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A COMPARATIVE STUDY ON THE SEISMIC VULNERABILITY OF 1970s vs MODERN TALL STEEL MOMENT-RESISTING FRAME BUILDINGS

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Abstract

This paper outlines the methodology followed in the risk-based assessment of two archetype tall buildings in downtown San Francisco: a 50 story steel moment resisting frame (MRF) office building designed following the requirements of the Uniform Building Code of 1973 and a 50 story steel MRF office building designed following modern code requirements (International Building Code 2012). The methodology enables the development of the vulnerability function for the archetype buildings under consideration, highlighting loss contribution from (1) collapse, (2) irreparable damage from excessive residual deformations and (3) reparable damage. The goal of this study is to benchmark the performance of older existing steel MRF buildings against modern designs, providing an overall comparison of their seismic vulnerabilities.

The results illustrate that existing tall steel MRF buildings from the 1970s are drastically more vulnerable to earthquakes than tall steel MRF buildings designed to modern standards. The vulnerability function for the 1970s archetype building highlights that collapse potential is the highest contributor to the losses, with a collapse fragility characterized by a relatively low median spectral acceleration. The resulting vulnerability function of the modern archetype building indicates that: i) at low ground motion intensities of shaking, losses are influenced by repairable damage; ii) at medium intensities of shaking losses are equally dominated by repairable damage and residual drift rendering the building irreparable; iii) collapse only starts contributing to the loss at large spectral amplitudes, but even then losses are largely dominated by residual drifts. The collapse fragility of the modern archetype building is in agreement with the design objective of modern building codes, which is to produce designs with low probability of collapse under a Maximum Considered Earthquake.

Keywords: Vulnerability Functions; Tall Buildings; Moment-Resisting Frames; Steel.

1. Introduction

Tall buildings play an important role in the socio-economic activity of major metropolitan areas. The resilience of these structures is critical to ensure a successful recovery after major disasters. Until the introduction of Performance Based Seismic Design (PBSD) in the 1990s, tall buildings were designed using conventional building code guidelines [1] which do not provide an explicit understanding of performance during major earthquakes. Researchers and engineers have raised concerns that the prescriptive approach of building codes is not suitable for tall building design due to the significant contribution of higher mode effects [2]. As a result of these shortcomings, several jurisdictions in areas of high seismicity throughout the Unites States (e.g. Los Angeles and San Francisco) have adopted a PBSD approach for the design of new tall buildings. While new designs follow a more adequate approach, little is known about the seismic performance of older existing tall buildings that were designed prior to the adoption of PBSD [3].

This study presents the results of a risk-based seismic performance assessment of two archetype tall buildings in a case study city, San Francisco, CA:

- 1973 Archetype: a 50 story steel moment resisting frame (MRF) office building designed following the requirements of the Uniform Building Code of 1973 [4]
- 2012 Archetype: a 50 story steel MRF office building design following modern code requirements (International Building Code 2012) [5]

The 1973 archetype is developed based on an inventory of existing tall buildings in San Francisco as representative of design and construction practice from the mid-1970s to the mid-1980s (prior to the introduction of PBSD). The 2012 archetype is a building of the same geometry and occupancy as the 1973 archetype, but designed following modern building code requirements. The 2012 archetype building, while following modern building code requirements, does not follow a PBSD approach, which would be expected to provide performance beyond that of the code. The objective of the assessment is to produce a vulnerability function for the archetype buildings under consideration. To that end, a Multiple Stripe Analysis (MSA) is conducted at 8 different intensity levels of ground motion shaking ranging from frequent to very rare seismic events, i.e. return period events from 25 to 5000 years. Non-Linear Response History Analyses (NLRHA) are conducted with ground motions representative of each intensity level considered. The results of the NLRHA results are used to assess the probability of earthquake losses, considering collapse potential and the probability of the buildings deemed irreparable due to permanent residual drifts in the structure.

2. Methodology

A risk-based assessment consists of the evaluation of a number of intensity-based performance assessments under a range of ground motion intensity levels which are then combined with the ground motion hazard curve to provide the annual rates of exceedance of a performance measure, e.g. losses [6]. The technical basis of the methodology followed to conduct the risk-based seismic performance assessment here presented is that developed by the Pacific Earthquake Engineering Research (PEER) Centre, which applies the total probability theorem to predict earthquake consequences in terms of the probability of incurring a particular value of a performance measure [7]. Under this framework, performance is computed by integrating: the probability of incurring an earthquake of different intensities over all possible intensities; the probability of incurring a certain building response (drift, acceleration, etc.) given an intensity of ground shaking; and the probability of incurring certain damage and consequences given a value of building response [8].

The implementation of such methodology to assess the performance of tall steel-framed archetype buildings in our case study city of San Francisco can be broken into the steps outlined in Fig. 1. This study is focused on the development of vulnerability functions as opposed to performance (loss functions), both direct outputs of the methodology here presented.

