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EVALUATION OF THE CONTRIBUTION OF PANEL ZONES TO THE GLOBAL PERFORMANCE OF MOMENT RESISTING FRAMES UNDER SEISMIC LOAD

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ABSTRACT

Before the 1994 Northridge Earthquake, the seismic design strategy for moment resisting frames considered the yielding of the panel zone when calculating the required level of ductility. Following the unacceptable performance of conventional moment resisting frame (MRF) connection details during the Northridge seismic event, prequalified connections were developed to concentrate beam yielding away from the column face, preserving the connection. With these new connection strategies, the panel zone deformation may not contribute as significantly to the overall behaviour of an MRF. Therefore, considering the increased use of advanced dynamic modeling techniques, it is important for both designers and researchers to know what level of modelling detail is required to properly capture the behaviour of an MRF. This paper examines the influence of the panel zone model on the global performance of a moment resisting frame. The nine-storey SAC building is used as a model to evaluate the influence of this variation. The beam-to-column connections use reduced beam sections and are modeled with OpenSees using nonlinear elements that that capture cyclic deterioration. In one case, the panel zones are modelled as rigid offsets with no shear yielding. In the other case, the panel zones are modeled using a rotational spring box, in which rigid links are arranged in a rectangle and connected at the four corners by three pins and one nonlinear spring that captures the shear distortion in the panel zone. An Incremental Dynamic Analysis with 7 ground motions is conducted to determine the differences in global performance. The more refined panel zone model results in a longer first mode period and less energy dissipation in the plastic hinges of the beams. However, the difference in engineering demand parameters at design level events is minimal and may not justify the increase in computational requirements unless collapse assessment is desired.

Keywords: Steel Moment Resting Frames, panel zone, nonlinear time history analysis, global behaviour, earthquake engineering

1. INTRODUCTION

The Northridge Earthquake had far reaching impacts on the design of moment resisting steel frames in seismic areas. The investigation conducted after that seismic event highlighted the poor performance of the beam-to-column connections, due to fractures in and around the weld connecting the beam flanges to the column. This indicated a need for modifications to the connection, so the SAC joint venture was given the task of developing a new design strategy for moment resisting frame connections. The SAC research program resulted in the development of six prequalified connections (FEMA, 2000) that have demonstrated adequate performance during cyclic component testing. Such connections are currently approved for use in seismic areas of Canada (CISC, 2009) and the United States (AISC, 2010). The pre-Northridge research into ductile design for special moment resisting frames highlighted the contribution of the panel zone behaviour during a seismic event (Tsai & Popov, 1991; Ramirez et al., 2012). This behaviour was determined to contribute enough to the behaviour of the frame that its influence should be considered when analysing special moment resisting frames (AISC, 2010). Testing of some of the newer prequalified connections demonstrated a reduction in the amount of panel zone yielding as a result of shifting the plastic hinge away from the column face by changing the moment capacity of the beam section (Han, Moon, &

Jung, 2014; Takhirov & Popov, 2002; Uang, 2007). For example, one finite element analysis demonstrated that a pre-Northridge connection panel zone undergoes significant yielding, while the same scenario designed with a Reduced Beam Section (RBS) connection demonstrates very little panel zone yielding (Swati & Gaurang, 2014). Regardless of the connection type, the AISC seismic provisions (2010) require that the panel zone deformations be considered when modeling the global performance of a moment resisting frame (MRF). However, more modern connections could reduce the contribution of the panel zone yielding to the total hysteretic energy dissipation of the frame. It is important to understand the impact of the panel zone yielding on the global performance of the frame to determine the required level of modeling detail when analyzing MRF. This paper will examine the impact of including panel zone yielding on the global performance of the MRF by comparing the results obtained from a global analysis of the same MRF model using two accepted methods of modeling panel zones.

2. METHODS OF MODELING MRF COMPONENTS

2.1 Yielding Beams

Models of MRFs often consist of wire frames constructed with beam-column elements that have concentrated plasticity hinges at the joints of beam-column connections (Deierlein et al. 2010). The hinges capture non-linear behaviour by using moment-rotation hysteretic relationships (Charney et al. 2009; Foutch & Yun, 2002). These hinges have been shown to adequately capture the behaviour of the beam and column elements in an MRF during seismic loading because the real nonlinear yielding is also concentrated at the joints (Charney et al, 2004). Recently, several hysteretic models have been developed to capture cyclic stiffness and strength deterioration and post-yielding stiffness deterioration. One such model, which is used in this research, is the Ibarra-Medina-Krawlinker (IMK) model (Ibarra, Medina, & Krawinkler, 2005). This model is implemented in OpenSees by the *MODIMKPeakOriented* zero length member. The element's parameters were adjusted to match the results from component tests for RBS sections (Popov, Blondet, & Stepanov, 1999).

