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CALTECH VIRTUAL SHAKER

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ABSTRACT

There is a renewed and growing interest on the seismic performance of the existing tall-building stock in Los Angeles during a major seismic event. Of particular interest is the seismic performance of existing high-rise buildings constructed before the 1994 Northridge Earthquake. These buildings were designed without the help of the state-of-the-art nonlinear analysis methodologies and the recently published performance-based seismic design guidelines (PEER, 2010; LATBSDC, 2011), and have been known to contain defects in the welded beam-to-column connections rendering them susceptible to large earthquakes. In order to assess the likely performance of these buildings in such earthquakes, Caltech and Arup have undertaken a simulation-based region-wide study of existing steel structures in Los Angeles. The data obtained from these simulations will not only give a detailed perspective of the analyzed building performance, but also provide a wider picture of the city-wide/region-wide impact of the imminent next major earthquake.

In the study's first phase, data on four existing high-rise steel buildings located in Los Angeles have been extracted from the structural drawings. The buildings have differing height-to-length aspect ratios and range from eleven to thirty stories. The four buildings, all of which were constructed pre-Northridge, are modeled using FRAME3D (Krishnan, 2009). The models are analyzed under seven two-component horizontal ground motion records and their performances are compared. The analysis results of this study and the accompanying seismic strengthening recommendations for high-rise buildings will fill an important knowledge gap and aid in the city's disaster mitigation plans.

INTRODUCTION

From 1960's to 1990's, moment-resisting frames were the most commonly used lateral load resisting system in California for medium to high-rise buildings. The performance of the concrete moment frame buildings of the era has been studied and has revealed significant deficiencies related to the non-ductile detailing practices of the time. Although there hasn't been a study of similar scope for steel moment-resisting frame buildings, their observed performance during the magnitude 6.7 Northridge Earthquake (January 17, 1994) has led the research and engineering community to expect similar deficiencies from these buildings.

The magnitude 6.7 Northridge earthquake of January 17, 1994 exposed the vulnerability of both older and newer steel moment frame structures to seismic-related damage. This damage was generally

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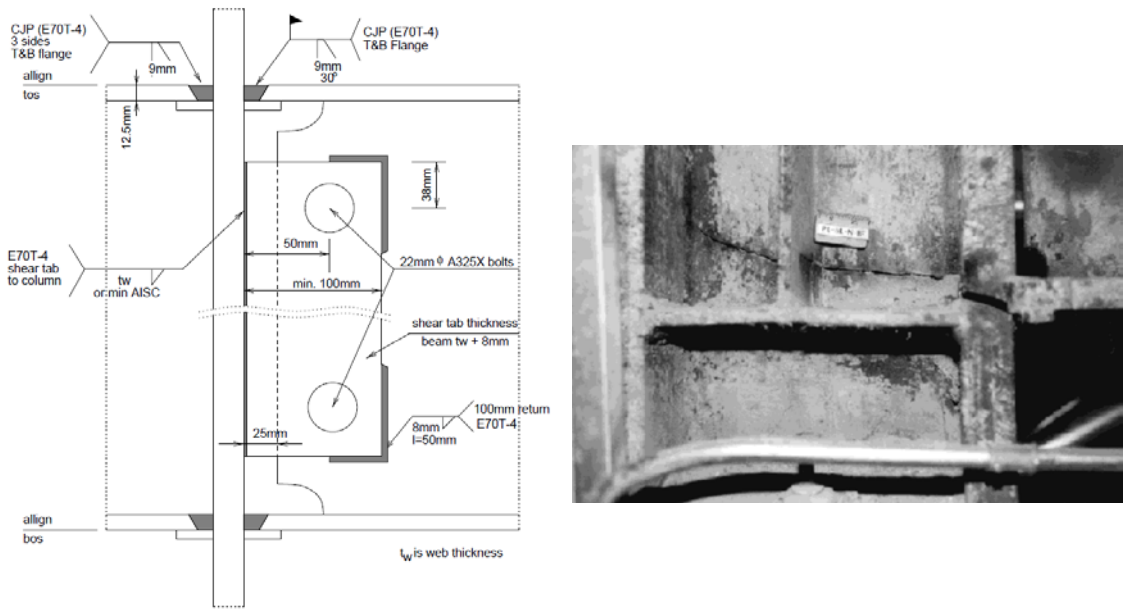
observed to be in the form of brittle failures at the interface of the beam and column members and was found in buildings ranging from 10 to 30 stories in height (Figure 1).