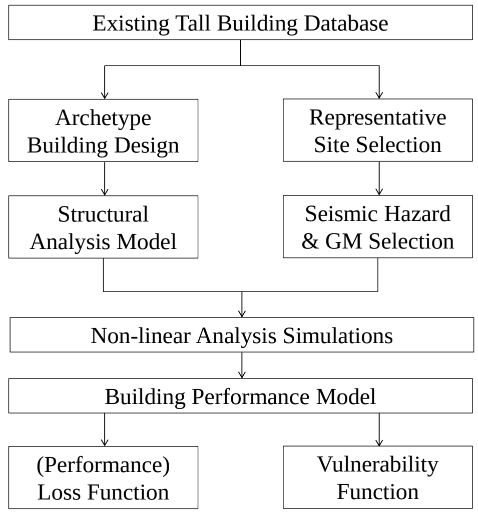


Fig. 1 – Methodology

2.1 Archetype Buildings and Representative Site Selection

The 1973 archetype building is developed based on a database of the existing tall buildings stock in San Francisco, CA. [3] provides a detailed review of the existing tall building database in the case study city, which reveals that the steel MRF system is the most prevalent type in pre-1990s construction for buildings greater than 35 stories in height. A 50-story steel MRF office building designed per the 1973 Uniform Building Code is selected as one of the archetype buildings for this study. The building is regular in plan and represents the state of design and construction practice from the mid-1970s to the mid-1980s. The occupancy of the building is that of a commercial office with two levels for mechanical equipment, one at mid-height, and one at the top floor. The building enclosure is assumed to be composed of precast concrete panels and glass windows, floor system composed of concrete slab (76.2 mm or 3 in.) over metal deck (63.5 mm or 2.5 in.) supported by steel beams of ASTM A36 (248 MPa or 36 ksi), and steel columns of ASTM A572 (345 MPa or 50 ksi). The lateral resisting system of the building is a space MRF composed of wide flange beams, built up box columns, and welded beam–column connections. Typical story heights are 3.8 m (12.5 ft), except at the lobby (6.1 m or 20 ft). The overall height of the structure is 192.8 m (632.5 ft) above ground. The building width is 51.2 m (168 ft), consisting of 6 bays of 8.5 m (28 ft) in each direction.

In order to benchmark the performance of the 50-story archetype building representative of 1970s construction against current standards, an additional archetype is developed for a building of equal dimensions and occupancy, but designed per the 2012 International Building Code. The lateral resisting system is also a steel MRF with a perimeter frame as opposed to a space frame. Fig. 2 illustrates the lateral resisting system for the 1970s archetype versus the modern archetype building. The 1973 archetype consists of 7 frames in each direction, whereas the 2012 archetype consists of only 2 in each direction (perimeter frame).

The design of the 1973 archetype is in accordance with the provisions of the Uniform Building Code (UBC) 1973 and SEAOC Bluebook of 1973 [9], which was commonly employed to supplement minimum design requirements. Lateral wind forces generally governed the design of tall buildings over seismic forces per UBC 1973, and member sizes would have been sized for wind demand and detailed to provide a ductile response under seismic excitation. While UBC 1973 does not specify drift limits, design offices would have implemented drift limits established by their firm’s practice or those obtained from the Bluebook. In this paper, the drift limit recommendations from Appendix D of the Bluebook for buildings taller than 13 stories are used, equal to 0.0025 for wind and 0.005 for seismic. Current seismic drift limits are slightly more stringent; 0.020 times the story height, which for a deflection amplification factor of 5.5 as prescribed for special steel MRF, is approximately 0.004 [10].

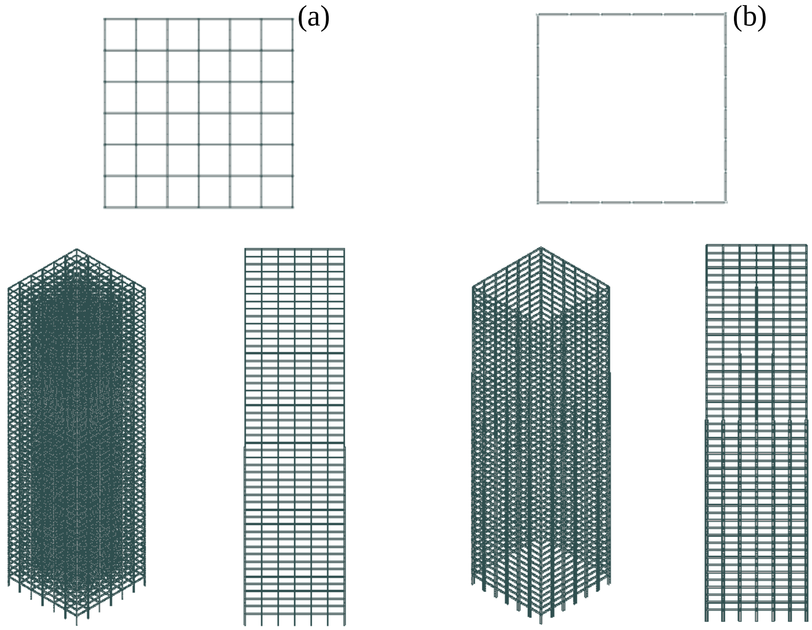


Fig. 2 – Lateral resisting system of archetype buildings: steel MRF representative of 1970s design practice (a) and modern design practice (b).

