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Comparison of Seismic Performance & Recovery Metrics for a 1970s vs Modern Tall Steel Moment Frame Building

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ABSTRACT

This study benchmarks the performance of older existing tall steel moment resisting frame (MRF) buildings designed following historic code-prescriptive requirements (1973 Uniform Building Code) against modern design standards (2012 International Building Code). The comparison is based on risk-based assessments of alternative designs of a 50-story archetype office building, located at a site in San Francisco, CA. The following metrics are compared: (i) mean annual rate of collapse, λ_c (ii) average annual loss (AAL), and (iii) average annual downtime (AAD). The mean annual frequency of collapse of the the 1973 archetype building is 28 times greater than the equivalent 2012 building ($28 \cdot 10^{-4}$ versus $1 \cdot 10^{-4}$), or approximately 13% versus 0.5% probability of collapse in 50 years. The expected AAL is 65% higher for the 1973 than the 2012 building (0.66% versus 0.40% of building replacement cost); and the AAD to re-occupancy is 72% greater for the 1973 than the 2012 building (8.1 vs 4.7 days). The AAD to functional recovery for the 1973 building is twice that of the 2012 building (10.4 vs 5.0 days). An evaluation of the results at various earthquake ground motion shaking intensities suggests that existing 1970s tall steel moment frames are far from complying with modern design requirements in terms of both collapse safety under extreme ground motions and damage control in small to moderate magnitude earthquakes. Furthermore, while modern building code requirements provide acceptable seismic collapse safety, they do not ensure a level of damage control to assure a swift recovery after a damaging earthquake.

Introduction

A major concern in earthquake disaster resilience is the large risks posed by existing buildings, which may have built-in deficiencies that are not permitted by current building codes. For some types of buildings, such as unreinforced masonry structures, the risks are so large, that mandatory laws have been enacted to assess and retrofit the buildings. However, in other cases, such as with

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non-ductile concrete buildings or older tall steel buildings, the risks and mitigation strategies are not as straightforward. The goal of this study is to benchmark the performance of older seismically vulnerable tall steel moment resisting frame (MRF) buildings, which constitute a significant portion of tall buildings constructed between 1960 and 1990 [1, 2] in San Francisco, Los Angeles and other high seismic regions of the western US, against modern designs. The comparison is focused on risk metrics that can help inform policy and decision making.

This study benchmarks the performance of older existing tall steel MRF buildings designed following historic code-prescriptive requirements (UBC 1973 [3]), against modern design standards (IBC 2012 [4]) by conducting a comparative risk-based assessment of alternative designs of a 50-story archetype office building, located at a site in San Francisco, CA. The following metrics are compared: (i) annual rates of collapse, λ_c; (ii) average annual losses (AAL); (iii) average annual downtime (AAD).

Methodology

The archetype buildings are developed based on a database of the existing tall building stock in San Francisco. Numerical models of the structure are developed to assess structural performance using nonlinear dynamic analyses. Performance is evaluated under sets of earthquake ground motions, selected and scaled to various intensities. Expected losses are determined from structural response following the FEMA P-58 methodology [5]. Downtime estimates follow the Resilience-based Earthquake Design Initiative’s (REDi) downtime assessment methodology [6].

The occupancy of the archetype buildings is assumed to be commercial office. Typical story heights and beam spans are 3.8 m and 8.5 m, respectively. In the 1970s, it was customary to have moment connections in all beam-to-column intersections, as illustrated in Figure 1b, but due to practical and economic factors, this practice evolved to perimeter frame structures in which only two frame lines in each direction are moment-resisting, as illustrated in Figure 1c. The resulting section sizes for a typical frame in each archetype building are shown in Figure 1a. The frames consist of wide flange beams, welded beam-to-column connections, and built-up box columns (denoted R in Figure 1a) in the 1973 building and built-up I sections (denoted I in Figure 1a) in the 2012 building. The total building dead load is approximately 800,000 kN. The design wind and seismic base shears are equal to 1.80% and 1.96% of total building dead load, respectively, per UBC 1973 requirements, versus 4.26% and 3.74% per IBC 2012. For a detailed description of the design method and assumptions refer to [2].