(a) (b)

Figure 1. Pre-Northridge Moment Connection; (a) Indicative detail (Stojadinovic, 2000), (b) Fracture observed at a column flange and web at a moment connection (by SEAOC)

The observations were mainly limited to the neighbourhoods that were affected by the Northridge Earthquake, and Downtown Los Angeles, where many of the medium to high-rise steel moment frame buildings were located, was outside of the mandatory inspection zone. After nearly two decades, the 2011 Christchurch Earthquake raised the question of the expected seismic performance of the buildings in Downtown Los Angeles again. The performance of individual medium-rise buildings in downtown Christchurch had a big socio-economic impact on the whole surrounding area (Kam et al. (2011)). With this motivation, the authors have undertaken a simulation-based region-wide study of existing medium to high-rise steel structures in Los Angeles. The data obtained from these simulations will not only give a detailed perspective of the analyzed building performance, but also provide a wider picture of the city-wide/region-wide impact of the imminent next major earthquake.

In the study's first phase, data on four existing high-rise steel buildings located in Los Angeles have been extracted from the structural drawings. The buildings have differing height-to-length aspect ratios and range from 11 to 30 stories. The four buildings, all of which were constructed pre-Northridge, are modeled using FRAME3D (Krishnan, 2009). The models are analyzed under seven two-component horizontal ground motion records and their performances are compared. The analysis results of this study and the accompanying seismic strengthening recommendations for high-rise buildings will fill an important knowledge gap and aid in the city's disaster mitigation plans.

SITE and BUILDING DESCRIPTION

As part of the study, four steel moment resisting frame buildings built before the 1994 Northridge Earthquake was modeled. Three of the buildings were located in Downtown Los Angeles, where as the fourth building was located in West Los Angeles (Figure 1). The buildings were bbuilt on stiff to very dense soil sites.

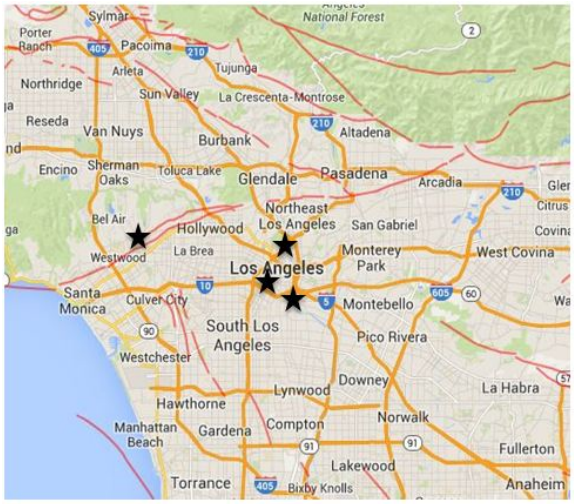
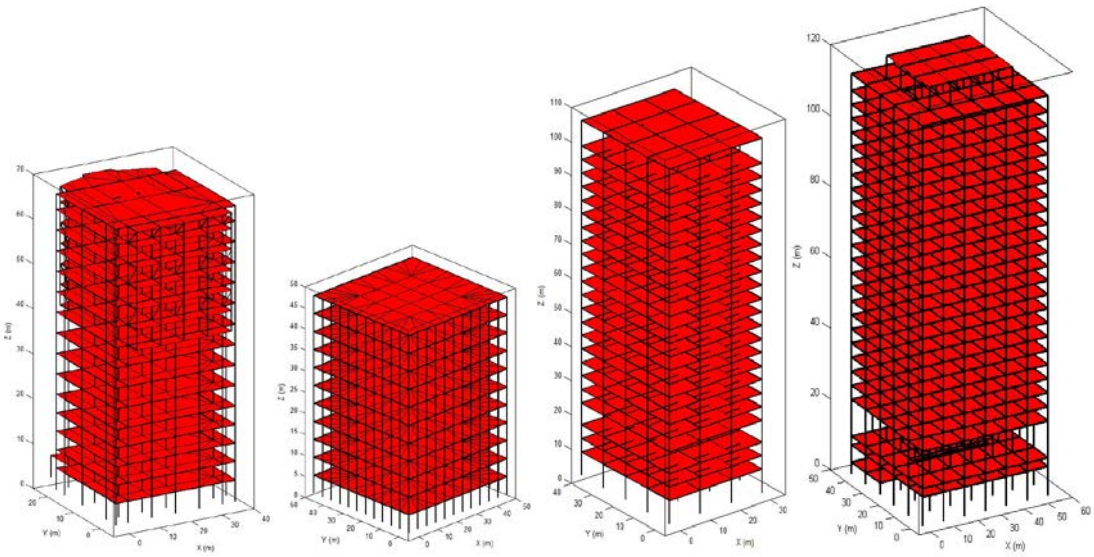


