Non-Linear Time History Analysis of Tall Steel Moment Frame Buildings in LS-DYNA

Carlos Molina Hutt

UCL

1 Summary

Non-linear time history analyses were carried out in LS-DYNA® (LSTC) in order to assess the seismic performance of existing tall steel moment resisting framed buildings. Ground motion earthquake records representative of the Maximum Considered Earthquake (MCE) hazard level defined in current building codes were used in the analysis. This paper focuses on the different component models utilized to capture the complex non-linear elements of the structure: beams, columns, panel zones, splices and moment connections. Both beam and column elements were modelled using the Belytschko-Schwer element formulation with lumped plasticity at both ends of the resultant beam. Columns elements captured interaction between bi-axial bending moment and axial force, buckling in compression and degradation parameters for response under cyclic loads calibrated to match experimental tests results. Beams elements captured implicit degradation in bending and random fracture at the connections. The random fracture was modelled such that plastic rotation at fracture occurred as a random variable characterized by a truncated normal distribution following results from experimental testing. Panel zones and column splices were modelled with discrete elements and general nonlinear translational and rotational springs. Panel zones were modelled using the Krawinkler model by means of an assembly of rigid links and rotational springs to capture the tri-linear shear force-deformation relationship of the joint. Column splices were modelled as non-linear springs capable of reaching their nominal capacity with a sudden brittle failure in axial tension and/or bending and full capacity in compression as observed in experiments. The paper briefly discusses the limitations of complex analytical models in trying to capture the non-linear dynamic response of structural systems and components.

2 Motivation

There are a large number of seismically vulnerable cities around the world due to their proximity to major active faults and their large number of older buildings. Until very recently, tall buildings were designed using only conventional building codes, which follow a prescriptive force-based approach based on the first mode translational response of the structure. Many researchers and engineers have raised concerns that the prescriptive approach of building codes is not suitable for tall buildings, which have significant responses in higher modes.

An inventory of the existing tall building stock in San Francisco revealed that most tall buildings in the city were built in the 1970s and 1980s and adopted a steel Special Moment Resisting Frame (SMRF) structural system. In order to assess the seismic performance of existing tall buildings in San Francisco, non-linear response history analyses of a representative 40-story building were carried out with ground motions representative of the Maximum Considered Earthquake (MCE) hazard level defined in current building codes. Under this level of shaking roughly 15% of the buildings are expected to be red-tagged and 70% are expected to sustain severe damage capable of causing loss of life. A small proportion of buildings may collapse.

This paper focuses on the properties of the archetype building and the analytical model developed in LS-DYNA (LSTC) to conduct the above mentioned study. For additional details on the inventory of the existing tall building stock in San Francisco, seismic hazard, ground motion selection and scaling, building performance predictions and conclusions please refer to the 15th World Conference of Earthquake Engineering paper titled *Seismic Assessment of Typical 1970s Tall Steel Moment Frame Buildings in Downtown San Francisco* presented in Lisbon on September 27, 2012.

3 Archetype Building

Based on an inventory of the existing tall building stock developed in downtown San Francisco, it was determined that the steel moment frame system is the most prevalent type in pre-1990s construction for buildings greater than 35 stories in height. Therefore, a 40-story steel SMRF was selected as a representative prototype building. The prototype building attempts to represent the state of design and construction practice from the mid-1970s to the mid-1980s. Based on examination of existing building drawings, the following use and layout was assumed for the prototype building: rectangular layout in plan; 38 levels of office space; 2 levels (one at mid-height and one at the top) dedicated to mechanical equipment; 3 basement levels for parking; building enclosure composed of concrete panels and glass windows; floor system composed of concrete slab (3 inches or 76.3 mm) over metal deck (2.5 inches or 63.5 mm) supported by steel beams; steel grade of columns A572 and steel grade of beams A36. Typical story heights are 10 feet (≈3 meters) for basement levels, 20 feet (≈6 meters) at ground level (lobby) and 12.5 feet (≈3.75 meters) for typical office levels. The overall height of the structure is 507.5 feet (≈153.75 meters) above ground and 30 feet (≈9 meters) below grade. The gravity loads, Superimposed Dead Load (SDL) and Live Load (LL), associated with the different spaces is summarized in Table 1 below.

