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SEISMIC PERFORMANCE SENSITIVITY OF A 9-STORY STEEL FRAME TO PLASTIC HINGE MODELING UNCERTAINTIES

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SUMMARY

The effects of different beam-column plastic-hinge modelling assumptions on the seismic behaviour of steel frames are studied through Incremental Dynamic Analysis (IDA). The well-known 9-story LA9 2D steel frame is used as a testbed by adopting multiple possible moment-rotation relationships for the beam plastic-hinges. These are modelled as full quadrilinear backbones with pinching hysteresis, including an elastic, a hardening, a negative stiffness and a residual plateau branch, terminating with a final drop to zero strength. The properties considered include the post-yield hardening ratio, the end-of-hardening rotation, the residual moment capacity and the ultimate rotation reached. The hinge parameters are varied one at a time, globally throughout the building, generating several plausible structural models which differ only in the adopted connection model, some being more brittle and others more ductile. The seismic performance of each resulting frame is then evaluated using IDA, i.e., by performing multiple nonlinear time history analyses for a suite of ground motion records appropriately scaled to several intensity levels. By appropriately post-processing the results the median IDA curves are estimated, forming a solid basis for comparing the different models. Thus, we are able to evaluate the influence of several plastic-hinge modelling assumptions to the seismic performance of the frame and understand the sensitivity of such results that are usually estimated on the basis of a single structural model.

1. INTRODUCTION

Modern seismic guidelines, e.g. FEMA-350 [SAC, 2000] have recognized the sensitivity of structural response to the uncertainties in the structure and the model itself. While we are usually content to use a single "best" model to estimate the seismic performance of a given structure, what we end up estimating is only one realization out of an infinite number of possibilities. While it is still an extremely difficult problem to correctly quantify the influence of such modelling uncertainties, there have been several attempts to isolate some useful cases and gain insight into the effect of the properties of a model to its estimated seismic performance.

Following the investigations after the Northridge earthquake, there has been a considerable effort to study the effect of steel frame moment-connection properties. For example, Luco and Cornell [1998, 2000] found that connection fractures have a detrimental effect on the dynamic response of steel moment-resisting frames while Foutch and Shi [1998] used different hysteretic models to show that the hysteresis of the moment connection has a relatively insignificant effect on the global demand. Perhaps the most exhaustive study on global collapse capacity has been performed by Ibarra [2003] who studied the dynamic instability of idealized single bay frames with beam-column connections having non-trivial backbones including both cyclic and in-cycle degradation.

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Still, the need remains for a full range estimation of the influence of plastic-hinge characteristics on the seismic performance of realistic steel frames.

As a first attempt to fulfil such needs we are proposing the use of Incremental Dynamic Analysis (IDA) [Vamvatsikos and Cornell, 2002] to evaluate the influence of plastic hinges on a realistic building for each performance level, from serviceability to global collapse. Using the well known LA9 9-storey steel frame [e.g. Foutch and Yun, 2002] as a reference structure we will study the sensitivity of seismic demand and capacity to various connection moment-rotation backbone properties by forming alternate structural models and comparing them using IDA fractile curves [Fragiadakis et al, 2006].

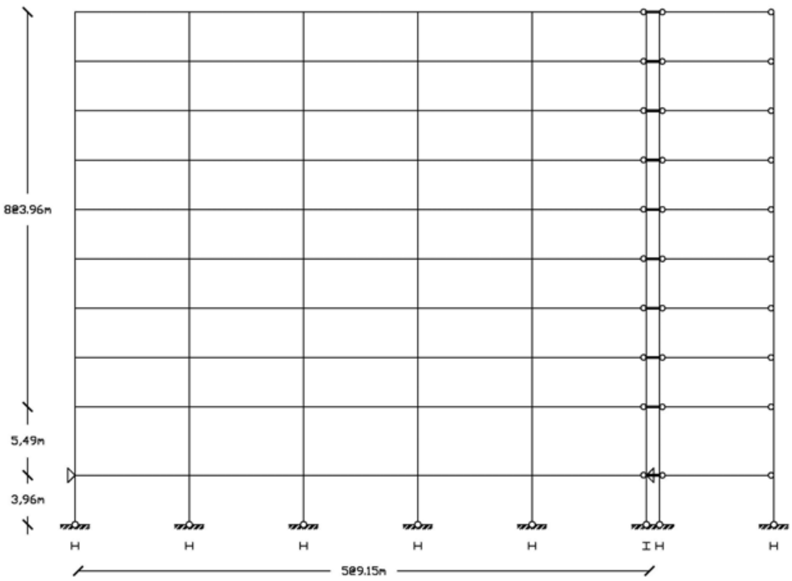

Figure 1: The LA9 steel moment resisting frame.