Figure 2. Building locations

The building models are shown in Figure 2. The building heights varied from 45 m to 120 m as shown on Table 1. Lateral load resisting system of the buildings consists solely of steel moment resisting frames. The gravity systems are structural steel columns and beams supporting light-weight concrete on metal decks.



(a) (b) (c) (d)

Figure 3. Building models

Table 1. Summary of the study buildings

Building ID	Number of Stories	Typical Story Height (ft)	Building Height (m)	Typical Grid Span (ft)	Plan Dimensions (ft x ft)
A	15	14	70	25	100x130
B	11	14	45	15	165x165
C	24	13	105	33	100x140
D	29	13	120	29	150x170

SEISMIC HAZARD

Seven time history pairs which spectrally match to the Maximum Considered Earthquake (MCE) hazard level (2475 yr) for a site in Downtown Los Angeles were used to analyse the four buildings (Melek et. al., 2014). The fault parallel direction was assumed to be parallel to the X axis of the buildings. Acceleration response spectra of the ground motions are provided in Figure 3. It is observed that for periods less than 4 seconds, the spectral accelerations of the records are closely matched, whereas a greater dispersion exists in the spectra for longer periods.

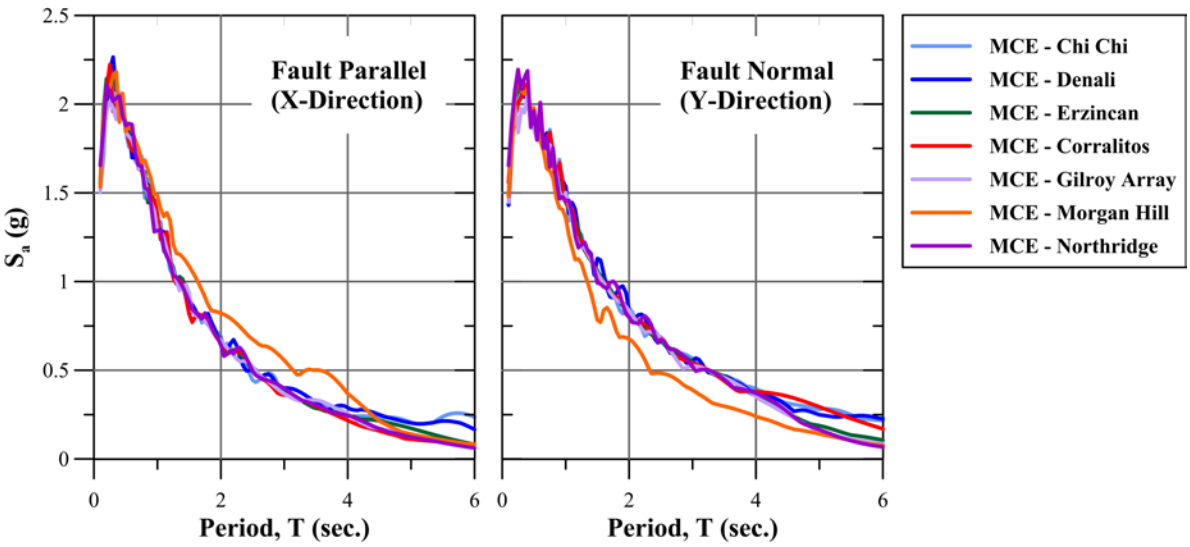


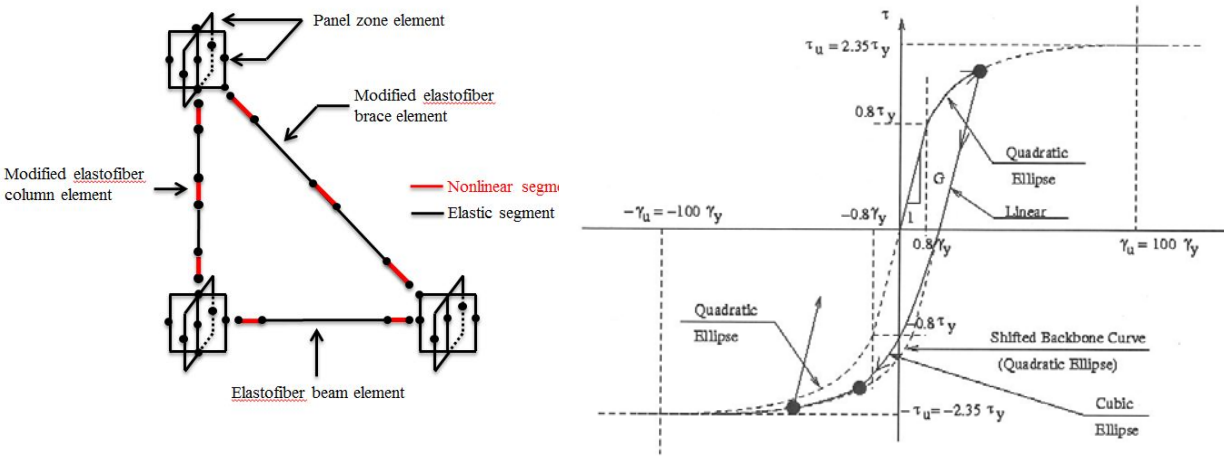
Figure 4. Acceleration response spectrum curves of Service Level and MCE scenario earthquake records for a site in Downtown Los Angeles (Melek et. al., 2014)

NONLINEAR MODELING