Table 1. Loading Assumptions

Use	SDL		LL		Use	SDL		LL	
	(psf)	(kPa)	(psf)	(kPa)		(psf)	(kPa)	(psf)	(kPa)
Parking	15	0.7	52	2.5	Mechanical	135	6.5	56	2.7
Lobby	90	4.3	100	4.8	Roof	85	4.1	32	1.5
Office	40	1.9	56	2.7	Façade	41.5	2.0	-	-

The prototype building was designed to the provisions of the Uniform Building Code 1973 Edition (UBC 73) and the 1973 SEAOC Blue Book, which was commonly employed to supplement minimum design requirements. As illustrated in Figure 1, the prototype system consisted of a space frame with 20 to 40 feet spans (≈6 to 12 meters) using wide flange beams, built up box columns, and welded beam-column connections. Typical member sizes and connection details were verified against construction drawings of existing buildings.

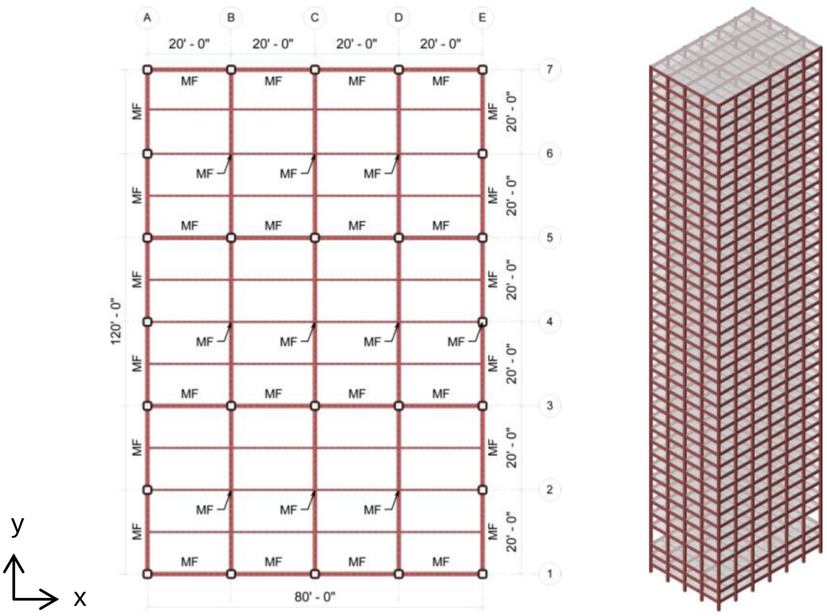


Figure 1. Prototype 40-Story Office Building

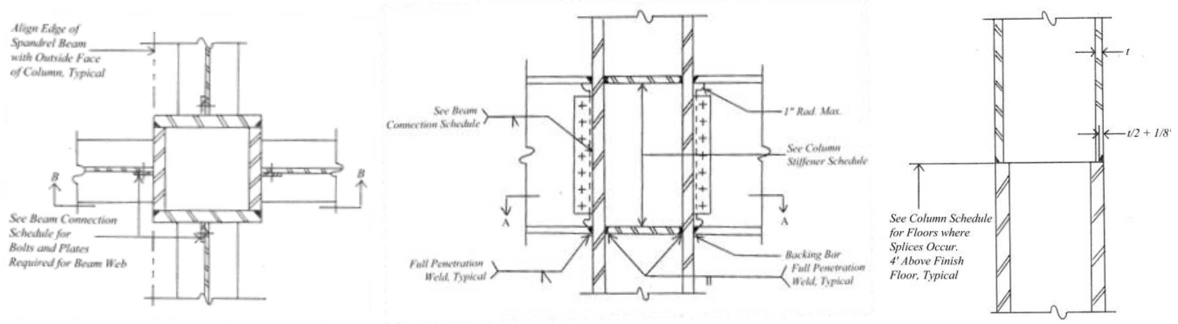
Per UBC 73, lateral wind forces generally govern over seismic for design of tall building. Per discussion with engineers practicing at this time, member sizes would have been sized for wind demand and detailed to provide a ductile response under seismic excitation. UBC 73 includes simple and concise prescriptive (equivalent static) strength design guidelines but does not specify drift limits. In the 1970s, design offices would have most likely implemented drift limits established by their firms practice or those obtained from the SEAOC Blue Book of the time. For this study, the drift limit recommendations from Appendix D of the SEAOC Blue Book are used, equal to 0.0025 for wind and 0.005 for seismic. The latter criterion is suggested for buildings taller than 13 stories. It is important to note that moment frame section sizes in the prototype building were governed by wind drift limits, resulting in low strength utilization ratios under code prescribed forces. Also worth noting is that such wind drift limits are similar to those currently used in the design of tall buildings.