2.2 Panel Zones

In addition to the yielding beams, several models have also been constructed to represent the panel zone in MRFs. Some methods, such as the rotational spring box model (Figure 3a) and the scissor model (Figure 3b), use a rotational spring to capture the shear deformations in the panel zone (Gupta & Krawinkler, 1999; Sazmand & Aghakouchak, 2012). Other models, such as the frame model (Figure 3c), use a linear diagonal spring to capture shear behaviour (Castro, Elghazouli, & Izzuddin, 2005). Finally, the simplest considered model uses rigid offsets (Figure 3d) to represent the offset of plastic hinges from the centreline intersection of the beam-column connection without capturing any panel zone flexibility or yielding. This paper will assess the global behavioural differences between the rotational spring box model and the rigid offset model, shown in Figures 3a and 3d, respectively. The rotational spring box model uses eight rigid links around the perimeter of the panel zone. Each link is connected to a node of a beam or column element and a pin node at one of the four corners. The only exception is that one corner contains a rotational spring. This spring's stiffness is calibrated to represent the shear stiffness of the panel zone and is implemented in OpenSees using the *rotPanelZone2D* element. In the case of the rigid offset model, the length of each offset is determined by the geometric properties of the perpendicular adjoining element; the vertical rigid links have a length corresponding to half of the beam depth, while the horizontal rigid offsets have a length that corresponds to half of the column depth in addition to the distance to the plastic hinge. Extending the rigid offset to the plastic hinge of the beam reduced the number of degrees of freedom in the model while having little effect on the global response because the reduced beam section connection precluded yielding between the plastic hinge and the column. The extra length in the rigid offset was implemented in both models to maintain consistency when comparing results.

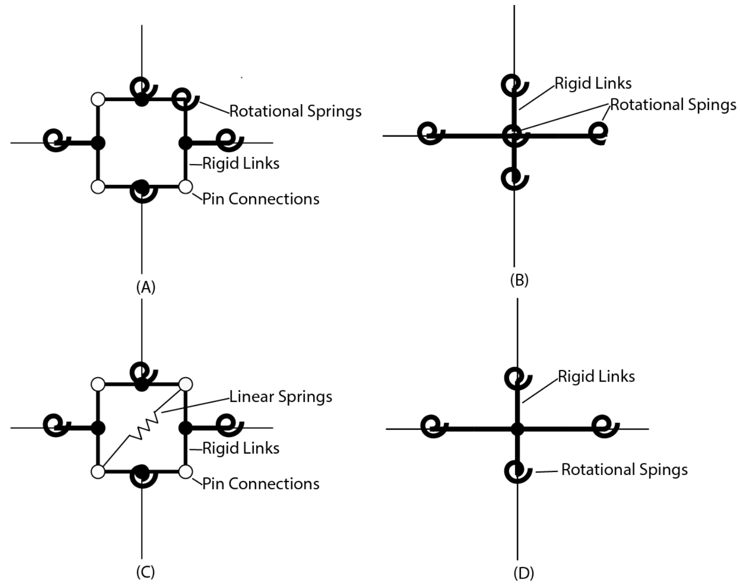

Figure 1: Alternative panel zone modelling techniques: (a) rotational spring box, (b) scissor links, (c) linear spring, (d) rigid offsets

3. DESIGN AND MODELLING OF PROTOTYPE STRUCTURE

To develop an understanding of the panel zone contribution to the global performance of the MRF, two models of the same building were created. The 9 storey SAC building in Los Angeles was chosen as the baseline for the models (Gupta & Krawinkler, 1999). This building was designed as part of the SAC venture in 1999 and uses post-Northridge MRFs as its seismic resisting system. Both models consisted of a basic wire frame with concentrated hinges, created in *OpenSees* and visualized in Figure 2. Rayleigh damping of 2% in the first two modes was used for both models (Gupta & Krawinkler, 1999). The rigid offset model has a fundamental period of 1.8 seconds, while the fundamental period of the rotational spring box model is 2.0 seconds. The rotational spring box model requires 3480 degrees of freedom, whereas the rigid offset model requires 1740 degrees of freedom. The increased number of degrees of freedom to capture the panel zone yielding increases the computation time by 67%. Such a difference in computational demand can become significant for incremental dynamic analysis or for larger buildings.

