Evaluation of the Influence of Vertical Irregularities on the Seismic Performance of a 9-storey steel frame

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SUMMARY

A methodology based on Incremental Dynamic Analysis (IDA) is presented for the evaluation of structures with vertical irregularities. Four types of storey-irregularities are considered: stiffness, strength, combined stiffness and strength, and mass irregularities. Using the well-known 9-storey LA9 steel frame as a base, the objective is to quantify the effect of irregularities, both for individual and for combinations of stories, on its response. In this context a rational methodology for comparing the seismic performance of different structural configurations is proposed by means of IDA. This entails performing nonlinear time history analyses for a suite of ground motion records scaled to several intensity levels and suitably interpolating the results to calculate capacities for a number of limit-states, from elasticity to final global instability. By expressing all limit-state capacities with a common intensity measure, the reference and each modified structure can be naturally compared without needing to have the same period or yield base shear. Using the bootstrap method to construct appropriate confidence intervals, it becomes possible to isolate the effect of irregularities from the record-to-record variability. Thus, the proposed methodology enables a full-range performance evaluation using a highly accurate analysis method that pinpoints the effect of any source of irregularity for each limit-state.

Keywords: incremental dynamic analysis, vertical irregularities, nonlinear dynamic analysis, seismic performance evaluation, bootstrap.

1. INTRODUCTION

Many buildings are in some sense vertically irregular. Some have been initially designed so, e.g., in the case of a soft first-storey. Others have become so by accident, for example due to inconsistencies or even errors during the construction process, while many have been rendered irregular during their lifetime because of damage, rehabilitation or change of use. Therefore, it is essential for structural engineers to obtain a better understanding of the seismic response of structures with irregular distributions of mass, stiffness or strength along their height, a need that has also been recognized by current seismic guidelines (e.g. FEMA-356 [1], Eurocode [2]).

Several researchers have provided useful insight into this issue, the most recent and comprehensive efforts including the work of Valmundsson and Nau [3], Al-Ali and Krawinkler [4] and Chintanpakdee and Chopra [5]. Valmundsson and Nau [3] mostly focused on comparing the adequacy of simplified seismic code design procedures when applied to vertically irregular frames. Al-Ali and Krawinkler [4] followed by Chintanpakdee and Chopra [5] performed systematic investigations on the effect of vertical irregularities on the seismic response of simple mid-rise single-bay frames. The structures used by the first investigators were based on a strong-beam-weak-column philosophy as opposed to the latter who adopted a more realistic strong-column-weak-beam approach. Despite some anticipated differences due to the different approaches used, all efforts reached relatively compatible conclusions. Still, in all cases several issues were left open.

First, all studies focused mostly on the influence of irregularities to the seismic *demands* rather than *capacities*. This in fact constitutes a broadband comparison that encompasses several limit-states of the structure. While this comparison is quite interesting in itself it raises the question whether the conclusions reached would hold when analyzed for each limit-state, or level of structural response, separately. Furthermore, the methodologies that have so far been developed to compare the reference and each modified structure [4,5] have to tune the modified structure to have the same period and yield base shear as the original in order to ensure a fair comparison. While valuable for idealized frames, such methods are not suitable for the direct comparison of different design alternatives of a realistic structure. Finally, the influence of the record-to-record variability has been neglected. In references [3], [4] and [5], timehistory analyses were performed for 4, 15 and 20 records, respectively, resulting in an increasingly improved but still unknown confidence in the derived conclusions.

To fulfil these needs we are proposing the use of Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell [6]) to evaluate the influence of irregularities on a realistic building for each performance level, from serviceability to global collapse. Using the LA9 9-storey steel frame (e.g. Foutch and Yun [7]) as a reference structure we study the effects of various patterns and types of vertical irregularities on the limit-state capacities without needing any modifications to match the irregular to the reference structure, simply by adopting a common intensity measure to express capacities. Then, by incorporating the bootstrap method [8] the effect of irregularities is isolated from any record-to-record variability: For each limit-state, confidence intervals are calculated and hypothesis testing is performed for the median and the dispersion of capacity, providing evidence of whether the observed differences are due to vertical irregularities or the randomness in the seismic loading.

2. STRUCTURAL MODELS

2.1 Reference Frame

The structure considered is a 9-storey steel moment resisting frame with a single-storey basement (Figure 1). The frame has been designed for a Los Angeles site, following the 1997 NEHRP (National Earthquake Hazard Reduction Program) provisions [7]. A centerline model with fracturing connections was formed using the OpenSEES [9] platform. It allows for plastic hinge formation at the beam ends while the columns remain elastic. The fracturing connections are modelled as rotational springs with 1% strain hardening and a strength drop to 60% of the plastic moment capacity at ten times the yield rotation. Similar behaviour was assumed for both positive and negative moments. Geometric nonlinearities in the form of P-Δ effects were considered while the effect of internal gravity frames has been incorporated. The fundamental period of the reference frame is $T_1^* = 2.25$ sec and accounts for approximately 84% of the total mass; it is essentially a first mode dominated structure that still allows for a significant sensitivity to higher modes. This design will become the reference frame and serve as the basis for comparing all irregular designs.

2.2 Frames with Vertical Irregularities

Practically an infinite number of vertically irregular designs can be obtained by selecting different properties and varying their distribution along the height. The irregularities that will be considered for the 9-storey frame are based on the three basic quantities, mass, stiffness and strength that can be modified separately or jointly, for single or multiple stories, along the building height. Following in the footsteps of Al-Ali and Krawinkler [4] only four types of vertical irregularities are examined: mass (MI), stiffness (KI), strength (SI) and combined stiffness-strength irregularities (KSI). Mass irregularities are very common and usually are due to different use of one floor compared to the use of adjacent floors, such as car parking floors, or floors with mechanical equipment. Typical stiffness irregularities appear as soft stories or in general when elements of the lateral-force-resisting system, such as braces, are present on one storey but not on adjacent stories. In many practical cases, strength changes occur together with stiffness, e.g., when the cross section of a member is changed both the moment of inertia and the plastic moment capacity are modified. Vertical irregularity cases of this type also appear when a structure has a set back, or when the number or size of structural members is reduced, for example when columns are stopped at the second floor instead of terminating at the basement. Stiffness and strength can also be modified unintentionally by non-structural elements, such as partition walls. Finally, strength only irregularities may be encountered, for example on reinforced concrete structures, where the stiffness is attributed to the dimensions of reinforced concrete sections while the strength is controlled primarily by the reinforcement.

In the cases considered the storey properties are modified by upgrading or degrading the properties of all of the storey's members, i.e., the beams and the supporting columns, by a single modification factor. Only one modification factor, equal to 2, was considered. Therefore, upgrading the stiffness of a storey by a factor of 2 refers to multiplying the stiffness of all members of that storey by 2, while when degrading the stiffness we divide by 2. In contrast, Al-Ali and Krawinkler [4] considered a wide range of modification factors, while Chintanpakdee and Chopra [10] considered only two values: 2 and 5. Each of the four types of irregularities considered, denoted as "KI", "SI", "KSI" and "MI", was applied to each of the nine stories of the frame separately. For brevity results will be shown only for odd-numbered stories. Additionally, several combined-stories irregularities were considered by modifying together with the given storey all stories beneath it. In total we examine 17 cases which are designated as follows: case "(n) KI" refers to the modification of stiffness of the n-th storey, while case "(1-n) SI" refers to the modification of the strength of stories 1 up to n. A superscript plus or minus sign will be added when necessary to distinguish between an upgraded and a degraded case. In total 136+1 IDA analyses were performed.

3. METHODOLOGY

Incremental Dynamic Analysis (IDA) [6] is regarded as one of the most powerful analysis methods available, since it can provide accurate estimates of the complete range of the model's response, from elastic to yielding, then to nonlinear inelastic and finally to global dynamic instability. IDA involves performing a series of nonlinear dynamic analyses for each record by scaling it to several levels of intensity that are appropriately selected. Each dynamic analysis is characterized by two scalars, an Intensity Measure (IM), which represents the scaling factor of the record, and an Engineering Demand Parameter (EDP) (according to current Pacific Earthquake Engineering Research Center terminology), which monitors the structural response of the model. An appropriate choice for the IM for moderate period structures with no near fault activity is the 5%-damped first-mode spectral acceleration $S_a(T_1, 5\%)$, while the maximum interstorey drift $\theta_{\max}$ of the structure is a good candidate for the EDP. Limit-states (e.g., immediate occupancy or collapse prevention according to FEMA-350 [11]) can be defined on each IDA curve and summarized to produce the probability of exceeding a specified limit-state given the IM level.

To perform IDA we used a suite of twenty records (Table 1) representing a scenario earthquake. These records belong to a bin of relatively large magnitudes of 6.5–6.9 and moderate distances, all recorded on firm soil and bearing no marks of directivity. Each of these records was appropriately scaled to cover the entire range of structural response for each irregular case. At each scaling level a nonlinear dynamic analysis was performed and a single scalar, the EDP, was used to describe the structural response. Using the hunt-and-fill algorithm [12] to select the IM-levels allowed the use of only fourteen runs per record to capture each IDA curve with excellent accuracy. Appropriate interpolation techniques [12] were applied in order to approximate each IDA curve in the IM-EDP plane from the discrete points obtained from the timehistory analyses, as shown in Figure 2a for the base case and the records of Table 1. Such results were in turn summarized to produce e.g. the median IDA curve in Figure 2a or the median storey drift demands along the height of the frame for a number of $S_a(T_1, 5\%)$-levels (Figure 2b).

In comparison, in order to evaluate the structural response, Al-Ali and Krawinkler [4] performed dynamic analyses for one elastic and two inelastic levels using strength reduction factors and some scaling by spectrum matching. In the terminology used by Jalayer [13] they have performed a three-stripes analysis, or a three-clouds analysis in $S_a(T_1, 5\%)$-terms, since in an $S_a(T_1, 5\%)$ versus θ_{max} plot the results will appear as three distinct clouds. Chintanpakdee and Chopra [5], on the other hand, have used what can be called a single-cloud analysis, where the records have been left unscaled. Our approach was to use a total of 14 levels of intensity non-uniformly dispersed to achieve maximum coverage for each record.