Built-up box columns and wide flange beams were selected for the prototype building consistent with existing building drawings of this time. Table 2 below summarizes the column and beam section sizes used in the prototype building.

Table 2. Beam and Column Section Sizes per UBC 73 Design

Level Range	Wide Flange Beams			Box Columns		
	Exterior L=20'	Interior L=20'	Interior L=40'	Interior	Ext. Short EL. (x)	Ext. Long EL. (y)
Base to 10	W36x256	W36x282	W30x124	22x22x3.0x3.0	26x26x3.0x3.0	20x20x2.5x2.5
11 to 20	W33x169	W36x194	W27x84	20x20x2.0x2.0	26x26x2.5x2.5	20x20x2.0x2.0
21 to 30	W33x118	W33x169	W27x84	18x18x1.0x1.0	24x24x1.5x1.5	18x18x1.0x1.0
30 to Roof	W24x62	W27x84	W24x76	18x18x.75x.75	24x24x1.0x1.0	18x18x.75x.75

Typical details from drawings of existing buildings were reviewed to assess potential deficiencies. Figure 2 illustrates some typical connection details. The fracture prone pre-Northridge moment connections were very common, and the switch in the weld process that led to welds with very low toughness, as evidenced by fractures observed in the 1994 Northridge earthquake, took place in the mid 1960s (FEMA 352). Therefore, fracture prone connections are an anticipated deficiency in the prototype building.



a. Plan Section Typical Connection   b. Typical Moment Connection   C. Typical Splice

Figure 2. Typical Details Observed in Existing Building Drawings

It appears that the designs of the 1970's did not include consideration of panel zone flexibility or strong column-weak beam principles. Krawinkler's panel zone model was not developed until 1978 (ATC-72-1) and strong column weak beam requirements were not introduced in the UBC until 1988 (SAC/BD-00/25). However, considering the large column sizes required to satisfy drift requirements in tall moment frames, weak panel zones or flexural strength of columns are not believed to be critical from a strength point of view, yet required to accurately capture the stiffness of the structure.

Column splices were typically located 4 feet (≈1.2 meters) above the floor level approximately every three floors. Based on the typical splice connection details observed, if subject to tensile forces, these splices would only be able to carry half the capacity of the smallest section size being connected. Similarly, if subject to pure bending, these splices would have only been able to carry a fraction of the moment demand of the smallest column. Furthermore, experimental tests on heavy steel section welded splices had illustrated sudden failures with limited ductility (Bruneau and Mahin 1990). Based on this evidence column splice failures are considered as a significant factor in the assessment.

4 Analytical Model

This section outlines the modelling assumptions used in the non-linear response time history analyses in LS-DYNA (LSTC).

4.1 Component models

The component models to represent non-linear columns, beams, panel zones and splices are illustrated in
Figure *3* below. Concrete slabs were modelled as elastic cracked concrete 2D shell elements to represent the flexible floor diaphragm and are hidden in the close-up image of the component models in
Figure *3* for clarity.

