Seismic assessment and retrofit recommendations for typical 1970s tall steel moment frame buildings in San Francisco

Carlos Molina Hutt

Thesis topic
Seismic assessment and retrofit of existing tall buildings in highly seismic regions

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Abstract

Most of the tall building stock in San Francisco was designed according to prescriptive guidelines that are now believed to be inadequate for tall building design. Furthermore, the lateral resisting system most commonly utilised in such buildings has shown significant vulnerability in past earthquake events. This document summarises the background and work conducted to date as part of a larger study, currently in progress, aimed at assessing seismic performance and providing retrofit recommendations where required for typical 1970s tall steel moment frame buildings in San Francisco.

Introduction

An inventory of the existing tall building stock in the San Francisco area has been carried out. Buildings were categorised per number of stories, type of lateral resisting system, location and year of construction. This survey revealed that most of the tall buildings were built in the 1970s and 1980s and adopted the steel Special Moment Resisting Frame (SMRF) structural system.

Most of the tall building stock in San Francisco was therefore designed to the provisions of the 1973 Uniform Building Code (UBC) of the time. Historic prescriptive code provisions are now believed not to provide the same level of performance as current procedures for tall building design in highly seismic regions. As a result, recommended alternatives have been published by researchers and practitioners within the earthquake engineering community for tall building design, such as Guidelines for Seismic Design of Tall Buildings published by the Pacific Earthquake Engineering Research Center (PEER) or Next Generation Performance Based Seismic Design Guidelines published by the United States Federal Emergency Management Agency (FEMA) among other publications.

Among other deficiencies, the 1973 UBC did not account for flexibility in the panel zones nor did it include provisions to ensure a 'strong column-weak beam' in the design of steel SMRFs. Furthermore, the 1994 Northridge earthquake in Southern California highlighted a serious vulnerability to brittle weld fracture in steel SMRF connections. Brittle weld fractures jeopardize the ductility of the steel SMRF system and consequently, the overall performance of buildings that adopted such system.

Methodology

In order to evaluate the impact of these issues, a 40-story steel SMRF building representative of the existing tall building stock has been designed per the code provisions and construction details of the time. The recommendations provided by the 1973 Structural Engineers Association of California (SEAOC) Blue Book were accounted for to supplement the design. The study was extended to also assess and compare the seismic performance of tall steel SMRFs designed to the current International Building Code (IBC). Table 1 illustrates the design of the representative 40 story building designed per UBC 73 and IBC 2006. Note that the reduction in section sizes from the IBC 2006 design to the UBC 73 design is a result from the lower strength demand requirements from the UBC 73 design with an overall design base shear of roughly 50% of that from IBC 2006.

The representative building models will be subjected to ground motions representative of the hazard levels currently recommended for the performance-based seismic design of such buildings. Non-linear response history analyses will be conducted in the finite element program, LS-DYNA. Prior to conducting the response history analyses in the overall building models, analytical component models for the column-beam joints were developed to capture fracture of the welds, flexibility of the panel zones, degradation of the plastic hinges, and buckling of the columns.

Results and discussion

Component models to represent the degradation of plastic hinges and the buckling of columns have been previously implemented in LS-DYNA and will be utilized accordingly in the analysis. The flexibility and nonlinearity of the panel zones was modeled per the recommendations of the PEER Report 2010/111 Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings. The analytical model used to represent the panel zone consists of an assembly of rigid links and rotational springs that represent the tri-linear shear force-deformation behavior of the panel zone. This analytical model captures the stable inelastic cyclic behavior as observed in experimental tests. A representation of the analytical model of the panel zone, its tri-linear shear force-deformation behaviour and inelastic cyclic behavior can be observed in Fig. 1.

Modeling of brittle fracture in beam-column connections is a complex issue as a result of the randomness and variability in the deformation capacity of connections. The approach followed in this study adopted the recommendations of Maison and Bonowitz, where fracture elements are introduced at the beam-column flexure connection. Fracture of connections was introduced as a random variable, which represents the rotation at which fracture takes place. Connections can achieve both ductile behavior and fracture, which can occur at top and bottom flanges. Rotational capacities are derived from a test database and surveys of damaged buildings following Northridge earthquake and can be represented as normal probability distributions. A representation of the fracture connection models for the top and bottom chords of a beam and their associated probability density functions is represented in Fig. 2.

IBC 2006 DESIGN						
Level	Beams			Columns		
Range	Exterior L=20'	Interior L=20'	Interior L=40'	Interior	Ext. Short EL.	Ext. Long EL.
Base to 10	W36x231	W36x231	W36x361	Box26x26x4.5	Box26x26x3.5	Box26x26x3
11 to 20	W33x201	W33x201	W36x282	Box26x26x3.75	Box26x26x2.5	Box26x26x2
21 to 30	W30x191	W30x191	W33x291	Box26x26x3.75	Box26x26x2.5	Box26x26x1.5
30 to Roof	W30x148	W30x148	W33x241	Box26x26x3.5	Box26x26x2	Box26x26x1.5

UBC 73 DESIGN						
Level	Beams			Columns		
Range	Exterior L=20'	Interior L=20'	Interior L=40'	Interior	Ext. Short EL.	Ext. Long EL.
Base to 10	W36x135	W36x135	W36x135	Box26x26x3.5	Box26x26x2.5	Box24x24x1.25
11 to 20	W33x118	W33x118	W36x135	Box26x26x3	Box26x26x2	Box24x24x1.25
21 to 30	W30x90	W30x90	W33x118	Box26x26x2	Box26x26x1.5	Box24x24x0.75
30 to Roof	W30x90	W30x90	W33x118	Box26x26x1.5	Box26x26x1	Box24x24x0.75