There are many requirements in modern design standards, which are not present in the 1973 designs, which drastically improve seismic performance, e.g. response spectrum analysis method as opposed to equivalent lateral force, minimum base shear requirements, p-delta effects, strong column weak beam provisions, capacity design principles, etc. In addition, the fracture prone-moment connections, which resulted from a combination of low toughness weld materials, large built-in initial flaws, and high stain demands are expected to negatively affect the seismic performance of the older steel frames. The switch in the weld process that led to welds with low toughness, as evidenced by fractures observed in the 1994 Northridge earthquake, took place in the mid-1960s [7]. Therefore, it is assumed that that fracture-prone pre-Northridge moment connections are common in designs from the 1970s.

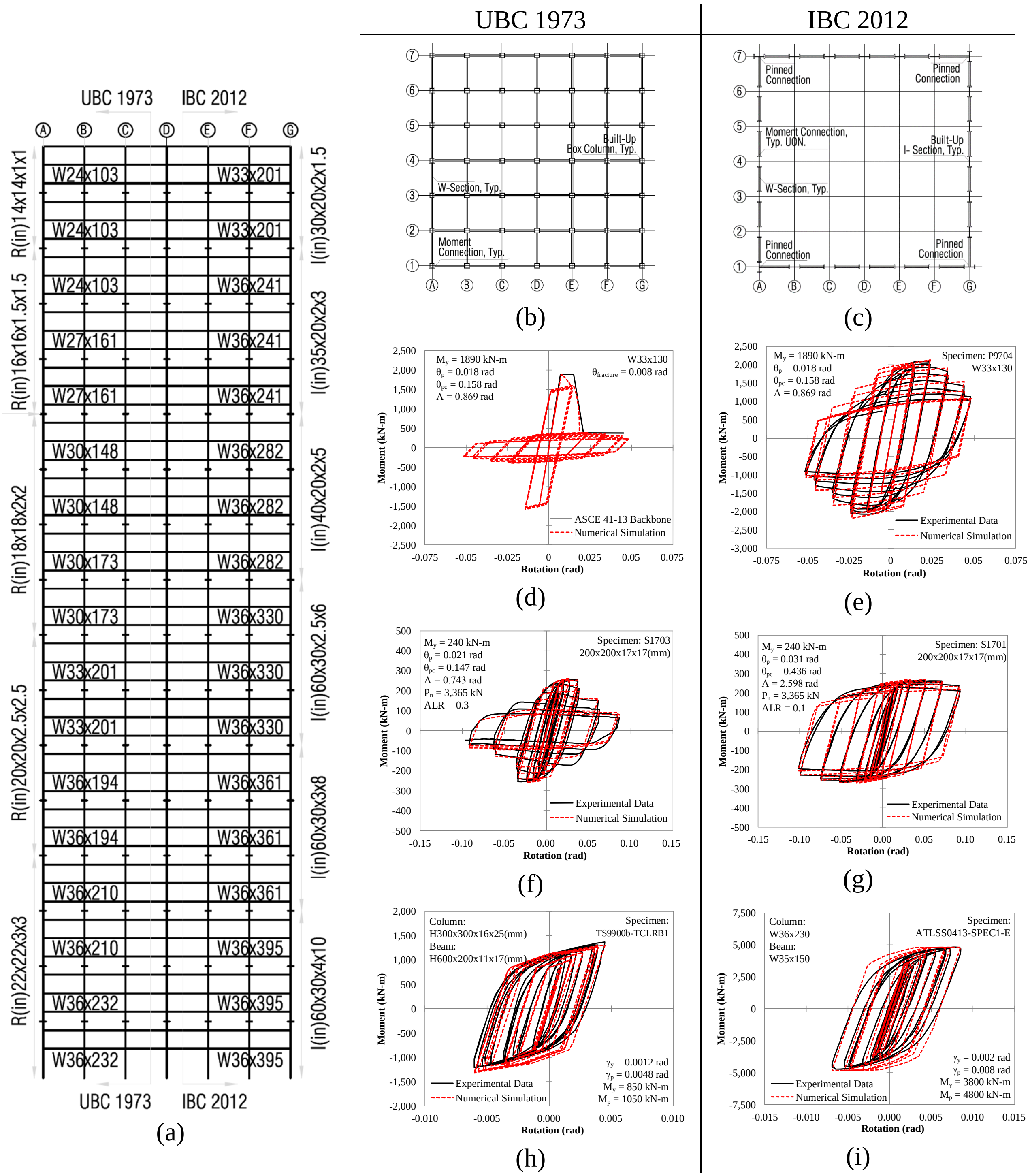
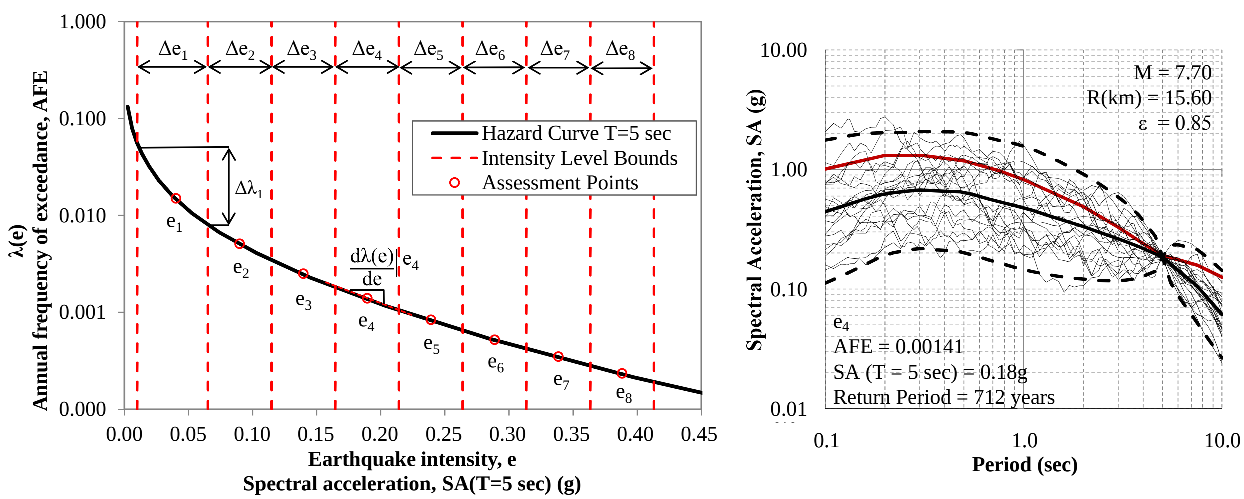