In order to compare the performance of the modified versus the reference frame, a continuum of limit-states was defined, each at a given value of θ_{max}, spanning all the structural response range from elasticity to global dynamic instability. For each limit-state (i.e., each value of θ_{max}) the corresponding $S_a(T_1, 5\%)$ values of capacity were obtained, one for each record [12], and they were appropriately summarized into their median value and dispersion. The standard deviation of the logarithms of the capacity values was used as a dispersion measure, which is a natural choice for data that is approximately lognormally distributed (e.g., Benjamin and Cornell [14]). By comparing the median and the dispersion of the capacities of the reference versus the modified frame for each limit-state (or value of θ_{max}) we gain an accurate and closely focused image of the effect of the modification on the structure's performance at each level of response.

It should be noted that such comparisons are made possible by expressing all capacities in a common IM, in our case the 5%-damped spectral acceleration at the first mode of the reference structure $S_a(T_1^*, 5\%)$. While this would pose no problem when dealing with strength modifications, it may not appear as simple for mass or stiffness irregularities which result in modified building periods. Then, objections may be raised owing to the fact that using each structure's $S_a(T_1, 5\%)$ seems a better choice and doing otherwise may introduce errors. In reality, this is only a matter of efficiency, i.e., how many records are needed to achieve a given confidence in the results (Luco [15]). Actually, it has been shown (Vamvatsikos and

Cornell [16]) that in the inelastic range (which is of interest) and for a relatively wide range of periods around T_1^*, similar values of efficiency are achieved. Comparisons of different designs were made possible in earlier studies [4,5] by tuning the irregular cases to have the same first mode period and/or yield base shear as the base case. While this method enables the comparison of idealized structures with only a few timehistory analyses it is not appropriate for the direct comparison of realistic design alternatives. Thus, the proposed methodology based on IDA allows us to perform comparisons in a much more realistic way that does not involve further modifying the irregular structure in any way since the selection of the IM is simply a post-processing issue. As an example, Figure 3 shows how the median IDA curves for the upgraded and the degraded cases compare with the median IDA curve of the base case when using $S_a(T_1^*, 5\%)$ as the common IM.

4. RESPONSE STATISTICS FOR SINGLE-STOREY IRREGULARITIES

Figures 4 up to 8 demonstrate the effect of vertical irregularities on the performance of the LA9 frame for the four types of irregularities considered. Figure 4, shows the median $S_a(T_1^*, 5\%)$ capacities for all limit-states (i.e., values of $\theta_{\max}$) considered, both for the upgraded and the degraded cases, normalized by the corresponding median capacities of the base case. Each column of Figure 4 refers to one of the four irregularities considered, while each of the five rows refers to the single storey (1, 3, 5, 7 or 9) whose properties where modified. Essentially each graph in Figure 4 compares the effect of a single-storey modification on the median $S_a(T_1^*, 5\%)$ limit-state *capacities* by scanning IDA curves, like the ones appearing in Figure 3, along the horizontal axis for constant values of $\theta_{\max}$. On the other hand, Figures 5 up to 8 show the distribution along the height of peak interstorey drift *demands* for each irregular case, normalized by the corresponding base-case values, for four intensity levels, namely $S_a(T_1^*, 5\%) =$ 0.25g, 0.5g, 0.75g and 1.0g.

4.1 Effects of single-storey Stiffness Irregularities

The effects of single-storey stiffness irregularities on the median $S_a(T_1^*, 5\%)$ capacities are shown in the first column of Figure 4 both for the stiffened and the softened cases. When the stiffness irregularity is at the lower stories (1-3) it mostly influences the limit-states near collapse, stiffer cases gaining up to a 10% increase and softer cases showing a similar decrease in capacity. Interestingly enough, a modified first storey inversely influences some early-inelastic limit-states, a soft first storey providing a small capacity bonus. Stiffness changes in the middle of the building seem to have a negligible influence (less than 5%) regardless of the limit-state. On the other hand when the irregularity appears at a higher storey (8-9), significant changes occur in the early limit-states. For $\theta_{\max}$ up to 4%, stiffer stories offer up to a 30% increase in capacity, while softer stories offer a similar decrease up to $\theta_{\max} = 7\%$.

In Figure 5 we can see the distribution of normalized median interstorey drift demands over the height of the building for given levels of $S_a(T_1^*, 5\%)$. First of all, it becomes apparent that single-storey modifications cause wide-spread changes all over the building. This is a direct consequence of the redistribution happening in a beam-hinge model [5]. At the lower intensity levels (0.25-0.5g) such effects are concentrated around the modified story, generating 30-50% changes in demands and in accordance with previous research results [5]. At higher intensities though, the changes become more uniformly distributed and the maxima often migrate to stories away from the modified one. Finally, close to collapse, the story demand profile becomes practically the same in all cases, leading to roughly 50% reduced demands regardless of the position of the modification and whether it is upgrading or degrading, indicating a robust collapse mechanism. This means that despite the initial differences all modified structures will fail in almost the same way and at similar intensity levels. The only exception appears for softer lower stories (1-3) which heavily increase the demands of all stories above, leading to an earlier global collapse.

4.2 Effects of single-storey Strength Irregularities

The influence of strength irregularities (SI) on limit-state capacities is shown in the second column of Figure 4, clearly showing that they have an overall greater impact compared to stiffness. Perhaps the only exception appears for $\theta_{\max}$ less than 2%, where little or no yielding has occurred yet and the results are not affected. Other than that the position of the modified storey leaves a distinct mark on the structure's capacity for each limit-state. When the modified storey is located close to the base (1-3) a powerful "fuse" effect appears for all limit-states away from collapse: the yielding of the soft lower story seems to isolate the higher ones and protect the building, thus leading to an improved performance (e.g. see Vamvatsikos and Cornell [6]). A stronger lower storey delays this phenomenon and has the exact opposite effect. Still, when close to collapse, this temporary relief vanishes and the modified structure practically gains little over the base case. Strengthening or weakening mid-height stories has a milder effect, stronger stories providing a 15% bonus to capacity for all higher limit-states. Similar, although much more pronounced effects appear when the modification is in the higher stories (7-9). Weakening any of these stories provides a mild disadvantage for lower limit-states that diminishes close to collapse, while strengthening offers up to a 25% performance improvement for any $\theta_{\max}$ greater than 3%. Obviously a weak top storey does not speed up global collapse, but curiously enough a stronger one seems to delay it.

As shown in the interstorey drift demand profiles of Figure 6, similarly to the stiffness irregularities, strength modifications have a widespread effect all over the structure that depends highly on the intensity level. For $S_a(T_1^*, 5\%) = 0.25$g introducing a weaker/stronger storey at any level of the structure will correspondingly increase/decrease the drift demands in the neighbourhood of the modification but it will have the inverse effect for other stories. Regardless of the storey that is modified, the most significant changes always appear at the top three stories, often reaching levels higher than 50%. At this level our observations are in complete agreement with Chintanpakdee and Chopra [5]. As $S_a(T_1^*, 5\%)$ increases these effects become less and less pronounced, especially for modifications in the lower and higher stories. On the other hand, for irregularities in the middle stories (5-7) the influence of the modifications changes character becoming more uniform over the height of the building. Making these stories stronger/weaker reduces/increases demands practically for all stories and not just in the neighbourhood of the irregularity. At 1.0g though the picture changes again: Whatever the storey and no matter if it is weakened or strengthened, the structure gains a rather uniform 50% reduction in demands for all stories. The single exception is the softening of the upper stories which leads to demands that are equal or higher than the reference structure. Again it seems that we have a rather consistent collapse mechanism for this building and any single-storey strength modification considered will not significantly affect the way the building fails, but only the $S_a(T_1^*, 5\%)$ level that this happens.

4.3 Effects of single-storey combined Strength and Stiffness Irregularities

The third column of Figure 4 shows the effect of combined stiffness and strength irregularities to the median limit-state capacities. Compared to the previous two types of irregularities we observe a much greater influence to the $S_a(T_1^*, 5\%)$ capacities. At least qualitatively, if not quantitatively, the KSI capacity effects, i.e., the third column of Figure 4, can be obtained by adding together the first and second columns (KI and SI) for any given value of $\theta_{\max}$. Essentially, the stiffness plus strength modification tends to have the combined properties of the corresponding separate stiffness and strength cases. Thus, for lower values of $\theta_{\max}$ we can identify in the KSI cases a significant positive effect when degrading lower stories or when upgrading the top ones. Obviously this is a manifestation of the effects associated with soft lower (KI-degraded) or strong higher (SI-upgraded) stories to the limit-state capacity, as mentioned earlier. In a similar way, for limit-states close to collapse it becomes apparent that upgrading any storey will result in a 10-25% increase in $S_a(T_1^*, 5\%)$ capacity, while degrading will have an adverse effect only when it happens at lower stories.

Figure 7 shows the distribution of drift demands over the height of the building for four IM levels. For the lower intensity levels, 0.25-0.75g, the simple combination rule that was established earlier seems to hold, at least qualitatively. The interstorey drift demands for any KSI case and any storey can be roughly obtained by adding up the KI and SI results. At the 0.25-0.50g levels, upgrading has again an important positive effect in the neighbourhood of the modification and the inverse effect elsewhere. At higher intensities though this effect is smeared out, where upgrading/degrading has a strong positive/negative effect everywhere, while the major changes still happen at the modified floor. Near collapse, at 1.0g, these patterns are amplified and almost all degraded cases reach global instability, while all upgraded ones gain a relatively uniform 50-60% decrease in demands. The only exception is degrading the top story which seems to have a positive effect and delay collapse. It is notable again that the distribution of drift demands near collapse remains almost the same for all the KSI-modified cases, echoing the similar findings for the KI and SI cases regarding the robustness of the collapse mechanism.

4.4 Effects of single-storey Mass irregularities

The effect of mass irregularities on the limit-state capacities is comparable to the stiffness irregularity cases (Figure 4). Actually, a close match can be observed between the cases with increased stiffness at a given floor and the corresponding cases with reduced mass and vice versa. Naturally, this is a direct outcome of the inverse effect that these two quantities have on the dynamic characteristics (e.g. the period) of the given substructure (the storey) and the whole building. Thus, the higher influence to capacities appears when the mass is modified either in the lower or in the top stories; changes in the middle ones have little, if any, significance.

