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Orientation dependence of ground motion and structural response of reinforced concrete space frames

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Abstract. Horizontal seismic demand is represented in seismic design codes by a single elastic response spectrum. Using a single spectrum conceals the difference between ground motions that are highly polarised and those for which the demand does not depend significantly on the orientation. In this study, two suites of ground motion were developed using the program *RspMatchBi*, with each suite characterised by different orientation dependence and seismic demand. Reinforced concrete space frames with various arrangements in plan were modelled in *OpenSees*. Non-linear time history analysis was carried out with the suites of records applied to the models at 9 degree increments, through 360 degrees of non-redundant orientations. An iteration process was implemented to determine the scaling factor for each ground motion at which storey drift exceeded a design level of 2.5% in 50% of applied orientations. Various performance criteria were also taken into account to investigate the changes in orientation dependence of seismic structural response. Results suggest that azimuth-dependent structures are more vulnerable to orientation-dependent seismic demand, and that the investigation of directionality for seismic design needs further consideration in terms of performance criteria and structural periods at each principle axis.

Keywords: Orientation dependence of ground motion, azimuth-dependent structures, seismic design

1 INTRODUCTION

Prior to the Next Generation Attenuation (NGA) project (Power et. al, 2008), most codes of practice expressed the horizontal components of ground motion in terms of a geometric mean (GM_{xy}) or envelope (Env_{xy}) spectrum. These measures do not take into account the fact that the amplitudes of horizontal components of earthquake records are not identical at all orientation angles. The NGA project employed a new geometric mean definition, GMRotI50, which does not depend on the arbitrary orientation of the instrument that recorded the ground motion (Power et al., 2008; Boore et al., 2006). Instead, it is based on a set of geometric means calculated for all possible non-redundant orientations of the as-recorded orthogonal horizontal components, and takes the median value over all the orientation angles. This measure is independent of sensor orientation and all spectral ordinates correspond to a single angle of rotation. However, it has been observed that the spectral ordinates can be sensitive to the period range used in its definition (Grant, 2011).

The US standard, ASCE 7-10, has adopted the maximum rotated spectral response as its seismic demand measure, which gives the orientation of maximum spectral demand for every period value considered. The orientation angle governing each ordinate may differ for different periods (Grant, 2011). The maximum rotated response gives the direct measure of peak elastic demand on axisymmetric (azimuth-independent) structures, such as simple bridge piers (Stewart et al., 2011).

It has been argued and investigated in several studies that the most appropriate seismic demand depends on the structural form (Grant, 2011; Padilla et al. 2010; Ghazizadeh, 2012). Structures (with regular plan configuration) are regarded as azimuth-dependent or independent; the former has different dynamic properties in its orthogonal principle axes and the latter corresponds to structures such as chimneys and circular bridge piers. In addition to this definition, in this study azimuth-dependent and independent structures are also assumed to have different and similar elastic response in each direction, respectively. Apart from the structure itself, performance criteria can introduce a source of orientation dependence. For example, a close adjacent structure may impose a tight displacement limit in a single direction, but not in other directions, meaning that the orientation of the peak structural response is important.

This paper aims to investigate how the nonlinear response of azimuth-dependent and independent space frame models subjected to two suites of orientation dependent and independent ground motions vary with orientation angles. Four reinforced concrete space frames are modelled in *OpenSees* and nonlinear time history analyses are carried out on the models using the record suites. Results from two suites are compared to understand the extent to which realistically matched ground motion records influence the structural response and seismic demand. The study is also extended for two of the frame models considering various performance criteria to investigate how the results of two suites change with the response quantity of interest.

