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SEISMIC PERFORAMNCE EVALUATION OF A 1980'S EXISTING TALL STEEL PERIMETER MOMENT RESISTING FRAME USING NON-LINEAR ANALYSIS

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Abstract

From 1960 to the mid-1990s, steel moment resisting frames were regarded as one of the most ductile systems in the building code and therefore widely implemented as the seismic resisting system of choice across buildings along the Western United States. During this period, tall buildings were designed to satisfy code-prescriptive requirements, now widely believed to be inadequate for the seismic design of tall buildings. Furthermore, concerns regarding potential for fracture-prone welded connections became apparent following the 1994 Northridge earthquake. This paper presents a seismic performance assessment of a mid-1980s steel perimeter moment resisting frame of 50 storeys designed to reflect the state of design and construction of the time. The archetype is designed using the 1985 edition of the Uniform Building Code for a site in San Francisco with characteristic connection details and architectural form of the period.

Non-Linear response time history analyses are carried out within ETABS 2016 with three ground motion suites representative of different shaking intensities: Service Level Earthquake (SLE), Design Basis Earthquake (DBE), and the Maximum Considered Earthquake (MCE), as defined in the PEER guidelines for performance-based seismic design of tall buildings. Under SLE shaking, the performance of the archetype is linear elastic and comparable to the response of modern tall buildings. However, simulations suggest that the archetype does not achieve life-safety objectives under DBE and MCE level events with probabilities of collapse of 36% and 91%, respectively. These results suggest that existing tall buildings consistent with the archetype have considerable collapse risk in moderate-to-large earthquakes.

Keywords: Tall Buildings, Non-Linear Analysis

1. Introduction

Tall buildings are typically clustered in hubs of major social and economic production and their damage during a seismic event could disproportionately affect the local economy and potentially displace thousands of residents and businesses, as seen in the 2011 Canterbury Earthquake [1]. San Francisco, CA is bounded East and West by major active faults. The city also has a significant number of older buildings, which pose a considerable seismic risk. The clustered concentration of existing tall buildings in the downtown area has been identified as a significant risk not only to the local neighbourhood but to city wide post event recovery as it represents a dominant portion of the City's business sector [2].

During the period between 1960 and 1990, tall buildings were designed using conventional code-prescriptive requirements, now considered inadequate by both the academic and engineering community [3]. Until the development of the Performance Based Seismic Design (PBSD) philosophy, which is now mandated for the design of tall buildings in San Francisco, the seismic performance of tall buildings was not explicitly verified as part of the design process. As a result, little is known about the seismic performance of existing tall buildings that were designed prior to the adoption of PBSD [4].

Early code provisions had several shortcomings in their applications to tall building design (e.g. low base shear requirements, no consideration of higher mode effects, lack of capacity design principles, etc.). These inherit vulnerabilities in the code design requirements are compounded by construction and design practices of the time. Welded Steel Moment Resisting Frames (SMRF) were widely employed as the structural engineer's tall building system of choice, with a belief that they guaranteed ductility with a proven track record in seismic events. This belief was challenged when brittle fractures were observed in the welded steel connections following the 1994 Northridge earthquake. These observations prompted a limited number of inspections in the City of San Francisco. However, no inspections were mandated.

As of 2018, San Francisco has 156 buildings taller than 240 ft (~70.0 meters) [3]. A report produced by United States Geological Survey (USGS) identified 39 SMRF buildings constructed between 1960 and 1994 which are potentially seismically vulnerable [4]. Whilst modern seismic design objectives strive to achieve a probability of collapse of no more than 10% under Maximum Considered Earthquake shaking, studies suggest that for pre-1994 SMRF buildings, the potential for collapse could be as high as 80% (Molina Hutt et al., 2019).

This study presents the results of an intensity-based seismic performance assessment of an archetype mid-1980s perimeter SMRF of 50 storeys in height. The archetype is designed to reflect the state of design and construction of the time and follows the 1985 edition of the Uniform Building Code or UBC 1985 [6] Structural response is evaluated via non-linear response history analysis in ETABS 2016 [7]. Performance is evaluated under three distinct shaking intensities: Service Level Earthquake (SLE), Design Basis Earthquake (DBE), and Maximum Considered Earthquake (MCE) with return periods of 43, 475 and 2,475 years, respectively.

