building code can be demonstrated to provide a satisfactory design that complies with the intent of the provisions of the code, such as stated in Section 104.11 of the International Building Code (International Code Council 2015).

Design and analysis guidance using PBEE in the United States has been advanced by documents such as the “Guidelines for Performance-Based Seismic Design of Tall Buildings Version 2.0” published by the Tall Buildings Initiative of the Pacific Earthquake Engineering Center (PEER 2016) and “An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region” published by the Los Angeles Tall Buildings Structural Design Council (LATBSDC 2015).

These PBEE procedures rely upon three-dimensional response history analyses to demonstrate a low probability of collapse when subjected to risk-targeted maximum considered earthquake ground motions. These ground motions are defined in both the International Building Code and ASCE 7 (ASCE/SEI 2016). For the response history analysis, a suite of horizontal orthogonal ground motion components (and a vertical ground motion component, if required) is needed. ASCE 7-16 requires a suite of 11 ground motion pairs or triplets. Ground motions are to be either amplitude scaled or spectrally matched in accordance with the requirements (and certain limitations) of ASCE 7-16. A deaggregation analysis of the source contributions conducted as part of the probabilistic seismic hazard analysis (PSHA) can provide insight into the most likely magnitude, M, and distance, R, contributing in a mean annual sense, to the probability of exceedance, given that M,R pair for a particular spectral ordinate.

Currently there is a lack of guidance on the selection of ground motions when there the hazard deaggregation from the PSHA indicates that there is not a single dominant seismic source, but there may a two or more sources with different predominant magnitudes and distances that have significant contributions to the seismic hazard at the site. With the specified suite of a minimum of 11 ground motions, how would one select ground motions? Should the 11 ground motions be divided between the sources, should there be 11 ground motions specified for each source, or should there be a minimum number of ground motions (less than 11) be specified for each source?

This paper provides motivating examples and some illustrative calculations to evaluate two candidate procedures for selecting ground motions and assessing structural responses when there are multiple “types” of ground motions of interest at a particular design intensity level.

2 EXAMPLES

An example of this situation occurs in Seattle, where the ground motion hazard has significant contributions from crustal sources at close distances as well as subduction zone sources at greater distances. Figures 1, 2 and 3 show the deaggregation of the hazard in downtown Seattle, Washington at spectral periods of 0.5, 2.0 and 5.0 seconds, respectively for the MCE ground motions; the USGS interactive deaggregation web tool (https://geohazards.usgs.gov/deaggint/2008/) was used to produce these figures. For comparison purposes, the average shear wave velocity in the upper 30 meters has been assumed to be 360 meters per second. These figures show that the hazard at different structural periods is governed by more than just a single seismic source. At short periods, a very local close-in crustal source (in this case, the Seattle fault) dominates the hazard; at distances of about 40 to 80 km, other crustal sources also have some contribution and there is a smaller contribution to the hazard from the Cascadia subduction zone earthquakes with distances beyond 80 km and magnitudes between 8 and 9. For a 5-second period, the contribution to hazard from the close-in crustal source is about equal to the contribution from the distant Cascadia subduction zone.

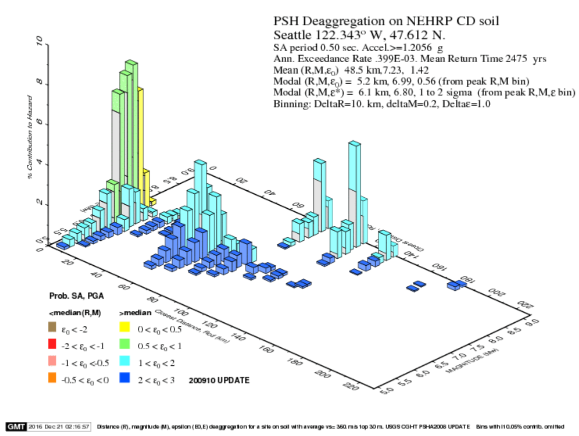

Figure 1. Deaggregation of spectral acceleration at spectral period of 0.5 seconds for Seattle, Washington (figure from USGS).