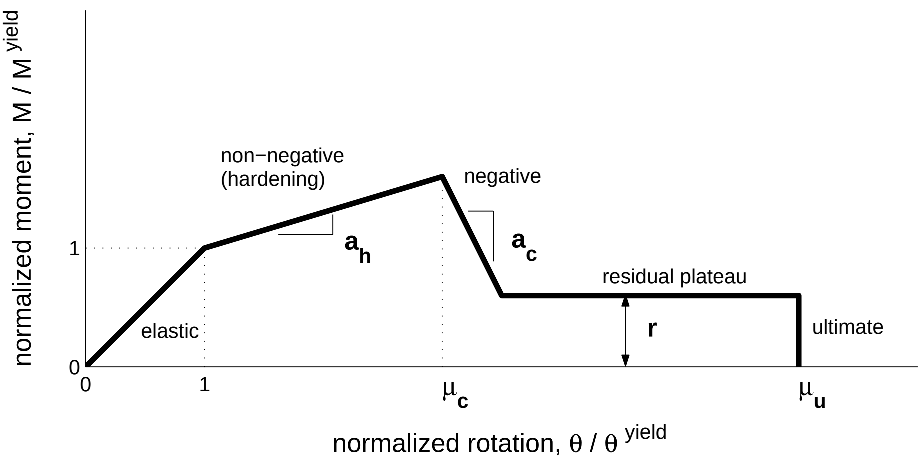

Figure 2: The moment-rotation relationship of the beam point-hinge in normalized coordinates.

2. MODEL DESCRIPTION AND GROUND MOTION SET

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 [Foutch and Yun, 2002]. A centerline model with fracturing connections was formed using the OpenSEES [McKenna and Fenves, 2001] platform. It allows for plastic hinge formation at the beam ends while the columns remain elastic. Geometric nonlinearities in the form of P-Δ effects were considered while the effect of internal gravity frames has been explicitly incorporated. The fundamental period of the reference frame is T_1 = 2.28 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.

The fracturing connections are modelled as rotational springs with moderately pinching hysteresis and a quadrilinear moment-rotation backbone, shown in normalized coordinates in Figure 2. The backbone initially allows for elastic behaviour, then hardens at a non-negative slope of a_h that terminates at a normalized rotation μ_c, where the negative stiffness segment starts, having a slope of a_c. Then a residual plateau at a normalized height r appears, delaying the failure of the connection until the ultimate normalized rotation μ_u. Similar behaviour is assumed for both positive and negative moments. This is in effect a complex backbone that is versatile enough to simulate the behaviour of numerous moment-connections and it will serve as the primary target for our sensitivity studies. Using a hinge with properties a_h = 5%, μ_c = 3, a_c = -200%, r = 50% and μ_u = 7, we have formed a reference frame that is used as the basis for comparing all modified models.

Table 1: The suite of thirty "ordinary" ground motions.

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
21	Imperial Valley, 1979	El Centro Array #13	230	C,D	6.5	21.9	0.139
22	Imperial Valley, 1979	Westmoreland Fire Station	180	C,D	6.5	15.1	0.11
23	Loma Prieta, 1989	Halls Valley	090	C,C	6.9	31.6	0.103
24	Loma Prieta, 1989	WAHO	000	–,D	6.9	16.9	0.37
25	Superstition Hills, 1987	Wildlife Liquefaction Array	360	C,D	6.7	24.4	0.2
26	Imperial Valley, 1979	Compuertas	015	C,D	6.5	32.6	0.186
27	Imperial Valley, 1979	Plaster City	045	C,D	6.5	31.7	0.042
28	Loma Prieta, 1989	Hollister Diff. Array	165	–,D	6.9	25.8	0.269
29	San Fernando, 1971	LA, Hollywood Stor. Lot	090	C,D	6.6	21.2	0.21
30	Loma Prieta, 1989	WAHO	090	–,D	6.9	16.9	0.638

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

3. METHODOLOGY

Incremental Dynamic Analysis (IDA) [Vamvatsikos and Cornell, 2002] is a powerful analysis method that 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 interstory 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 [SAC, 2000]) can be defined on each IDA curve and summarized to produce the probability of exceeding a specified limit-state given the IM level.