Figure 1: Archetype steel MRF building elevation (a), plan (b, c) and sample hysteretic response of key structural components: beams (d, e), columns (f, g) and panel zones (h, i).

The nonlinear dynamic analyses are performed using finite element models capable of capturing the response of all structural elements that significantly contribute to the strength and stiffness of the system. A 2D numerical model of a representative frame, as illustrated in Figure 1a, is developed in LS-DYNA [8]. The first (T_1), second (T_2) and third (T_3) modes of the representative 2D frame are 5.5, 2.1 and 1.3 seconds, respectively, for the 1973 archetype building versus 5.1, 2.0 and 1.2 seconds, respectively, for the 2012 archetype building.

Key structural elements include beams, columns and panel zones. Beams are modeled as lumped plasticity beam elements following recommendations in [9], which propose modeling parameters based on a large database of experimental tests. To account for fracture in the moment connections, a plastic rotation threshold at which fracture is set to occur in the connections is introduced according to ASCE 41 recommendations [10]. The impact of the plastic rotation threshold at fracture is illustrated in the hysteretic responses for a pre- versus post-Northridge sample moment connection, as shown in Figure 1d versus 1e. Columns are modeled as lumped plasticity beam elements with yield surfaces capable of capturing interactions between bending moment and axial force following the recommendations in [11], which account for different rates of degradation in the moment-rotation response as a function of axial load-to-capacity ratios (ALR). Sample moment-rotation responses of columns with an ALR of 0.3, representative of the 1973 building, and applied ALR of 0.1, representative of the 2012 archetype, are illustrated in Figures 1f and 1g, respectively. Panel zones are modeled using the Krawinkler model as outlined in [12], which incorporates an assembly of rigid links and rotational springs to represent the finite dimensions and flexibility of the panel zone. A sample shear force-deformation response of a panel zone is illustrated in Figures 1h and 1i. The analytical component models are verified against available test data, as seen in Figures 1e to 1i.