Figure 8 shows that the correspondence of stiffness increase and mass decrease holds roughly for the drift demands over the height of the building. For low intensities the largest change in drift demands happens for top-storey modifications (also observed in [4]). Again the major changes seem to occur in the neighbourhood of the modification, and the inverse effects appear elsewhere. For higher intensities these differences are smeared out, practically all stories having increased demands when the mass is increased anywhere. Finally, for the 1.0g intensity level, the drift demands are smaller by 30-50% when compared to the base case for all nine cases both for increasing and decreasing the mass. As observed in all previous irregularities the shape of drift demands is not affected substantially near collapse. As a final note, it should be stressed that, contrary to previous findings [4], the effects of mass modifications are not less than those of stiffness modifications; they are roughly similar. It is possible that the stiffness-only tuning used in [4], both for the stiffness and the mass irregular cases, may have biased such results to underplay the mass effects.

5. BOOTSTRAP AND HYPOTHESIS TESTING

The IDA curves display significant record-to-record variability, as becomes obvious in Figure 2a. It is only natural to investigate the accuracy of the results given the limited sample size of 20 records. Looking just at the ratio of the medians in Figure 4 can often be misleading: Small changes in the elastic range, where the record-to-record variability is generally low, may be statistically more significant than larger differences in the highly variable near-collapse range. Thus we need to calculate confidence intervals and perform some hypothesis testing on whether the results that we see are indeed an effect of the irregularities or simply an artefact of the record-to-record variability.

Analytical formulas are usually not available when percentile values are involved; hence we turn to the bootstrap method (Efron and Tibshirani [8]) to fill this gap. It offers a reliable way to compare the median capacities between the reference frame and any modified structure for each limit-state. In essence it allows a direct comparison of two fractile IDA curves, namely the curve of the irregular case against the curve of the base case, on the basis of capacity. Since in both cases the randomness is induced by the record suite, which for the sake of a fair comparison should be kept common, we have a classic case of paired samples [17]. By sampling with replacement from the original 20 accelerograms to generate alternate 20-record suites we can calculate the ratio of the median $S_a(T_1^*, 5\%)$-capacities of the modified over the base case for every record suite and for every limit-state. If we perform this process for numerous random samples (say 1000), confidence intervals can be generated for the ratio of the median capacities for each $\theta_{\max}$ value. If we let $\hat{S}_{\text{irreg}} / \hat{S}_{\text{base}}$ represent the sample of ratios of median $S_a(T_1^*, 5\%)$ capacities for a given $\theta_{\max}$ and use the superscript "(x)" to denote the sample's x% fractile, then the $(1\text{-}x)\cdot 100\%$ confidence interval of the ratio for the given limit state can be formally calculated as:

$$\left[\left(\frac{\hat{S}_{irreg}}{\hat{S}_{base}} \right)^{(x/2)}, \left(\frac{\hat{S}_{irreg}}{\hat{S}_{base}} \right)^{(1-x/2)} \right] \quad (1)$$

If for a given $\theta_{\max}$ the interval contains unity we do not have significant evidence at the $(1\text{-}x)\cdot 100\%$ level to accept that the differences in the median capacities are produced from the irregularity rather than the earthquake records for the specified limit-state.

The upper and lower bounds of these intervals at the 90% confidence level (x = 0.10) appear in Figure 9 for all the irregularities, all storeys and all the limit-states considered. There are two features worth our attention in these plots: The width of the confidence intervals and the symmetry of the bounds with respect to the unity-line. The first provides a measure of the sensitivity of the results to the record selection and the second measures how likely it is that a random record suite will cause an increase or decrease of the median capacity for the particular modification. Obviously, if the unity-line is outside the confidence interval, then we cannot reject the hypothesis at the 90% level that the irregularity causes the observed difference. Still, even if the interval contains one, we have reasonable evidence supporting our conclusions when the unity appears closer to one of the bounds, but clearly little or no evidence if it is in the middle of the interval: It becomes a roughly 50-50 chance then that the modification will cause an increase or decrease. Thus, in order to draw objective conclusions one must treat Figure 9 as a necessary complement to Figure 4.

Indeed, all of the conclusions we have reached are verified by Figure 9. It can be seen that when the influence to the median capacity (Figure 4) is small the confidence intervals (Figure 9) generally contain one. When the effect of irregularities is stronger, the width of the confidence interval may be quite large but it usually does not contain one. Additionally, the width of the intervals tends to become larger at limit states closer to collapse due to the fact that the IDA curves show larger dispersion for higher damage levels. A more efficient IM that is better related to the high inelastic deformations developed at the higher intensity levels would reduce this dispersion (e.g. Vamvatsikos and Cornell [16]).

Furthermore, we can identify several cases where the confidence interval is quite wide and is almost equally divided above and below one, such as case (9) MI^+ for drift values of 6% up to 20% or case (3) SI^- for drift values beyond 10%. Figure 4 correctly led us to the conclusion that the effects of these two cases are small and thus the influence of irregularity is insignificant. However, the bootstrap results imply that the irregularity may appear insignificant in a median sense but there so much sensitivity to the record selection that we may easily find important influences in some cases where the median capacity ratios are equally likely to appear larger or smaller than one. Generally speaking, the influence of irregularities observed in Figure 4 can be easily increased or decreased depending on the choice of records.

To complete the picture, an examination of the ratio of the achieved capacity dispersions of the modified over the reference frame is also necessary. For this purpose bootstrap percentile confidence intervals are calculated as in Eq. (1) for the ratio of the standard deviations of the natural logarithm of the capacities. These appear in Figure 10, providing evidence on whether one can focus only on the influence of irregularities to the median capacities or should also consider changes in the dispersion. Fortunately, for most cases the width of the intervals is small and contains the unity line. However, there are cases, e.g., (1) KI, (9) KI and (7) MI, where significant changes to the dispersion are observed for the limit states near collapse. In general most of the influence to capacity dispersion is observed when changing stiffness or mass and by proxy in the combined stiffness-strength case. When mass and stiffness are modified the irregular frame undergoes a period change that can probably account for most of these effects, thus even when looking at mild changes in the median capacities we may have to be alert for large changes in their dispersion.

6. EFFECTS OF MULTI-STOREY MODIFICATIONS

To conclude our investigation we will also examine cases where more than one adjacent stories are modified. From the numerous combinations of stories available we chose to present only three: (1-2), (1-4) and (1-9). Naturally, case (1-9) cannot be considered irregular since all stories are modified by the same factor. Still it will be presented for comparison and as an example of applying the proposed method to meaningfully compare different design alternatives. Clearly the methods adopted in [4,5] would find no difference between the base frame and the (1-9) modified cases; they were not designed with this application in mind.

The ratios of the median capacities for every limit-state and type of modification are shown in Figure 11. Comparing them with the results for single-storey modifications, show that the influence of multi-storey modifications can be qualitatively recreated by adding up the corresponding single-storey influences. Thus the resulting curves often end up looking like a largely amplified version of the most influential single-storey modification included in the multi-storey combination. For example, all modifications involving stories (1-2) actually resemble a blown up version of the corresponding (most influential) first-storey irregularity.

Therefore, a (1-2) KI stiffness upgrade provides the usual bonus to higher limit-states, while a corresponding SI strength degrade enables an early "fuze" effect that helps post-yield limit-states (but not close to collapse). The KSI case again resembles a combination of the KI and SI ones, and the mass modification MI case tends to have the inverse effect compared to KI, especially close to collapse. Perhaps the only surprise comes from the unexpected bonus that a stiffness degrade provides to early inelastic limit-states for θ_{max} around 1-4%, a trend that does exists in the first-storey KI modification but only faintly.

Similarly, the (1-4) storey modifications do resemble a combination of the corresponding first and third storey cases, the highly influential first-storey leaving a very recognizable pattern on the results, and making them look a lot like the (1-2) cases. What probably best signifies the involvement of the higher stories in this case happens at the SI (and by proxy in the KSI) modification, where the fuze effect may appear again for lower limit-states, but near collapse we see what common sense would dictate: The strengthened case gets a 10% improved collapse performance, while the weakened case a similar reduction.

Finally, when the whole structure is modified, we can identify the strong influence of both the first and top storey modifications. Actually the combined effect is more accurately described by stating that all intricacies are gone and the picture clears: Stiffening the structure helps mostly in the early limit-states but softening hurts everywhere. Strengthening or weakening have a positive/negative effect mostly for near-collapse limit-states, while the combined stiffening plus strengthening obviously has the positive effects of both, helping the performance of the structure everywhere (the opposite happening for combined softening-weakening). Finally, mass modifications are true to their stiffness-reciprocal nature. Any mass decrease largely improves performance only in the pre-collapse limit-states, while mass increase hurts the structure practically everywhere.

The corresponding 90% bootstrap confidence intervals appear in Figure 12. Despite the magnitude of the effects caused by multi-storey modifications, the bootstrap results show that they are often highly dependent on the record selection. For example, for all KI cases and for θ_{max} values within 4-12% the confidence intervals (Figure 12) contain the unit-line and therefore the (small) influences observed in Figure 11 could easily swing the other way. In general though, when observing a relatively large impact from any modification, the confidence intervals verify that it is also statistically significant. Thus, one should focus on the more prominent effects shown in Figures 4 and 11 and disregard the less important characteristics which are usually statistically insignificant.

7. CONCLUSIONS

A methodology based on Incremental Dynamic Analysis for comparing the capacities of different structural designs has been proposed to study the effect of vertical irregularities on the seismic performance of a multi-storey building. It offers three distinct advantages over previous research: (a) It achieves a focused view by examining the effects on the capacities of each limit-state separately, from elasticity to global dynamic collapse, (b) it does not need the "tuning" that previous methods had to use in order to match the fundamental period and/or yield base shear for the structures compared, and (c) by taking advantage of the bootstrap it provides a sanity check for our results.

Four types of irregularities have been examined: stiffness, strength, combined stiffness-strength and mass irregularities, while their effect was studied for both single-storey and multi-storey modifications. In brief it was found that:

- The effect of any single or multi-storey modification significantly differs depending on the limit-state or level of intensity considered. In the elastic or early inelastic range, most of the findings of previous researchers are verified, but as damage increases in the structure such conclusions do not hold. Especially, when approaching global dynamic instability, any single-storey modification considered has no effect on the collapse mechanism, only on the intensity level that it is activated.