2. Archetype

Almufti et al. in collaboration with the Structural Engineers Association of Northern California (SEANOC), developed a database of the existing tall building stock in San Francisco [8]. The study revealed a large concentration of existing tall buildings constructed between 1960 and 1990, with SMRFs as the most prevalent type of seismic force resisting system in structures over 35 storeys in height. This survey was further refined and validated by the more recent Applied Technology Council – San Francisco Tall Buildings Study which has identified 78 SMRF buildings taller than 240 ft (~70.0 meters), which represents 50% of the existing tall building stock [2].

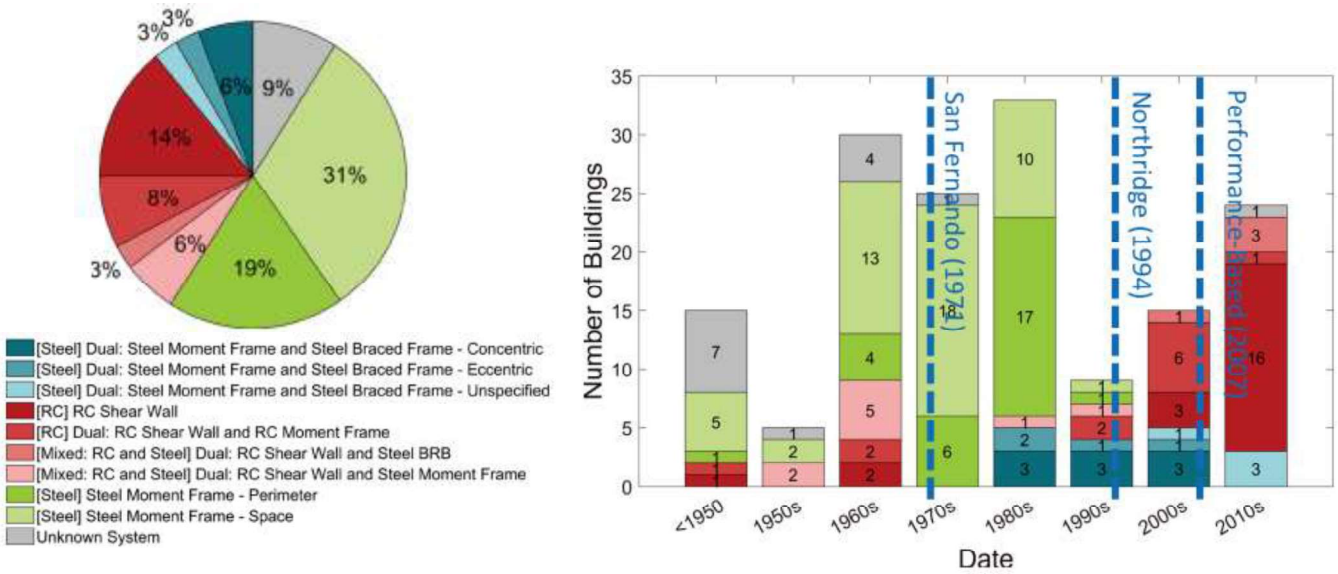


Fig. 1 - Seismic force resisting system in San Francisco's tall buildings by: (a) percentage of buildings, and (b) number of buildings by decade of construction [2].

A hypothetical 1980s 50-storey perimeter SMRF is developed in this study. The design, illustrated in Fig.2, is intended to enable an evaluation of expected seismic performance of existing tall SMRF buildings of this era. The archetype attempts to represent the state of design and construction from the mid-1980s. From 1960 to 1990, engineers typically implemented either a space or perimeter moment resisting frame configuration. The use of a perimeter moment resisting frames was employed more frequently during the 1980s, as illustrated in Fig.1b, to maximise net internal floor areas [10].