2 ORIENTATION OF GROUND MOTION AND STRUCTURAL RESPONSE

The intensity of ground motion varies with orientation angle, and is not often equally strong in all directions. Ground motion records provided in databases are based on the orientation of the accelerometer installed in the field, and two instruments with different recording angles will exhibit different acceleration histories. Orientation-dependent records are more apparent in the near-fault region where ground motions can potentially be polarised in one direction. Fig. 1 shows the bidirectional spectral response orbit with fundamental period of 1.3 seconds and damping ratio of 5% of critical for Imperial Valley earthquake PEER record 183. As can be seen, the peak response is approximately two times the response in the orthogonal orientation, and the amplitude is not uniformly distributed in all directions. Orientation-independent and dependent ground motions impose different demand in seismic structural design. This has been inappropriately considered in seismic codes and can introduce biased results in bidirectional time history structural analysis. Three dimensional structural models should be analysed using consistent pairs of ground motion records such that they are simultaneously imposed along two horizontal structural axes (the vertical component is often neglected for regular structures), and are compatible with the seismicity of the site in terms of both the level and variation of intensity with orientation angle.

Structural form and configuration can define a source of orientation dependence in seismic structural response. Nonlinear response of structures that are azimuth-dependent is different compared to those azimuth-independent, subjected to a similar ground shaking. In different cases of azimuth-dependent structures, the effect of orientation-dependent ground motions would vary depending on the angle in which the structure is subjected to the shaking (Padilla et al., 2010). Different structures decompose differently the ground motions in their principle axes. For instance in the case of current study, longitudinal and transverse frames of a moment-resisting reinforced concrete structure would impose different demands for a polarised ground motion depending on what orientation the shaking is experienced. If the direction of strong component is aligned with the frame in the x direction, the demand would be higher for that frame compared to when the ground motion is applied to the structure at 45˚, as the shaking would be decomposed between the x and y frames equally. As both components of ground motions are modified or scaled to match the peak rotated response in structural analysis, structures would require the peak value in their principle axis for seismic design, and therefore impose conservatively high demand.

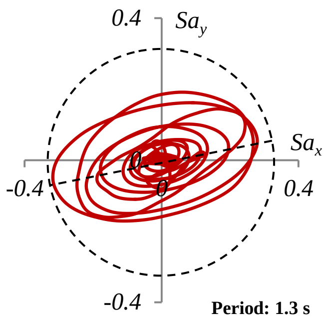

Figure 1. Bidirectional response PEER records 183 from Imperial Valley

Many codes require the most critical orientation of ground motion and the angle of maximum structural response to be taken into account for seismic structural design. It has been argued by several researchers that codes do not consider the probability of exceedance of the structural response and seismic demand consistently (Beyer and Bommer, 2007; Grant, 2011). Hence, the maximum response from all possible angles of incidence between ground motion and structural axes cannot represent the appropriate seismic demand. At least, when there is not enough information about the local faults, the orientation of two orthogonal structural axes would not coincide with the strongest horizontal components of ground motions, and therefore, it is vital in most cases to consider that the probability of the strong shaking is identical in all orientation angles. No matter whether the structure is azimuth-dependent or independent, it is more constructive to consider the performance of structure holistically over all orientation angles.

Structural response can still be orientation-dependent even if both ground motion and structure are orientation and azimuth-independent. This source of directionality comes from the way in which engineers are interested in performance criteria. For structures that are adjacent only in one direction, the lateral displacement is essential to be in a limited range for that particular direction, whereas it is not that much of concern for the orthogonal direction. For instance, a structure in the middle of two buildings in *x* direction would require a different performance criteria compared to a case where one of the adjacent structures did not exist. The former needs consideration in both positive and negative directions of *x*, whereas the latter only require checking in the positive direction. It would also be different for the case where the structure is also surrounded by another building in *y* direction. If the strong component of ground motion aligned with the frame in *x* direction, the structure demand more in *x* than *y* to not exceeding the limit state of lateral displacement. In this study, these essential considerations have been investigated analytically.