- Combined stiffness and strength irregularities have the largest effect among the irregularities considered. Strength comes second while mass and stiffness are the least influential.
- The combined stiffness and strength irregularity cases can be (at least qualitatively) decomposed to a stiffness and a strength component, as they combine the effects of both.
- Mass irregularities tend to have a reciprocal influence when compared to the stiffness irregularities for the same stories. Contrary to previous research the magnitude of their influence was found to be comparable to the corresponding stiffness irregularities, not less, an effect that may be connected to the "stiffness-tuning" used in earlier research.
- The behaviour observed for single-storey irregularity cases can be extended in a straightforward manner to qualitatively study the influence of multi-storey irregularities.
- The dispersion of $S_a(T_1^*, 5\%)$ limit-state capacities can be greatly affected by mass or stiffness irregularities as they substantially modify the first-mode period.
- The effects of irregularities are highly dependent on the record selection. In each case only the most prominent effects observed stand out from the record-to-record variability and are thus found to be statistically significant.

In conclusion, vertical irregularities have been shown to produce different effects that depend upon the type of irregularity, its position and most importantly, the intensity of the earthquake, or equivalently the response level or damaged state of the structure. While some consistent trends have been identified these only hold for the summarized values of many records. Individual records will often go against this "central" behaviour, something that engineers should always keep in mind.

REFERENCES

[1] American Society of Civil Engineers. Prestandard and Commentary for the Seismic Rehabilitation of Buildings, FEMA-356, Federal Emergency Management Agency, Washington, DC, 2000.

[2] CEN. Eurocode 8 - Design of structures for earthquake resistance, Part 1, European standard prEN 1998-1, Draft No. 6, European Committee for Standardization, Brussels, 2003.

[3] Valmundsson EV, Nau JM. Seismic Response of building frames with vertical structural irregularities, *Journal of Structural Engineering* 1997; **123**(1): 30-41.

[4] Al-Ali AAK, Krawinkler H. Effects of Vertical Irregularities on Seismic behaviour of building structures, Rep. No 130, John A. Blume Earthquake Engineering Center, Stanford Univ., Stanford, Calif., 1998.

[5] Chintanapakdee C, Chopra AK. Seismic Response of Vertically Irregular Frames: Response History and Modal Pushover Analysis, *Journal of Structural Engineering* 2004; **130**(8):1177-1185.

[6] Vamvatsikos D, Cornell CA. Incremental dynamic analysis. *Earthquake Engineering and Structural Dynamics* 2002; **31**: 491-514.

[7] Foutch DA, Yun S-Y. Modelling of steel moment frames for seismic loads. *Journal of Constructional Steel Research* 2002; **58**: 529-564.

[8] Efron B, Tibshirani R. 1993. *An Introduction to the Bootstrap*. Chapman and Hall: New York.

[9] McKenna F, Fenves GL. The OpenSees Command Language Manual - Version 1.2. Pacific Earthquake Engineering Research Centre, University of California, Berkeley, 2001.

[10] Chintanapakdee C, Chopra AK. Evaluation of the modal pushover procedure using vertically ‘regular’ and irregular generic frames, *Report No. EERC 2003-03*, Earthquake Engineering Research Center, Univ. of California at Berkeley, Berkeley, Calif., 224.

[11] SAC Joint Venture. Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, prepared for the Federal Emergency Management Agency, FEMA-350, Washington D.C., 2000.

[12] Vamvatsikos D, Cornell CA. Applied Incremental Dynamic Analysis, *Earthquake Spectra* 2004, **20**(2): 523–553.

[13] Jalayer F. Direct probabilistic seismic analysis: Implementing non-linear dynamic assessments. *PhD Dissertation,* Department of Civil and Environmental Engineering, Stanford University, Stanford, CA, 2003.

[14] Benjamin JR, Cornell CA. *Probability, Statistics, and Decision for Civil Engineers*. McGraw-Hill: New York, p. 684, 1970.

[15] Luco N. Probabilistic seismic demand analysis, SMRF connection fractures, and near-source effects. *PhD Dissertation,* Department of Civil and Environmental Engineering, Stanford University, Stanford, CA, 2002.
http://www.stanford.edu/group/rms/Thesis/Luco_Dissertation.zip [Feb 12th, 2005].

[16] Vamvatsikos D, Cornell CA. Developing efficient scalar and vector intensity measures for IDA capacity estimation by incorporating elastic spectral shape information. *Earthquake Engineering and Structural Dynamics* 2005 (accepted).

[17] Rice JA. *Mathematical Statistics and Data Analysis (2nd edn)*. Duxbury Press: Belmont CA.

Table 1. The set of twenty ground motion records used.

No	Event	Station	φ^{o} [1]	Soil [2]	M [3]	R [4] (km)	PGA (g)
1	Loma Prieta, 1989	Agnews State Hospital	090	C,D	6.9	28.2	0.159
2	Northridge, 1994	LA, Baldwin Hills	090	B,B	6.7	31.3	0.239
3	Imperial Valley, 1979	Compuertas	285	C,D	6.5	32.6	0.147
4	Imperial Valley, 1979	Plaster City	135	C,D	6.5	31.7	0.057
5	Loma Prieta, 1989	Hollister Diff. Array	255	–,D	6.9	25.8	0.279
6	San Fernando, 1971	LA, Hollywood Stor. Lot	180	C,D	6.6	21.2	0.174
7	Loma Prieta, 1989	Anderson Dam Downstrm	270	B,D	6.9	21.4	0.244
8	Loma Prieta, 1989	Coyote Lake Dam Downstrm	285	B,D	6.9	22.3	0.179
9	Imperial Valley, 1979	El Centro Array #12	140	C,D	6.5	18.2	0.143
10	Imperial Valley, 1979	Cucapah	085	C,D	6.5	23.6	0.309
11	Northridge, 1994	LA Hollywood Storage FF	360	C,D	6.7	25.5	0.358
12	Loma Prieta, 1989	Sunnyvale Colton Ave	270	C,D	6.9	28.8	0.207
13	Loma Prieta, 1989	Anderson Dam Downstrm	360	B,D	6.9	21.4	0.24
14	Imperial Valley, 1979	Chihuahua	012	C,D	6.5	28.7	0.27
15	Imperial Valley, 1979	El Centro Array #13	140	C,D	6.5	21.9	0.117
16	Imperial Valley, 1979	Westmoreland Fire Station	090	C,D	6.5	15.1	0.074
17	Loma Prieta, 1989	Hollister South & Pine	000	–,D	6.9	28.8	0.371
18	Loma Prieta, 1989	Sunnyvale Colton Ave	360	C,D	6.9	28.8	0.209
19	Superstition Hills, 1987	Wildlife Liquefaction Array	090	C,D	6.7	24.4	0.180
20	Imperial Valley, 1979	Chihuahua	282	C,D	6.5	28.7	0.254

[1] Component
[2] USGS, Geomatrix soil class
[3] Moment magnitude
[4] Closest distance to fault rupture

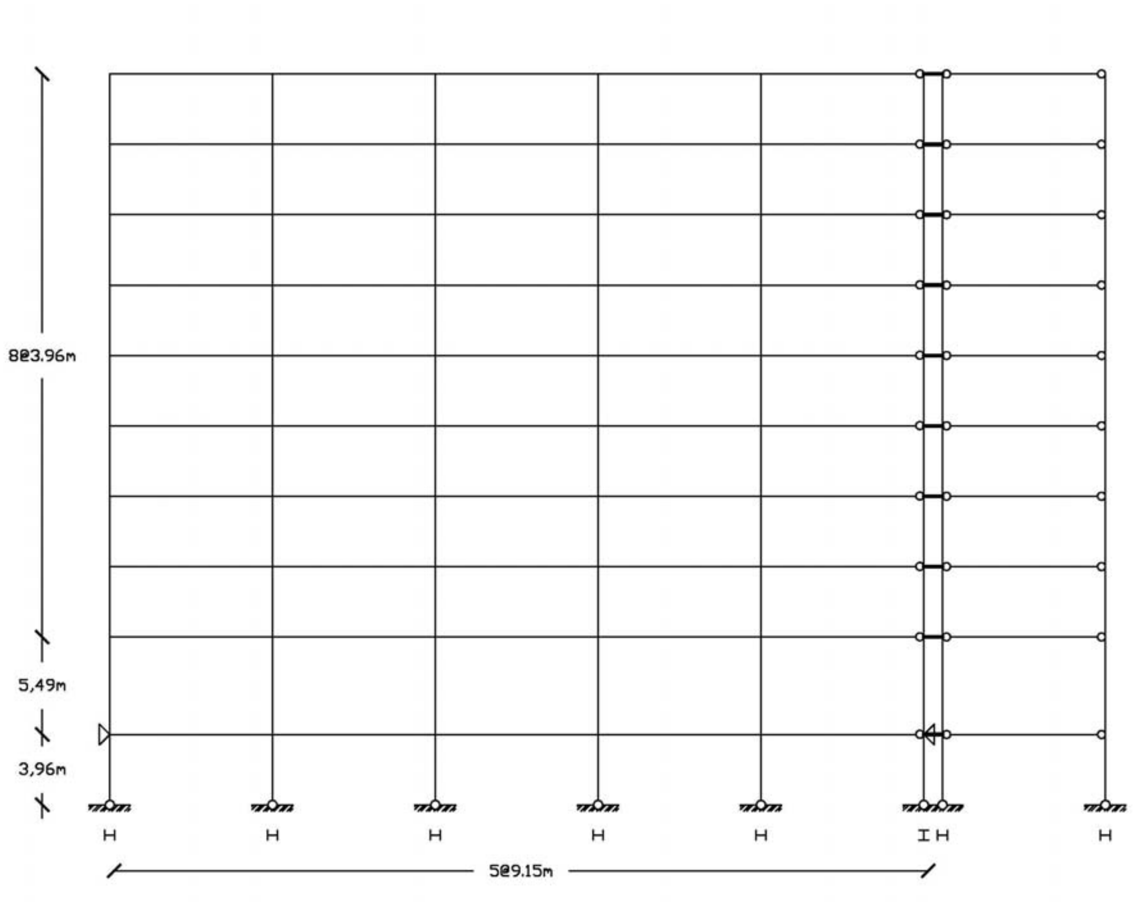
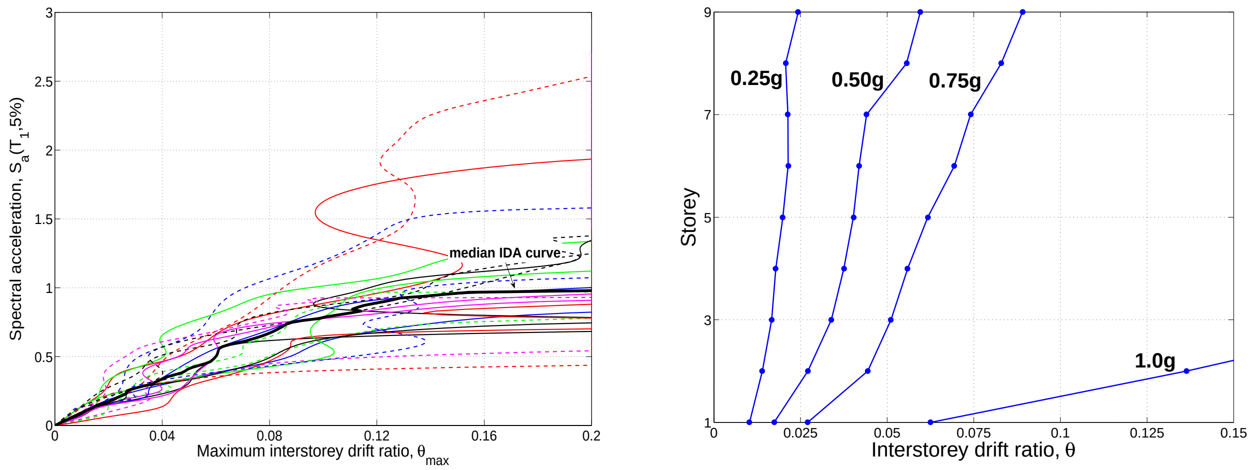


Figure 1. LA9 frame model.