Table 1   Design Comparison of Representative Building to IBC 2006 and UBC 73

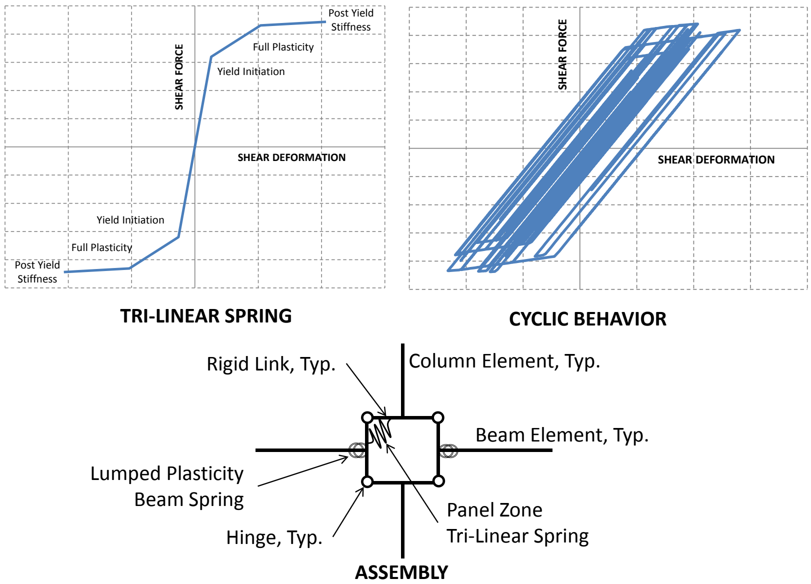

Fig 1   Panel Zone Analytical Model, Force-Deformation Relationship and Cyclic Response

Conclusion and next steps

Component models for the column-beam joints demonstrate the analytical capabilities to capture the deficiencies of steel SMRF systems designed per 1973 UBC. Once these component models are implemented in the overall buildings models the response history analyses will be carried out. The results of these analyses will identify the weaknesses in existing tall buildings with steel SMRF systems designed in the 1970s. Potential retrofit techniques to enhance building performance to current targets will then be assessed. Time has been allocated for the completion of this study and will be delivered by April 2012.

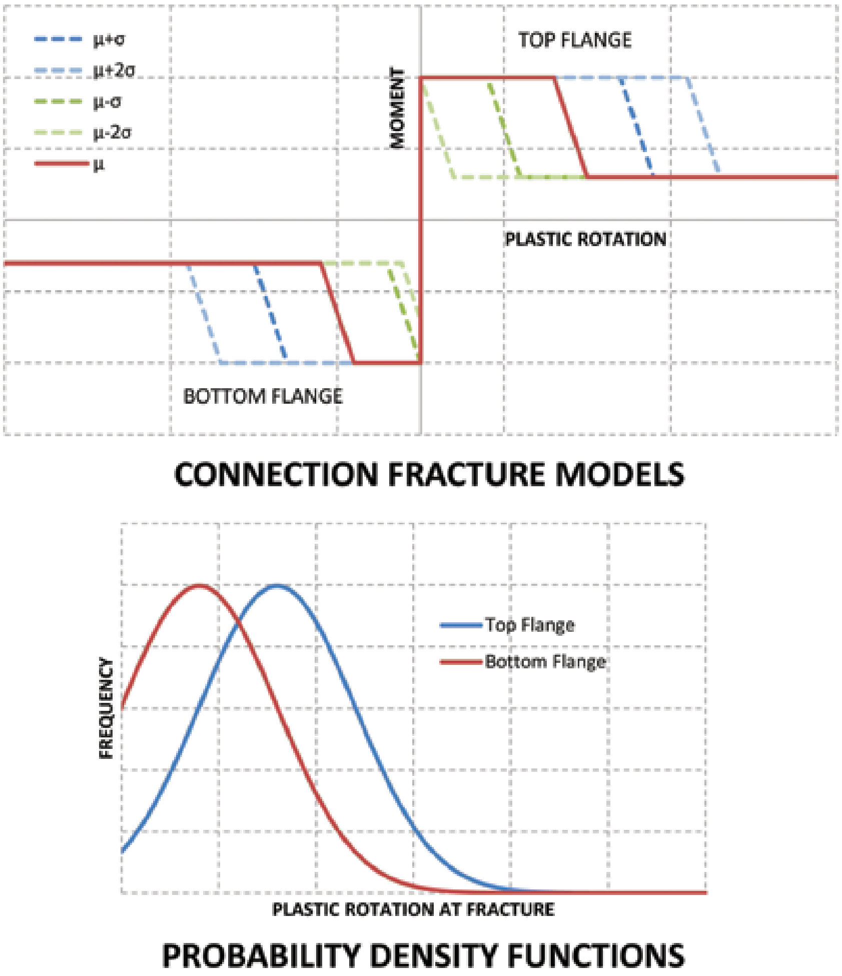

Fig 2 Connection Fracture Moment-Rotation Relation and Probability Density Functions

The evaluation of existing steel SMRF in tall buildings can be regarded as an initial iteration of the proposed thesis: Seismic Assessment and Retrofit of Existing Tall Buildings in Highly Seismic Regions. The next steps of this research study are to identify typical building systems utilised in the past for tall buildings. For those buildings, carry out the design of such systems per historic and current codes and evaluate their response to seismic excitation per non-linear time history analyses. An important aspect of this research will consist of assessing the sensitivity of results to the seismic hazard and modeling criteria.

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