3 ANALYSIS METHODOLOGY

3.1 Ground motion records

Two suites of ground acceleration, each comprising twenty two-component records, are used to study the variation of structural response with orientation of ground motion. The original records were selected from the PEER NGA database (Chiou et al., 2008) and matched spectrally to target spectra. Spectral matching of ground motion records are often used for time history analyses. This method matches records’ response spectra to a target spectrum such that they will be consistent with the target level of demand. *RSPMatchBi* (Grant, 2010) is considered for the purpose of this study as it takes into account the bidirectional demand, and therefore it does not operate only with one horizontal component of ground motion. The first suite is orientation-independent for all periods, and was matched to a target geometric mean spectral response which was developed from ground motion prediction equation by Boore and Atkinson (2008) and with the parameters applied in Grant (2010). The second suite is orientation-dependent such that records are adjusted to match two target spectra:

one is the maximum rotated demand, referred to as the major axis demand in this study, and the other is the demand in the orthogonal direction, referred to as the minor axis demand. Major and minor spectra were developed previously in Grant (2010) and were acquired from mean spectra using a factor developed in Watson-Lamprey and Boore (2007), and a period-dependent factor by Hong and Goda (2007), respectively. The original records used in this study are those adopted previously in Grant (2010) and applied later in the study of bridge piers by Padilla et al. (2010).

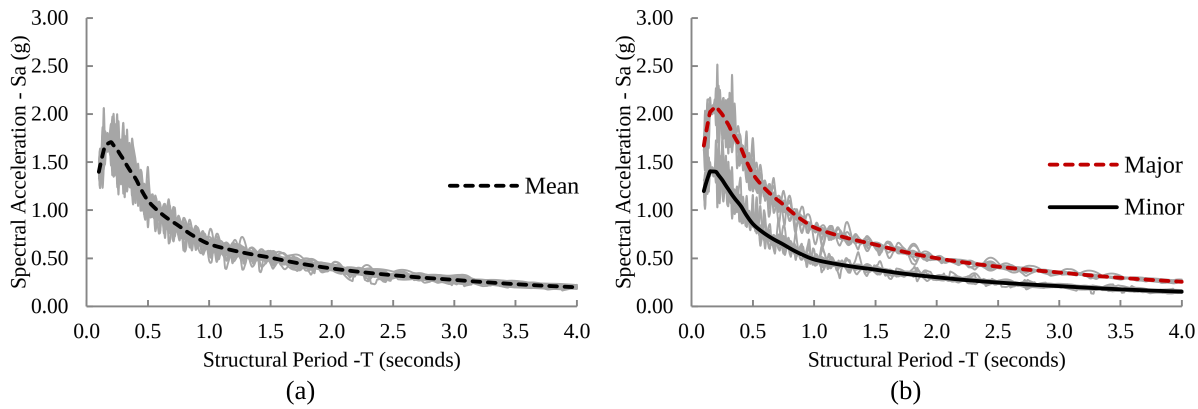

Figure 2. (a) Suite 1: records matched to mean target – dashed black line, (b) Suite 2: records matched to major and minor targets – dashed and solid lines

Two spectrally matched suites were developed for a 2% damping ratio for use in nonlinear time history analysis. Fig. 2(a) and 2(b) represents the suites 1 and 2 which in former both principle axes of ground motion matched to the mean spectrum, and in latter major axis of records (strong component) matched to the major axis of target spectrum, and minor axis of records (weak component) matched to the minor axis of target spectrum. Each record from each suite is then used as seismic input in *OpenSees* program and applied to structures for dynamic analysis. Two suites have relatively the same geometric mean and therefore, it should be informative for the aim of this study to compare the two suites results from time history structural analysis.

3.2 Structural modelling and analysis

Four one-storey reinforced concrete space frames are chosen as structural models in this study. In both longitudinal and transverse directions, the lateral system comprises moment-resisting frames. Variable numbers of bays are defined to better investigate the response of structural models with various plan configurations to the suites of ground motions. Fig. 3(a) and 3(b) show the four proposed structural models and a sample of plan configuration for model M.3. The length of each bay and height of the structure are considered to be constant with the values of 6 m and 4 m, respectively. The *xy* plane geometry of different models are summarised in Table 1. The weight of 103.5 kN, including both live and dead loads, is distributed uniformly on the beams. It is also assumed that the centre of mass and stiffness are at the same location and hence there is no torsional effect on the columns. Nodes at the first floor are assigned to a rigid diaphragm. A damping ratio of 2% is also considered in the *OpenSees* models with tangent stiffness proportional Rayleigh damping.