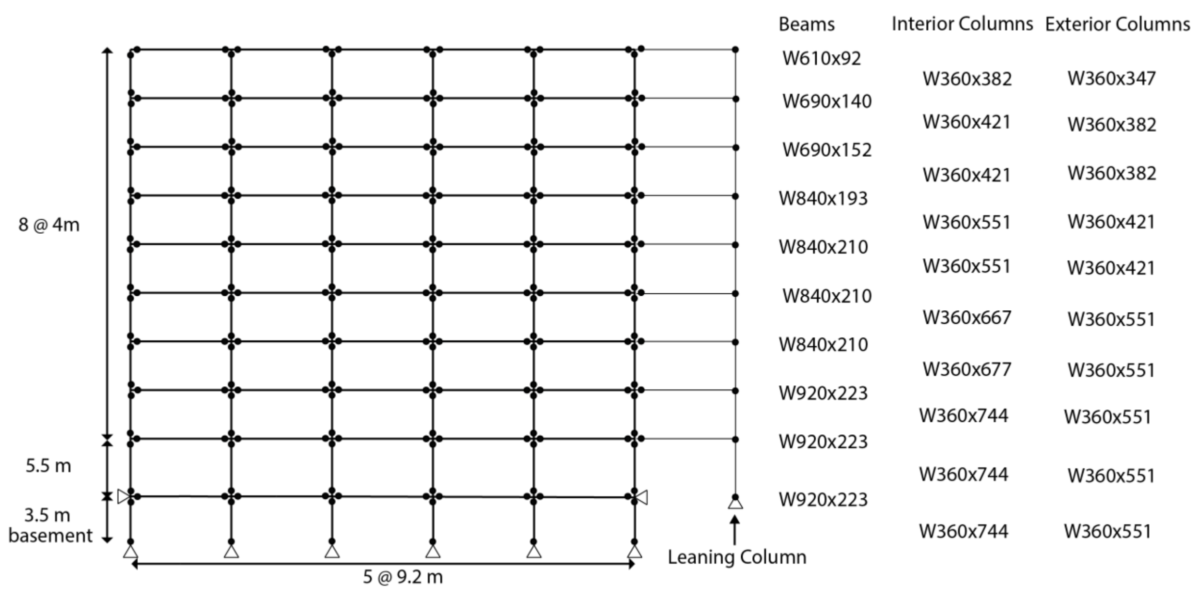

Figure 2: Wire frame model in OpenSees for 9 Storey SAC building with leaning column

4. INCREMENTAL DYNAMIC ANALYSIS

Seven ground motions where selected using the target spectrum scaling tool on the *PEER NGA West 2 Ground motion* database (Bozorgnia et al, 2012). The response spectra of the seven scaled ground motions are shown in Figure 3, scaled to match the design spectrum at the first-mode period of the rigid offset model. Scaling factors applied to the ground motions range from 0.69 to 3.19, as summarized in Table 1. An Incremental Dynamic Analysis with all seven scaled ground motions was conducted on both models, scaling the ground motions using the first-mode period of each model. The Engineering Demand Parameter of interest is selected as the maximum inter-storey drift, and failure is defined as an interstorey drift greater than 8% or an observed joint failure consisting of all beam hinges of a particular floor having deteriorated fully (i.e. rotations greater than 0.05 rad).

Table 1: List of Ground Motions and Scale Factors

Ground Motion	Location (Year)	Magnitude	Scaling Factor for Design Spectrum	
			Rotational Spring Box	Rigid Offset
Kern	Kern County, Cal (1952)	7.4	2.32	2.09
San Fernando	San Fernando Valley, Cal (1971)	6.6	1.57	1.41
Point Mugu	Point Mugu, Cal (1973)	5.7	2.24	2.02
Gazli	Gazli, USSR (1984)	6.8	0.77	0.69
Tabas	Tabas, Iran (1978)	7.4	0.95	0.86
Taiwan_13	Hualien, Taiwan (2013)	5.9	2.72	2.45
Lazio-Abruzzo	Basilicata, Italy (1990)	5.8	3.19	2.87

