Representation of Bidirectional Ground Motions for Design Spectra in Building Codes

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The 2009 NEHRP *Provisions* modified the definition of horizontal ground motion from the geometric mean of spectral accelerations for two components to the peak response of a single lumped mass oscillator regardless of direction. These *maximum-direction* (MD) ground motions operate under the assumption that the dynamic properties of the structure (e.g., stiffness, strength) are identical in all directions. This assumption may be true for some in-plan symmetric structures, however, the response of most structures is dominated by modes of vibration along specific axes (e.g., longitudinal and transverse axes in a building), and often the dynamic properties (especially stiffness) along those axes are distinct. In order to achieve structural designs consistent with the collapse risk level given in the NEHRP documents, we argue that design spectra should be compatible with expected levels of ground motion along those principal response axes. The use of MD ground motions effectively assumes that the azimuth of maximum ground motion coincides with the directions of principal structural response. Because this is unlikely, design ground motions have lower probability of occurrence than intended, with significant societal costs. We recommend adjustments to make design ground motions compatible with target risk levels.
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INTRODUCTION

The 2009 NEHRP *Provisions* and *Commentary* (BSSC 2009) introduced a new definition of horizontal ground motions for use in the seismic design of structures. As illustrated in Figure 1 (Huang et al. 2008), bidirectional ground motions in the horizontal plane are represented by the response of a single lumped mass oscillator with a given period and viscous damping ratio. The *maximum-direction* (MD) ground motion is the peak response of the oscillator regardless of azimuth, which in Figure 1 occurs in direction Y. This MD ground motion departs from past practice in engineering, which has specified design spectra based on the geometric mean of response spectral accelerations for the two horizontal components of ground motion.

In this article, we explain why this changed definition of ground motion is controversial. We begin by reviewing alternative definitions of bidirectional horizontal ground motions. We then describe the concept of directionality in structural response, differentiating structures with azimuth-independent and azimuth-dependent dynamic properties. We then explain the source of the controversy with the new ground motion definition, which is that the seismic demand applied to structures with azimuth-dependent properties is expected to be biased relative to the target risk level in the NEHRP *Provisions*. The bias is toward overestimation of ground motion by amounts ranging from 10 to 30% depending on period. We then conclude with a discussion of the societal costs of this change in ground motion definition and recommendations for code-adopting agencies that wish to forgo these biased ground motions within their jurisdiction.

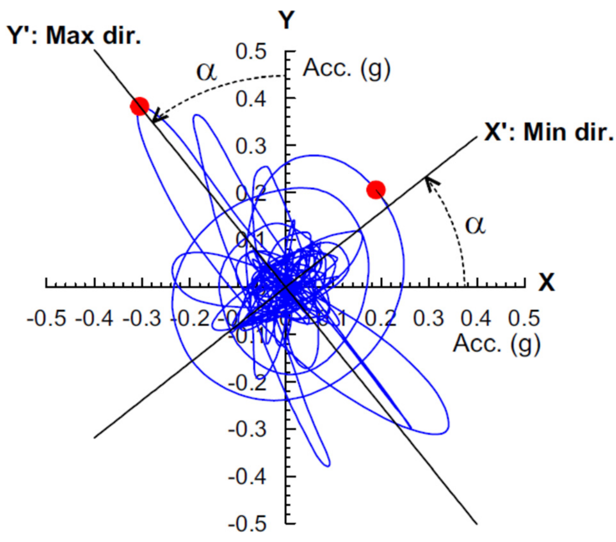

Figure 1. Trace of acceleration orbit of single lumped mass oscillator with direction-independent stiffness to bidirectional ground motion (from Huang et al. 2008). The two axes (X and Y) refer to the directions in the horizontal plane in which ground motion is recorded. Angle α represents the rotation of those axes to the direction of minimum and maximum ground motion.

This article is an opinion piece and not a research article because the bidirectional nonlinear response of structures has not been adequately researched to form professional consensus on how ground motion directionality affects collapse. Changes to the NEHRP *Provisions* and building codes normally follow exhaustive technical research and vetting of the results by design professionals. We are confident that research will take place in the years to come, with outcomes that are unknown at this time. In the meantime, this article presents the view that the proposed new ground motion definitions introduce overconservative bias to design ground motions.