Even through these seismic drift limits do not appear to be drastically different between UBC 73 and modern standards, it is important to note that the design forces are significantly larger in current standards than they were back in the 70s. The effective wind base shear under the forces prescribed by UBC 1973 is 1.84%, whereas the overall effective seismic base shear is 1.96%. The effective wind base shear with the forces prescribed by IBC 2012 is 4.26%, whereas the overall effective seismic base shear is 3.74%. It is also important to note that the IBC 2012 design base shear is controlled by minimum base shear requirements, which were not existence in the 1970s design regulations. Furthermore, there are a number of additional important considerations in modern design standards that were not present in designs of the 1970s and which can result in drastically improved seismic performance:

-response spectrum analysis method as opposed to equivalent lateral force procedure

-consideration of lateral forces acting simultaneously in both building directions

-consideration of accidental torsion

-minimum base shear requirements (scaling of forces and displacements)

-p-delta effects (scaling of forces and displacements)

-consideration of vertical and horizontal irregularities

-strong column weak beam consideration

-panel zone consideration

-capacity design principles

-prequalified seismic connection details

Typical member sizes and connection details for the 1970s archetype building were verified against available existing building drawings. Consistent with these records, built-up box columns and wide flange beams are selected for the prototype building. For the modern building design, built-up I sections are selected for the columns and wide flange sections are selected for the beams, both of the same steel grade specification as the 1970s design.

[3] illustrates some of the typical details frequently observed in existing building drawings. Since 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 [11], it is assumed that that fracture-prone pre-Northridge moment connections are common. Designs of the 1970s did not include consideration of panel zone flexibility or strong column-weak beam principles. The panel zone model proposed by Krawinkler was not developed until 1978 [12] and strong column-weak beam requirements were not introduced in the UBC provisions until 1988 [13]. Column splices are typically located 1.2 m (4 ft) above the floor level approximately every three floors. Observed typical splice connection details consist of partial joint penetration welds of roughly half the thickness of the smaller section being connected. When subject to tensile forces, these splices can only carry a fraction of the moment capacity and/or axial tension capacity of the smallest section size being connected. Furthermore, experimental tests on heavy steel section welded splices have illustrated sudden failures with limited ductility [14]. Based on this evidence, column splice failures are considered in the assessment.

For the modern building design, pre-qualified Reduced Beam Section (RBS) moment connection details are assumed in the design. Typical RBS connections are illustrated in [15]. Current design standards also require that, in steel Special MRF, columns splices are capable of developing the full capacity of the smallest section being connected. Therefore, failure of column splices is not considered in the assessment.

The overall seismic weight of the 1973 archetype design is 784,220 kN (176,300 kips), whereas the seismic weight of the modern design is 825,145 kN (185,500 kips). The 5% discrepancy in seismic weight between the two archetypes is a reflection of the differences in the steel self-weight in each design. The dynamic properties of the archetype building models are summarized in Fig. 3 including periods, mode shapes and effective over total mass for the first 4 modes.

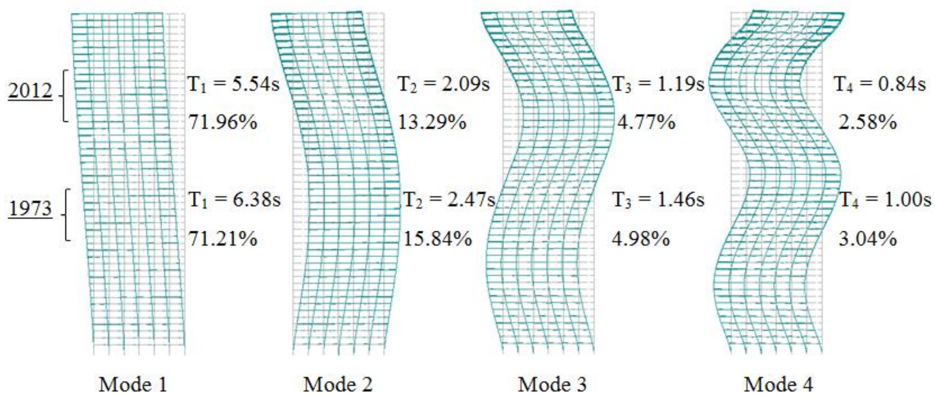


Fig. 3 – Dynamic properties of the archetype buildings.

The majority of tall buildings in San Francisco are clustered in the downtown area, located approximately 14 km from the San Andreas Fault and 16 km from the Hayward Fault. A representative site is selected in close proximity to most of the existing tall buildings in downtown San Francisco with soil properties consistent with ASCE 7-10 Site Class D [12].

2.2 Structural Analysis Modeling

NLRHA are conducted in LS-DYNA [16]. A sample 2D model and its components are illustrated in Fig. 4. Columns are modeled as lumped plasticity beam elements with yield surfaces capable of capturing interaction between bending moment and axial force. For the 1970s archetype building, degradation parameters for response under cyclic loads are calibrated based on experimental tests of tubular steel columns [17] in accordance with the guidelines for tubular hollow steel columns under varying levels of axial load [18]. For the modern archetype building, degradation parameters for response under cyclic loads are assumed to be equivalent to those outlined in ATC-72-1 [12] for steel beams due to the low axial demands in the columns under expected gravity loads, which are consistently below 20% as opposed to the 1970s design, where the applied load ratio under expected gravity loads ranges from 20 to 40%.

Beams are modeled as lumped plasticity beam elements. For the 1970s archetype building, beams capture fracture at the connections through a moment-rotation backbone curve with implicit degradation parameters. The fracture model, consistent with pre-Northridge moment connections, is developed based on ASCE 41 [19] recommendations. For the modern archetype building, beams follow the modelling parameters recommended in ATC 72-1 [12] for RBS connections, which are based on a large database of experimental tests. Panel zones are modeled using the Krawinkler model as outlined in [12] by the use of an assembly of rigid links and rotational springs that capture the trilinear shear force-deformation relation.