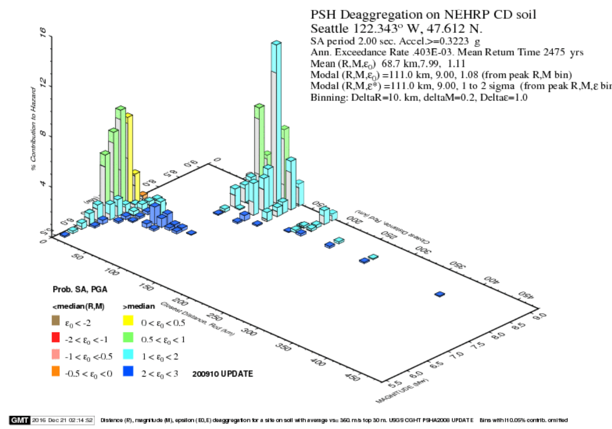

Figure 2. Deaggregation of spectral acceleration at spectral period of 2.0 seconds for Seattle, Washington (figure from USGS).

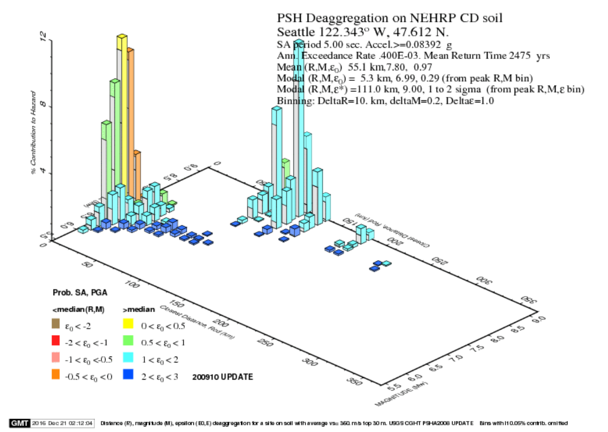

Figure 3. Deaggregation of spectral acceleration at spectral period of 5.0 seconds for Seattle, Washington (figure from USGS).

For tall buildings in Seattle with fundamental periods of 4 seconds or greater, consideration of different types of ground motion is needed, because of the contributions to MCE shaking from different seismic sources. For scenario earthquake ground motions, a conditioning period of 4 to 5 seconds will have to consider both types of sources. For a shorter conditioning period, presumably addressing the significant higher modes of vibration, the hazard is more dominated by the local crustal earthquake events rather than by the distant subduction zone events and ground motions.

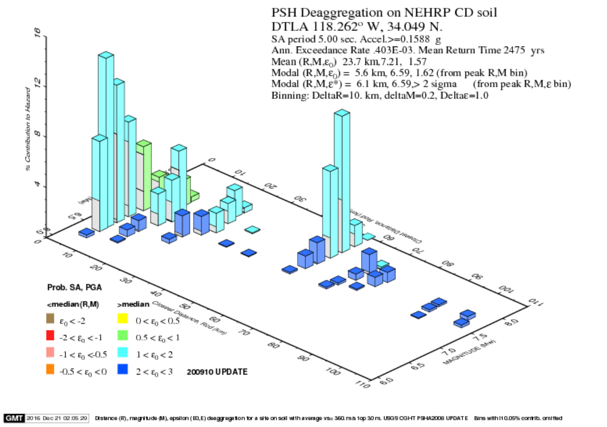

Figure 4. Deaggregation of spectral acceleration at spectral period of 5.0 seconds for Downtown Los Angeles, California (figure from USGS).

In the Los Angeles downtown area, a deaggregation analysis for a period of 5 seconds gives the results shown in Figure 4. From Figure 4, it can be seen that the hazard is due to contributions from the close-in local faults within the Los Angeles Basin and the more distant San Andreas fault system. At a period of 5 seconds, the local faults contribute more hazard than the San Andreas fault, but the San Andreas fault is significant enough to require consideration. For periods greater than 5 seconds, the relative contribution from from the San Andreas fault would further increase.

By contrast, the hazard deaggregation of a site in downtown San Francisco shows that almost all of the hazard is due to one source, the San Andreas fault, as shown in Figure 5.

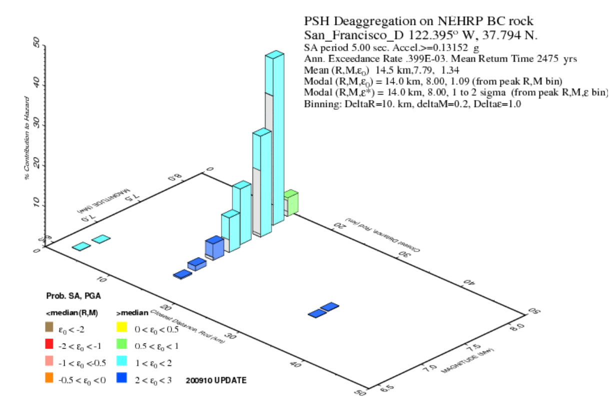

Figure 5. Deaggregation of spectral acceleration at spectral period of 5.0 seconds for Downtown San Francisco, California (figure from USGS).