Structural design is based on the direct displacement-based design (DDBD) procedure described in Priestley et al. (2007) for moment-resisting buildings. It should be noted that DDBD method calculates design moments for two-dimensional frames and, therefore, calculation is carried out separately for each frame of three-dimensional models, and columns sections are designed consistently with the demand capacity in each orthogonal direction. Spectral displacement corresponding to geometric mean target spectrum (Suite 1) is used to find the effective period required in DDBD. For the purpose of section verification, moment-curvature analysis was carried out in *OpenSees*. Typical beam and column sections with a fixed number of bars are proposed and shown in Fig. 3(c) for the current study. The core concrete is composed of 16 integration points (4 × 4) with the confined concrete property and cover concrete is 12 integration points of unconfined concrete. The number of steel bars is assumed to be 12 and 16 for columns and beams respectively. The section steel ratio is fixed in all cases and controlled to be in an acceptable range of 0.5%-2.5% for columns and 0.25%-1.5% for beams.

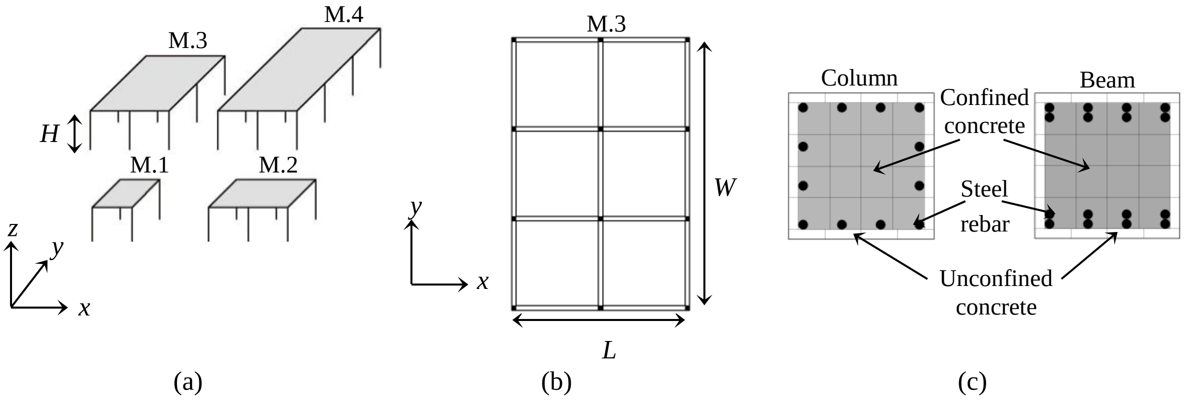

Figure 3. (a) *OpenSees* structural models, (b) Plan configuration, (c) Typical column and beam cross section

In both moment-curvature and nonlinear time-history analyses of models, beam and column sections are built based on the *FibreSection* command in which each cross section is divided into a number of sub-regions where they are independent in their geometry and material properties. The concrete material is modelled by command *Concrete01*, which is a uniaxial material model with degraded unloading/reloading stiffness and no tensile strength. Steel material property is modelled with the command *Steel02*, which is a uniaxial steel material with isotropic strain hardening. Beams and columns are modelled with three integration points to consider the spread of plasticity along the element. Ground motion records are then successively transformed and applied to the assembled models at 9˚ increments of orientation angle for a total of 40 orientations between 0˚ to 360˚. Since structures are regular in plan configuration, numbers of analyses reduced to only orientations between 0˚ to 180˚.