(a) (b)

Figure 2: Seismic hazard curve illustrating preliminary assessment points (a), target spectra and associated ground motion suite for a sample earthquake ground motion intensity level (b).

The performance of the archetype buildings is evaluated through a Multiple Stripe Analysis (MSA) approach, where assessments are performed at a series of ground motion shaking intensities spanning from high to low probabilities of occurrence. The upper and lower bound intensity levels considered result in a range of damage from negligible to complete loss. Preliminary bounds for assessment are applied to the seismic hazard curve at a representative site in downtown San Francisco, as shown in Figure 2a, with soil properties consistent with site class D [1]. When necessary, additional assessment points are introduced to obtain the desired range of damage. The hazard curve is obtained using the USGS hazard curve calculation tool [13]. A seismic hazard curve for a 5 second period is selected as it is close to the fundamental period of the archetype buildings and is the longest period for which USGS provides seismic hazard data. To select ground motion records for the dynamic analyses, a conditional spectrum (CS) [14], conditioned at a 5 second period, is selected as the target spectrum for each intensity level, using seismic hazard de-aggregation results. Suites of 20 ground motions are selected to collectively match the entire distribution of the CS, as illustrated in Figure 2b. The selected ground motions are input at the fixed base supports of the frame analysis models. A critical damping ratio of 2.5% is assumed in the analysis [15]. For further details of the model and ground motion selection procedure, refer to [2].

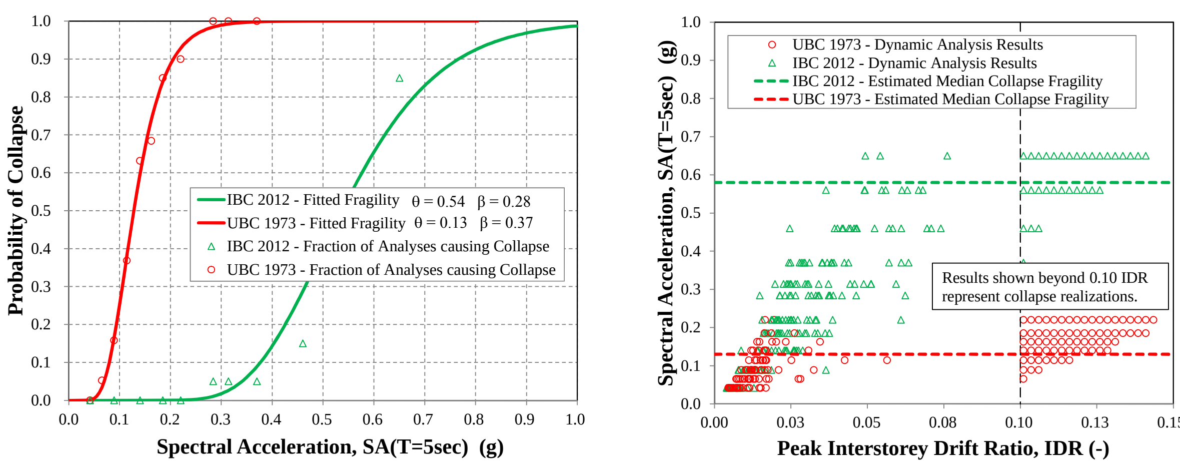
Owners, insurers and financial institutions often use quantitative statements of probable building repair cost expressed as a percentage of building replacement value. This metric is used in this work, where future repair costs are converted to present dollars. Repair costs are expressed as a percentage of building replacement cost, which is estimated based on the gross square footage at a rate of $3,550/m^2 ($330/ft^2) [1]. A building performance model is developed to evaluate the loss and downtime metrics of interest. At each earthquake ground motion intensity level considered in the MSA, 1000 loss and associated downtime estimates are calculated. Results from nonlinear dynamic analysis 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. 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 [5]. The components employed in the building performance model for the archetype buildings are assumed to be the same for the IBC 2012 and UBC 1973 designs, but fragilities are adjusted to account for modern seismic design requirements of structural and non-structural components. This adjustment is possible because the fragility library developed for the FEMA P58 project includes variations of the same component adjusted for different seismic design categories. The library also includes variations of a same component, e.g. beam-to-column connection, to account for important changes in design and construction practice, i.e. pre- and post-Northridge moment connection detailing.