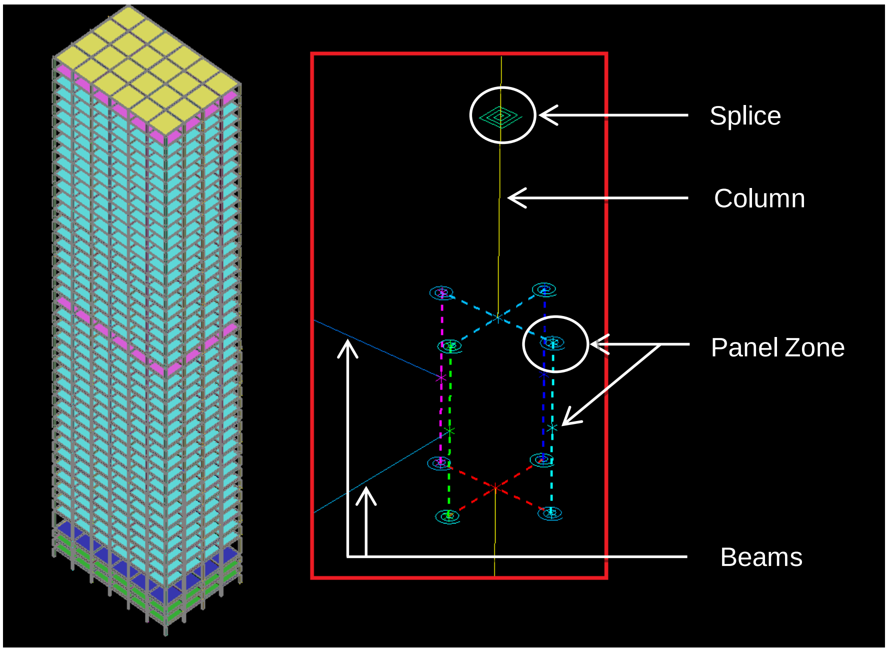


Figure 3: Isometric of Analytical Building Model and Close Up of Component Models (Boxed in Red)

4.1.1 Columns

Columns were modelled as lumped plasticity beam elements using the Belytschko-Schwer element formulation and material type 209 (MAT_HYSTERETIC), which enables yield surfaces capable of capturing interaction between bi-axial bending moment and axial force. Buckling in compression is also captured. Degradation parameters for response under cyclic loads were calibrated based on experimental tests of tubular steel columns (Nakashima et al. 2007) following the guidelines for tubular hollow steel columns under varying levels of axial load (Lignos and Krawinkler 2010). Figure 4 below illustrates the component deterioration calibration results for two column samples with an applied load ratio of 0.1 (left) and 0.3 (right) respectively.

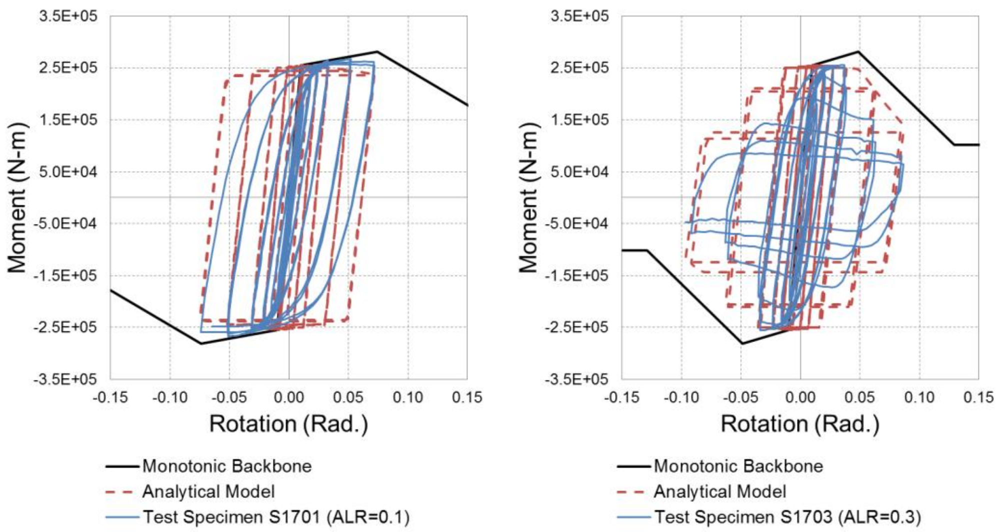


Figure 4. Calibration of Column Component Ceterioration under Varying Levels of Axial Load

Typical axial load to axial capacity utilization ratios were tracked though a nonlinear response history analysis for a small sample of columns. It was determined that an applied load ratio of 0.3 was a good representation for our prototype building design and the seismic intensity level under consideration.

4.1.2 Beams

Beams that form part of the moment frames were modelled as lumped plasticity elements with implicit degradation in bending to capture random fracture at the connections using the Belytschko-Schwer element formulation and material type 209 (MAT_HYSTERETIC). The random fracture model follows the methodology proposed by Maison and Bonowitz (1999), in which the plastic rotation at which fracture occurs is a random variable characterized by a truncated normal distribution following tests designed for typical pre-Northridge practice. Top and bottom capacities are modelled as a single random variable with a mean of 0.006 radians and a standard deviation of 0.004 radians. The truncated normal distribution and sample hysteretic behavior of beams with random fracture are shown in Figure 5.