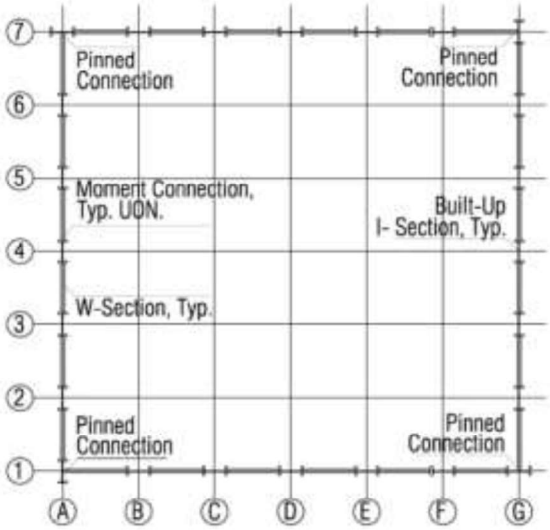
The structural system assumes a three-inch concrete decking spanning between secondary beams onto primary beams, which transfer load to gravity columns at grid line intersections in the interior of the building. The seismic force resisting system is a perimeter moment frame consisting of built-up I-section columns of ASTM A572 steel grade (345MPa/50 ksi) and wide flange beams of ASTM A36 (248MPa/36ksi) with welded beam-to-column connections. Perimeter beams connecting to the minor axis of corner columns are assumed to have pinned connections.

The archetype structure is designed as per the provisions of UBC 1985 with additional provisions from the Structural Engineers Association of California (SEAOC) Bluebook, commonly used to supplement minimum design requirements [4]. SEAOC did not specifically release an edition of the Bluebook to supplement UBC 1985 with its fourth and fifth editions released in 1980 and 1988, respectively.

For floors designated for office occupancy, the Superimposed Dead Load (SDL) consists of an allowance for services, raised floor and partitions at 40 psf (~1.9 kPa). The 25th and 49th floor have an additional allowance for plant equipment and plinths equivalent to 135 psf (~6.5 kPa). An allowance of 41.5 psf (~4.1 kPa) is considered for a façade composed of precast concrete panels and windows. The Live Load (LL) is assumed to have an 80-20 split between office and corridors respectively with 56 psf (~2.7k Pa) as specified in Section 2304 Table 23.A of UBC 1985.

The design of the archetype structure is largely dominated by drift requirements, as is typical in tall building design. Unlike earlier editions, UBC 1985 specifies prescriptive storey drift limits of 0.5% under its seismic provisions. Under wind loading, drift limits provisions within Appendix D of the 1980 SEAOC Blue book of 0.25% are adopted for this study as UBC 1985 did not specifically mandate drift limits under lateral wind loading. The resulting base shear under the UBC 1985 provisions for wind and seismic loading are 3,607 kips (16 MN) and 6,442 kips (28.6 MN) respectively, equivalent to 1.8% of the total building dead weight under wind loading and 3% under the seismic provisions. As a result of the structure being dominated by the drift requirements, the frame section sizes resulted in low strength utilisations under the prescribed code forces. UBC 1985 did not prescribe any design requirements for panel zone flexibility or strong

column-weak beam principles. However due to the large size of the column sections, required to comply with drift requirements, weak panel zones are not believed to be a potential source of vulnerability in the design. A summary of the dynamic properties of the building in one of its principal building directions is presented in Fig 2c.