For situations like these described in Seattle and downtown Los Angeles, multiple "types" of ground motions associated with the MCE shaking would need to be considered. The question is how to consider them when selecting ground motions and evaluating the structural responses produced by each type of motion.

3 DEALING WITH MULTIPLE "TYPES" OF GROUND MOTIONS ASSOCIATED WITH MCE SHAKING

The above referenced design and assessment guidelines all currently specify that the mean responses from a pooled set of ground motions should be considered, even when there are multiple types of ground motions. But an alternative proposal that has been advocated by some practitioners and researchers is to evaluate acceptance criteria separately when there are multiple "types" of ground motions associated with Maximum Capable Earthquake (MCE) shaking. (The procedure could be utilized more generally, but this paper is currently motivated by discussion related to MCE-level evaluations.) For example, if deaggregation associated with MCE-level shaking indicates contributions from two earthquake sources with differing magnitudes, or from both directivity and non-directivity ground motions, these subsets of ground motions would be considered separately when checking the resulting structural responses against acceptance criteria. Specifically, any limits on average displacements of member forces would have to be satisfied for each subset of ground motions, rather than satisfying the limits for a full set of ground motions that included both types.

The argument *for* considering the subsets of motions separately is roughly as follows: if the motions produce distinct demands on a structure, then pooling all the responses and looking at mean values may "average out" important demand features seen in one type of motion. More specifically, if one type of motion caused failure of the acceptance criteria on its own, while the pooled set did not, then looking at the pooled set might obscure the presence of unacceptable collapse risk.

The argument *against* considering subsets of motions separately is roughly as follows: PBEE acceptance criteria are focused on mean responses from already-small sets of data, and allow suppression of response variability (e.g., by allowing spectrum compatibilization); taking ground motion "type" as a special exception and looking at variations in structural responses from this specific source of uncertainty would be counter to the general approach of the acceptance criteria, and in fact would not be informative. A second argument against considering motions separately is that it would result in estimating response statistics from smaller numbers of ground motions, and that the separate response statistics would not actually provide greater insight regarding the structure's collapse risk.

To evaluate the above arguments quantitatively, we next consider some idealized examples to understand the circumstances when separate versus pooled treatment of responses would lead to different decisions, and to understand whether those decisions make sense.

3.1 BASIS FOR EVALUATION

It is proposed that the primary factor to consider in evaluating this potential approach is as follows: In a circumstance where there are two “types” of motions associated with the MCE amplitude, does the current approach (evaluating the mean response from the pooled set of motions) provide an accurate indication of the structure’s collapse risk? As a baseline for this evaluation, it is assumed that the code intends for motions subject to multiple types of ground motions to have the same collapse risk as buildings subject to a single type of ground motion. It is also assumed that the ASCE 7 target of 10% probability of collapse given the MCE is the nominal goal.

A secondary factor to consider is: are potential insights from evaluating each type motion separately worth the added complexity and increased estimation uncertainty associated with evaluating smaller subsets of ground motions?

3.2 BASELINE CASE: A SINGLE TYPE OF GROUND MOTION

To begin, consider the simple case where all MCE ground motions are of the same “type” (e.g., only crustal non-pulse-like motions contribute to MCE motions at the site, and so all structural responses can be grouped together as is typical practice today). This case, representative of the San Francisco case from Figure 5, will serve as a benchmark, and to illustrate the simplified calculation procedure we can use to study this problem.

Assume we have a structure with safety governed by peak drift demands, and that drift demands and capacities have the following properties:

- The drift capacity is lognormal with a mean of μ_C=0.07 and a log standard deviation (dispersion) of β_C=0.45.
- The drift demands are also lognormal, with a mean of μ_D=0.03 and a log standard deviation of β_D=0.5.

These values are consistent with those estimated by Gokkaya et al. (2016), when studying ductile reinforced concrete frame buildings from 1 to 20 stories in height (i.e., the Haselton buildings from FEMA P695, 2009). These distributions are illustrated graphically in Figure 6.