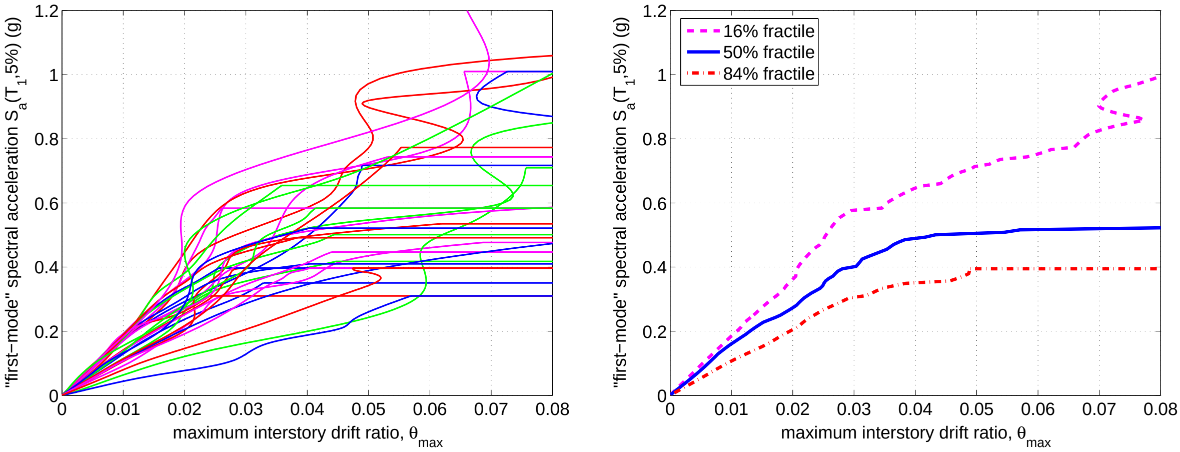

Figure 3: (a) The thirty IDA curves for the base case and (b) their summary in 16%, 50% and 84% fractiles.

To perform IDA we used a suite of thirty 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 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 [Vamvatsikos and Cornell, 2004] to select the IM-levels allowed the use of only twelve runs per record to capture each IDA curve with excellent accuracy. Appropriate interpolation techniques 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 3a for the base case and the records of Table 1. Such results were in turn summarized to produce the median and the 16%, 84% IDA curves in Figure 3b.

Having such a powerful, albeit resource-intensive tool at our disposal, we are left with the selection of the alternate models to evaluate. There is obviously an inexhaustible number of variations one could try with just the five parameters of the adopted plastic hinge, not including the possibility of having different hinge models in each story, or even for each individual connection. In the course of this limited study we chose to vary only four parameters, namely a_h, μ_c, r and μ_u, one at a time and independently of each other. In essence we performed a sensitivity study by perturbing four of the five parameters, using the values a_h = 0% or 10%, μ_c = 2 or 4, r = 20% or 80%, and μ_u = 5 or 9. The results, evaluated using both static pushover analysis and IDA appear in the following sections.

4. STATIC PUSHOVER ANALYSIS RESULTS

While IDA is indeed a powerful tool, using a static pushover (SPO) is always desirable. The pushover is a much cheaper method that despite its numerous problems can still provide useful information on the expected behaviour of a structural model. It has been shown to correspond to the fractile IDA curves [e.g., Vamvatsikos and Cornell, 2002], thus helping us to understand and qualitatively predict the dynamic behaviour of the structure. The SPO curves for the reference frame versus the frames having modified hinges are shown in each of the four Figures 4a-d. Therein, the base case appears always as a solid blue line in the middle, while the cases with the single upgraded and degraded parameter fan around on its sides.