3.3 Processing of results

In this study, the assessment of structural failure, considering various orientation angles in which ground motion applied to the structure, is considered holistically over all 360 degrees. As discussed before, the probability of shaking is identical for all orientation angles, and therefore the suitable response quantity to consider for design is the median over all 360˚. This means that the structural response should exceeds a proposed limit state in 50% of possible orientations. In this study, the orientation angles in which the response quantity of interest exceeds the limit state are regarded as failure. The percentage of orientations that exceed the failure limit is defined as the probability of failure over all directions, P_f. The desired probability of failure over orientation angles, as also defined in Padilla et al. (2010), is 50% (corresponding to median value), and this study aims to find the level of shaking that represents this target.

The quantity of concern for which the structural response is analysed over 40 orientations is proposed to be the inter-storey drift. The maximum inter-storey drift is calculated by the square root of the sum of squares (over time) in *x* and *y* directions. The limit state drift value of 2.5% is used to control the performance limits in the analysis. Models M.1 and M.2 are considered additionally for three cases in which response quantity in not checked for all 40 orientations but rather 1, 2 and 4 equally spaced directions. For each aforementioned cases, the peak positive drift in *x*, peak absolute drift in *x* (i.e.

positive and negative directions) and maximum and minimum drifts in both x and y directions are considered to find the shaking level representing the 50% probability failure in all orientations that ground motion applied to the structure.

Time history analysis is carried out for each suite of ground motions to obtain the probability of failure. In each case the results for the initial run may not be 50% probability of failure and, therefore records are scaled up or down using an iterative approach to acquire the 50% target. To converge on the 50% probability of failure over all orientations, the Bisection iteration process is used, assuming that any scaling factor that leads to a probability of failure between 49% and 51% will be acceptable.

4 RESULTS & DISCUSSION

Nonlinear time history analyses were performed in *OpenSees* for the models and assumptions described in the previous section. For each ground motion, the iteration procedure determined a scaling factor that could lead to failure in 50% of all orientations. For each structural model and each suite of ground motion, 20 scaling factors were attained from the iterative approach. The median values of scaling factors are reported in Table 1 for each structural model. In each suite of ground motion, a record which has the closest median scaling factor to the model was as representative of the suite, for the purpose of demonstrating the trends below. Variation of response with orientation is then demonstrated for that record in Fig. 4(a) to 4(d) as a typical variation of maximum storey drift versus orientation. In the top plots, the red line represents the maximum inter-storey drift described previously in section 4, and the shaded area demonstrates the regions where response quantity exceeds the limit state of 2.5% inter-storey drift. In the bottom plots, $P_f(GM,\theta)$ shows the probability of failure as the function of ground motion and orientation angle. Shaded areas take the value of 1 which means the drift limit is exceeded and other areas 0 which shows the drift value of less than 2.5%.

Table 1. Reinforced concrete structural models and scaling factors

Models	Dimensions		Median SF for $P_f = 0.5$ for all GM		
	X (m)	Y (m)	Suite 1	Suite 2	Suite 2/1
M.1	6	6	1.249	1.161	0.915
M.2	12	6	1.497	1.360	0.880
M.3	12	12	1.423	1.316	0.898
M.4	12	18	1.463	1.350	0.888

As it was the objective of this study to compare the response from two suites of records, ratios of suite 2 to suite 1 are also calculated and the median values are reported in Table 1. The scaling factors required for suite 1 is higher than suite 2 which points out that a lower scaling factor is needed in suite 2 to achieve the same probability of failure in suite 1. Therefore, ground motions in suite 2 impose marginally higher demand on the structure. In models M.1 and M.3, which are less azimuth-dependent and have similar numbers of bays in their principle axes, the ratios of suite 2 to suite 1 are approximately 0.9. Padilla et al. (2010) also obtained similar scaling factors for circular and square bridge piers, suggesting that suite 2 imposes around 10% higher demand on structures which are less azimuth-dependent. However, in their study, the response quantity of interest to control performance limits was peak material strain. This implies that the ratios of two studies are not significantly influenced by changing the response quantities. For the case of models M.2 and M.4, the factors are slightly less than 0.9, and are similar to each other. This suggests that for structural frames which are more azimuth-dependent and therefore have different fundamental periods in each direction of their principle axes, suite 2 imposes higher demand. Hence, azimuth-dependent structures are more sensitive to ground motions with realistic orientation dependence in their elastic spectral response.