BIDIRECTIONAL GROUND MOTION DEFINITIONS

Earthquake ground motions are recorded by triaxial accelerographs with two components in the horizontal plane and one vertical component. The azimuths of the horizontal components are arbitrary, but are often 000 (north positive) for one component and 090 (east positive) for the other. By definition, the acceleration response spectrum is computed for a single direction of ground motion. Spectral ordinates computed for the two components of horizontal ground motion (here termed $S_{a\text{-}x}$ and $S_{a\text{-}y}$) are combined as follows to compute the geometric mean ($S_{a\text{-}gm}$):

$$S_{a-gm} = \sqrt{(S_{a-x}) \times (S_{a-y})} \tag{1}$$

In the development of empirical ground motion prediction equations (GMPEs), use of the geometric mean has traditionally been preferred over a particular component (or both components) because (1) this averaging process somewhat reduces the data dispersion (as represented by logarithmic standard deviation term, σ_{ln}) (e.g., Baker and Cornell 2006; Watson-Lamprey and Boore 2007) and (2) the geometric mean is a good estimate of the central value of randomly oriented individual components, the variability of which can be dealt with through modification of the standard deviation term (as described further below).

Recognizing the dependence of geometric mean spectra on the original, arbitrary orientation of horizontal accelerometers, Boore et al. (2006) developed an orientation-independent geometric mean parameter denoted GMRotI50. However, whether individual ground motions are represented by the geometric mean of the as-recorded horizontal components or by GMRotI50, the resulting GMPEs provide nearly identical predictions of the median and standard deviation of intensity measures. The Next Generation Attenuation (NGA) GMPEs (Power et al. 2008) use the GMRotI50 parameter, which for practical purposes is consistent with the established practice of using the geometric mean, which dates to the 1980s (Douglas, 2003). More recently, Boore (2010) introduced an orientation-independent 50th-percentile single-component spectrum as an alternative to GMRotI50, but this parameter has not yet been used in the development of GMPEs.

Two additional measures of horizontal spectra are important for the discussion that follows. The first is the arbitrary component spectrum ($S_{a\text{-}arb}$), which is the response spectrum of the horizontal motion that would occur in any arbitrary azimuth. By definition, $S_{a\text{-}arb}$ will be smaller than the GMRotI50 spectral ordinate for 50% of the possible orientations. Beyer and Bommer (2006), Campbell and Bozorgnia (2007, 2008), and Watson-Lamprey and Boore (2007) found that GMPEs derived from arbitrary-component motions have similar medians but larger standard deviations relative to GMPEs derived from the geometric mean. The second additional measure is the maximum-direction ground motion defined in the *Introduction* and illustrated in Figure 1. This spectral ordinate, which is computed using input motions oriented in two orthogonal directions, matches the spectral ordinate that would be computed from the single-component ground motion oriented in the critical azimuth (marked as Y in Figure 1) for a linear oscillator. The orientation of this critical azimuth varies with oscillator period. The medians of maximum-direction ground motions are systematically higher than those for the geometric mean by factors ranging from 1.2 to 1.35 depending on oscillator period (Beyer and Bommer 2006; Watson-Lamprey and Boore 2007; Campbell and Bozorgnia 2007, 2008). Using different procedures, modification factors of 1.1 to 1.5 were found by Huang et al. (2008). Moreover, the standard deviation is higher for maximum-direction ground motions than for geometric mean ground motions (e.g., Watson-Lamprey and Boore 2007).

The USGS probabilistic ground motion maps (e.g., Petersen et al. 2008; http://earthquake.usgs.gov/hazards/designmaps/) are based on the GMRotI50/geometric mean representation of horizontal ground motion as are the design maps used in NEHRP *Provisions* prior to 2009 and in building codes derived from those prior versions of the *Provisions*. The ground motion design maps utilized in the 2009 NEHRP *Provisions* are modified from the ground motion hazard maps by (1) limiting the probabilistic ground motions through application of a deterministic cap in near-fault areas; (2) application of a risk coefficient (Luco et al. 2007) that is intended to provide a uniform probability of collapse across the U.S. and that replaces the two-thirds factor in previous versions of the code (i.e., Section 3.1.3 of 2003 NEHRP *Commentary*, BSSC 2003); and (3) multiplication by 1.1 (short periods) and 1.3 (mid-periods) to approximately convert from GMRotI50 to maximum-direction ground motions (using factors from Huang et al., 2008). Note that this conversion does not account for the higher standard deviation of maximum-direction ground motions.