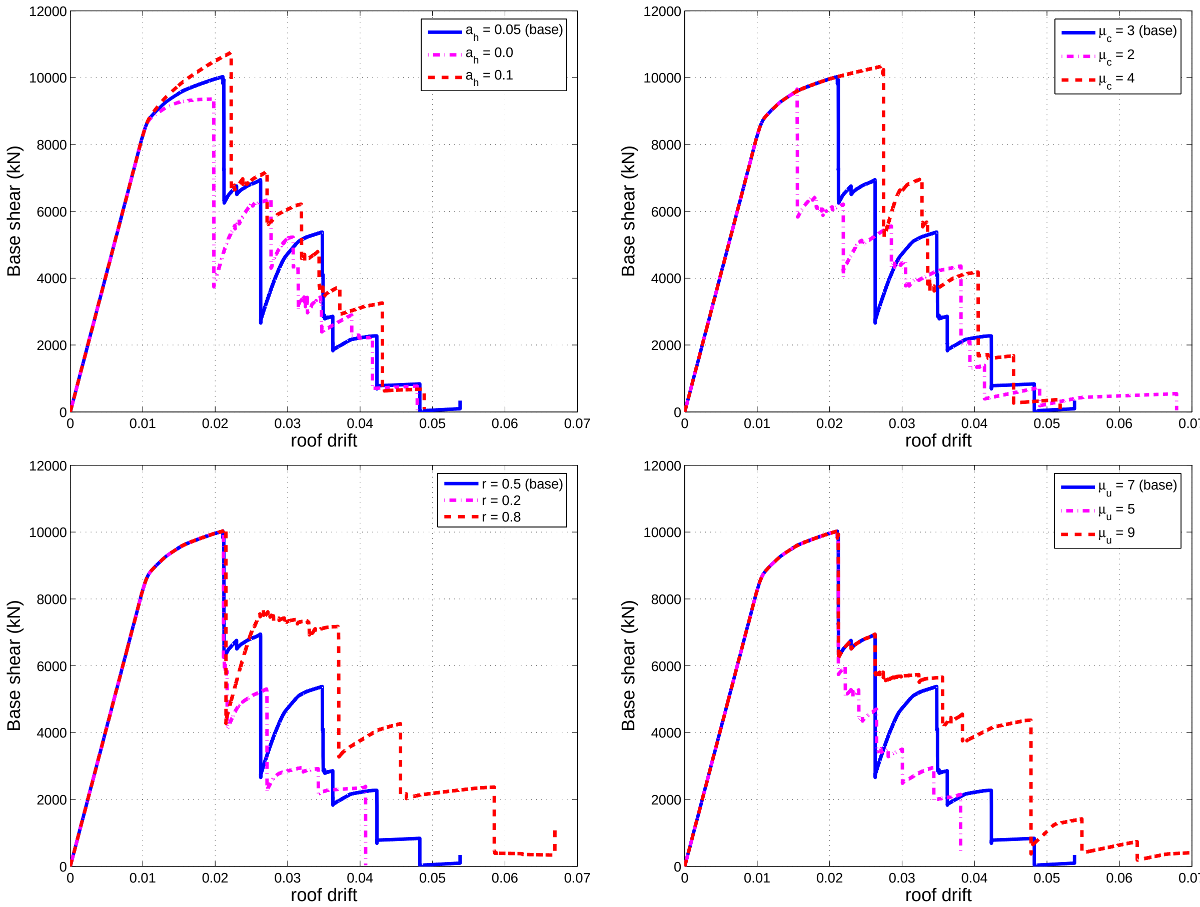

Figure 4: Sensitivity of static pushover curves to plastic hinge properties: (a) a_h, (b) μ_c, (c) r and (d) μ_u.