Each of these structural and non-structural building components has a component fragility function, i.e. a statistical distribution that indicates the conditional probability of incurring damage at a given value of demand. 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. SP3 [16], which adopts the FEMA P58 methodology, is used to conduct the loss assessment. When estimating losses, the impact of excessive residual story drifts is considered by means of a typical building repair fragility [5], to account for scenarios in which the building is damaged beyond repair. If irreparable, repair costs are taken as the building replacement values. A similar approach is followed to consider collapse contribution to losses by means of a collapse fragility, developed from the nonlinear dynamic analysis results.

While seismic loss estimates associated with direct economic losses enable discussions with building owners and investors about expected building performance, they do not provide a quantitative measure of resilience. In addition to direct economic losses, there is significant vulnerability to indirect economic losses due to downtime, defined as the time required to achieve a recovery state after an earthquake. Downtime to re-occupancy and functional recovery are considered in this work. Re-occupancy occurs when the building is deemed safe enough to be used for shelter; and functional recovery occurs when the building regains its primary function, i.e. it is operational. These estimates follow the REDi downtime assessment methodology [6], which identifies the extent of damage and criticality of building components that may hinder achieving a recovery state. It provides a logical approach for labor allocation and repair sequencing including structural, interior, exterior, mechanical, electrical, elevator and stair repairs on a floor per floor basis. Furthermore, the methodology includes delay estimates associated with impeding factors, which may impede the initiation of repairs such as post-earthquake inspection, engineering mobilization for review or re-design, financing, contractor mobilization, permitting and procurement of long lead items. Lastly, utility disruptions are also considered when estimating downtime for functional recovery. For a more detailed description of the building performance model refer to [2].

Collapse Risk

Assessing the collapse risk of a structure entails combining information related to the behavior of the structure with seismic hazard data at the site. The response of the structure is characterized by a collapse fragility, which uses nonlinear dynamic analysis results to describe the increasing probability of collapse as a function of the ground motion shaking intensity, typically the spectral acceleration (SA) at the fundamental period of the structure. Following the MSA approach, nonlinear dynamic analyses are performed at various intensity levels ('stripes') and hazard consistent ground motions are selected at each intensity level. At the higher earthquake ground motion shaking intensities, the fraction of ground motions that cause collapse are recorded and used to obtain the collapse fragility for the building, as illustrated in Figure 3.



(a) (b)

Figure 3: Multiple stripe analysis results (a) for collapse fragility derivation (b) of the 1973 and 2012 archetype buildings.

The resulting collapse fragility of the 1973 building has an estimated median of 0.13g and a dispersion of 0.37, while that of the 2012 building has an estimated median of 0.54g and a dispersion of 0.28, as illustrated in Figure 3a. The mean annual frequency of collapse, λ_c, is obtained by integrating the collapse fragility with seismic hazard curve, i.e., mean annual frequency of exceeding the ground motion shaking intensity [17]. The calculated, λ_c is equal to $28 \cdot 10^{-4}$ for the 1973 building, versus $1 \cdot 10^{-4}$ for the 2012 building. The results for the modern building are consistent with the λ_c calculated by [18] for modern, code-conforming concrete moment frames, which is estimated to range from $0.7 \cdot 10^{-4}$ to $7 \cdot 10^{-4}$. Comparing the two sets of results, the annualized collapse risk of the 1973 building is 28 times greater than that of the 2012 archetype building and about 14 times larger than the maximum risk target of 1% in 50 years (equivalent to λ_c equal to about $2 \cdot 10^{-4}$), which is implied in the risk-targeted MCE_R design value maps.

Loss and Downtime Risk

The results of the MSA can also be used to develop tools and metrics related to the expected economic losses and downtime associated with the performance of the building, such as the annualized loss, i.e. AAL, or annualized downtime, i.e. AAD. These parameters can be inferred from the loss and downtime function, respectively, and can be calculated as outlined in [2]. These metrics provide information in for risk management and recovery planning. For instance, the AAL can easily relate to annual insurance payments, and the AAD can be a useful tool for estimating business disruption and other indirect losses associated with building closures due to seismic damage.