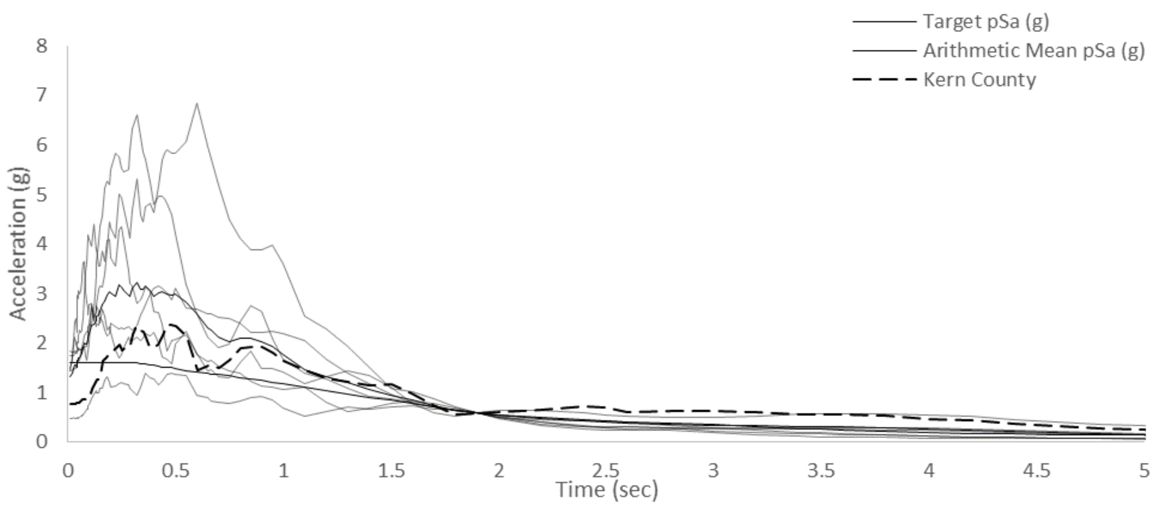

Figure 3: Target spectrum and ground motion spectra scaled at period of rigid offset model (1.8 seconds)

The IDA results shown in Figure 4 demonstrate generally similar characteristics. Both models reached failure for the same 5 ground motions, and each failure occurred at an amplification (relative to the design level) difference of less than 14%, with an average difference of 8.9%. The results from individual ground motions were even more similar at amplifications of less than 1.0 (i.e. the design ground motion intensity). However, the differences become much larger when the frame approaches collapse, increasing to a relative average difference of 10% at the last data point. This indicates that at design level earthquakes, the yielding of panel zones has relatively little impact on the global performance of the frame, but at larger intensities that approach collapse, the panel zone model is much more significant. The rigid offset model consistently produces larger median drifts when compared to the rotational spring box model, providing a more conservative result in general.

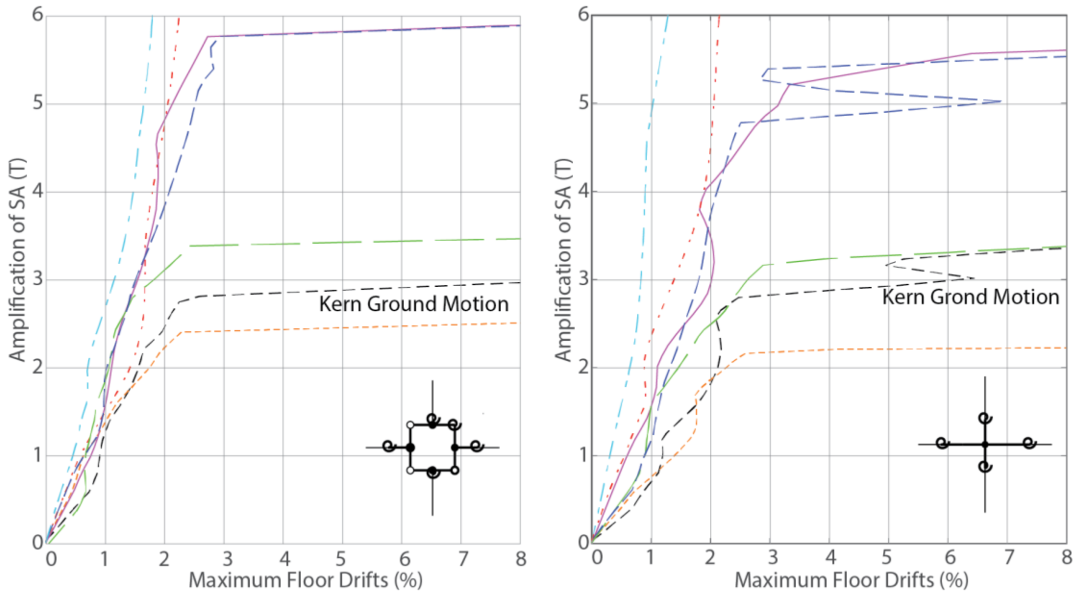

Figure 4: Incremental Dynamic Analysis for rotational spring box model (left) and rigid offset model (right)

4.1 Global performance comparison of both models during Kern ground motion

To better understand the behaviour of the models, the Kern ground motion was selected for a more in-depth analysis. This ground motion demonstrates trends that were typical in all 5 ground motions that reached failure. Both models withstood this ground motion at an amplification of 2.8 but failed at a multiplier of 3.0, as highlighted in the IDA in Figure 4.