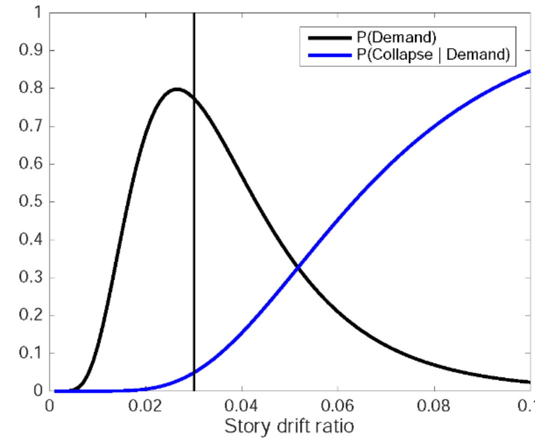

Figure 6. Illustration of assumed MCE demand and collapse capacity for the baseline base. *P*(*Demand*) is a probability density function, while *P*(*Collapse*|*Demand*) is a probability, conditional on the specified demand level. The mean value of the demand distribution is shown with a thin vertical line.

For these demand and capacity distributions, the probability of collapse given MCE is 0.098. This is a useful baseline, as the building barely satisfies a 3% mean story drift ratio requirement, and barely satisfies the 10% probably of collapse requirement, at the MCE level. This analysis ignores for the moment the uncertainty resulting from estimation error, and assumes that the 3% mean drift is accurate. We next compare some alternative cases with two types of ground motions.

3.3 CASE 1: A SUBSET OF MOTIONS ARE EXTREMELY AGGRESSIVE

Now consider a case where some of the ground motions are of a particular type that produces *much* larger structural demands. Specifically, consider the following:

- 1/3 of motions are “aggressive,” and produce a mean demand of 0.05.
- 2/3 of motions are “’benign,“ and produce a mean demand of 0.02.
- All motions have response dispersions of β_D=0.5 (the same as for the baseline case).
- The building’s drift capacity is the same as the baseline case above.

These distributions are illustrated graphically in Figure 7. This case is designed to be an extreme situation of the multi-source cases described above (e.g., Seattle or Los Angeles). The factor of 2.5 difference between the aggressive and benign responses is anticipated to be far larger than anything that could be observed from real motions, and so should provide a bound on real-world situations.

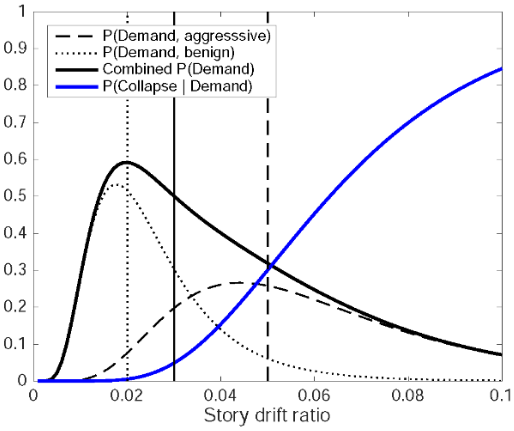

Figure 7. Illustration of Case 1 (some extremely aggressive motions). Mean values for each demand distribution are shown with vertical lines.

Next we consider collapse risk in this case. Given an aggressive motion, the probability of collapse is 0.3. Given a benign motion, the probability of collapse is 0.03. Given an MCE motion of any type, the probability of collapse is

$$P(C|MCE) = P(C|A,MCE) \times P(\mathrm{A}|MCE) + P(C|B,MCE) \times P(\mathrm{B}|MCE) \quad (1)$$

$$= 0.3 \times (1/3) + 0.03 \times (2/3) \quad (2)$$

$$= 0.12 \quad (3)$$

where *A* and *B* denote aggressive and benign ground motions, respectively.

So, relative to the baseline case, in this case the pooled mean Story Drift Ratio is the same, but the probability of collapse is slightly higher (and no longer satisfies the ASCE 7 target of 0.1). Keep in mind, however, that this was an extreme case where the mean demands from the aggressive motions were 2.5 times that of the benign motions. Let us next consider a likely-more-realistic example.

3.4 CASE 2: A SUBSET OF MOTIONS ARE MODERATELY AGGRESSIVE

Now consider the case of more interest: some of the ground motions are of a particular type that produces larger structural demands. Consider the following case:

- 1/2 of motions are moderately "aggressive," and produce a mean demand of 0.035.
- 1/2 of motions are "benign," and produce a mean demand of 0.025.
- All other parameters are the same as Case 1.

These distributions are illustrated graphically in Figure 8. In this case the responses from the two types of motions more similar than in Case 1, but they still differ by 40%, which is substantial.