DIRECTIONALITY IN STRUCTURAL RESPONSE

Some structural systems, such as flagpoles and circular tanks, do not have preferred directions of response and have equivalent dynamic properties in all horizontal directions. We refer to such structures as having *azimuth-independent* properties. Many other structural systems have preferred response directions, perhaps with different dynamic properties in those principal directions, which we refer to as *azimuth-dependent* structures. For example, bridges and dams have distinctly different strengths and stiffnesses transverse to their principal axis than other directions, and the design is often controlled by the transverse response. In general, buildings have different stiffnesses and strengths depending upon the orientation of the axes along which these properties are determined. Partially for this reason, structural engineering design practice for lateral loads includes analysis with respect to two principal axes (e.g., longitudinal and transverse). The use of response spectra in design directly implies this approach because differences in stiffness generally lead to differing fundamental periods of vibration for different azimuths (or principal axes) of the structure. Even structures with identical stiffnesses along each principal axis (hence identical periods) tend to have preferred directions of response associated with their vibration modes, so even those structures are azimuth-dependent in that respect.

For structures with azimuth-independent properties, the single lumped mass oscillator model that is the basis for maximum-direction ground motions is a good analogue to the real system. Structures with azimuth-dependent properties will generally have different phasing of the modal responses along the two principal axes, particularly when the vibration periods are distinct. This causes peak responses in the various modes to occur at different times. This is the reason for modal combination rules in the building code (e.g., Section 12.9.3 of ASCE-7; ASCE 2010). Very little research has been undertaken to investigate the complex three-dimensional (3-D) nonlinear response of structures with azimuth-dependent properties. Rather, previous work has undertaken two-dimensional (2-D) analyses along the two principal axes of a building and attempted to infer the 3-D response (e.g., Christovasilis et al. 2009, FEMA 2009). Due to the lack of suitable research, there is no scientific basis at this time for declaring any particular component of ground motion as controlling the collapse risk of structures with azimuth-dependent properties.

RATIONALE FOR THE BIAS CONCERN

Studies of ground motion directionality have shown that the azimuth of the maximum direction ground motion is arbitrary for fault distances (R_{rup}) larger than approximately 3–5 km (Campbell and Bozorgnia 2007, Watson-Lamprey and Boore 2007). At closer fault distances, the azimuth of the maximum direction motion tends to align with the strike-normal direction, but let us assume for this discussion that we are working with a structure located at $R_{rup} > 3$–5 km.

Let us next assume that the subject structure has azimuth-dependent properties, such as a building with lateral force resisting systems oriented in the transverse and longitudinal directions. Because the response is likely to be dominated by lower modes producing vibration aligned with the principal axes of the building (Clough and Penzien 1993), it stands to reason that the ground motion that should be provided for design purposes should be appropriate for those same axes. Because the alignment of the building and the azimuth of the maximum component of ground motion are both arbitrary, the ground motions that should be used to evaluate the response in a particular direction is the arbitrary component.

As noted previously, that arbitrary component can take on a range of values, with the median being GMRotI50 and with the maximum possible value being the maximum-direction ground motion. Hazard analyses performed on the arbitrary component of ground motion with its relatively high standard deviation (compared with GMRotI50) will account for the arbitrary directionality probabilistically; as return period increases, the ground motion will draw closer to (although never reach) the maximum-direction ground motion. As shown by Baker and Cornell (2006), hazard analyses performed for the geometric mean (similar to GMRotI50) provide ground motions that are slightly lower than those for the arbitrary component at long return periods. This suggests that GMRotI50-based ground motions (such as those on the USGS probabilistic ground motion maps) may be slightly unconservative relative to the more desirable arbitrary-component ground motions because of the lower σ_{ln} for GMRotI50. However, there is a counter- balance, which is that σ_{ln} values for the soft rock or soil site conditions in most urban regions of the western U.S. are lower than those for the relatively stiff reference site conditions used in the USGS maps. Hazard analyses performed with relatively high σ_{ln} values for a reference rock site and then modified by deterministic site factors overestimate ground motions (Goulet and Stewart 2009). In summary, while the GMRotI50 σ_{ln} is arguably too low relative to σ_{ln} for the arbitrary component (difference in variance of 0.03–0.05 for PGA to 1.0 sec S_a; Campbell and Bozorgnia 2007, Watson-Lamprey and Boore 2007), it is also too high for common site conditions (e.g., for a change of V_{s30} from the common reference value of 760 m/s to a typical soil site condition of 270 m/s, the difference in variance is $\sim$0.1 to 0.03 for PGA to 1.0 sec S_a for M>7 earthquakes at distances $< \sim$15 km; Abrahamson et al. 2008). While this trade-off of compensating errors is hardly ideal, the use of the existing USGS probabilistic ground motion maps combined with NEHRP site and risk factors represents the most reasonable (probabilistically most consistent) basis currently available for evaluating design ground motions along the principal axes of structures.