Figure 4a shows the effect of the post-yield hardening slope a_h. Clearly, increasing the slope to 10% seems to raise the SPO curve to the highest base shear observed in any of the cases studied, while it slightly delays its drop due to fracturing. Lowering a_h to zero has the exact opposite effect. Still, when the negative slope takes over and the SPO starts dropping, it becomes hard to find any considerable differences. Connections in stories different from the one where fracturing has occurred may indeed be enjoying the benefits of a higher (or lower) hardening, but the sheer loss of strength seems to nullify this effect anywhere beyond 2% roof drift. Increasing the μ_c in Figure 4b to a value of 4 instead of 3 seems to have a much more substantial effect. The fracturing drop is delayed at least by 0.6% roof drift, while there is a small increase in the maximum load. Similarly, lowering μ_c to only 2 has the inverse influence, forcing the SPO to lose strength earlier, both in force and deformation terms. Still, once again, when fracturing occurs the changes in μ_c do not matter any more: The three curves converge to practically a similar degrading path. Unsurprisingly, both a_h and μ_c expend their influence in the pre-fracturing nonlinear range, as they both control the hardening, positive stiffness segment but have little, if any, control on the segments that follow.

The situation is reversed for the next two parameters studied, namely r and μ_u. These clearly influence only the hinge behaviour after the negative drop, therefore in Figures 4c and 4d we see no change at all in the SPO curves before the loss of strength occurs. Afterwards, it is obvious that either increasing the height of the residual plateau r or increasing its length μ_u before the ultimate failure, are two practically equivalent ways to extend the curve of the building to roof drifts close to 7%, while maintaining somewhat higher strengths than the base case. Similarly, reducing these values is a sure method to force an earlier collapse of the structure: The drop to zero strength now appears at about 4% versus the 5% of the base case.

5. INCREMENTAL DYNAMIC ANALYSIS RESULTS

Having studied the SPO curves, we have now gained an idea on what to expect qualitatively when we process the IDA results. Still, there is only so much that we can discern using just static results.