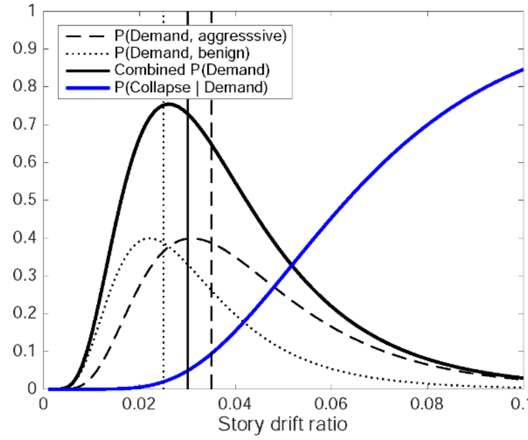

Figure 8. Illustration of Case 2 (some moderately aggressive motions). Mean values for each demand distribution are shown with vertical lines.

Again, if the motions were pooled, this building would barely satisfy a 3% mean story drift ratio requirement, but if the two types of motion were treated separately, the building would fail the check.

Next we consider collapse risk. Given an aggressive motion, the probability of collapse is 0.14. Given a benign motion, the probability of collapse is 0.06. Given an MCE motion of any type, the probability of collapse is 0.10

So, relative to the baseline case, this case has identical mean (pooled) drift, and essentially identical probability of collapse given MCE. This indicates that a building subjected to two types of motions, and having an acceptable mean (pooled) drift, may not have more collapse risk than a building subjected to a single type of motion. And if true, that implies that no change to the standard acceptance criterion is required if the building is subjected to two types of motions.

3.5 ESTIMATION ERROR

An additional issue to consider is that, by looking at subsets of the structural responses independently, there is a loss of estimation precision. Assuming a demand dispersion of 0.4 (a typical number[1] when using amplitude scaled motions),

- The mean response estimate obtained from 11 analyses has a standard error of 13%. This means that two thirds of the time the analyst will get a mean response estimate within +/- 13% of the true value, and 95% of the time the analyst will get a mean response estimate within +/- 26% of the true value.
- The mean response estimate obtained from 5 analyses has a standard error of 20%. This means

[1]This number is smaller than the β_D=0.5 used in the above calculations because the above number also considered modeling uncertainty in addition to record-to-record variability.

that two thirds of the time the analyst will get a mean response estimate within +/- 20% of the true value, and 95% of the time the analyst will get a mean response estimate within +/- 40% of the true value.

These intervals are cut in half for a demand dispersion of 0.2 (a typical number for spectrum compatibilized motions).

If the analyst is to split the 11 ground motions into two (or more) sets of ground motion types, the above results indicate the degree of estimation error introduced by considering subsets of ground motions separately.

3.6 IMPACT OF MULTIPLE SOURCES ON MCE AMPLITUDES

An argument raised by some analysts is that separate checks of ground motions should be required in two-source situations, in order to account for the increased seismic risk present when two (or more) sources are present near a site. In this subsection we briefly note that this issue is already accounted for in the determination of the MCE spectrum.

To illustrate, Figure 9 shows a schematic ground motion hazard curve that is typical of sites where more than one seismic source contributes to hazard. The horizontal axis of the plot is a metric of ground motion intensity (e.g., spectral acceleration with a given period and damping), and the vertical axis is the annual rate of exceeding that intensity level at the site of interest. Ground motion hazard curves from two seismic sources are shown in red and blue, and the overall hazard is computed by summing the exceedance rates from the two sources (and indicated in black).

For the design and assessment documents discussed above, the MCE ground motion amplitude is typically the amplitude with a $4*10^{-4}$ annual rate of exceedance (i.e., 2% probability of exceedance in 50 years).

For the example in Figure 9, the MCE amplitude would be 0.6g. Note that if one of the two seismic sources were removed from consideration, the hazard curve would be lower (because the removed source would not contribute to hazard) and the MCE would also be correspondingly lower. For this example, if Source 2 were removed then the Source 1 hazard would also be the total hazard and the MCE would be MCE_1=0.48g. Or, if Source 1 were removed, then the MCE would be MCE_2=0.52g. So we see that the two sources at this site do create a larger MCE amplitude than if only a single source were present. There is no need to further compensate for the two sources at the structural response acceptance criterion stage.