USGS design maps are not based directly on the probabilistic ground motion maps, but include the aforementioned adjustment factors for maximum-direction ground motions. Figure 2 illustrates schematically one reason why this biases the design ground motions relative to the intended risk level. The ground motion hazard curve provides the indicated "probabilistic ground motion intensity" at 2% probability of exceedance in 50 years (that ground motion can be adjusted subsequently using mapped risk factors for the target collapse risk level). However, as indicated in Figure 2, the NEHRP design maps use a ground motion that is scaled up from the probabilistic ground motion intensity, and this scaled-up motion provides a target risk level having lower probability, which is equivalent to longer return period. Accordingly, the design-basis ground motions are biased relative to the target risk level.

We recognize that the assumption of structural response being dominated by lower modes associated with the principal axes of the building has not been verified by 3-D nonlinear response history analyses taken to collapse. Accordingly, we cannot cite scientific research to support this part of our argument. However, given that structural response at lower shaking levels is generally dominated by lower modes (Clough and Penzien 1993), this approach is more plausible than to assume the response is dominated by the maximum

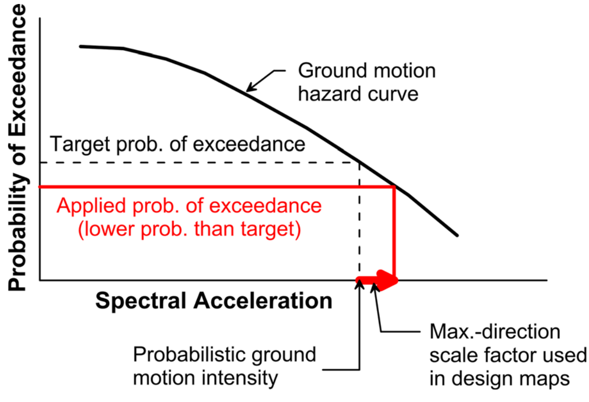

Figure 2. Schematic ground motion hazard curve.

direction of ground motion, which is almost certainly not aligned with the principal axes of a structure.

A second reason why the maximum-direction ground motion definition introduces bias is related to the fact that the critical azimuth associated with maximum-direction motions changes with period. A uniform hazard spectrum (UHS) is already an envelope of ground motions from many earthquakes that is not realizable in any one event. The maximum-direction UHS combines ground motions in different directions that cannot occur. It is a non-realizable case of a UHS that is already non-realizable, which is a step away from the realistic ground motions that should be the basis for risk-based analysis of structural performance.

DISCUSSION AND RECOMMENDATIONS

EFFECT OF NGA AND UPDATED SOURCE MODELS ON MAPPED GROUND MOTIONS

Petersen et al. (2008) compared probabilistic ground motion maps developed in 2002 and 2007 (the 2007 maps are the probabilistic ground motion maps used in the 2009 NEHRP *Provisions*). The maps apply for a 2% probability of exceedance in 50 years hazard level and the V_{s30}=760 m/s site condition. The result is shown in Figure 3 for 1.0 sec S_a. The changed ground motions result from adoption of the NGA relations as the GMPEs for active regions from updated source models. While ground motions increase in some areas (e.g., Northern California, Oregon), the ratios of new/old values range from approximately

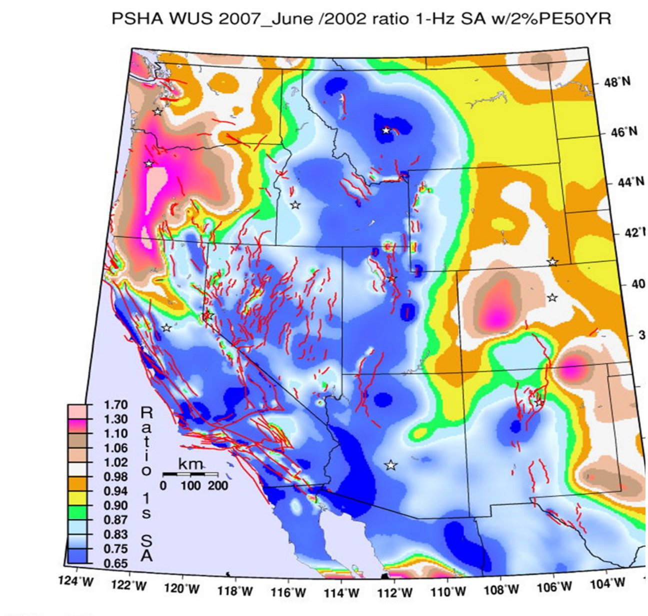

Figure 3. Changes is mapped 1.0 sec Sa at the 2% probability of exceedance in 50 years hazard level in the western United States (Petersen et al. 2008).