Figures 4a and 4b illustrate the probabilities of observing (i) repairable damage, (ii) residual IDRs rendering the building irreparable or (iii) collapse at each earthquake ground motion intensity for both archetype buildings. These results indicate that collapse risk is the greatest contributor to the expected losses in the 1973 building. In contrast, residual story drifts rendering the building irreparable are the greatest contributor to the expected loss in the 2012 building. The likelihood of observing these outcomes needs to be considered when computing the AAL and the AAD. In the event of collapse or residual deformations rendering the building irreparable, losses and downtime are essentially equal to the building replacement cost and time. The building replacement cost is denoted by a loss ratio of 1. The building replacement time is estimated as 1000 days based on 300 days for demolition and re-design, and 700 days for reconstruction (approximately two weeks per story).

The AAL for the 1973 building is estimated at 0.66% of building replacement cost, whereas the AAL for the 2012 building is estimated at 0.40%. The contribution of repairable damage, irreparable damage and collapse to the AAL is illustrated in Figures 4c and 4d. These data are consistent with previous observations of collapse and irreparable damage controlling the overall loss in the 1973 and the 2012 buildings, respectively. As a reference point, [20] evaluated the performance of a set of modern concrete-framed 20-story buildings to have AALs on the order of 0.4 to 0.7% of building replacement cost. These results are consistent with the results for the 2012 50-story archetype considered in this study, particularly when considering that normalized seismic losses tend to be lower in tall buildings due to significant damage concentrated in a few stories rather than distributed throughout the building height [2].