Mode	Period (sec)	Effective/Total Mass (%)
1	6.09	73.15
2	2.20	11.94
3	1.25	4.57

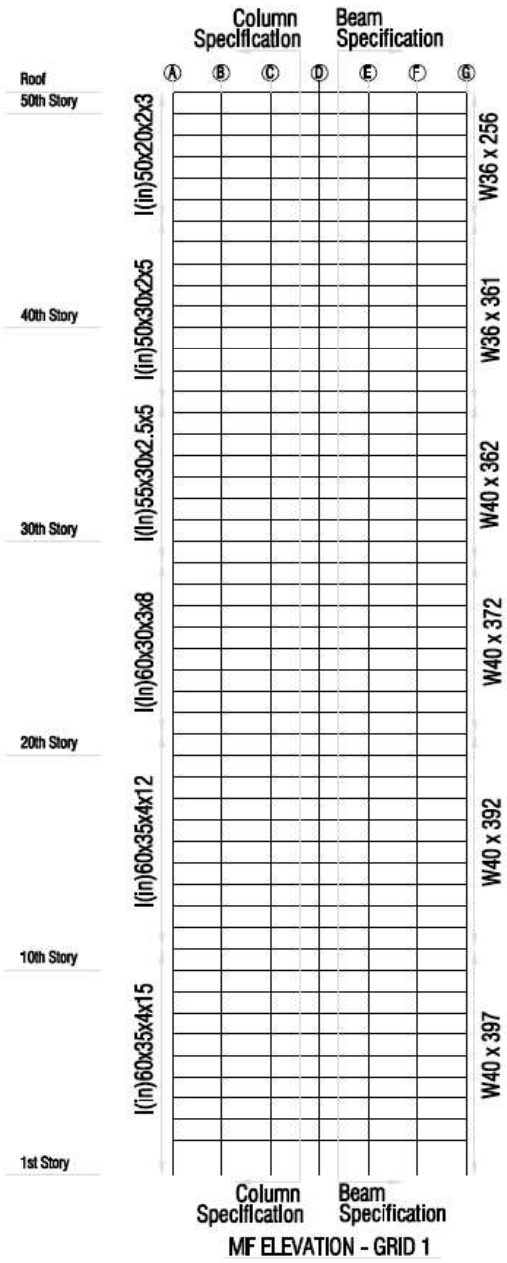


Fig. 2 - Archetype 50 storey perimeter steel moment frame office building: (a) plan, (b) elevation (illustrating key beam and column sizes up the building height) and (c) summary of the dynamic properties of the archetype building in one of its principal building directions.

3. Analytical Model

The following outlines the three-dimensional numerical model developed for the seismic performance assessment carried out in ETABS 2016. Component models to capture the non-linear behaviour of beams, columns and panel zones are included in the analysis. A rigid diaphragm is assumed at every storey, and column splices are not considered in the analysis.

3.1 Beams

Beams are modelled with lumped plasticity at beam ends on the perimeter moment resisting frame. The archetype is assumed to be constructed with fracture prone connections that would prevent the development of a full plastic response post-yield. The cyclic backbone definition which accounts for the fracture is based on ASCE 41-17 [11] recommendations for modelling Welded Unreinforced Flange (WUF) moment connections, which limits the post-yield rotation to simulate fracture in pre-Northridge connections as illustrated in Fig.3. It is assumed that the bending moment capacity of the beam reduces to zero when rotations exceed the maximum value defined in ASCE 41-17. Fig.3 presents an analytical simulation for a beam section with a WUF moment connection which undergoes fracture in accordance with the backbone definition within ASCE 41-17.

3.2 Column Components

Columns are modelled with lumped plasticity hinges capable of capturing the interaction between axial load and applied moments in both axis. Cyclic deterioration of columns are not explicitly captured and therefore use a cyclic envelope, in which degradation is implicitly captured, as defined by NIST, for wide flange columns. Similar to the beam hinge definition, the equations define the backbone curve with respect to the yield moment but with consideration of the axial demands in the column.

