MSc Earthquake Engineering with Disaster Management
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A Seismic Performance Assessment of a 1980s Perimeter Steel Moment Resisting Frame Using Non-Linear Analysis

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Introduction

San Francisco is at significant risk of a major seismic event, with the US Geological Survey estimating there is a 72% probability of a seismic event greater then 6.7Mw occurring by 2045 (USGS, 2017). The city is considered to be one of the world's most seismically vulnerable cities, with the neighbouring Hayward fault having the potential to disrupt more than seven million people and damage two million buildings, resulting in loses approaching $30 billion (USGS, 2017).

Figure 1: A map showing known geological faults in the San Francisco Bay Region highlighting faults likely to cause an 6.7Mw event by 2045 (Source: USGS)

Figure 2: A map of San Francisco's Downton District illustrating the period in which the tall buildings were constructed (Source: CommericalCafe)

The post war era saw a boom in construction, particularly in tall buildings along the western coast of the United States between 1950 to 1990. This period saw rapid changes in seismic building codes as a result of research and damages observed in major seismic events, the most significant of which being the 1994 Northridge Earthquake. The 1994 Northridge Earthquake in Los Angeles revealed significant vulnerabilities in the design and construction of beam to column connections in steel moment resisting frames, which is the most prevalent type of lateral resisting structural system for buildings over 35 stories in San Francisco constructed between 1960-1990.

Aims and objectives

The seismic performance of existing tall buildings has recently been brought into question (Lat et al. 2017, Molina Hutt, 2017). This project aims to quantify the expected performance of a 1980's 50 storey perimeter steel moment resisting frame using advanced non-linear time history analysis.

Objectives:

- Develop a tall building archetype structure and numerical model to represent a tall building in San Francisco's existing building stock.
- Carry out non-linear time history analysis against three defined seismic intensity levels.
- Evaluate the performance of the archetype structure under the seismic intensity against industry best practice guidance.

Methodology

Figure 3: Analytical model used to design archetype to UBC 1985

Develop the representative archetype structure

- UBC 1985 perimeter steel moment resisting frame

Define non-linear modelling parameters

- Fracture prone beam connections
- Column axial-moment hinges
- Panel zone hinges

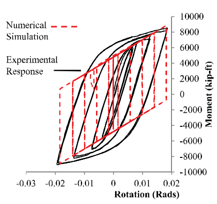

Figure 4: A comparison between analytical and experimental beam deformation of a W36x150 wide flanged beam under a defined loading protocol

Define seismic intensity level and ground motion selection

- Service Level (43 year return period)
- Design Level (475 year return period)
- Maximum Considered event (2475 year return period)

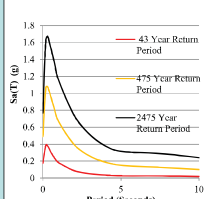

Figure 6: Target hazard response spectrum for the site of interest in San Fransico with 5% damping

Run non-linear time history analysis and evaluate results

- ETABS 2016 Analysis
- PEER Tall Building Intiative Guidelines

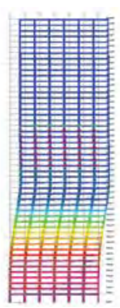

Figure 5: Residual displacement of the archetype after non-linear time history analysis

Archetype

The archetype is a 50 storey perimeter steel moment resisting frame designed using the UBC 1985 design provisions. The structure is rectangular in plan consisting of 6 bays of 28ft in each direction. The overall height of the structure is 632.5ft with a typical storey height of 12.5ft and ground storey of 20ft as is typical of tall building design in this period. Figure 8 (right) presents the lateral resisting elements as designed to UBC 1985

The numerical model for this performance assessment was carried out using ETABS 2016.2.0 which has the capability to capture geometric and material non-linearity of beams, columns and panel zones. Figure 7 presents the 3d model with a example of the non-linear hinge employed to model fracture of the pre-Northridge connections as per recommendations from ASCE 41.