The truncated tails at zero plastic rotation denote fracture prior to yield, which is supported by data from the SAC studies. In these cases, fracture is set to occur at 70% of the moment capacity of the beam. The residual moment capacity after fracture is set at 25% of the beam capacity.

For each analysis run, a different random fracture sample was obtained for each of the moment connections in the building model. Therefore, all analysis runs have a unique distribution of plastic rotation capacities throughout the structure. However, for any model, all samples of plastic rotation at fracture fit the distribution presented in Figure 5.

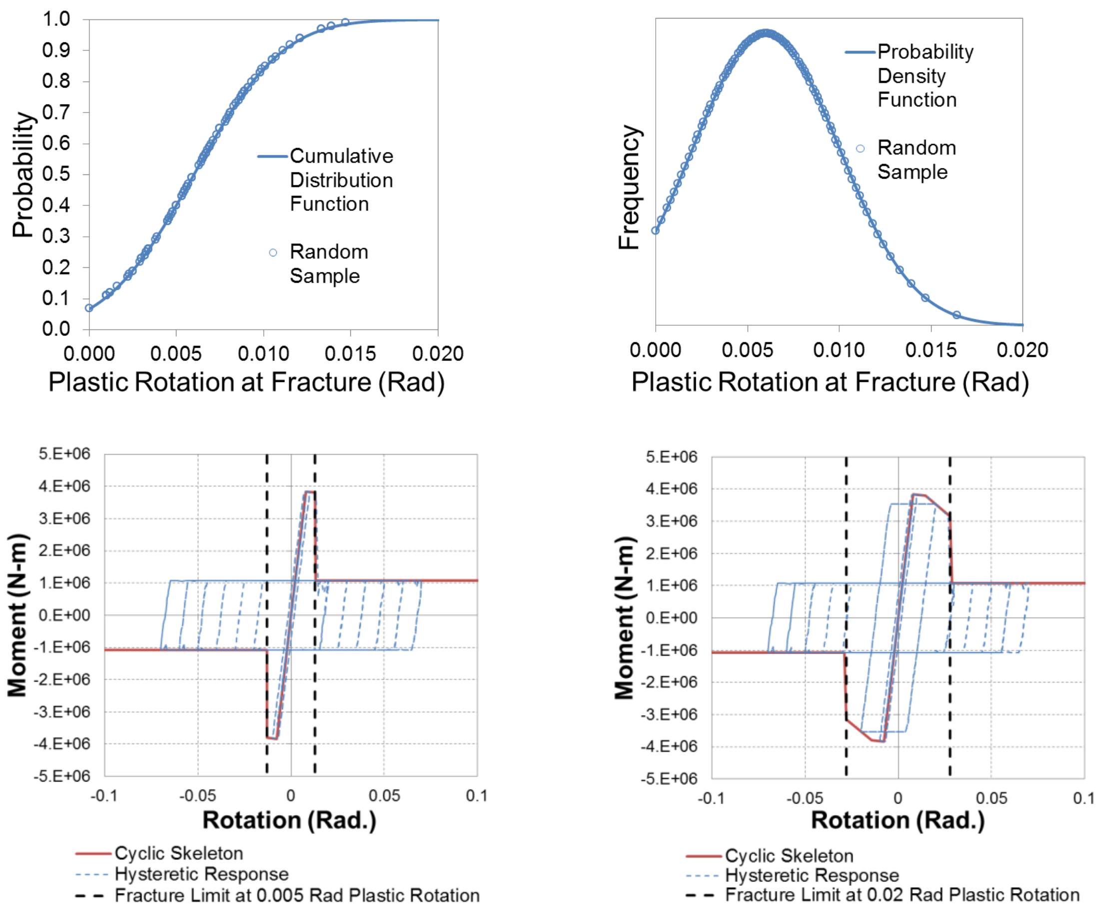


Figure 5. Probability Distribution for Random Fracture in Connections (Top Figures) and Sample Hysteretic Response for 0.005 Rad and 0.02 Rad of Plastic Rotation (Bottom Figures)

4.1.3 Panel zones

Panel zones were modelled using the Krawinkler model as outlined in ATC-72-1 by the use of an assembly of rigid links and rotational springs that capture the tri-linear shear force-deformation relation. Rigid links were achieved by means of CONSTRAINED_NODAL_RIGID_BODY formulations that defined two parallelograms, one for each axis of the column cross section. The geometry of the parallelogram illustrates the actual extents of the panel zone, while the force-deformation relation is calibrated by means of non-linear springs (SPRING_GENERAL_NONLINEAR) at the four corners of the parallelograms. Since the prototype building model is three dimensional and columns are built-up box sections, the shear force-deformation relationship in each direction was assumed decoupled.