Figure 5 compares the storey displacement time histories for both models at an amplification of 2.8. The models reach similar maximum roof displacements, and both show a similar influence of the higher modes through the time history. When comparing the maximum interstorey drifts, the models demonstrate very similar drift shapes, which is shown more clearly in Figure 7. While the rotational spring box model undergoes slightly larger peaks when compared to its rigid offset counterpart, it does not undergo drift values higher than 2.5%. The model with rigid offsets has 6% smaller residual displacements at roof level.

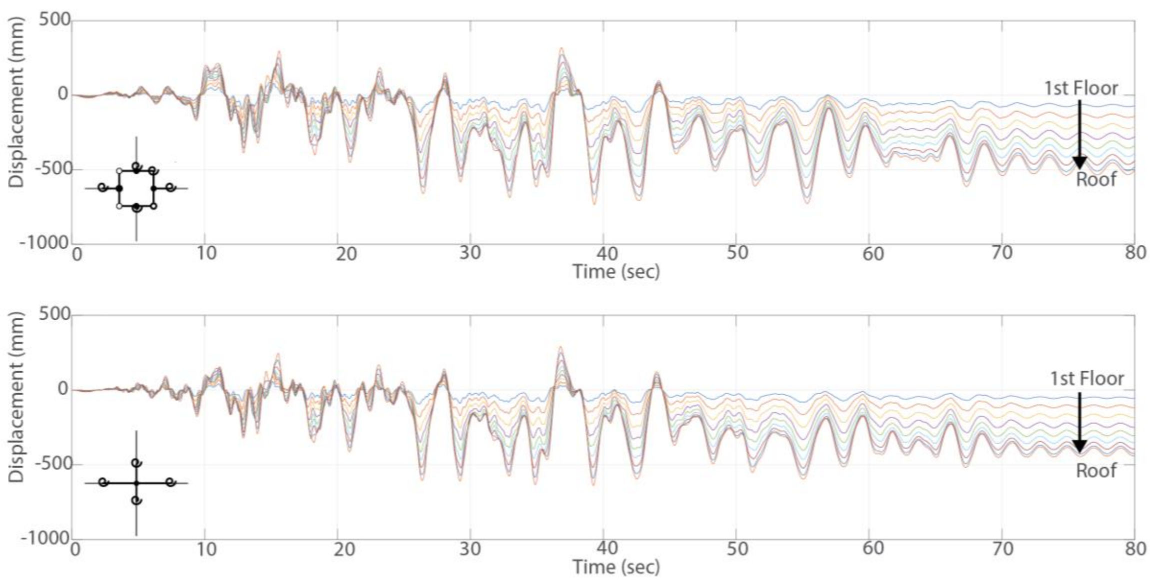

Figure 5: Total floor displacements under Kern ground motion at 2.8 times design amplitude: (top) rotational spring box model; (bottom) rigid offset model

Both models respond similarly to each other when the Kern event is amplified by a factor of 3.0, although the differences are larger in this case undergoing collapse. The time histories of storey displacements for both models are displayed in Figure 6. While the first 35 seconds of the response are generally similar (differences in floor displacements are generally less than 5%), the final seconds before collapse are different. The rotational spring box model demonstrates a slightly higher displacement peak than that achieved at failure, an indicator of full cyclic deterioration of the hinges. By comparison, the time history of the rigid offset model demonstrates a return past the neutral point of the structure, before returning to the same direction of failure as observed with the rotational spring box model.