Nonlinear modeling and analysis of the buildings are conducted through FRAME3D (Krishnan, 2009), a program for the three-dimensional non-linear analysis of steel buildings. Figure 5a provides the element arrangement in the program, including the nodes, attachment points of the elements; beams, columns, braces and panel zones.

Panel Zones

Panel zone elements consist of two orthogonal panels which always remain planar and deform only in shear as a result of the end moments and shears of the attached beams and columns. A linear quadratic ellipsoidal model (Figure 5b) was used to represent the shear stress-strain behaviour of the panel zones (Challa, 1992).



(a) (b)

Figure 5. (a) Element arrangement in Frame3d, and (b) panel zone shear model

Columns and Braces

Modified elastofiber elements (Figure 6) which consist of three nonlinear segments (two at the element ends and one at the middle section of the element to capture buckling) was used to model the braces and the columns in the buildings. The nonlinear segments are divided into twenty fibers along the cross-section and each fiber runs the full length of the segment. Lengths of the nonlinear segments are taken as 3% and 11% of the clear spans for elements with low (less than 1.4) and high (greater than 1.4) ultimate stress to yield stress ratios, respectively, based on calibration studies on beams in double-curvature. The axial stress-strain behaviour of each fiber is governed by the hysteretic uniaxial cubic-ellipsoidal law proposed by Hall and Chan (1992, 1995). Monotonic stress-strain behavior and sample hysteresis loop of the model are provided in Figure 7a and Figure 7b, respectively. More details can be found in Challa (1992) and Hall and Challa (1995).