0.65 to 0.75 in southern California and the San Francisco Bay Area. It is interesting that this reduction is nearly completely offset by the maximum-direction/GMRotI50 factor of 1.3 applied to develop the NEHRP 2009 design maps.

If the intent of the modified ground motion definition was to avoid significant changes in the design maps, that could have been achieved in a more transparent manner by weighting new and old GMPEs and source models with the stated objective of not allowing changes in mapped ordinates beyond a selected threshold. Because the use of maximum direction ground motions is not justified for the vast majority of structures, our view is that changing the ground motion definition in that manner is not an appropriate means by which to achieve stability in design ground motions.

COSTS AND BENEFITS OF INCREASED DESIGN GROUND MOTIONS

The 10% to 30% increase of design ground motions introduced by the changed ground motion definition in the 2009 NEHRP *Provisions* (relative to ground motions that would have been computed for GMRotI50) affects the costs of new construction and retrofits if put into law in the form of building codes. The increased design ground motions translate to increased base shear forces, which in turn lead to increased member sizes and potentially more costly detailing. The effect of the change is dependent on the nature of individual projects. Structure costs as a percentage of total construction costs for new construction are usually in the range of 15% to 20%. The portion of this cost directly attributable to seismic resistance in areas of high seismicity might range from one-quarter to one-third of the total structural cost. It is not unreasonable to estimate that the increased design ground motions in the *Provisions* would add costs on the order of magnitude of 1% to new construction in seismically active areas such as California. Using an estimate of non-residential building construction spending in California of approximately $60 billion annually, the added premium is over $500 million per year in California alone.

Experienced earthquake engineering practitioners know that increases in base shear capacity do not necessarily translate into proportionally higher margins of safety, capital preservation, and post-event revenue. By increasing design ground motions, the new ground motion definition in the 2009 NEHRP *Provisions* will result in increased elastic base shear strength. It is not, however, a foregone conclusion that the higher elastic design forces would improve earthquake performance. A structure is designed and detailed to perform inelastically once its elastic strength has been reached. Available ductility can be reduced when the elastic strength of a component is increased unless other adjustments are made to the design. Increased elastic design forces actually could worsen performance in some cases.

More to the point, the benefit of the increase in strength is not quantified in any way. The design and engineering community consequently cannot inform their clients or the general public of the result of this mandated investment. This lack of transparency is diametrically at odds with fundamental principles of performance based engineering.

In addition to concerns about construction cost, we also are concerned about unnecessary use of resources and increases in the carbon footprint of building construction. When sustainability trends are generally toward more efficient use of materials, it seems contrary to adopt new ground motion definitions that purposely bias the outcomes toward increased materials consumption.

We are strong proponents for the best seismic safety possible given the constraints of our current state of knowledge and available design and construction resources. We support the NEHRP 2009 concept of defining a level of risk (in this case 1% probability of collapse in 50 years) and that is applied nationwide. We further recognize that the selected risk level is to some extent arbitrary, different constituencies may set different risk thresholds, and the extent to which design-basis risk levels are achieved in constructed structures is uncertain. That being said, once a tolerable risk level is chosen, our position is that it should be maintained without bias in the analysis and design process. The increase in ground motions introduces bias relative to the stated risk objective, which we do not support.

RECOMMENDATIONS

For structures with azimuth-independent properties, we support use of the 2009 NEHRP *Provisions* and subsequent ASCE-7/10 document, including the existing ground motion design maps. For structures with azimuth-dependent properties, we recommend use of the 2009 NEHRP *Provisions* and subsequent ASCE-7/10 document with the exception of the ground motion design maps. Until new design maps are prepared, the existing design maps can be used with reductions by the factors of 1.1 (short periods) and 1.3 (mid periods) along with the NEHRP site factors and risk factors. For most of the United States, the application of the 1.1 and 1.3 factors should reduce the spectral ordinates to values consistent with the 2007 USGS probabilistic ground motion maps for the 2% in 50 years hazard level, adjusted for uniform collapse risk.

In the long term, if use of maximum direction design ground motions persists in the design community, GMPEs for that ground shaking parameter will need to be developed. Those GMPEs will have higher medians and also higher standard deviations. The larger standard deviations are not currently accounted for in the development of the USGS hazard or design maps, and will cause the mapped ordinates to increase relative to current values in seismically active areas.

We close by noting that opposition to the changed ground motion definition in the 2009 *Provisions* was voiced by 11 member organizations that either voted “No” or expressed reservations about the proposal. Hence, the opinions expressed in this article are widely held among design professionals.

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