(a) (b)

Figure 2. IDA results for the reference frame: (a) twenty IDA curves and their median, (b) median interstorey drift demands for different $S_a(T_1^*, 5\%)$ values.

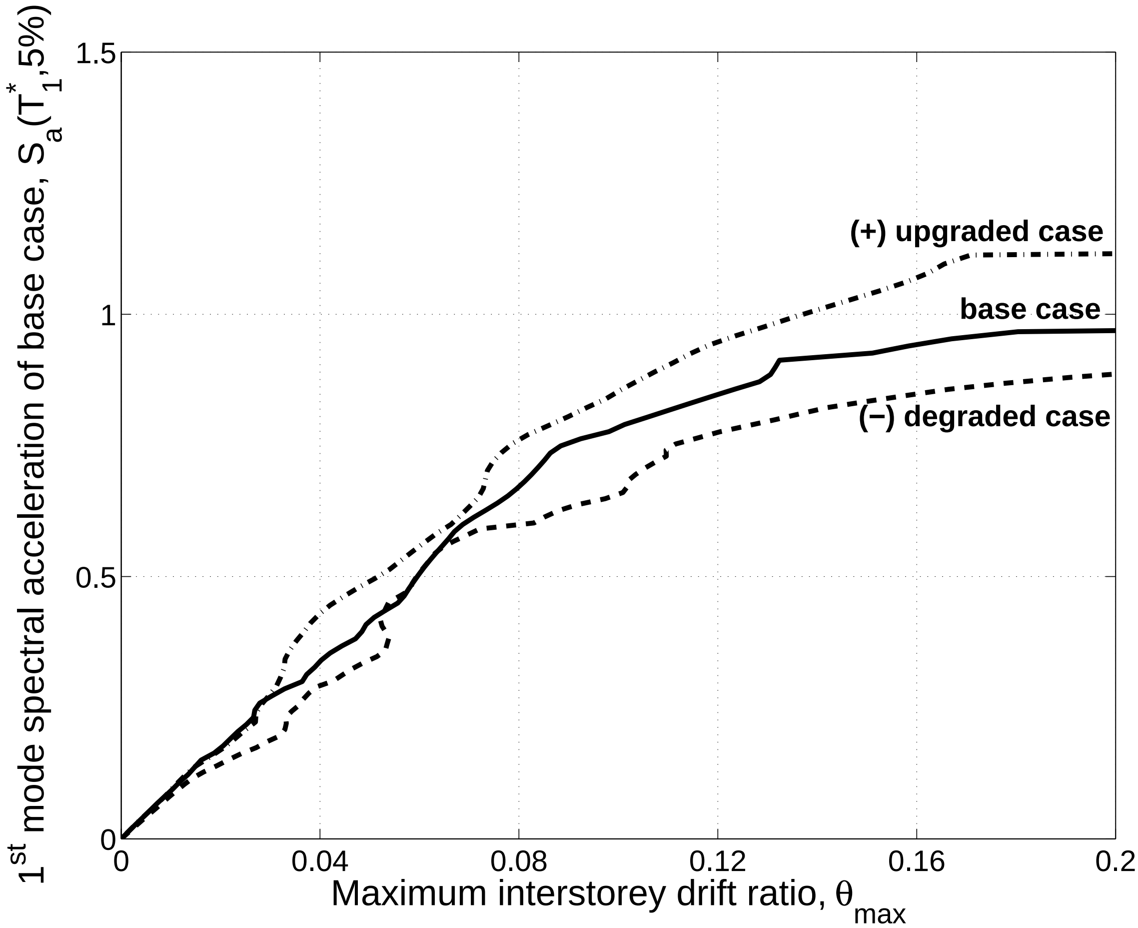


Figure 3. Using a common IM to compare median IDA curves of the base case versus the degraded and upgraded ones for strength and stiffness irregularities in the 6^{th} storey (6 KSI).

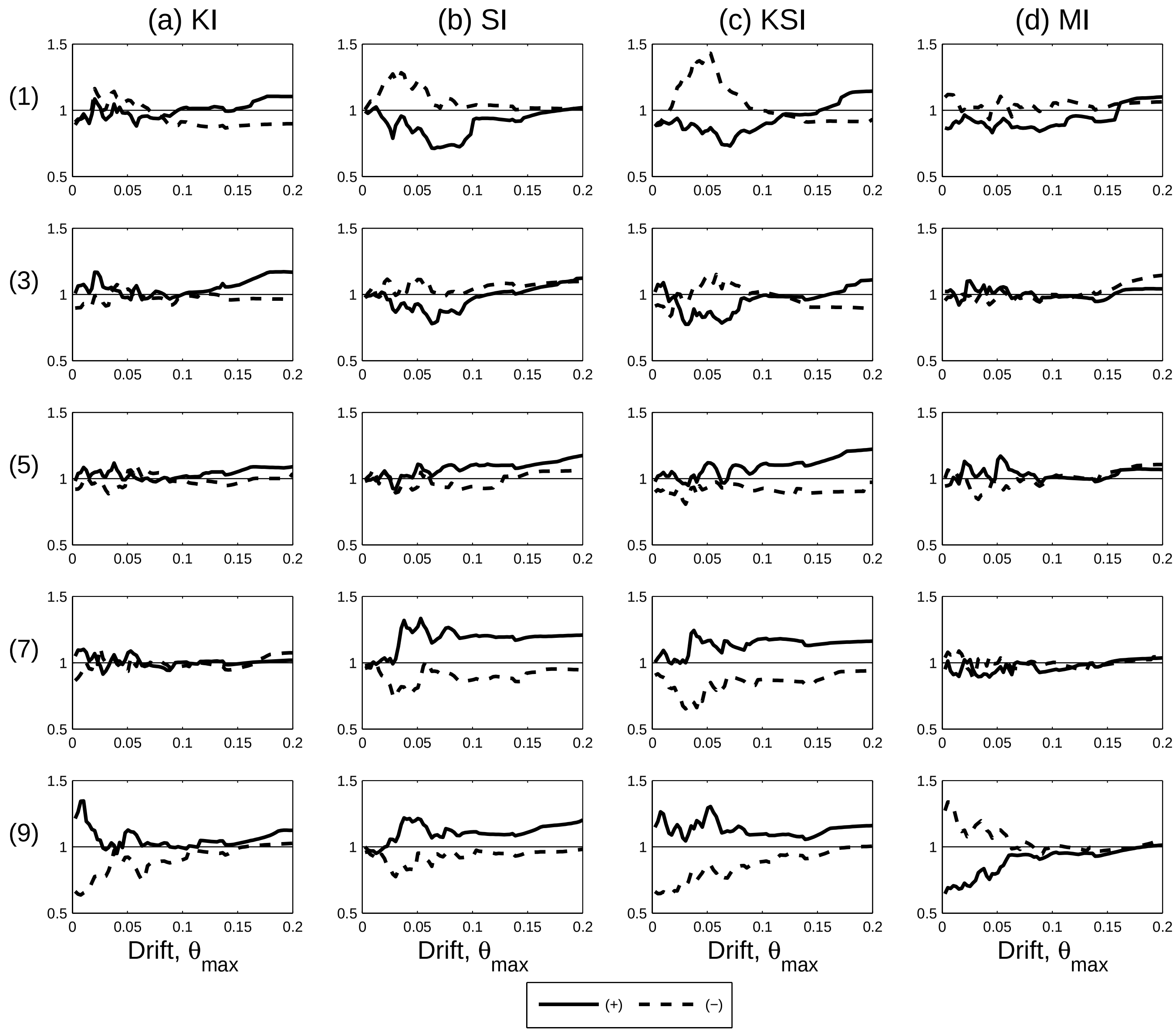


Figure 4. The median $S_a(T_1^*, 5\%)$-capacities given $\theta_{\max}$ for the upgraded and degraded cases normalized by the corresponding base case values.