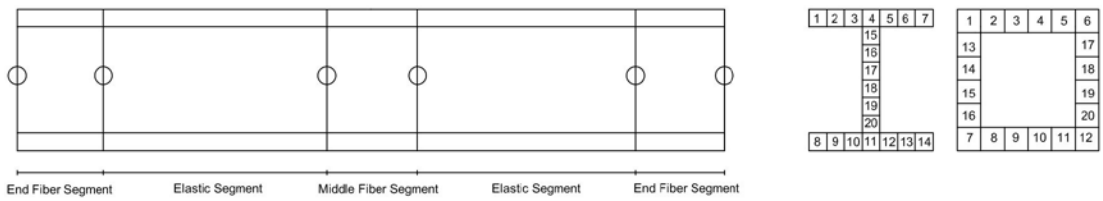


Figure 6. Frame3d modified elastofiber element

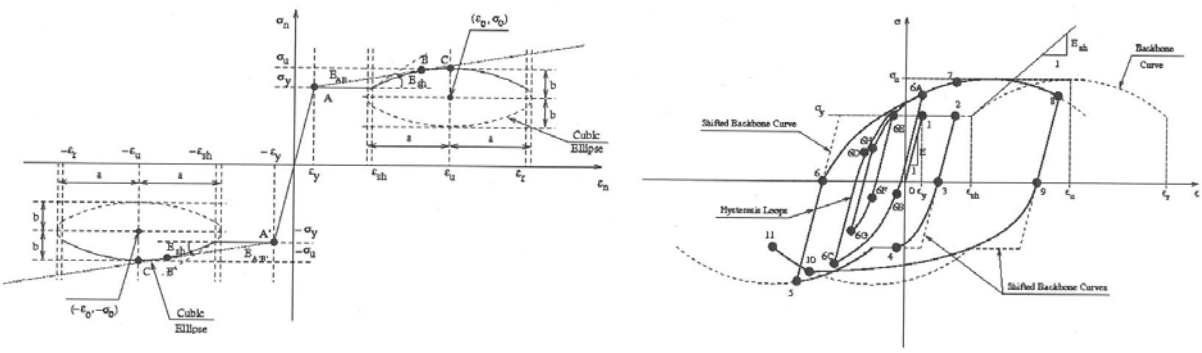
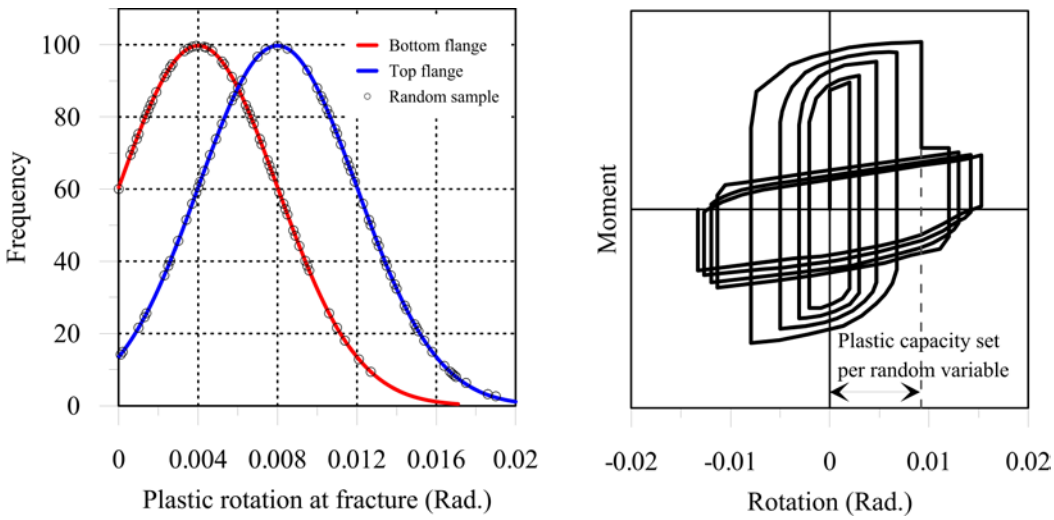