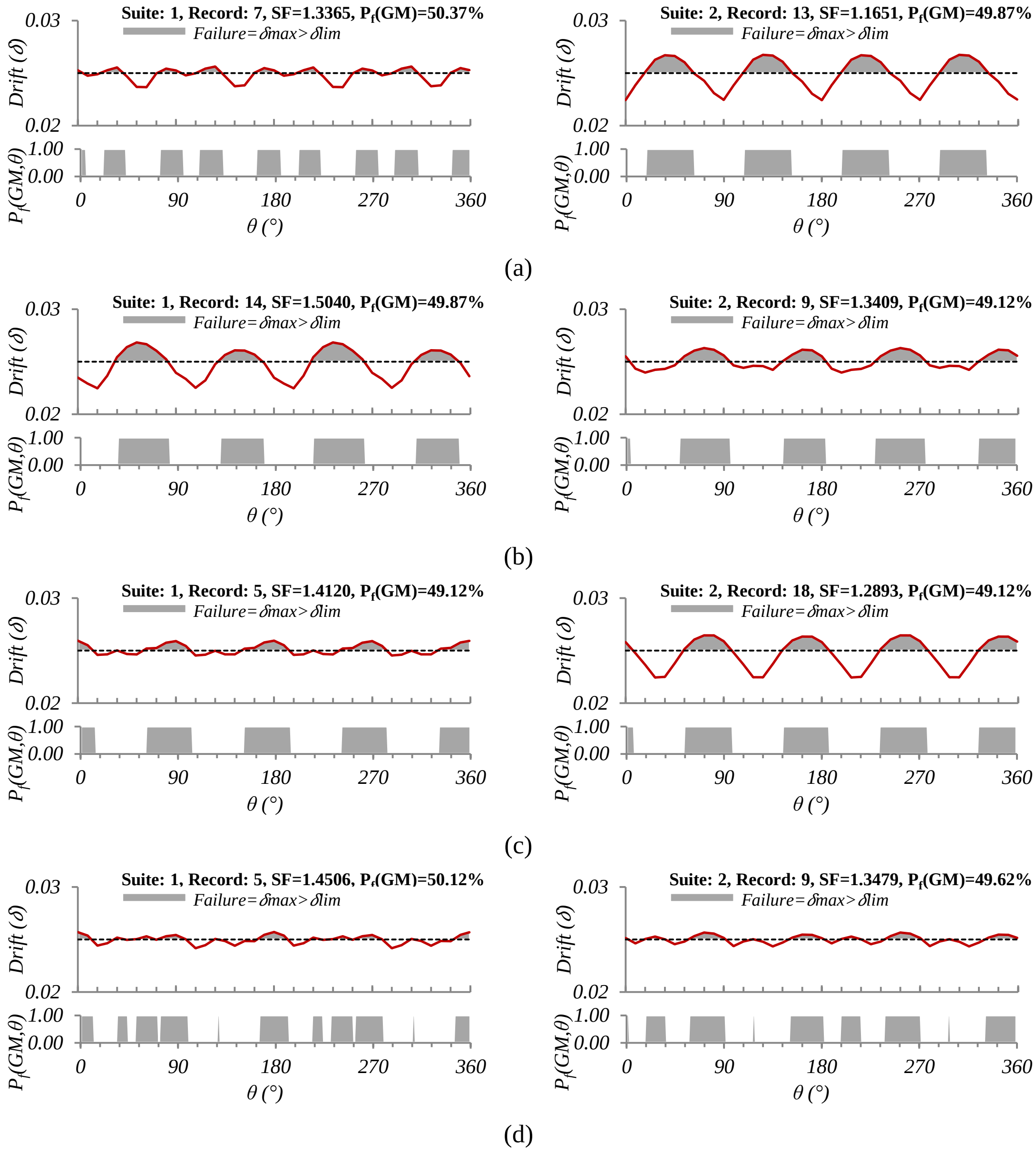

Figure 4. Maximum inter-storey drifts vs. orientation angles for suite 1&2: (a) M.1 (b) M.2 (c) M.3 (d) M.4