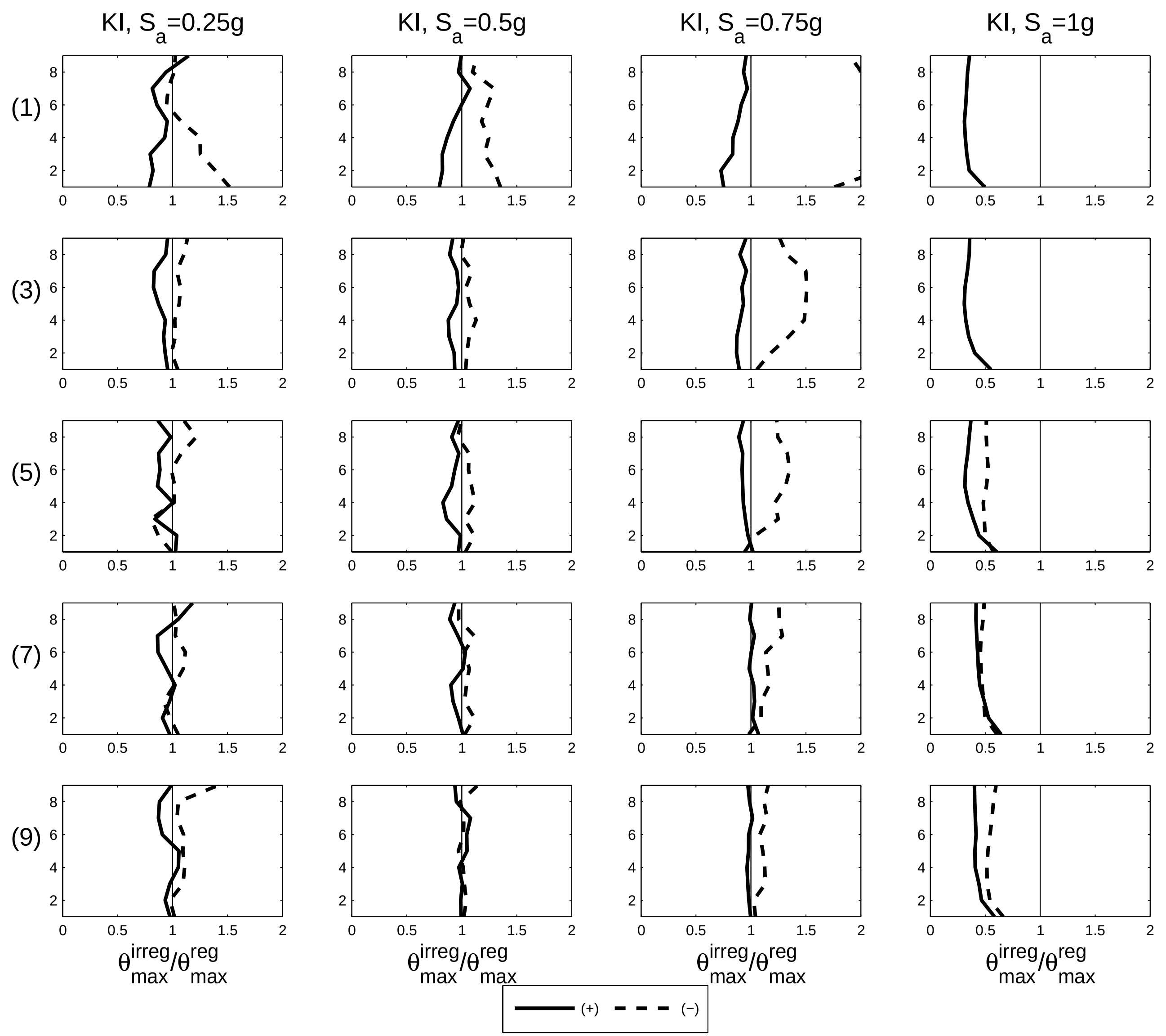


Figure 5. Normalized peak interstorey drift demands for single-storey stiffness irregular (KI) cases for four $S_a(T_1^*, 5\%)$ levels.

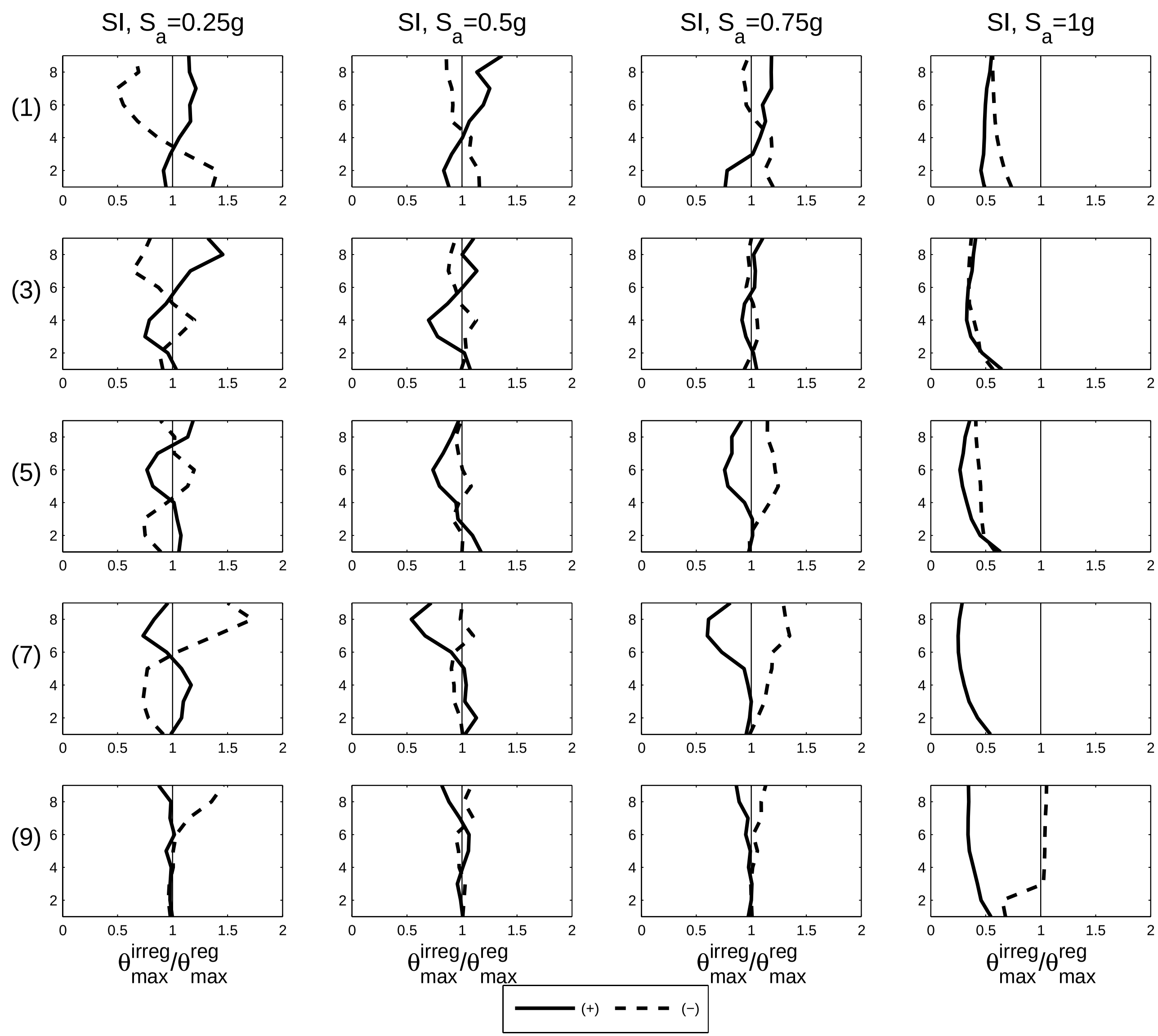


Figure 6. Normalized peak interstorey drift demands for single-storey strength irregular (SI) cases for four $S_a(T_1^*, 5\%)$ levels.

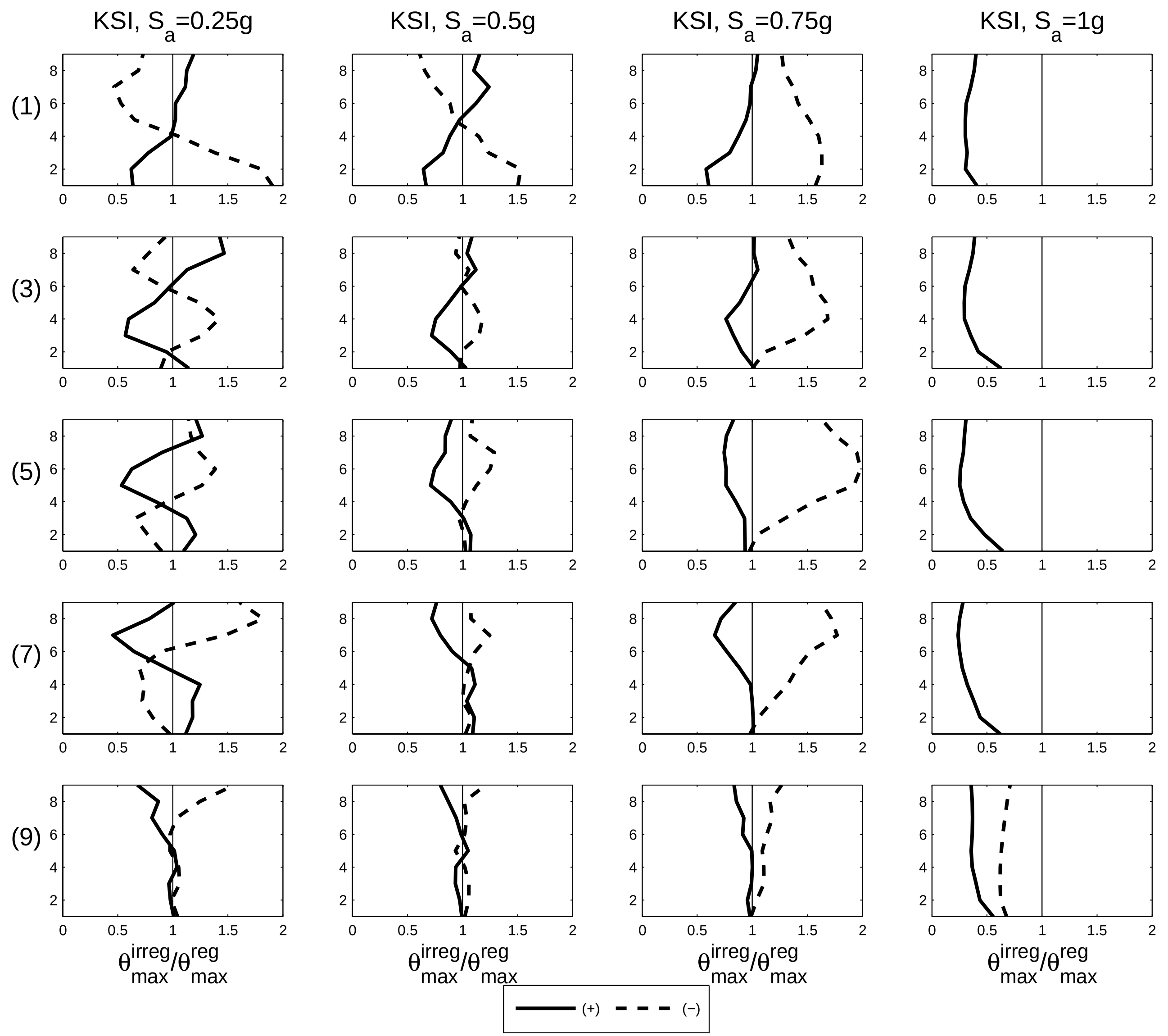


Figure 7. Normalized peak interstorey drift demands for single-storey stiffness and strength irregular (KSI) cases for four $S_a(T_1^*, 5\%)$ levels.