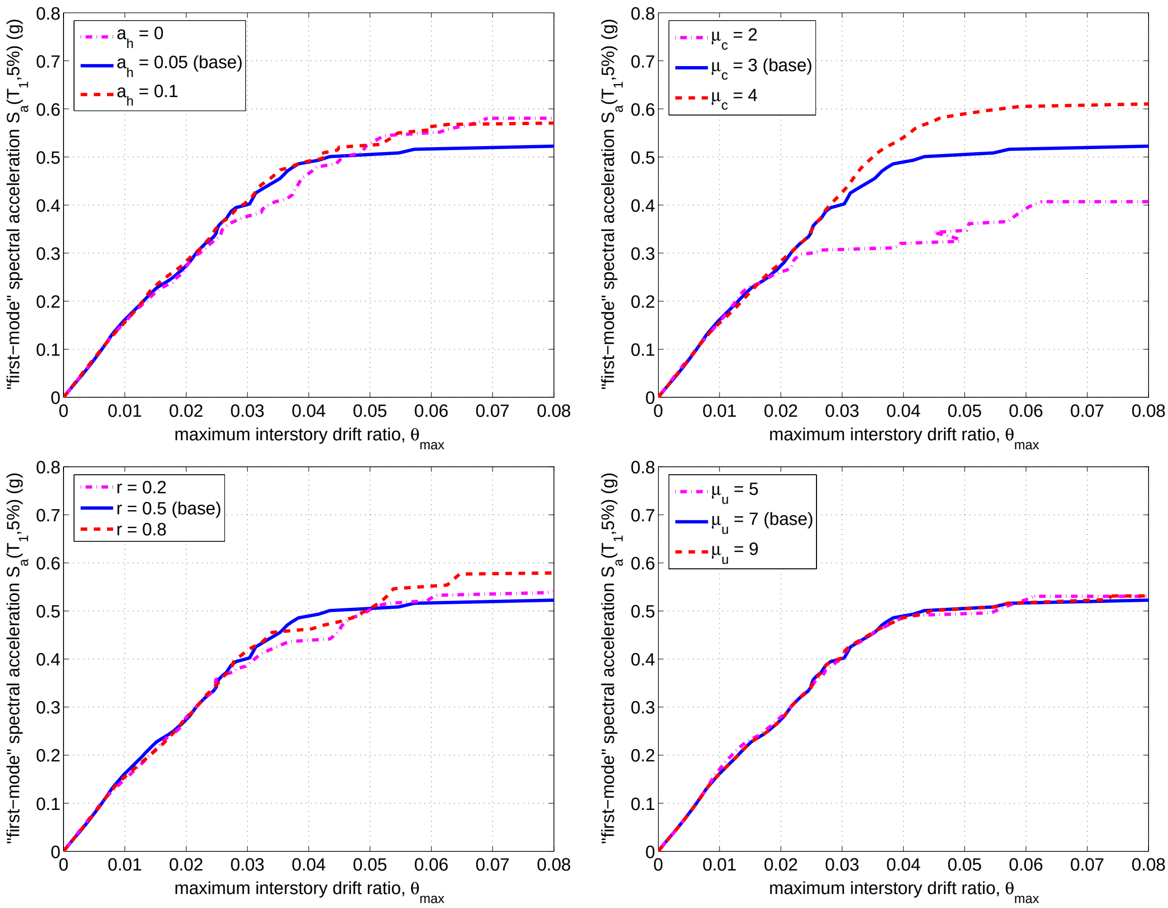

Figure 5: Sensitivity of the median IDA curves to plastic hinge properties: a_h, μ_c, r and μ_u.

The median IDA curves of the base case versus the modified ones appear in Figures 5a-b and at a first glance they seem to only partially confirm our SPO-based intuition. Before we jump to conclusions though, it is important to take into account the limitations of our investigation. Only thirty records were used to trace the median IDA curves shown. Given the enormous record-to-record variability that is evident in Figure 3a, there is some significant uncertainty around the resulting curves that is bound to increase for higher levels of the intensity. Therefore, when the median curves have only a small difference there is not significant evidence to state that one curve is higher in IM terms than another, something that should normally indicate an immediate gain in performance [Fragiadakis et al, 2006].

Such a situation appears in Figure 5a, where both the increase and the decrease of the hardening slope a_h seem to offer a gain in global collapse capacity, as they both terminate in IM-values that are almost 5-10% higher than the base case. Such differences are indeed insignificant given the dispersion around the median IDAs induced by the record-to-record variability. Therefore the only conclusion that we can draw confidently is that there is not much difference at all. Practically speaking, increasing or degreasing a_h within the tested limits is not enough to influence performance, at least not by an amount that would make any difference when using thirty ground motion records.

On the other hand, accelerating or delaying the occurrence of the strength drop is of decisive importance (Figure 5b). Increasing μ_c to 4 has produced an almost 20% improvement practically everywhere in the median capacities after 3% interstory drift. Global collapse, as indicated by the flatline of the median IDAs occurs now at $S_a(T_1,5\%) = 0.61$g versus 0.52g in the base case. Reducing μ_c to 2 has a very negative impact on the structural performance, dropping the collapse capacity to $S_a(T_1,5\%) = 0.4$g and moving the onset of softening to a much earlier intensity of 0.3g versus 0.4g. The structure now begins accumulating serious damage much earlier and has become a lot more vulnerable due to this change.

The effects of r and μ_u are shown in Figures 5c and 5d, where it appears that the post-negative-stiffness part of the plastic hinge has little if any influence on the predicted performance of the LA9 structure. The height of the residual plateau modifies the IDA curves only by a small amount, rendering such changes practically undetectable when using thirty records. For μ_u there can be no objection that the median IDAs are actually coincident. It seems that the strength loss caused by the fracturing connections dominates the response of the building and does not allow the beneficial or detrimental effect of any subsequent backbone changes to manifest itself. The marginal changes that were observed for these two parameters in the SPO curves (Figures 4c-d), translate to practically no changes in the dynamic behaviour.

6. CONCLUSIONS

The effect of four parameters of fracturing beam-column connections on the global seismic performance were investigated for a nine-storey steel moment-resisting frame using both incremental dynamic analysis (IDA) and static pushover methods. The observations made by comparing static pushover curves only qualitatively translate to the IDA results. Increasing the hardening slope of the beam point-hinge appears to provide some useful strength gains in the pushover, but these practically disappear in the dynamic analyses. Similarly, the marginal gains in the pushover realized by increasing the height of the post-fracture residual plateau or by extending its length are practically undetectable when performing IDA. On the other hand, delaying or accelerating the onset of fracturing is an extremely significant issue that directly improves or degrades the response of the model, especially for the higher limit-states reaching up to global dynamic instability. It thus becomes obvious that it is not easy to predict the influence of such characteristics of a structure to its seismic performance. Changes that seem simple enough to be plainly predictable actually challenge our intuition when seen from the proper point-of-view. There is still much to understand even for simple, regular 2D frames.

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