For the 1970s archetype building, column splices are modeled as nonlinear springs capable of reaching their nominal capacity with a sudden brittle failure and then 20% residual capacity when subject to axial tension and/or bending. Modeling of brittle failure is intended to capture the limited ductility observed in experimental tests on heavy steel section welded splices as observed by [14]. Full column capacity is assumed in compression since this is achieved by direct bearing. For the modern archetype building, splices can develop the full capacity of the smallest section being connected.

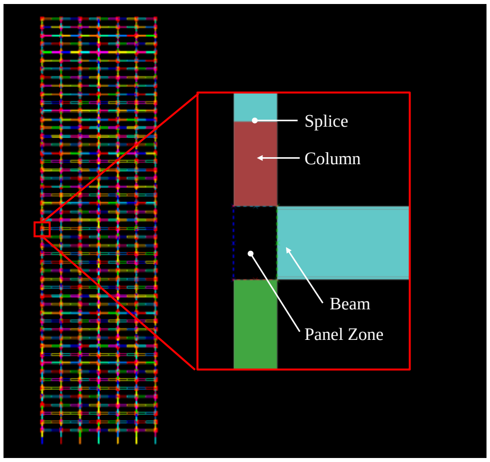


Fig. 4 – Elevation view of analytical model and close-up of component models (boxed in red).

Analytical models are subject to ground motions in conjunction with expected gravity loads associated with the seismic weight of the structure. Seismic weight includes self-weight, superimposed dead load, and 25% of the unreduced live loads. For the archetype building design, the seismic weight is approximately that of the corresponding tributary area of the frame (space frame). However, for the modern building design (perimeter frame), the seismic weight corresponding to a representative frame is greater than the tributary area of the frame. Therefore, a leaning column is modeled to support the corresponding seismic weight of the frame and include relevant p-delta effects. A value of 2.5% damping is assumed in the analysis [2]. A fixed base is assumed at ground level and soil-structure interaction is not considered.

2.3 Seismic Hazard and Ground Motion Selection

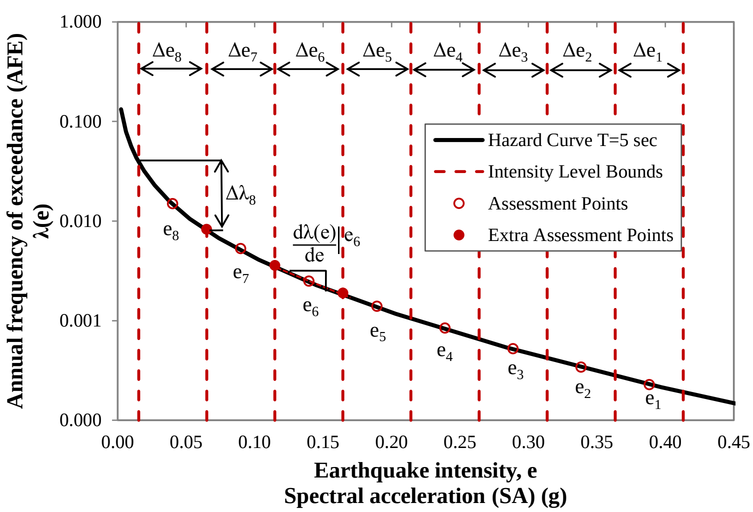
Seismic hazard data is obtained from Probabibilistic Seismic Hazard Analysis (PSHA) results at the site of interest using the USGS hazard curve calculation tool [20]. In order to perform structural analysis, a series of ground motion intensities spanning from low to high probability of occurrence are selected. The minimum and maximum annual frequencies of exceedance (AFE) and corresponding spectral accelerations (SA) are:

Minimum: AFE = 0.04 and corresponding SA_{MIN}

Maximum: AFE = 0.0002 and corresponding SA_{MAX}

The upper and lower bound intensity levels are considered to cover a range from negligible damage to complete loss. These bounds are obtained from the seismic hazard curve at a period of 5 seconds, as shown in Fig. 5. A 5 second period is selected as it is in close proximity to fundamental period of the archetype buildings considered and the longest period for which USGS provides seismic hazard data.

The lower bound corresponds to a ground motion intensity level that does not result in significant damage to structural or non-structural components whereas the upper bound corresponds to a ground motion intensity beyond the level that triggers collapse. Once the bounds of spectral accelerations SA_{MIN} to SA_{MAX} are determined, the range is split into a number of equal intervals for assessment. Based on the recommendation of [8], 8 intensity level intervals are selected to capture a wide range of responses. The midpoint SA of each one of these intervals is then computed and its corresponding AFE. This process is graphically illustrated in Fig. 5, where the earthquake ground motion intensity intervals and the assessment points are denoted by Δe_i and e_i respectively. The AFE and SA associated with each earthquake ground motion intensity level considered in the assessment are summarized in table adjacent to Fig. 5. Note that additional assessment points are considered in the assessment of the 1970s archetype building due to the high probability of collapse associated with intensities of ground motion shaking e_1 to e_4.



e_i	SA (g)	AFE (-)	Return Period (years)
e_1	0.3882	0.0002	4162
e_2	0.3385	0.0004	2631
e_3	0.2888	0.0005	1913
e_4	0.2391	0.0009	1124
e_5	0.1893	0.0014	714
e_{5-6}*	0.1645	0.0019	526
e_6	0.1396	0.0025	400
e_{6-7}*	0.1148	0.0036	278
e_7	0.0899	0.0053	189
e_{7-8}*	0.0650	0.0083	120
e_8	0.0402	0.0150	67

Extra assessment points.