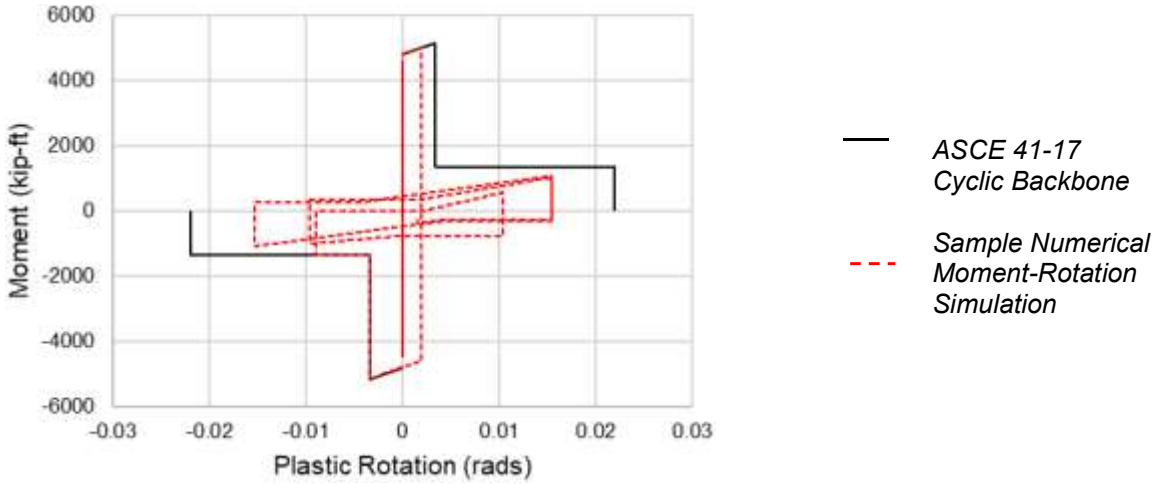


Fig. 3 - Analytical simulation of a Welded Unreinforced Flange (WUF) beam-to-column connection with the cyclic backbone as defined in ASCE 41-17 (2017).

3.3 Panel Zones

Steel panel zones are modelled as per guidance provided in ATC-72 [12] based on the Krawinkler tri-linear shear force-deformation relationship model which utilises rigid elements that are connected with rotational springs at each corner. The panel zones within ETABS were modelled using multilinear link elements in the joints between the columns and beam elements in which a moment-rotation relationship is defined as per the relationships defined in ATC-72-1.

3.4 Additional Assumptions

The seismic weight of the structure is assumed to include the self-weight, superimposed dead loading and 25% of the unreduced live loads. Equivalent viscous damping is set as 2.5% of the critical damping as per PEER guidelines [12]. Linear elastic gravity beam and column elements are included in the three-dimensional model. The structure is assumed to have a fixed base at foundation level with no consideration for soil-structure interaction.

4. Seismic Hazard and Ground Motion Selection

The majority of tall buildings in San Francisco within the downtown area are SMRFs as illustrated in Fig.4 [2]. This study considers a site situated near the San Francisco Transbay Transit Centre with ground conditions consistent with Site Class D as defined in ASCE 7-10 [13]. The site in consideration is approximately 14km from the San Andreas Fault and 16km from the Hayward Fault [4].

Fig. 4 - Rendering of tall buildings constructed from 1960 to 1994 in downtown San Francisco classified according to seismic force resisting system [2].

Three return periods are selected to carry out intensity-based seismic performance assessments: a 43-year, a 475-year and a 2,475-year return period corresponding to a Service level Earthquake (SLE), Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) as defined in ASCE 7-10 and PEER TBI [12]. No consideration is made of vertical ground motion components, as in common in seismic design practice.

Ground motions are obtained from the PEER NGA-West-2 database [14]. At each intensity level a set of linearly scaled ground motion pairs are selected such that their geometric mean spectral acceleration matches the target spectrum within the period range of interest between $0.2T_1$ and $1.5T_1$, from approximately 1 to 9 seconds. Eleven ground motions are selected for the DBE and MCE evaluations, and seven are selected for the SLE assessment in accordance with the PEER TBI recommendations. Fig.5 illustrates the comparison between the target spectrum and geomean average for each of the ground motions suites. Average spectral accelerations at the fundamental period of the structure are 0.03g, 0.17g and 0.28g for SLE, DBE and MCE, respectively.