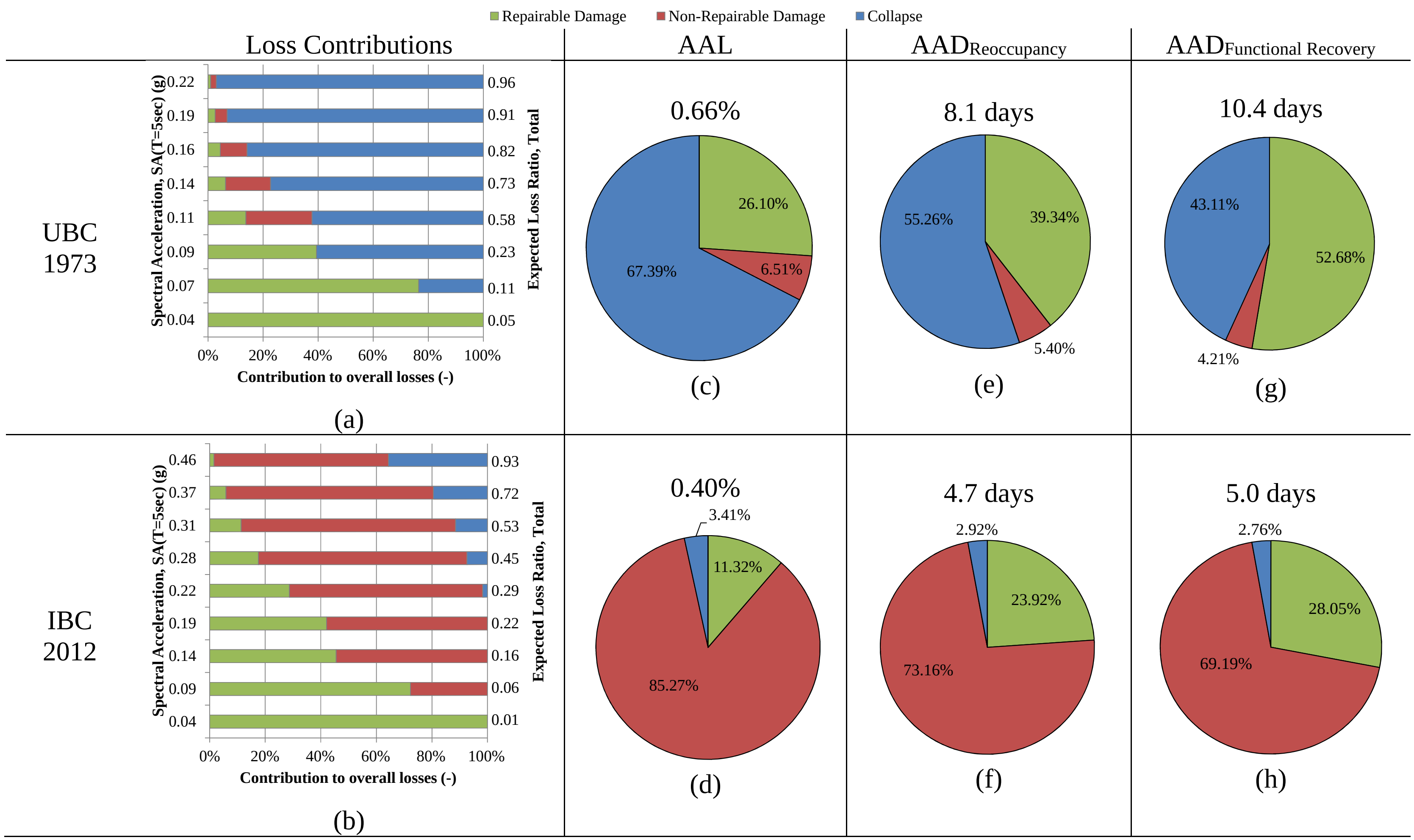


Figure 4: Contribution of repairable damage, irreparable damage from residual drifts and collapse to overall loss, average annual losses, and average annual downtime to re-occupancy and functional recovery for UBC 1973 and IBC 2012 archetype buildings.

The estimated AADs to re-occupancy for the 1973 building is 8.1 days versus 4.7 days for the 2012 building. The estimated AAD to functional recovery for the 1973 building is 10.4 days versus 5.0 days for the 2012 building. The contribution of repairable damage, irreparable damage and collapse to the AAL is illustrated in Figures 4e to 4h, coinciding with previous observations of collapse and irreparable damage controlling the loss in the 1973 and the 2012 buildings, respectively.

While annualized loss and downtime metrics are useful for risk management and recovery planning, they are not as intuitive as an intensity based assessment. For reference, the expected losses and downtime under the intensity level closest to the 10% in 50 year hazard (SA at a 5-second period equal to 0.14g in Figures 4a and 4b) are briefly discussed. This intensity level is selected as it is consistent with DBE shaking and with that used by building rating systems. At the selected intensity level, the 1973 building has an expected loss ratio of 0.73, and expected downtime of 778 and 790 days to re-occupancy and functional recovery, respectively. These estimates are based on a 31% probability of observing repairable damage, 12% of irreparable damage and 57% probability of collapse. In contrast, the 2012 building has an expected loss ratio of 0.16, and expected downtime of 227 and 268 days to re-occupancy and functional recovery, respectively. These estimates are based on a 91% probability of observing repairable damage, 9% of irreparable damage and 0% probability of collapse.

Conclusions

The following metrics are compared for a 1970s versus a modern 50-story steel MRF archetype building in San Francisco, CA: (i) mean annual rate of collapse, λ_c (ii) average annual loss (AAL), and (iii) average annual downtime (AAD). The mean annual frequency of collapse of the the 1973 archetype building is 28 times greater than the equivalent 2012 archetype building ($28\cdot10^{-4}$ versus $1\cdot10^{-4}$), or approximately 13% versus 0.5% probability of collapse in 50 years. The expected AAL is 65% higher for the 1973 than the 2012 50-story archetype building (0.66% versus 0.40% of building replacement cost). The AAD to re-occupancy for the 1973 archetype building is 72% greater than that of the 2012 archetype building (8.1 vs 4.7 days). The AAD to functional recovery for the 1973 archetype building is twice that of the 2012 archetype building (10.4 vs 5.0 days). An evaluation of the results at different earthquake ground motion shaking intensities suggests that existing 1970s tall steel moment frames are considerably more vulnerable to both collapse and damage than modern code-conforming buildings. Furthermore, while modern building code requirements provide acceptable seismic collapse safety, they do not ensure a level of damage control to enable a swift recovery after a damaging earthquake.

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