Fig. 5 – Seismic hazard curve at representative site in downtown San Francisco (V_{S30}=260m/s, T=5sec.) illustrating the earthquake ground motion intensities (e_i) considered in the risk-based assessment.

A Conditional Spectrum (CS), conditioned at a 5 second period, is selected as the target spectrum for each of the intensity levels considered in the assessment. USGS deaggregation data at each intensity of ground motion shaking is used to construct the target conditional mean spectrum and variance as outlined in [21]. The target conditional spectrum mean and variance as well as the ground motion records obtained for assessment are checked against the corresponding Uniform Hazard Spectrum (UHS) obtained from the USGS hazard data to ensure the spectral acceleration of the mean, variance, ground motion record spectra and UHS are coincident at the conditioning period. The effect of ground motion pulses and soil structure interaction are not explicitly considered in the analyses. The selected ground motions are input at the base of the structural model, which is assumed to have a fixed support at its base.

2.4 Building Performance Modeling

Communicating performance as the probable consequences in terms of direct economic losses to repair earthquake damage is the metric used in this study to communicate performance, where the costs are expressed in present dollars. Losses are expressed as a percentage of repair cost, i.e., the cost required to restore a building to its pre-earthquake condition, over total building cost, i.e., the cost required to rebuild with a new structure of similar construction. In this report, total replacement cost includes replacement of basic building structure, exterior enclosure; mechanical, electrical, and plumbing (MEP) infrastructure; as well as all tenant improvements and contents. Demolition and site clearance are not included in the total replacement cost since the intent is to estimate the direct losses. Based on a Class 5 rough order-of-magnitude cost estimate based on the Association for the Advancement of Cost Engineering (AACE), the most likely estimated cost for the archetype building in San Francisco in present dollars is \$3,550=m^2 (\$330=ft^2) with an accuracy range of -5 to +30%.

Engineering demand parameters (EDPs), including maximum story drift ratios and peak floor accelerations are obtained from the NLRHA at every story in the building under consideration. Fig. 6 illustrates sample input demand parameters for the archetype buildings for a sample earthquake ground motion intensity, e_8, with an AFE of 0.0150. These parameters are used as input demands to the building performance model, which contains structural and non-structural components at each story level for all components in the building that are susceptible to earthquake damage. The building performance model for this study is developed in SP3 (Seismic Performance Prediction Program) [22], which follows the same loss calculation methodology of FEMA's Performance Assessment Calculation Tool or PACT [8].

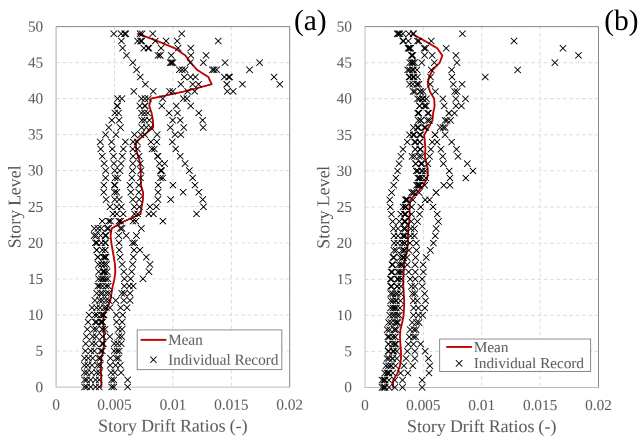


Fig. 6 – EDPs used as inputs to the building performance model: story transient drift ratios for the 1970s (a) and modern (b) building archetypes for sample intensity level e_8 (AFE = 0.015).

Structural component quantities are based on the structural design of the archetype buildings. Non-structural component quantities are estimated based on typical quantities found in buildings of similar occupancy by use of the Normative Quantity Estimation Tool [8]. Normative quantities are an estimate of the quantity of components and contents likely to be present in a building of a specific occupancy based on gross square footage. These quantities were developed based on a detailed analysis of approximately 3,000 buildings across typical occupancies [8]. This study assumes estimates of quantities at the 50th percentile level. Each one of these structural and non-structural building components has a component fragility function. A component fragility function is a statistical distribution that indicates the conditional probability of incurring damage at a given value of demand, which is typically assumed to be lognormal distribution.

All non-structural components included in the building performance model of the 1973 archetype, can be found in [3], including fragility number, description, quantity, units and demand parameter. Component quantities are identical between the 1970s and modern building archetypes. However, the 1970s components are more susceptible to earthquake damage than modern building components. This discrepancy is intended to capture the lack of seismic design considerations for non-structural building components in the 1970s. Component fragility functions contain unique fragilities for each possible damage state in the component. Each damage state has an associated consequence function, from which the repair cost and repair time associated with the level of damage in the component is estimated. The occurrence of damage states is predicted by individual demand parameters, as determined from the NLRHA. For each realization, fragility functions are used in conjunction with demand parameters to determine a damage state for each component. Consequence functions are then used to translate damage states into repair or replacement costs [8]. The direct economic losses for each realization are estimated by conducting this calculation for every component at every story throughout the building, for intensities of shaking in which collapse doesn't occur and the building is not deemed irreparable due to the presence of large residual drifts.