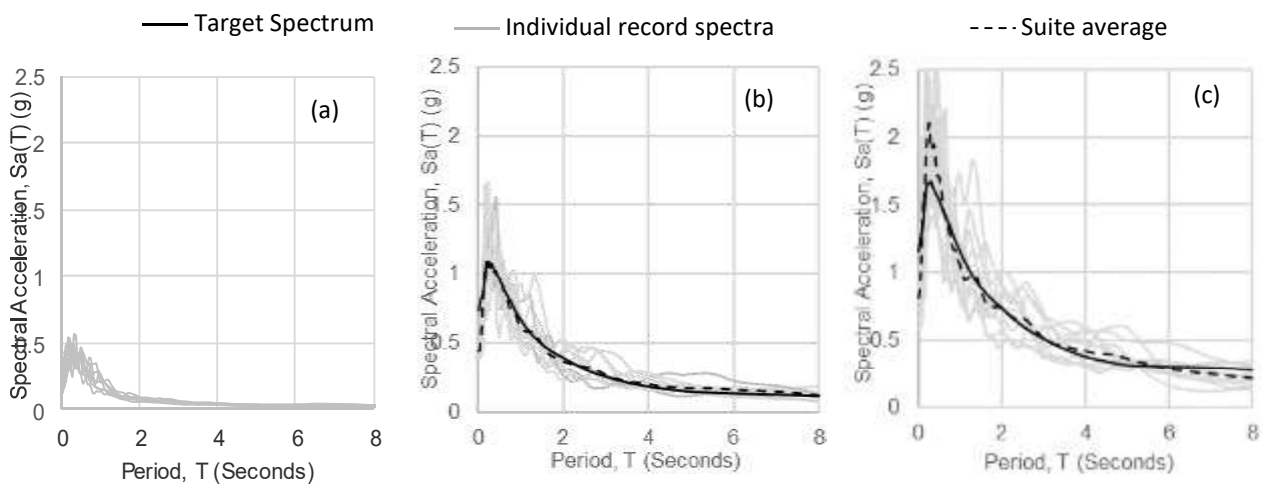


Fig. 5 - Target and individual ground motion spectra at (a) SLE, (b) DBE, and (c) MCE.

5. Results and Performance Evaluation

The performance of the archetype is evaluated against the PEER TBI guidelines which set out performance limits in terms of global response and component deformations. Global response is evaluated based on Inter-Storey Drift (ISD), both transient and residual. Component response is evaluated in beams, columns and panel elements. Peak Floor Acceleration (PFA) is also provided as a proxy of expected non-structural component damage. Fig.6 provides a summary of the average, median and individual ground motion simulation results for non-collapses simulations (100%, 64% and 9% of ground motions at SLE, DBE and MCE shaking intensities, respectively).

5.1 Service Level Earthquake (SLE)

PEER TBI limits ISDs in any storey under the SLE evaluation to 0.5%. Fig.6a illustrates that 71% of the SLE ground motions exceed this limit. Peak ISD deformations are generally fairly uniform up the height of the structure. Results indicate a response within the linear elastic range, with no significant concentrations of deformation on any one storey. However, the absolute maximum ISD of all simulations is 0.96%, at storey 20, and the maximum of the mean of peak ISD results is 0.64%, at storey 19, far exceeding the 0.5% limit.

At the component level, the beams, columns and panel zones remain elastic, as can be seen in Fig.6d to 6f. Because the structural system response is within the linear elastic range, there are no residual drifts, as observed in Fig.6b. The absolute maximum PFA of all simulations is 0.23g, at the top storey, and the maximum of the mean PFA results is 0.17g at that same storey.

This ground motion intensity level reflects the upper bound seismic events that have occurred in the San Francisco Bay Area within the last 100 years [15]. Overall, the results suggest that under a SLE event, the archetype structure will achieve a performance somewhat consistent with the PEER TBI performance objectives for Risk Category II structures. However, direct economic losses are anticipated from non-structural damage due to the observed drifts exceeding those recommended for modern tall building design.

5.2 Design Basis Earthquake (DBE)

The results indicate a 36% chance of collapse at the DBE intensity level with four out of 11 ground motion records resulting in collapse. These results are consistent with those reported by Molina Hutt et al. (2019) for a similar building, which adopted a space frame configuration and was designed per UBC 1973, reported to have a 40% chance of collapse at DBE. These results suggest that both 1970s and 1980s, space or perimeter tall SMRFs, fail to achieve the life-safety objective of modern building codes and pose a considerable collapse risk under code-level ground motion shaking.

Collapse realisations are not included in the response plots illustrated in Fig.6g to 6l. The maximum of the mean of peak ISD results is 2.8% at storey 17 for non-collapse simulations. The absolute maximum ISD is 5.6%, at storey 20 far exceeding the 2% limit in ASCE 7-10. The absolute maximum PFA of all simulations is 1.26g at the storey 17, and the maximum of the mean of peak PFA results is 0.93g at storey 19.