A final note on this topic is that the above calculations assume that the MCE computation is ”probabilistic.” The above-discussed guidelines do also allow for some sites to have a ”deterministic” MCE, in which the largest median-plus-one-standard-deviation amplitude from any source is used at the MCE amplitude. With a deterministic calculation, the presence of multiple seismic sources does not increase the hazard. But if this is deemed a problem (and there is a good argument to be made that it is a problem), the solution lies in revising or removing the deterministic MCE calculation procedure, and not in revising the acceptance criterion checks.

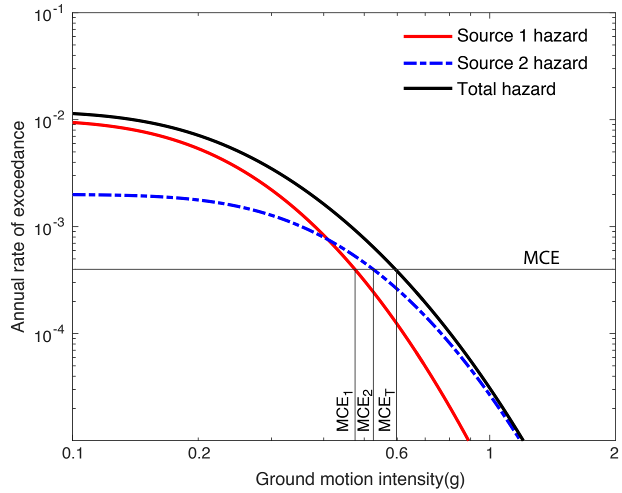

Figure 9. Hazard curves and potential MCE amplitudes for an example site with two earthquake sources.

3.7 TREATMENT OF MULTIPLE SOURCES VERSUS MULTIPLE CONDITIONING PERIODS IN CONDITIONAL MEAN SPECTRUM-TYPE CHECKS

The proposal evaluated above (to treat ground motion subsets separately) looks at first glance like the accepted approach of using multiple conditioning periods in a Conditional Mean Spectrum (CMS) calculation, and checking the results from these cases separately against acceptance criteria (e.g., Baker 2011; American Society of Civil Engineers 2016; Loth and Baker 2015). But there is a key difference in the two cases that leads to the difference in treatment.

In the CMS case, the distinct conditioning periods are associated with distinct hazard analyses. So the response results cannot be combined as in Equation 2. In the case considered here, the distinct types of ground motions are subsets of a single hazard analysis, and the distinct types of ground motions are simply two (or more) ways of representing a single loading amplitude.

Because of this difference, it does not follow that distinct types of ground motions should be considered separately in acceptance criteria checks.

4 CONCLUSIONS

This paper presented discussion of sites in which more than one type of ground motion may be associated with MCE amplitude motions. Primary discussion focused on sites such as Seattle, where both subduction and crustal ground motions are of interest, but the arguments are largely relevant for other situations such as when directivity-pulse-like motions and ordinary motions are of interest.

The question of interest was whether standard acceptance criteria checks (which compare the mean response from a suite of ground motions versus some allowable response level) are sufficient, or whether each type of ground motion should be separately evaluated using the acceptance criteria checks.

Idealized calculations were used to evaluate collapse risk given MCE level shaking, for three cases with differing levels of contribution from two seismic sources, and from differing levels of demand associated with MCE shaking from each source.

The calculations suggested that, for plausible assumed values of demands and capacities, having subsets of motions that produce differing demands does not invalidate standard acceptance criteria checks using mean demands from all ground motions. A circumstance where standard acceptance criteria may break down somewhat is when one type of ground motion is unlikely (so that it is not well represented in a pooled ground motion set) and extremely aggressive in producing structural demands (e.g., it produces 2.5x larger demands than other motions). Even in such a case above (Case 1), the collapse risk of the structure was not dramatically increased. A remaining question is whether limits on peak drift demands from individual motions already adequately address cases like this.

Discussion was also provided of several related issues, such as the increase of statistical estimation uncertainty associated with the alternative to standard acceptance criteria checks, the impact of multiple sources on MCE amplitudes, and the relationship of this issue to the acceptance criteria checks for Conditional Mean Spectra with multiple conditioning periods.

On the whole, the results all support the current procedure of grouping all ground motions together when computing mean responses for acceptance criteria checks. Unless we obtain evidence of real-world buildings where extreme response differences are observed among grounds motion of multiple types but having the same MCE amplitude, requiring acceptance criteria checks on individual subsets of motions adds analysis complexity that is not justified by any potential insights that might be provided.

ACKNOWLEGEMENTS

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