It is clear from the ratios that structural configuration that is more independent in stiffness would lead to higher ratios. In Padilla et al. (2010) study on bridge piers with circular, square and various rectangular cross sections, suite2/suite1 ratios are higher in the case of azimuth-dependent rectangular sections (0.935 at the highest), whereas in this study ratios decreased to 0.88 for azimuth-dependent models. This suggests a different regime of vulnerability to orientation-dependence seismic demand for each structural type. However, both in this study and Padilla (2010), there is no statistically significant difference in the ratios obtained from different structural models, and they all lie between 0.8 and 1. Although ratios are slightly different, results in Fig. 4 show the cases (M.1 suite 2, M.2 suite 1, M.3 suite 2) where the value of maximum inter-storey drift is twice as much as the limit state value, and for the purpose of conservative applications in design, a failure probability of higher than 50% may be appropriate.

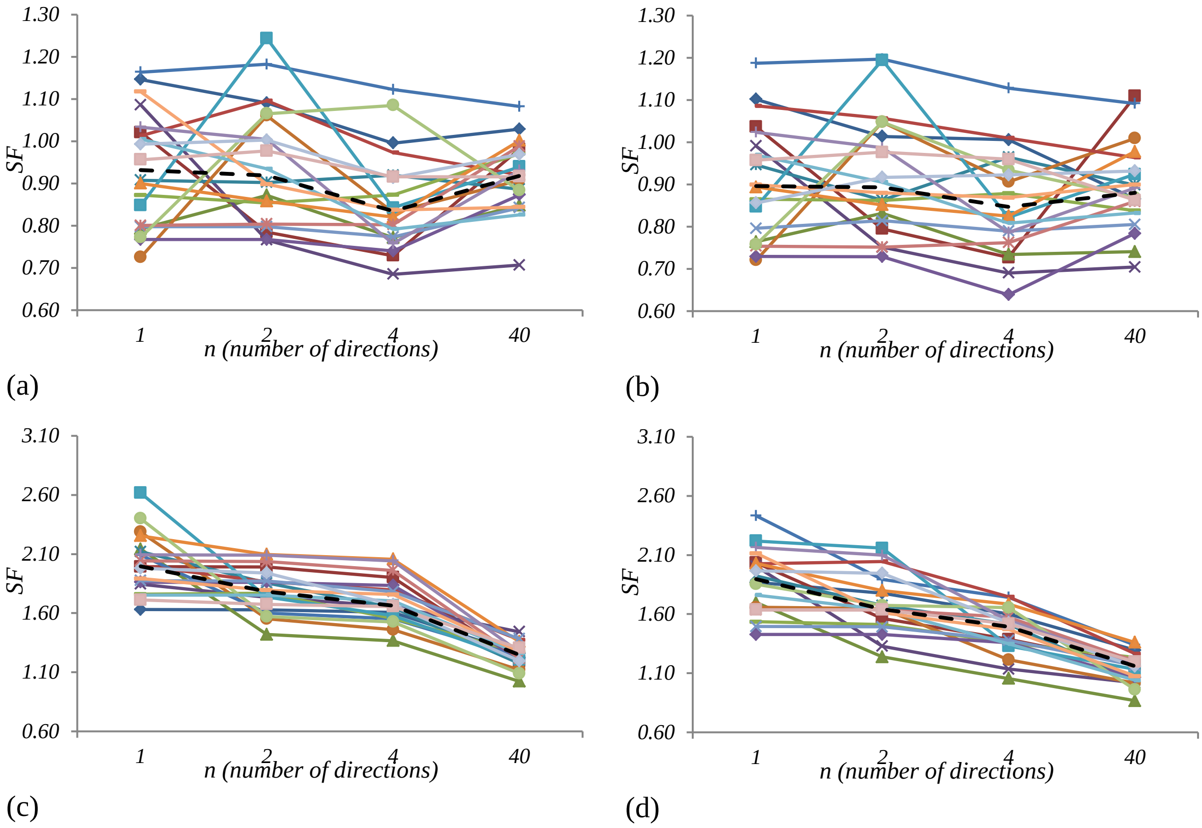

Figure 5. Scaling factors versus various directions of response checking: (a) Ratios of suite2/suite1 for Model M.1 (b) Ratios of suite2/suite1 for Model M.2; (c) & (d) Scaling factors of suite 1 & 2 for model M1