Residual drifts are an important consideration when estimating losses. Typical building repair fragility as a function of residual drifts is a lognormal distribution with a median value of 1% residual drift ratio and a dispersion of 0.3 [8]. Residual drifts predicted by nonlinear analysis are highly sensitive to component modeling assumptions. Accurate statistical simulation of residual drift requires the use of advanced component models, careful attention to cyclic hysteretic response, and a large number of ground motion pairs. Therefore, residual drifts are estimated as a function of peak transient response of the structure and the median story drift ratio calculated at yield based on FEMA recommendations [8]. For each realization, the PACT analysis uses the maximum residual story drift together with the building repair fragility to determine if the building is deemed irreparable. If irreparable, repair cost and repair time are taken as the building replacement values.

2.5 Vulnerability Functions

The risk-based assessment conducted in this study enables the development of a seismic a vulnerability function for the archetype buildings highlighting loss contribution from (1) collapse, (2) irreparable damage from excessive residual deformations and (3) reparable damage. The vulnerability function provides the damage ratio, total loss over total building cost, versus spectral acceleration at the conditioning period, selected for this study at 5 seconds as it is in close proximity to the fundamental period of the structures of interest. These functions enable quick estimation of losses for a given spectral amplitude at a 5 second period. The probability of exceeding a certain value of loss at a given ground motion intensity of shaking can be denoted as P(L>x|E=e) and has the following key components:

$$P(L>x|C) \cdot P(C|E=e) \tag{1}$$
$$P(L>x|NC, R) \cdot P(NC|E=e) \cdot P(R|NC, E=e) \tag{2}$$
$$P(L> x|NC, NR) \cdot P(NC|E=e) \cdot P(NR|NC, E=e) \tag{3}$$

Eq. (1) , $P(L>x|C) \cdot P(C|E=e)$, denotes the probability of observing a value of loss greater than x, given that collapse has occurred, multiplied by the probability of observing collapse at a given intensity level. Eq. (2), $P(L>x|NC, R) \cdot P(NC|E=e) \cdot P(R|NC, E=e)$, denotes the probability of observing a value of loss greater than x, given that no collapse has occurred and residual drifts deem the building irreparable, multiplied by the probability of observing no collapse at a given intensity level, multiplied by the probability of residual drifts rendering the building irreparable given no collapse has occurred. Lastly, Eq. (3), $P(L> x|NC, NR) \cdot P(NC|E=e) \cdot P(NR|NC, E=e)$, denotes the probability of observing a value of loss greater than x, given that no collapse has occurred and residual drifts do not deem the building irreparable, multiplied by the probability of observing no collapse at a given intensity level and the probability of residual drifts not rendering the building irreparable given no collapse has occurred.

This study followed a Multiple Strip Analysis (MSA) approach in which NLRHA are performed at 8 initial intensity levels and where different ground motions are used at each intensity level under consideration. At the higher earthquake ground motion intensities, the fraction of ground motions that cause structural collapse are recorded and used to obtain the collapse fragility for the building. Additional ground motion intensities are considered for the 1970s archetype building due to the high probability of collapse at half of the intensity levels originally considered in the assessment. The statistical fitting technique for this data follows the method of maximum likelihood as described in [23]. The resulting collapse fragility of the 1970s archetype building has an estimated median of 0.11g and a dispersion of 0.38 as illustrated in Fig. 7a. The resulting collapse fragility of the modern archetype building has an estimated median of 0.41g and a dispersion of 0.26 as illustrated in Fig. 7b. In the event of collapse, total building loss is assumed. The collapse fragility of the modern archetype building is in agreement with the design of modern building codes, which is to produce designs with low probability of collapse (in the order of 10%) under a Maximum Considered Earthquake [2]. At a period of 5 seconds, the ASCE 7 MCE spectral ordinate is approximately 0.30g.

The probabilities of observing no collapse are derived from the collapse fragility whereas the probabilities of residual drift rendering the building irreparable are obtained from the SP3 analysis results. Similar to collapse realizations, total building loss is assumed when residual drifts deem the building irreparable. At intensity levels where a large number of realizations trigger collapse, there is difficulty in developing accurate estimates of loss given no collapse. For these cases ($e_{5\text{-}6}$ to e_8 for UBC 73) it is assumed that permanent deformations in the structure would deem the building irreparable. At those intensity levels considered in the assessment where low probabilities of collapse are observed, losses due to repairable damage are computed.

The vulnerability function provides the damage ratio, total loss over total building cost, versus spectral acceleration at a period of interest. As discussed earlier, this study followed a Multiple Strip Analysis (MSA) approach in which NLRHA are performed at 8 different ground motion intensities, defined from the seismic hazard curve at a period of 5 seconds. For each intensity of shaking, the damage ratio is computed considering the following components: (1) collapse, (2) non-collapse, non-repairable and (3) non-collapse, repairable. Knowledge of the damage ratio, and contribution of each of these components, at each intensity of shaking and its relevant spectral acceleration at the selected period enable producing the resulting vulnerability function for the 1970s archetype building as shown in as illustrated in Fig. 7a and for the modern archetype building as shown in as illustrated in Fig. 7b.