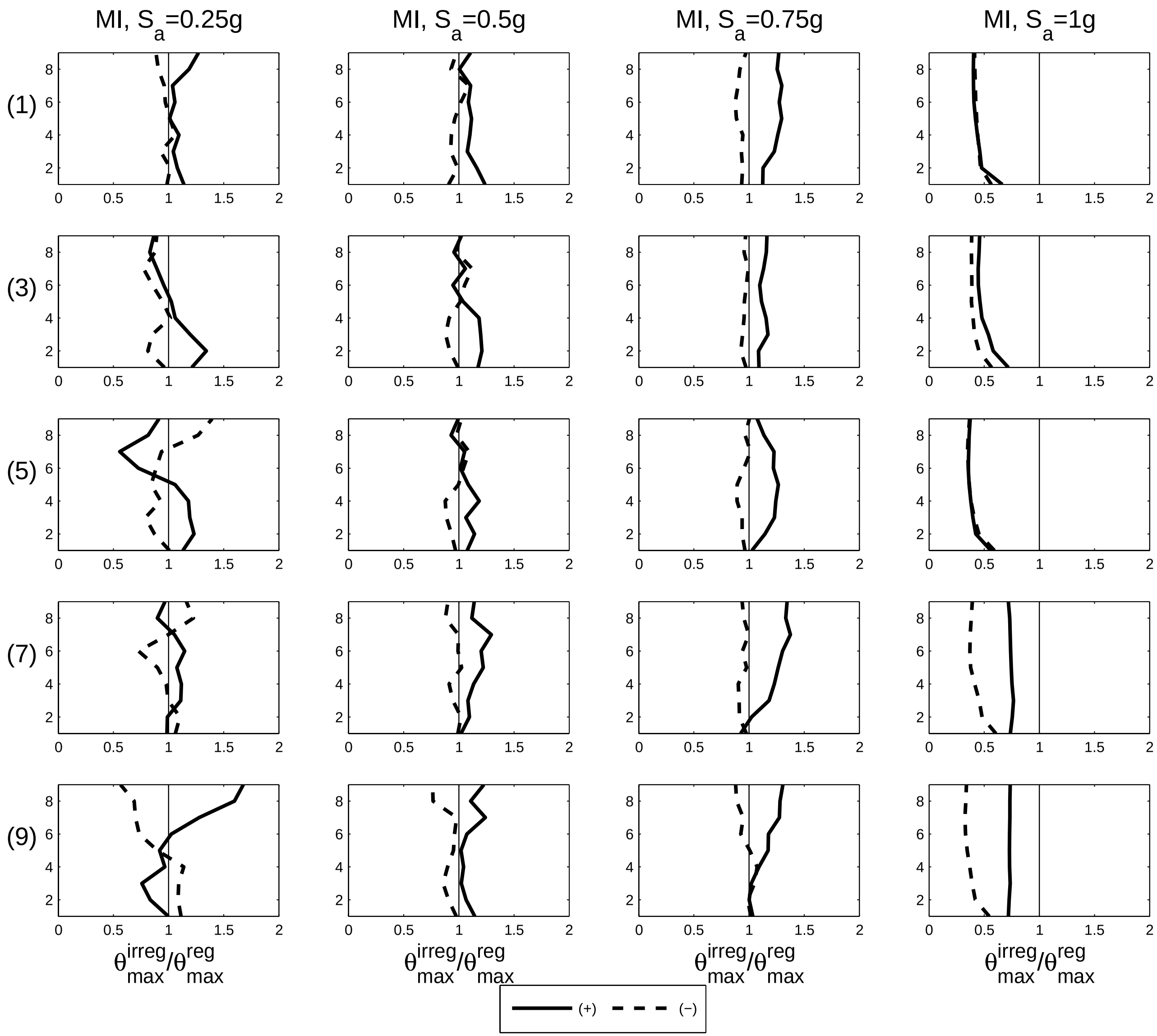


Figure 8. Normalized peak interstorey drift demands for single-storey mass irregular (MI) cases for four $S_a(T_1^*, 5\%)$ levels.

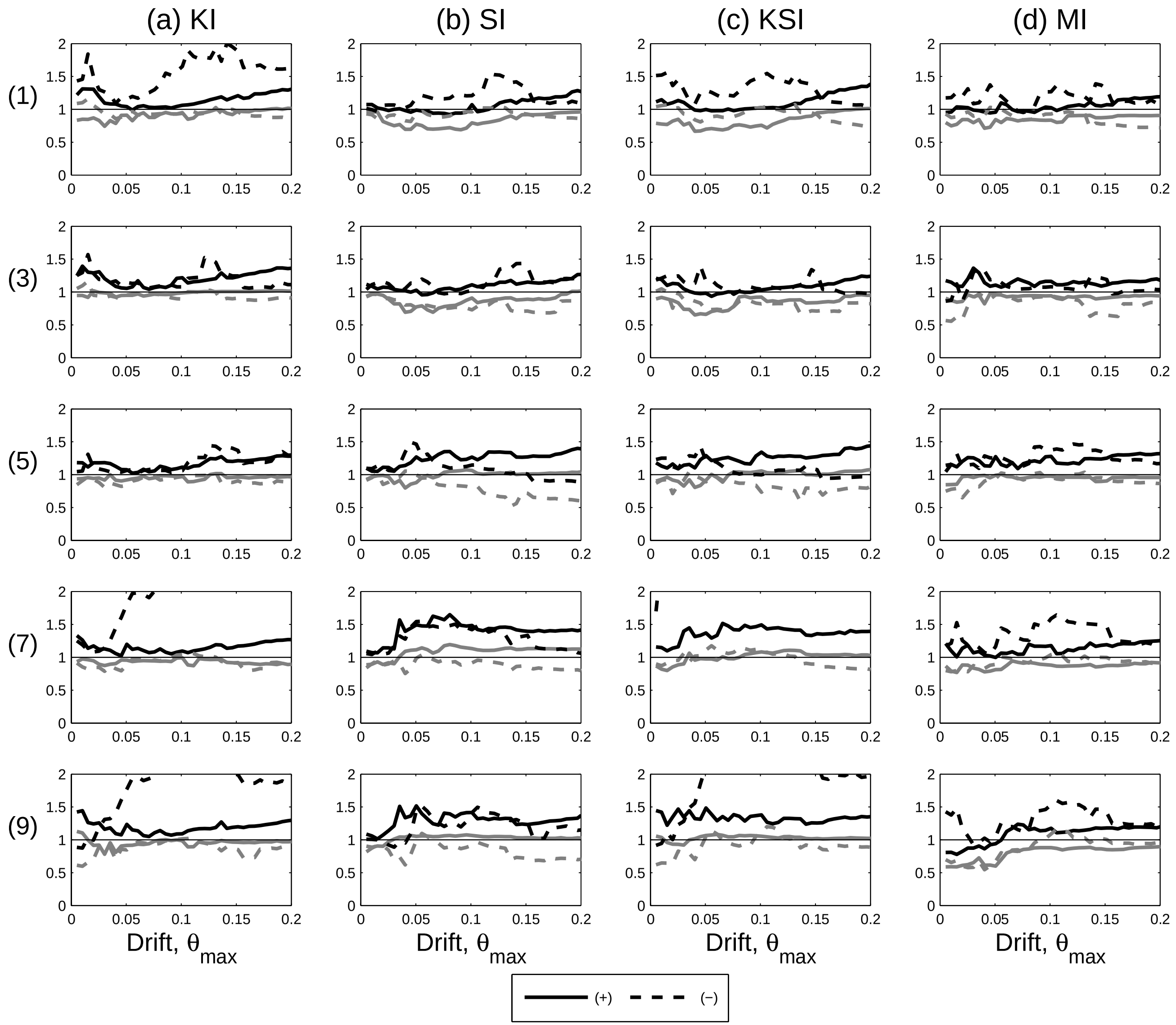


Figure 9. Bootstrap 90% confidence intervals on the ratio of the median $S_a(T_1^*, 5\%)$-capacities given $\theta_{\max}$ of the modified over the base case. Light gray lines indicate the lower bound and darker ones the upper bound.