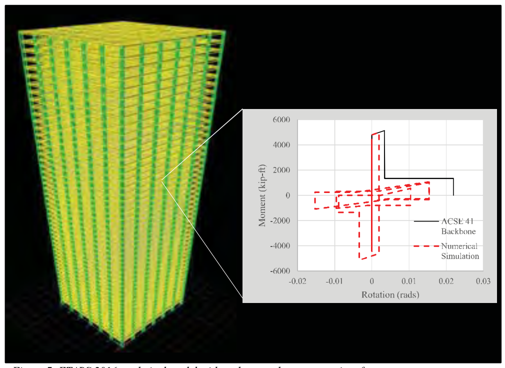

Figure 7: ETABS 2016 analytical model with an beam-column connection fracture response output of the pre-Northridge beam column connections as defined in ASCE 41

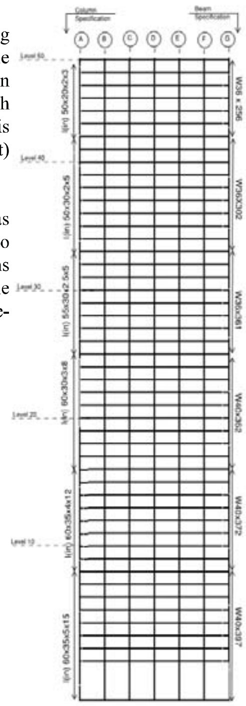

Figure 8: Elevation of the archetype structure presenting designed lateral resisting sections in accordance with UBC 1985

Results

The followings figure presents the peak storey global and component response deformations with the Percentage of Non-Collapsed (PNC). Global response is presented in as Inter Storey Drift (ISD), Peak Floor Acceleration (PFA) and Residual Storey Drift (RSD) deformations. Component behaviour is evaluated through beam and column hinge rotation deformation.

Service Level Intensity Results

100% PNC

Design Level Intensity Results

63% PNC

Maximum Considered Event Intensity Results

9% PNC

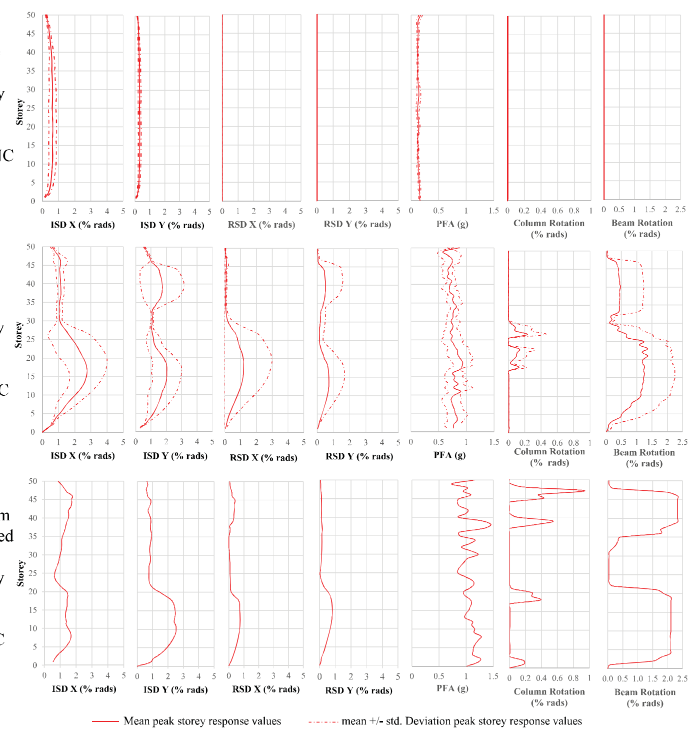

Figure 9: The peak story results of the non-linear dynamic analysis: Inter Storey Drift (ISD), Residual Sorey Drift (RSD), Peak Floor Acceleration (PFA), Column Hinge Rotation and Beam Hinge Rotation at the three seismic intensity levels

Conclusion

- The simulations suggest the 1985 archetype will not achieve life safety objectives. Failure is expected to occur with a soft storey mechanism at mid height of the structure.
- In cases of non-collapse, the residual deformations will likely result in demolition of the structure resulting in total losses for the building owner at a design level intensity or greater.
- Service level seismic events suggest the building will perform well, which may be why building owners have not had cause for concern with regards to their property performance.

References

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