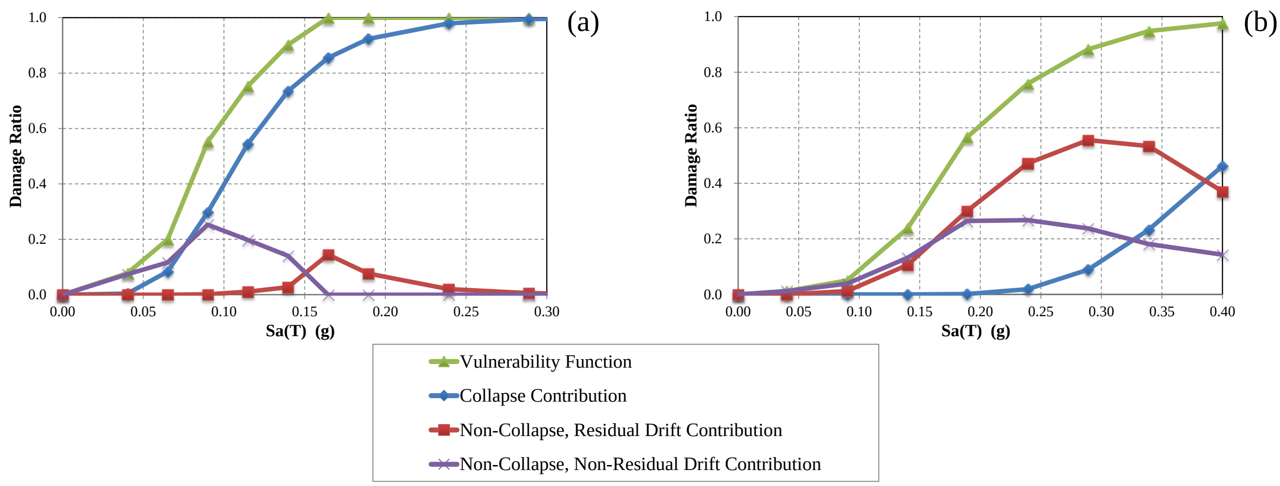


Fig. 7 – Vulnerability function for 1970s (a) and modern (b) archetype buildings illustrating overall contributions of collapse, non-collapse non-repairable and non-collapse repairable damage.

3. Conclusions

The objective of the study is to benchmark the performance of older existing steel MRF buildings designed following historic code-prescriptive requirements against modern design standards. To this end, a comparative risk-based assessment of two archetype 50-story tall steel MRF office buildings designed following the requirements of the 1973 UBC and the 2012 IBC is carried out using San Francisco, CA as a case study due to the large number of existing 1970s-era tall steel MRF buildings.

The results illustrate that existing tall steel MRF buildings from the 1970s are drastically more vulnerable to earthquakes than tall steel MRF buildings designed to modern standards. The estimated collapse fragility of the modern archetype is characterized by an estimated median spectral acceleration at a 5 second period of 0.41g. The resulting collapse fragility is in agreement with the design of modern building codes, which is to produce designs with low probability of collapse under a Maximum Considered Earthquake [2]. The estimated collapse fragility of the 1970s archetype has an estimated median spectral acceleration at a 5 second period of 0.11g, which is 27% of that estimated for the modern archetype building.

The resulting vulnerability function of the modern archetype building (Fig. 7b) indicates that at low ground motion intensities of shaking, losses are influenced by repairable damage. At medium intensities of shaking losses are equally dominated by repairable damage and residual drift rendering the building irreparable. Collapse starts contributing to the loss only at large spectral amplitudes. However, even at those intensities of shaking, losses are still largely dominated by residual drifts.

The vulnerability function for the 1970s archetype building (Fig. 7a) highlights that collapse potential is the highest contributor to the losses. The resulting vulnerability function indicates that at low ground motion intensities of shaking, losses are influenced by repairable damage, whereas at medium and high intensities of shaking losses are largely dominated by collapse potential. While the overall vulnerability function shows similar patterns to those presented in [24] for a 1970s 40 story archetype building, the 50 story archetype in this study has a median spectral amplitude of collapse fragility at a period of 5 seconds of 75% of the value estimated for the 40 story building.

Under the same intensity of ground motion shaking, expected damage ratios are drastically larger for the 1970s archetype than for the modern archetype building. For instance, under a 189-year return period earthquake intensity, the expected damage ratio of the 1970s archetype is 7.8% versus only 1.2% for the modern archetype. Similarly, at a 400 year return period, the expected damage ratio of the 1970s archetype is 55% versus only 5% for the modern archetype. For a return period of 714 years, the damage ratio of the 1970s archetype is 90.2% versus 23.8% for the modern archetype. Overall, the results indicate that 1970s tall steel MRF buildings are far from complying with modern design requirements, not only in terms of collapse safety, but in terms of damage control.

Future work should explore the influence of the following aspects in the resulting vulnerability functions presented in this study:

- Conditioning Period:

The use of conditional spectrum with mean and variability is computed on the basis of a conditioning period. However, the archetype buildings considered in this study are sensitive to response spectral amplitudes at multiple periods. Recent studies [25] have shown that risk-based assessments are relatively insensitive to the choice of conditioning period when the ground motions are carefully selected to ensure hazard consistency because the distributions of response spectra of the selected ground motions are consistent with the site ground motion hazard curves at all relevant periods; this consistency with the site hazard curves is independent of the conditioning period. However, while in the development of a performance (loss) function, loss results are integrated with the seismic hazard curve, such integration is not required in the development of the vulnerability function. It is therefore important to consider the impact of the conditioning period in the overall vulnerability function and loss contributions.

- Number of Ground Motions:

The study here presented is based on ground motions suites consisting of 8 records. While as little as 7 ground motions are sufficient, if carefully selected to match the target spectrum, to capture median response, it is recommended that ground motion suites are expanded to up to 40 ground motion records per intensity level. This is of special relevance considering the conditional spectrum (mean and variance) was selected as the target spectrum for the assessment.