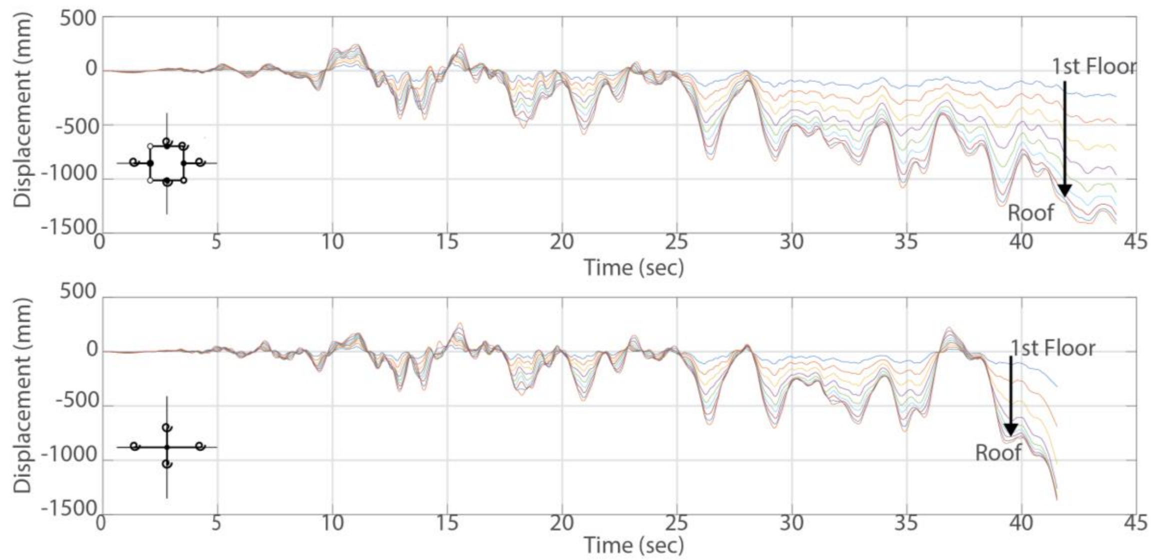

Figure 6: Total floor displacements under Kern ground motion at 3.0 times design amplitude: (top) rotational spring box model; (bottom) rigid offset model

The similarities observed when comparing total floor displacements are also apparent when comparing interstorey drifts. In both models, the lower floors experience larger interstorey drifts when compared to the upper floors, a behaviour that is highlighted in Figure 7. However, the models demonstrate some noticeable differences in interstorey drifts when the Kern event is amplified by 3.0: the peak interstorey drift over the height of the building is larger for the model with rigid offsets, and that peak occurs at the second storey rather than the third. Although a soft-storey failure mechanism forms in both models, the inter-storey drift is distributed more evenly across the height of the frame for the rotational spring box model.

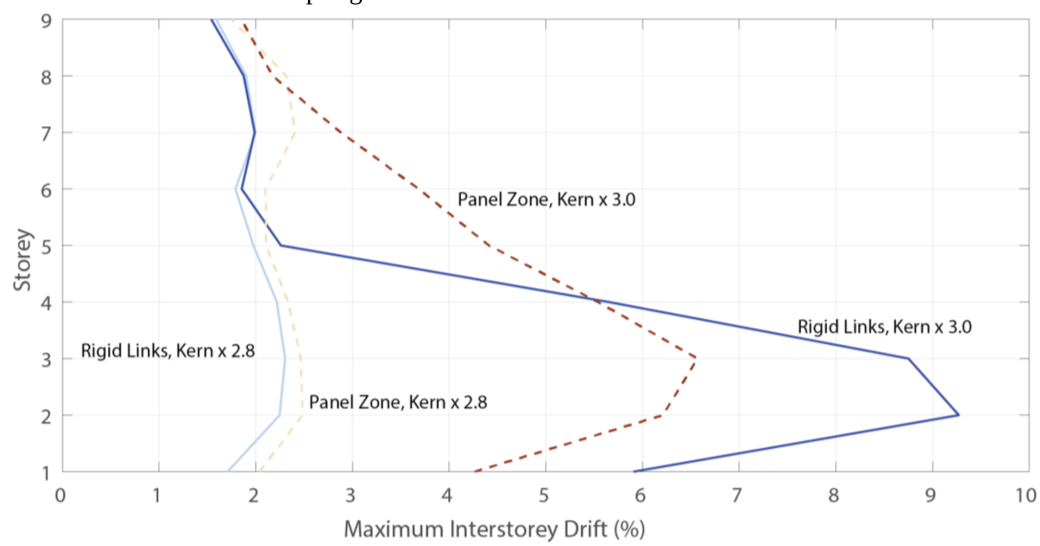

Figure 7: Maximum interstorey drift of both models

4.2 Local Performance Comparison of Critical Joint

Considering that both models had their peak demands at the second and third floors, a more localized analysis aims to better understand the interactions at this elevation. Figure 8 shows the hysteretic behaviour of a typical third floor joint with the rotational spring box model (Figure 8a) and with the rigid offset model (Figure 8b) during the Kern event scaled to 2.8 times the design intensity.