Figure 7. (a) Monotonic and (b) cyclic behavior of fibers in the modified elastofiber elements

Beams

Elastofiber elements which are essentially modified elastofiber elements (Figure 6) without the middle fiber segments, were used to model the moment frame beams in the buildings. Random fracture model by Maison and Bonowitz (1999) was used to assign fracture strains to the fibers. The model incorporates the use of a truncated normal distribution function (which has been derived based on tests on typical pre-Northridge connections) to randomly assign plastic rotation capacities to the fibers. Figure 8(a) provides the normal distribution functions used with mean plastic rotations of 0.004 (μ_{BF}) and 0.008 (μ_{TF}) radians for bottom and top flanges, respectively, both with standard deviations, σ, of 0.004 radians. A sample hysteresis curve of a connection with a randomly assigned plastic rotation capacity of 0.009 radians is provided in Figure 8(b).



(a) (b)

Figure 8. Distribution of plastic rotation capacities for bottom and top flanges (a), and sample hysteresis curve for a connection with plastic rotation capacity of 0.009 radians

Diaphragms

Four-noded linear-elastic plane elements connecting the moment frames were used to represent the floor slabs in the buildings.

NONLINEAR ANALYSIS RESULTS

Simulation results for Buildings B and C are presented in this paper. Twenty-four stories tall Building C, sustained heavy damage during the analysis where inter-story drifts up to 16% were recorded. The observed high inter-story drift ratios were not on limited to the first floor level. According to the analysis results, high inter-story drift ratios associated with the forming of a soft story was observed on Level 19 of the building (Figure 9).