Due to significant non-linear response in the structural components, there is considerable residual ISDs expected in non-collapse simulations. A single non-collapse DBE simulation, which nearly induced collapse, saw a peak residual drift of 5.5% at storey 20 which is significantly greater than the maximum of the mean and median of peak residual drift at 1.2% and 0.72% respectively. While ASCE 7-10 does not explicitly define limits on residual ISD under DBE shaking, the results far exceed those in PEER TBI under MCE shaking, which are limited at 1.5% as the absolute maximum of any simulation in any storey, and 1% as the maximum of the average in any storey.

These results indicate that, if there is no structural collapse, there will be significant damage to structural and non-structural components. FEMA provides a classification of damage state categories associated with residual ISD [3]. Based on the average residual ISD results at the DBE intensity, the building is expected to fall under a Damage State 3 category. This damage state indicates a need to realign the structural frame to restore a margin of safety for lateral stability, which may not be economically or practically feasible [3].

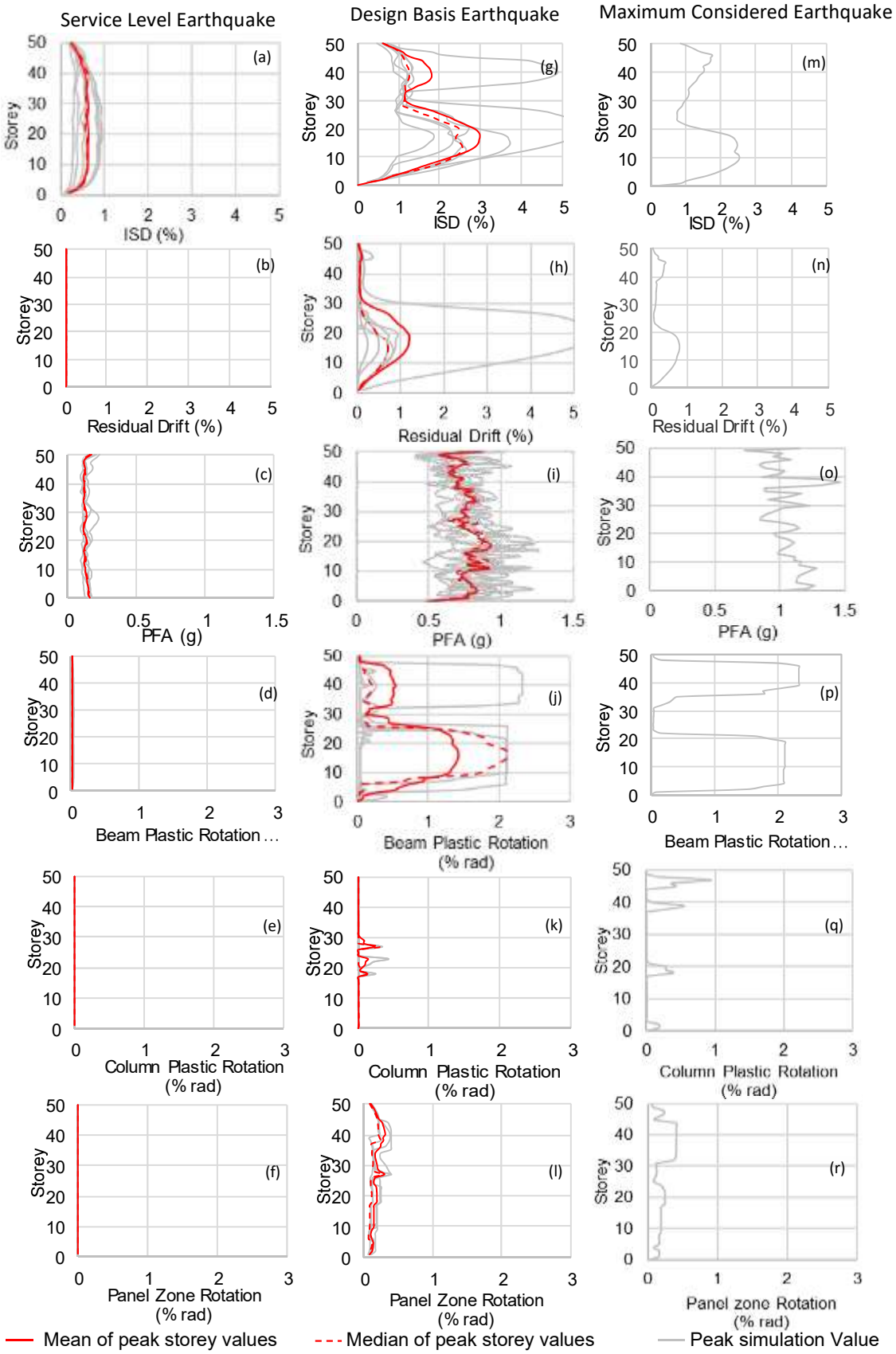


Fig. 6 - Peak storey results in non-collapse nonlinear dynamic analysis simulations at different shaking intensities: Inter-Storey Drift Ratio (ISD), Residual ISD, Peak Floor Accelerations (PFA), and plastic rotations of beams, columns and panel zones.

In cases of non-collapse simulations, there are significant beam connection fractures expected to occur, particularly in the lower proportion of the structure as seen in Fig.6j. Fractures are typically concentrated between storeys 11 to 26, with three simulations causing rotations that exceed the limits for WUF connections as defined is ASCE 41-17. Within each non-collapse simulation, columns are observed to yield between storeys 16 to 30 as seen Fig.6k. Overall deformations did not exceed the post capping moment capacity of the columns. As a result, the overall stability of the structure was not compromised. However, the large number of fractures and column yielding could constitute considerable collapse risk in aftershocks.

The ground motions that induced collapse of the archetype building at DBE exhibited significant velocity pulse characteristics. In a study carried out by Almufti et al.[8], 90% of ground motion records representative of the MCE intensity level for a site in San Francisco were expected to have velocity pulse-characteristics, hence the use of pulse-like ground motions in this study is believed to be consistent with the seismic hazard in San Francisco.