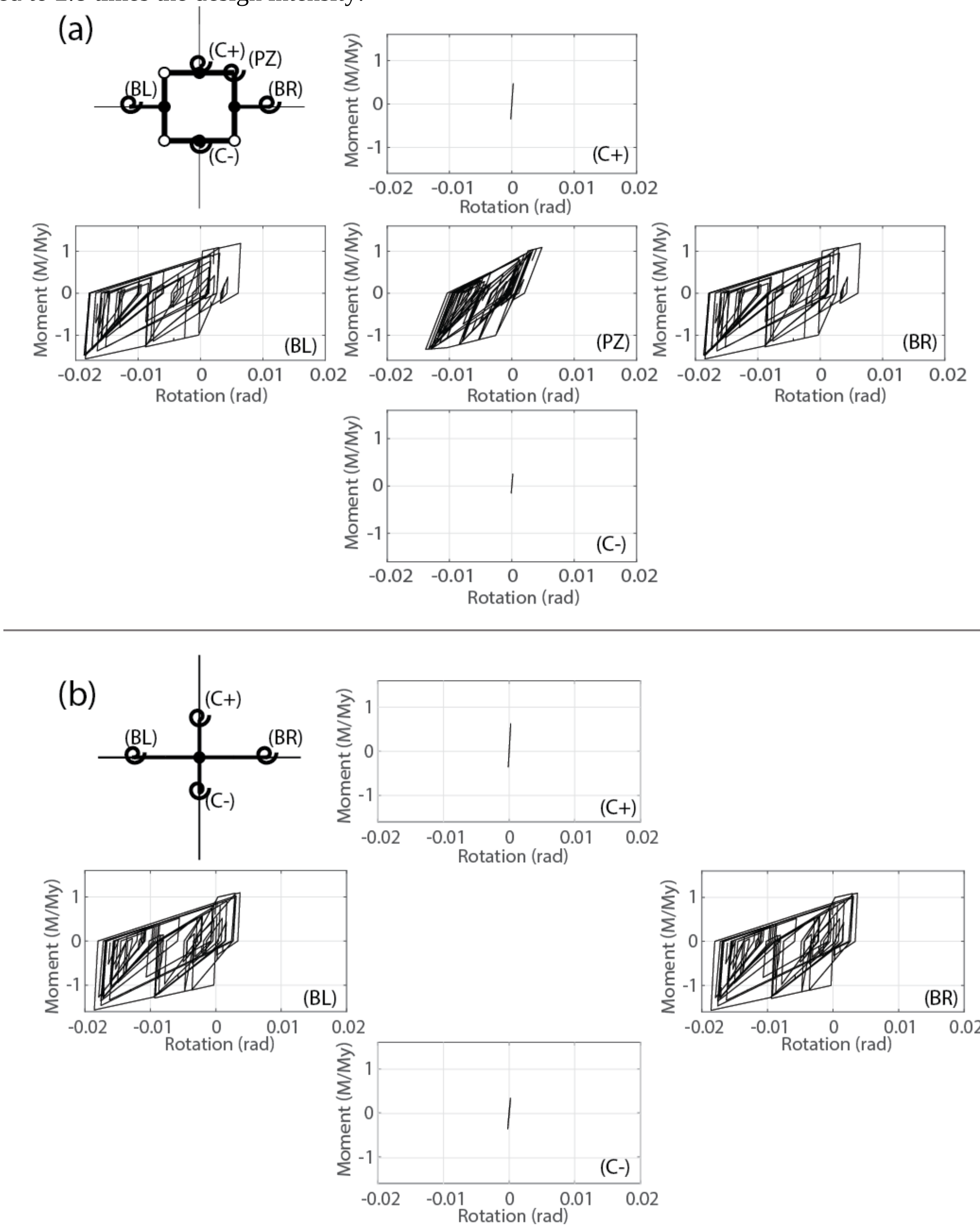

Figure 8: Typical third floor joint hystereses under Kern ground motion at 2.8 times design amplitude: (a) Rotational spring box model (b) Rigid offset model

In both events, the beam hinge graphs, shown on the left and right of Figure 8a and b, dominate the inelastic deformations and energy dissipation. The beam hinges in the rigid offset model undergo more inelastic behaviour than the beam hinges in the rotational spring box model, resulting in 6.1% more energy dissipated. This is to be expected because the shear yielding of the panel zone, shown by the panel zone hysteresis in the center of Figure 8a, contributes a portion of the inelastic energy dissipation. The column hystereses, at the top and bottom of the figures, indicate that the columns respond entirely in the elastic range with both models.