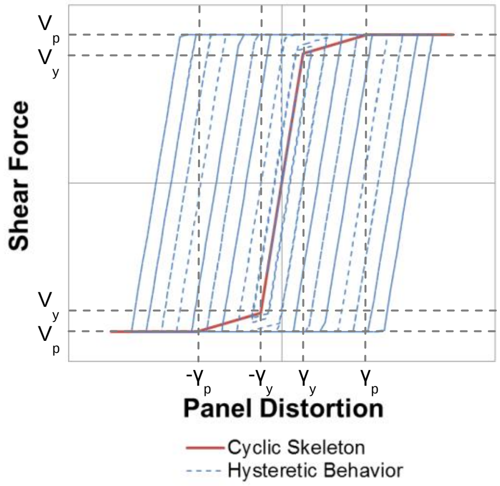


Figure 6. Hysteretic Response of Panel Zones

4.1.4 Column splices

Column splices were modelled as non-linear springs (SPRING_GENERAL_NONLINEAR) capable of reaching their nominal capacity with a sudden brittle failure followed by 20% residual capacity when subject to axial tension and/or bending. This enabled capturing brittle failure of the partial penetration welded splices, as shown in Figure 7 below, which were typically observed in existing building drawings. Full column capacity was assumed in compression since this is achieved by direct bearing.

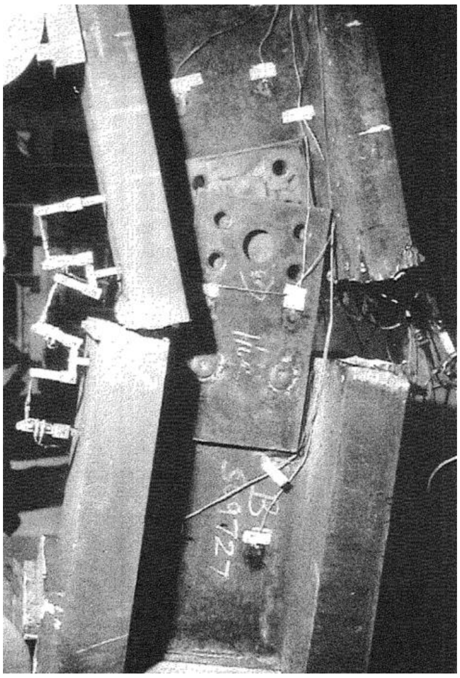


Figure 7: Failure of Partial-Penetration Welded Splice (Bruneau and Mahin 1990)

4.2 Loads, damping and boundary conditions

Analytical models were subject to the ground motions records in conjunction with expected gravity loads associated with the seismic weight of the structure. Seismic weight was assumed to include self-weight, superimposed dead load and 25% of the unreduced live loads (PEER 2010). Since the hazard level under consideration corresponds to that of the code MCE, 2.5% damping was assumed in the analysis (PEER 2010). DAMPING_FREQUENCY_RANGE_DEFORM, the damping model used in the analysis, applies damping to deformation excluding rigid body motion. The damping is adjusted based on tangent stiffness- which is believed appropriate for non-linear seismic analysis. A fixed base is assumed at foundation level and soil-structure interaction is not explicitly considered based on preliminary recommendations from the ATC-83 project, Improved Procedures for Characterizing and Modelling Soil-Structure Interaction for Performance-Based Seismic Engineering (Report Release Pending).