5.3 Maximum Considered Earthquake (MCE)

At MCE shaking, the results suggest a large probability of collapse, with only one out of 11 simulations not resulting in collapse. Failure is typically observed in middle section of the structure with collapse occurring around storey 20. The results are similar to that observed by Molina Hutt et al. (2019) for an equivalent structure designed as per UBC 1973, which is reported to have a probability of collapse of 80% under MCE ground motions.

In the non-collapsed simulation, significant structural damage is observed in critical elements, though not sufficiently to cause collapse. The maximum ISD was 2.53% at storey 10 with a residual ISD of 0.8% as shown in Fig.6m and 6n, respectively. There are significant component deformations, with only eight storeys without fractures in beam-to-column connections. Column deformation are also significant.

6. Conclusions

This study presents the results of an intensity-based seismic performance assessment of a 1980s tall perimeter SMRF designed per UBC 1985 in San Francisco, CA. Performance is evaluated via non-linear response history analysis under three distinct shaking intensities: SLE, DBE and MCE.

Under SLE shaking, the structure response is within the linear elastic range though drift results suggest there may be non-structural damage. Under the DBE and MCE shaking, results suggest a 36% and 91% probability of collapse, respectively, failing to meet the life-safety objectives of modern building codes. In cases where collapse is not observed, permanent deformations suggest the building would present considerable collapse risk in aftershocks. Two major seismic deficiencies are identified: (1) a high percentage of pre-Northridge connection fractures within the seismic force resisting system, leading to (2) a tendency to form a soft-storey mechanism in the middle portion of the structure.

As tall buildings in San Francisco have not experienced significant seismic demands from earthquakes in recent years (e.g. 1989 Loma Prieta exhibited low spectral accelerations at long periods), tall building owners have little appreciation for their seismic risk. The result of this study supports cause for concern that existing tall SMRF buildings may not achieve the life-safety objective of modern building codes under moderate-to-large seismic events.

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