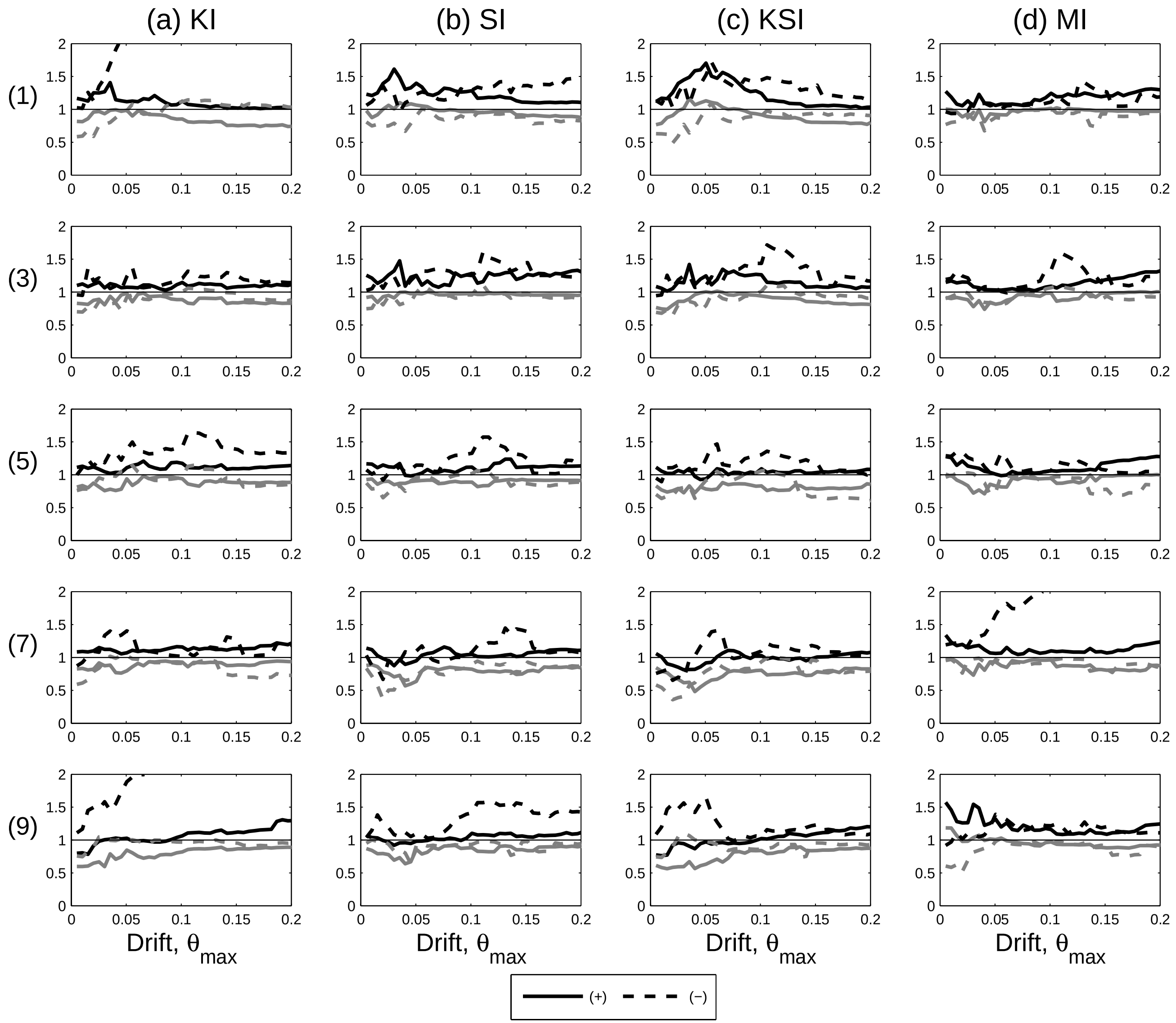


Figure 10. Bootstrap 90% confidence intervals on the ratio of the dispersion of $S_a(T_1^*, 5\%)$-capacities given $\theta_{\max}$ of the modified over the base case.

Light gray lines are used for the lower bound and darker ones for the upper bound.

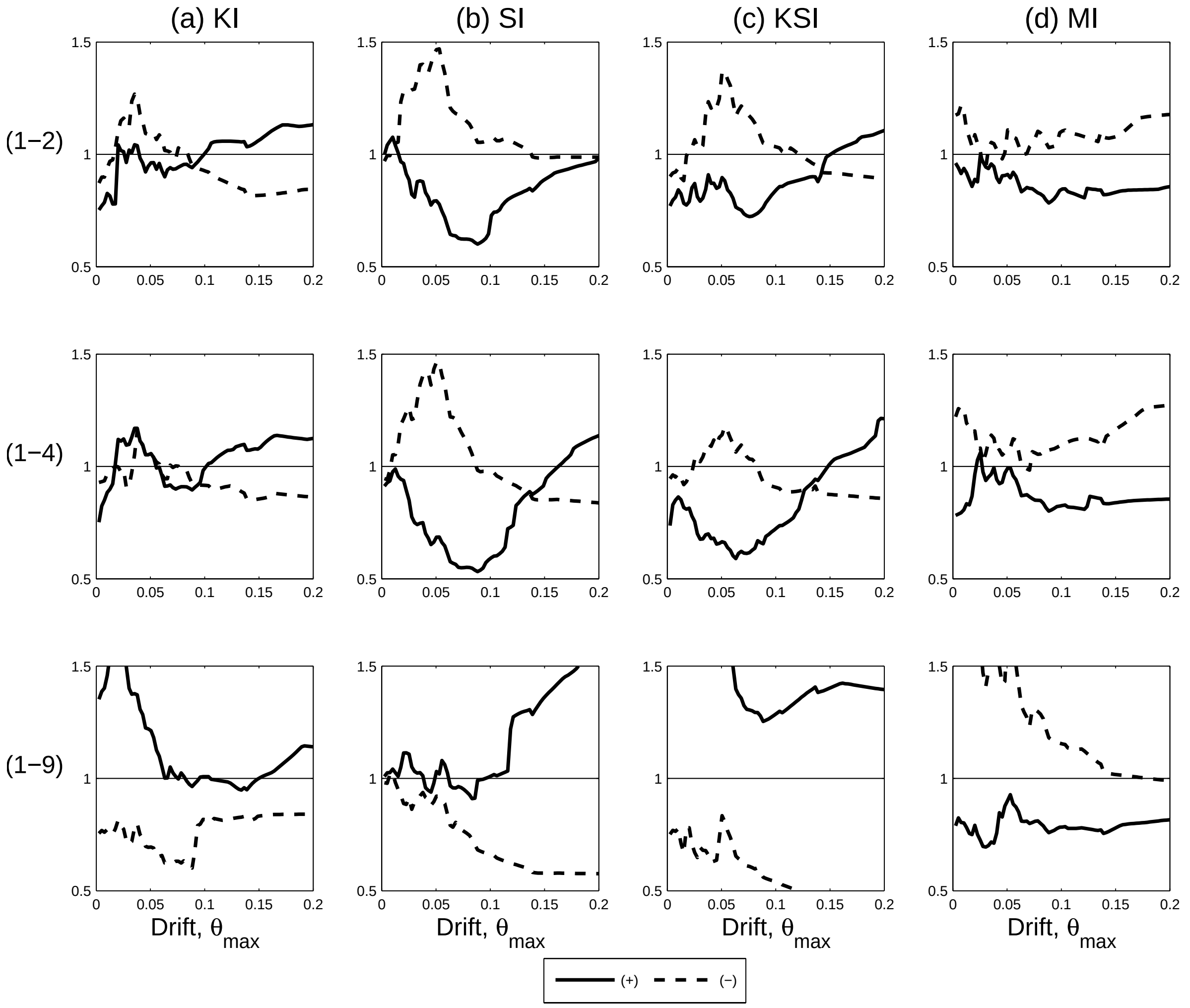


Figure 11. Median $S_a(T_1^*, 5\%)$-capacities given $\theta_{\max}$ for the upgraded and degraded cases of multi-storey modifications normalized by the corresponding base case values.

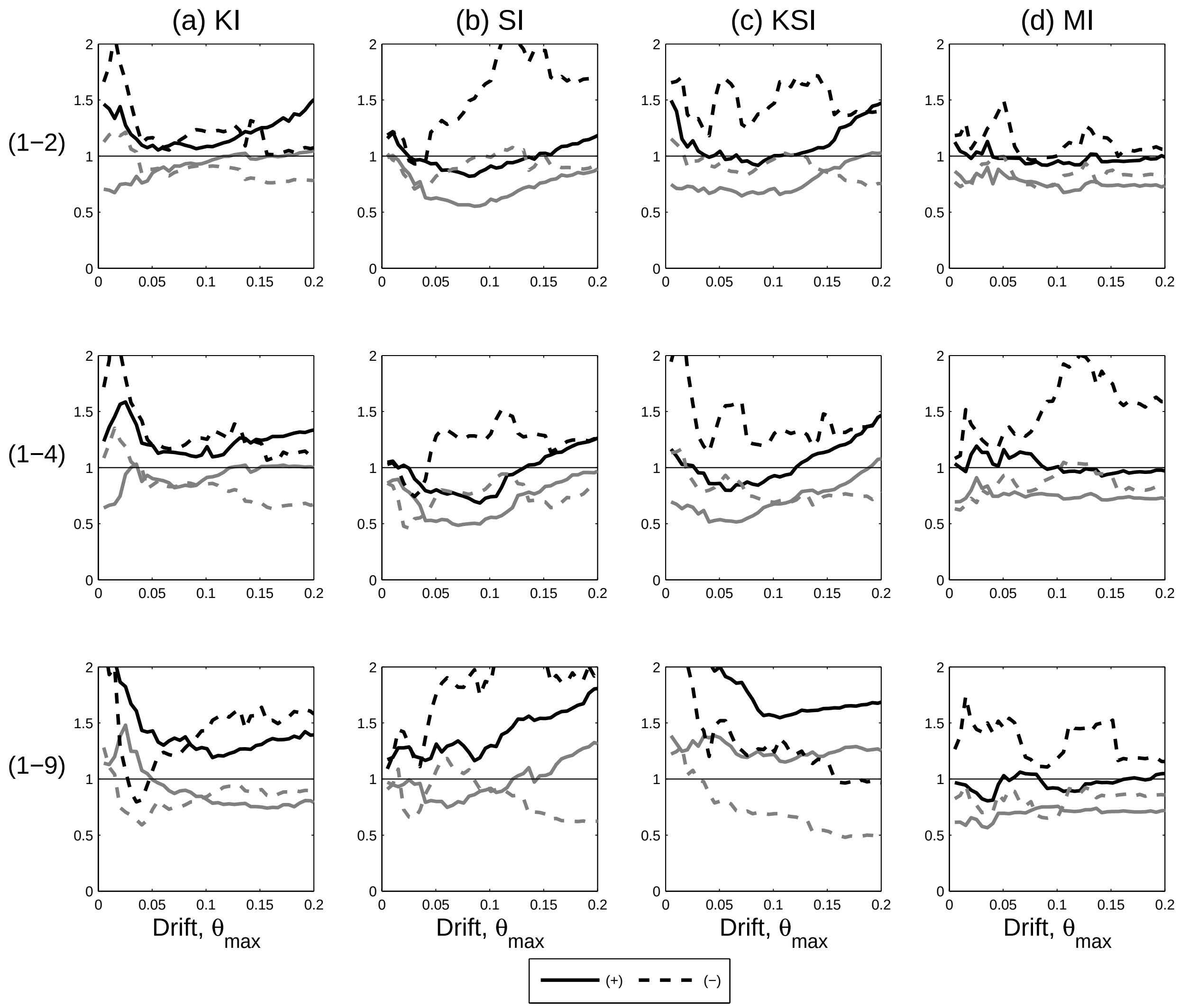


Figure 12. Bootstrap 90% confidence intervals on the ratio of the median $S_a(T_1^*, 5\%)$ capacities given $\theta_{\max}$ of the modified over the base case for multi-storey modifications. Light-colored lines are used for the lower bound and darker ones for the upper bound.