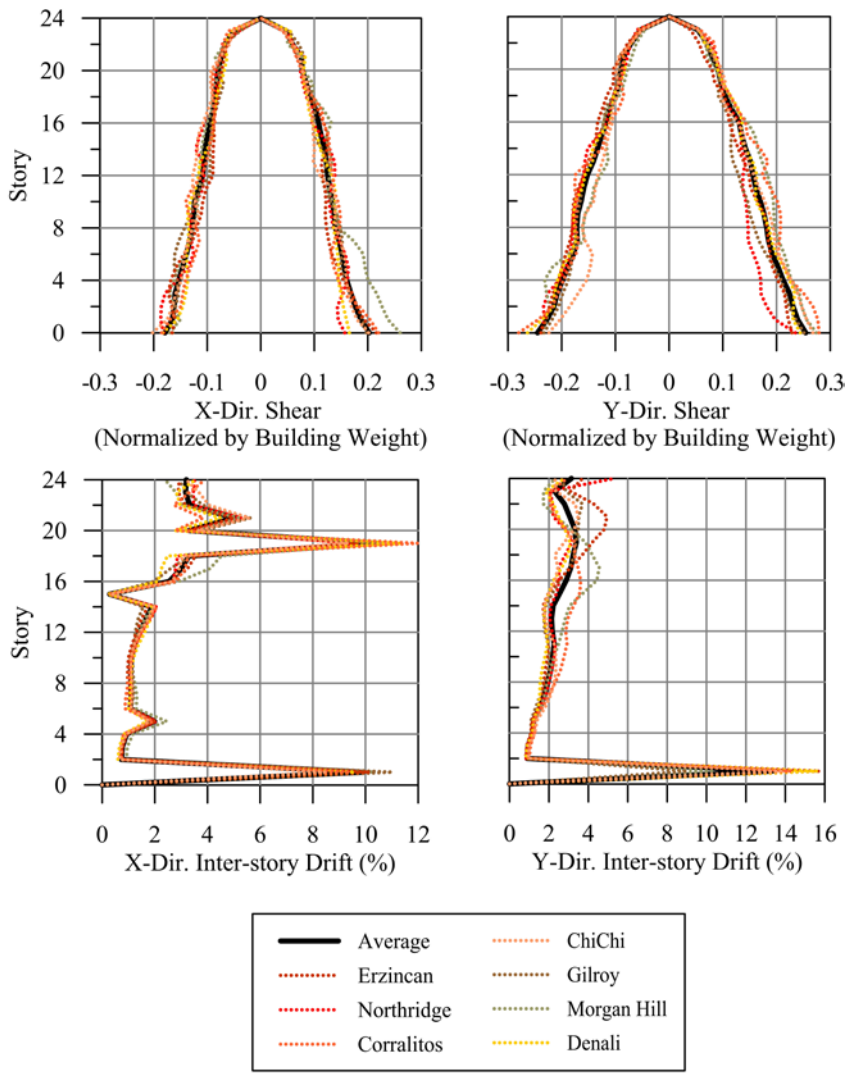


Figure 9. Story shears and inter-story drift demands on Building C

Figure 10 shows the simulated plastic rotations at the column and beam ends and at the panel zones. Beam and column plastic rotation results show a good correlation with the inter-story drift results given on Figure 9. At the bottom eight floors, a significant portion of the calculated plastic rotations at column ends exceeds the mean (μ) plastic rotation capacity values of 0.004 and 0.008 radians for bottom and top flanges, respectively. These plastic rotation demands are in line with the observed development of the soft story mechanism at lower levels of the building during the earthquake simulation. Similarly, substantial number of beam connections undergoes plastic rotations higher than the defined mean and mean + standard deviation capacities not only on lower floors but also at higher levels. The accumulation of the high rotation demands on level 14 to 19 is indicative of the rupture failure at beam connections and is in line with the measured high drift demands at upper levels in the X direction.