- Analytical Model:

Two-dimensional analytical models were developed to represent the lateral resisting system of the archetype buildings presented in this study. The assumptions of the models are intended to capture median response. However, sensitivity studies should be conducted to assess influence of modelling assumptions on the resulting vulnerability functions.

- Building Performance Model:

The building performance model presented in this report was developed using SP3 [18]. While the loss assessment methodology implemented in SP3 is equivalent to FEMA P-58's PACT, the generation of outputs is limited to relevant information pre-selected by the developers. This results in difficulty in conducting detailed checks of the building performance model, e.g. the contribution of a component to the overall loss or concentration of losses in a certain story.

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5. References

[1] FEMA (Federal Emergency Management Agency). (2006). "Next generation performance based seismic design guidelines. Program plan for new and existing buildings." Rep. FEMA 445, Washington, DC.

[2] PEER (Pacific Earthquake Engineering Research) Center. (2010). "Tall buildings initiative: Guidelines for performance-based seismic design of tall buildings." Rep. No. 2010/05, College of Engineering, Univ. of California, Berkeley, CA.

[3] Molina Hutt, C., Almufti, I., Willford, M., Deierlein, G. (2015). "Seismic Loss and Downtime Assessment of Existing Tall Steel-Framed Buildings and Strategies for Increased Resilience." American Society of Civil Engineers, Journal of Structural Engineering. (10.1061/(ASCE)ST.1943-541X.0001314).

[4] ICBO (International Conference of Building Officials). (1973). Uniform Building Code (UBC) 73, Whittier, CA.

[5] ICC (International Code Council). (2012). International Building Code (IBC) 2012, Country Club Hills, IL.

[6] NEHRP (2011). "Selecting and scaling earthquake ground motions for performing response-history analyses." NIST GCR 11-917-15 prepared by NEHRP Consultants Joint Venture for the National Institute of Standards and Technology, Gaithersburg, MD.

[7] Moehle, J., and Deierlein, G. (2004). "A framework methodology for performance-based earthquake engineering." Proc., 13th World Conf. of Earthquake Engineering, International Association for Earthquake Engineering, Tokyo.

[8] FEMA (Federal Emergency Management Agency). (2012). "Seismic performance assessment of buildings." Rep. FEMA P-58, Washington, DC.

[9] SEAOC (Structural Engineers Association of California). (1973). Recommended lateral force requirements and commentary, Sacramento, CA.

[10] ASCE. (2010). "Minimum design loads for buildings and other structures." ASCE/Structural Engineering Institute (SEI) 7-10, Reston, VA.

[11] FEMA (Federal Emergency Management Agency). (2000). "Recommended postearthquake evaluation and repair criteria for welded steel moment-frame buildings. Program to reduce the hazards of steel moment frame structures." Rep. FEMA 352, Washington, DC.

[12] PEER (Pacific Earthquake Engineering Research) Center. (2010). "Modeling and acceptance criteria for seismic design and analysis of tall buildings." Rep. No. 2010/111, College of Engineering, Univ. of California, Berkeley, CA.

[13] SAC. (2000). "Performance prediction and evaluation of steel special moment frames for seismic loads." Rep. No. SAC/BD-00/25, Richmond, CA.

[14] Bruneau, M., and Mahin, S. (1990). "Ultimate behavior of heavy steel section welded splices and design implications." J. Struct. Eng., 10.1061/ (ASCE)0733-9445(1990)116:8(2214), 2214–2235.

[15] Kochalski G. and Ericksen J. (2007). Prequalified Seismic Moment Connections, Steel Solutions Center, Modern Steel Construction.

[16] LS-DYNA 971 [Computer software]. Livermore, CA, Livermore Software Technology Corporation (LSTC).

[17] Kurata, M., Nakashima, M., and Suita, K. (2005). "Test on large cyclic deformation of steel tube columns having fixed column bases." J. Struct. Constr. Eng., 598, 149–154 (in Japanese).

[18] Lignos, D., and 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." Proc., 7th Int. Conf. on Urban Earthquake Engineering and 5th Int. Conf. on Earthquake Engineering, Tokyo Institute of Technology, Tokyo.

[19] ASCE 41 (2013). “Seismic Evaluation and Retrofit of Existing Buildings.” ASCE/Structural Engineering Institute (SEI) 41-13, Reston, VA.

[20] USGS Hazard Curve Application (http://geohazards.usgs.gov/hazardtool/application.php).

[21] Jayaram, N., Lin, T., and Baker, J. W. (2010). A computationally efficient ground-motion selection algorithm for matching a target response spectrum mean and variance, Earthquake Spectra. DOI: 10.1193/1.3608002

[22] SP3 [Computer software]. Seismic Performance Prediction Program. Haselton and Baker Risk Group (www.hbrisk.com).

[23] Baker, J. W. (2015). “Efficient analytical fragility function fitting using dynamic structural analysis.” Earthquake Spectra, (in press).

[24] Molina Hutt, C., Almufti, I., Willford, M., Deierlein, G. (2015b). “Risk-based Seismic Performance Assessment of Existing Tall-Steel Framed Buildings in San Francisco.” 2015 SECED Conference: Earthquake Risk and Engineering towards a Resilient World, Cambridge, UK.

[25] Lin T., Haselton, C. and Baker J.W. (2013). Conditional spectrum-based ground motion selection. Part I: Hazard consistency for risk-based assessments. Earthquake Engng Struct. Dyn DOI: 10.1002/eqe.2301