As described in section 3.3, performance criteria were changed for models M.1 and M.2 in order to consider three extra cases in which response quantity in not checked for all 40 orientations. For the case n = 1, the peak positive drift in x direction was recorded for each orientation that ground motion applied to the models to find the shaking level representing the 50% probability failure in all directions. The same was carried out for n = 2 and n = 4 where peak absolute drift in x (i.e. positive and negative directions) and maximum and minimum drifts in both x and y directions are considered respectively. Fig. 5(a) and 5(b) demonstrate the distribution of ratios of suite 2 to suite 1 scaling factors for 20 ground motion records and correspond to structural models M.1 (a single bay in each orthogonal axis) and M.2 (a single bay in one axis and two bays in orthogonal), respectively. The median values of 20 records are presented as a dashed line in each figure and tabulated in Table 2. Although in these figures there is no regular pattern on whether ratios in general decreased or increased, scaling factors and median values associated with each suite in Fig. 5(c) and 5(d) present a decreasing regime from n = 1 to 40. This means that for response checking restricted to a lower number of orientations, higher seismic input is required to obtain 50% failure probability. In the case of ratios which are proposed as directionality factor to be later implemented in seismic design, no general regime can be seen although median values in both models M.1 and M.2 have a similar pattern. Based on median values, when the response is checked for the maximum and minimum drifts in both x and y directions, suite 2 is increased more in demand compared to the case where drift is checked for all 40 orientations. Padilla et al. (2010) did not investigate the variation in ratios with number of orientations considered for response checking, and therefore no comment can be made in comparison to this study. Further investigation is required in order to understand how changing the interest in checking response with orientation would influence the directionality factor.

Table 2. Median SFs for various numbers of response checking

Models	Median of Suite2/Suite 1 for P_f=0.5 for all GM and n directions			
	n=1	n=2	n=4	n=40
M.1	0.932	0.918	0.834	0.915
M.2	0.896	0.893	0.847	0.880

5 CONCLUSIONS

This paper discusses different sources of orientation dependence in seismic structural response. Ground motion records, structural form and performance criteria are introduced as the major sources which need consideration in seismic design. Two target measures of geometric mean and major-minor spectral response (developed by Grant, 2011) were used to conduct a series of structural analysis on four one-story reinforced concrete moment-resisting space frames. Ground motion records previously used in Padilla et al. (2010) are transformed and applied to the structural models in *OpenSees* in 9˚ increments from 0˚ to 360˚. An iteration procedure was defined such that each ground motion record was scaled up or down to achieve failure (exceeding the 2.5% inter-storey drift limit state) in 50% of all orientations. Scaling factors were reported in this study with a median value for each suite of ground motion and structural model. Results are also compared with the previous study on the effect of directionality on bridge piers. Additionally, three extra cases were studied in which response quantity checked for only 1, 2 and 4 equally spaced directions rather than all 40 orientations.

Results suggest that scaling factors are within a limited range in all cases and there is no statistically significant difference in them, although changing structural form from azimuth-independent to azimuth-dependent has changed the factors. It was found that changing the response quantities and performance criteria of interest also influences the directionality factor; however more investigations are needed to generalise this conclusion.

Ground motion records that are realistic in their orientation dependence are found to impose more demand in seismic design. Looking at structures designed for particular periods in their longitudinal and transverse axes rather than various numbers of bays would be more informative in the investigation of directionality effect and also defining a scaling factor as a proxy in seismic design. Results suggest that the effect of directionality depends on whether structure is azimuth-dependent or independent.

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