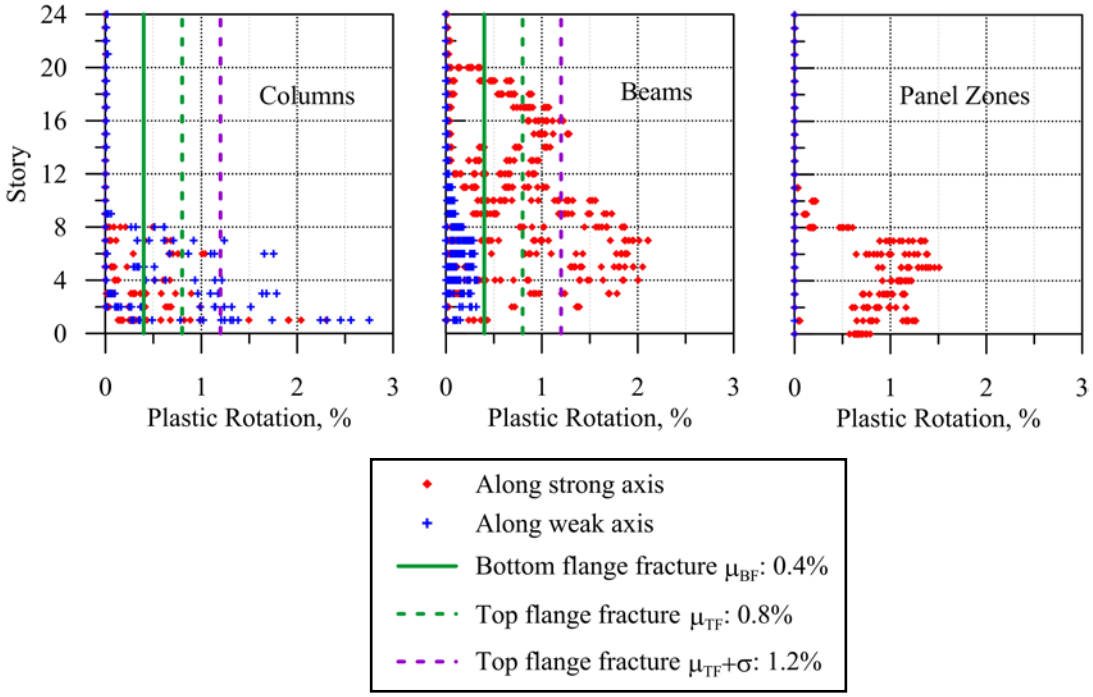


Figure 10. Average plastic rotations obtained for structural elements of Building C

Preliminary results of Building B are also presented (Figure 11). A failure pattern similar to Building C could be observed from the inter-story drift plots. High plastic rotation demands followed by the beam to column connection rupture failure results in high inter-story drift demands not only on lower levels but also at levels 7 and 10. As Building C, Building B has exhibited lateral load resisting system failure potentially resulting in collapse of the building.

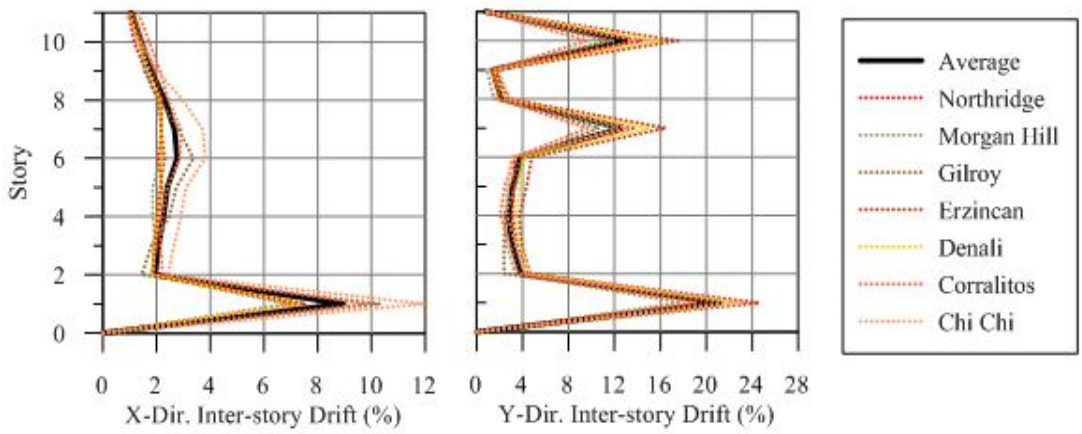


Figure 11. Inter-story drift demands on Building B

CONCLUSIONS

Expected seismic performance of medium to high rise pre-Northridge steel moment resisting frame buildings located in Los Angeles has been studied by modeling and analyzing four buildings in Frame3D. Preliminary results of the first two buildings are presented in this paper, and the analyses of the remaining two buildings are currently underway.

Preliminary results of the two buildings show damage patterns similar to the ones observed after the 1994 Northridge earthquake. Brittle fracture failures at beam-column connections lead to a significant lateral strength and stiffness degradation at several floors yielding excessive story drifts and initiating an overall loss of stability which eventually would cause collapse. Suggested retrofit schemes for these buildings would be addition of dampers and/or introducing new lateral systems to the buildings such as braced frames.

With the completion of the rest of the building analysis, this collaborative study Arup has undertaken with Caltech will provide detailed information on the expected seismic performance of the existing medium to high-rise structural steel building stock. This information helps us to confirm the worries related to the structural deficiencies of pre-1994 steel buildings which will help to reinforce City of Los Angeles's vision of increasing the seismic resiliency of the city in disaster mitigation plans of Los Angeles as well as be a motivation for other areas in the world which are susceptible to high seismic risks that would cause significant socio-economic impacts on the society.

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