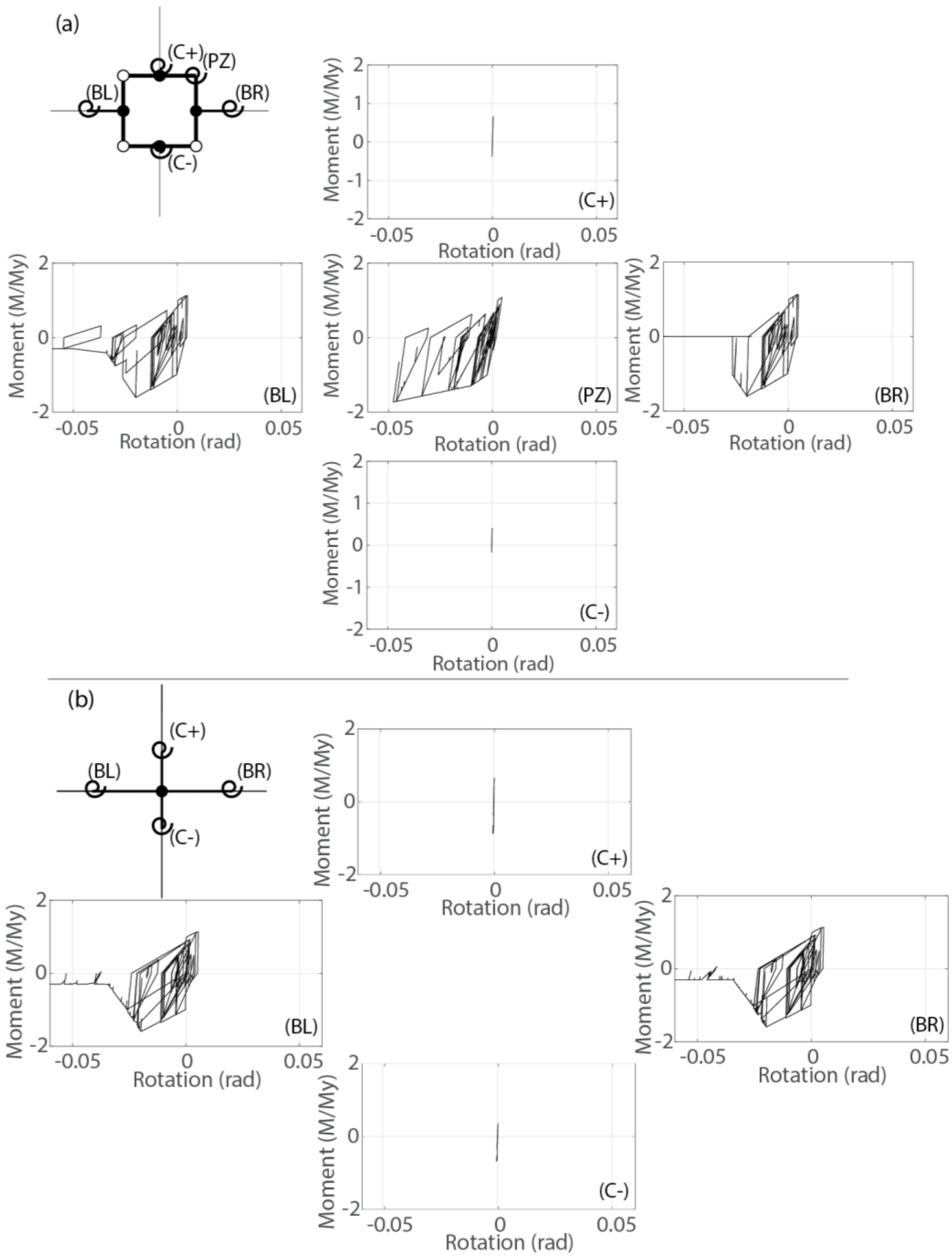

Figure 9: Typical third floor joint hystereses under Kern ground motion at 3.0 times design amplitude: (a) Rotational spring box model (b) Rigid offset model

The hystereses at the same joint during the Kern x 3.0 event (Figure 9) highlight the previously mentioned cyclic deterioration of the beams, leading to the failure of model. These graphs demonstrate that both models reached failure through deterioration of the beam hinges, shown by the beam springs undergoing large rotations without providing significant moment resistance. As was previously observed during the Kern event amplified by 2.8, the beam hinges behave similarly in both models, but the model with rigid offsets results in slightly more energy dissipation in the beams than the rotational spring box model.

5. CONCLUSIONS

The research presented in this paper indicates that modelling panel zone yielding has relatively little impact on the global performance of this MRF with RBS prequalified connections under design-level ground motions. Using a rotational spring box instead of rigid offsets has a greater influence on the interstorey drifts when the ground motions were scaled beyond design levels and towards collapse. Therefore, the joint model that is most appropriate for a numerical model may depend on the amplitude of the earthquake that will be considered and the expected seismic response. However, the predicted failure mode in this study was always deterioration of the beam hinges, regardless of which panel zone modeling technique was used. For design, using the rigid offset model results in a lower first mode period, increasing the estimated accelerations by increasing the first-mode frequency. These differences indicate that the rigid offset model is likely to be conservative for design of buildings that are similar to the 9-storey SAC frame designed using RBS connections. Using other connections in this frame could lead to different amounts of panel zone yielding, so further study is underway to determine the effects of panel zone yielding with other connections.

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