5 Limitations

It is important to acknowledge the limitations of complex analytical models. In the analytical model described in this paper there are a number of limitations that apply to some of the component models used to represent the non-linear behaviour of the structure. For instance, no distinction was made between the probability distribution of plastic rotation at fracture between top and bottom flange welds in moment connections, but rather a single joint distribution was assumed. Additionally, a representative axial load utilization ratio was assumed for the columns, based on tracked parameters throughout a sample non-linear time history analysis run, to establish degradation parameters for response under cyclic loading. Similarly, no moment-axial interaction effects were considered in the modelling of the column splices since the axial and flexural behaviour was represented by decoupled non-linear springs. Some of these limitations could have been easily avoided by slight enhancements of material models i.e. different moment-rotation behaviour in positive and negative bending (fracture) and automatic adjustment of degradation parameters as a function of axial load ratio (axial-bending degradation parameters) in MAT_HYSTERETIC_BEAM or enabling axial moment interaction between discrete non-linear springs in SPRING_GENERAL_NONLINEAR. Lastly, a simplified approach was taken to represent complex soil structure interaction effects. In this case, complex soil structure interaction effects could have been introduced in the model by explicitly modelling the foundation and relevant soil layers. However, based on preliminary recommendations from ATC-83 the increased level of complexity in the model does not outweigh additional accuracy of the results. Therefore, the proposed simplified approach can be regarded as acceptable for this specific study, which is intended to assess the deficiencies in the structure.

When using LS-DYNA for structural and seismic engineering applications, it is important to note that many of the guidelines developed by researchers and practitioners are most frequently intended for use in implicit analysis. Therefore, it is important to take into consideration the time step implications associated with strictly following recommended guidelines versus evaluating alternate modelling techniques that yield the same behaviour without compromising the analysis time step. In this particular study, the panel zone assembly developed following the Krawinkler model contained small elements with very large stiffness and very small stiffness. This had a direct impact in the time step of the analysis and required close consideration. Nevertheless, studies like these demonstrate the capabilities of utilizing an explicit analysis to solve structural and seismic engineering problems.

6 Summary

Robust non-linear component models can be used in LS-DYNA (LSTC) to accurately represent the complexities of the structural components of tall steel moment resisting framed buildings in order to assess seismic performance. LS-DYNA’s wide range of elements and library of material models enables the representation of complex components such as non-linear beams, columns, panel zones or fracture prone moment connections and splices. Nevertheless, it is important to acknowledge the limitations of complex analytical models and the trade-offs between increases in modelling complexity against increased accuracy of results.

7 References

Almufti, I., Molina Hutt, C., Willford, M., Deirelein, G., “Seismic Assessment of Typical 1970s Tall Steel Moment Frame Buildings in Downtown San Francisco”, In Press, 15th World Conference of Earthquake Engineering, 2012.

Bruneau M., Mahin, S. (1990). Ultimate Behavior of Heavy Steel Section Welded Splices and Design Implications. *Journal of Structural Engineering*, Vol. 116, No. 8.

FEMA. (2000). Program to Reduce the Earhquake Hazards of Steel Moment-Frame Structures: Recommended Postearthquake Evaluation and Repair Criteria for Welded Steel Moment-Frame Buildings. FEMA 352.

International Conference of Building Officials. (1973). Uniform Building Code 1973 Edition.

Lignos D. G., Krawinkler H. (2010). A Steel Database for Component Deteriorration of Tubular Hollow Square Steel Columns Under Varying Axial Load for Collapse Assessment of Steel Structures under Earthquakes. Joint Conference Proceedings 7CUEE & 5ICEE.

LS-DYNA. (2012). Keyword User’s Manual, Volume I, August 2012, Version 971 R6.1.0, Livermore Software Technology Corporation (LSTC).

LS-DYNA. (2012). Keyword User’s Manual, Volume II, Material Models, August 2012, Version 971 R6.1.0, Livermore Software Technology Corporation (LSTC).

Maison F. M., Bonowitz D. (1999). How Safe Are Pre-Northridge WSMFs? A case Study of the SAC Los Angeles Nine-Story Building. *Earthquake Spectra,* Vol. 15, No. 4.

Nakashima M. et al. (2007). Test on Large Cyclic Deformation of Steel Tube Columns Having Fixed Column Bases. *Journal of Structural and Construction Engineering,* Vol. 598, pages 149-154.

OASYS CEAP. CEAP Extra Keywords. MAT_HYSTERETIC_BEAM.

PEER. (2010). Tall Buildings Initiative: Guidelines for Performance-Based Seismic Design of Tall Buildings. PEER Report No. 2010/05.

PEER/ATC. (2010). Modelling and Acceptance Criteria for Seismic Design and Analysis of Tall Buildings. PEER/ATC-72-1.

SAC Joint Venture. (2000). Performance Prediction and Evaluation of Steel Special Moment Frames for Seismic Loads. SAC Steel Project Report No. SAC/BD-00/25.

SEAOC Seismology Committe. (1973) Recommended Lateral Force Requirements and Commentary.