Canadian Conference - Pacific Conference on Earthquake Engineering 2023

Vancouver, British Columbia

June 25th – June 30th, 2023

Functional Recovery Assessment of Tall Buildings with pre-Northridge Welded Steel Moment Frames

Francisco A. Galvis[1*], Gregory G. Deierlein[2], Omar Issa[3], Adam Zsarnoczay[4], Carlos Molina Hutt[5], Jack W. Baker[2]

[1] Project Engineer, Thornton Tomasetti, San Francisco, CA, USA*
[2] Professor, Department of Civil and Environmental Engineering, Stanford University, CA, USA
[3] Ph.D. Student, Department of Civil and Environmental Engineering, Stanford University, CA, USA
[4] Research Engineer, Department of Civil and Environmental Engineering, Stanford University, CA, USA
[5] Assistant Professor, Department of Civil Engineering, The University of British Columbia, BC, Canada
*fgalvis@thorntontomasetti.com (Corresponding Author)

ABSTRACT

Many tall buildings on the west coast of the United States rely on welded steel moment frames (WSMFs) of the kind that experienced premature brittle fractures during the 1994 Northridge and 1995 Kobe earthquakes. Despite their known vulnerabilities, the vast majority of existing tall WSMF buildings have neither been evaluated nor retrofitted. To support the development of risk mitigation policies for these structures, this paper presents the results of detailed evaluations of seismic economic losses and functional recovery time associated with these buildings. Recognizing that the structural performance of tall buildings strongly depends on their unique characteristics, we collected detailed design information of about 40 buildings in downtown San Francisco and developed their high-fidelity structural models in OpenSees. These models use a novel fiber-section approach to accurately simulate the fracture initiation and post-fracture behavior in welded connections. We use these models to quantify the expected economic loss from direct physical damage and the functional recovery time of tall WSMF buildings subjected to an earthquake scenario with 30% chance of occurring in the next 50 years (i.e., an event with a 140-year return period). We combined hazard-consistent ground motion record selection, nonlinear response history analyses, FEMA P-58 performance assessment, and ATC-138 functional recovery time evaluation procedures into an automated computational workflow for the first time to assess existing buildings. The results indicate that tall WSMFs vary widely in their performance. Their expected loss ratio (repair cost divided by replacement cost) ranges from 20% to 80%, with a median of approximately 40% loss ratio. The average functional recovery time ranges from 8 to 50 months, significantly exceeding the four-month target from San Francisco's Resilient City document (SPUR 2009). These consequences far exceed those estimated for modern tall buildings by other studies (10% loss ratio and 4 to 6 months functional recovery time for a design level earthquake). This study promotes the use of high-fidelity, building-specific models by showcasing the insights a state-of-the-art risk assessment methodology can provide to building owners and policy makers interested in understanding and managing the earthquake risk of a portfolio of buildings.

Keywords: 5 keywords separated by commas.

INTRODUCTION

The threat from seismically deficient WSMF has recently gained the attention of the general public [1,2], which has prompted discussions on risk mitigation investments capable of reducing the potential impacts of damage to WSMFs on life safety, economic development, and community functionality. To inform these risk mitigation decisions, stakeholders (e.g., policy makers, building owner, tenants, etc.) need reliable estimations of the risk of collapse for life safety, repair cost for economic development, and recovery time for community functionality. In the context of tall buildings, these metrics can be quantified using the performance-based earthquake engineering (PBEE) methodology [3,4], which can capture the unique structural characteristics of tall buildings in the risk assessment procedure. However, recent enhancements to the PBEE methodology for estimating functional recovery time [5,6] have not been applied to existing buildings, and they have the potential to provide important understanding on the complex behavior of tall WSMF.

Recognizing the aforementioned knowledge gap, NIST funded a 5-year project focused on advancing the seismic risk assessment of tall WSMF buildings. This project includes the development of a detailed database of structural characteristics for most tall WSMF buildings in downtown San Francisco to enable PBEE assessments. This paper presents the highlights of this project as well as the risk and loss assessment results for three representative buildings from the database.

The insights from this study can inform policy for assessment and retrofit of WSMF buildings that complement existing ordinances pioneered by two moderately sized cities [7,8]. San Francisco's Earthquake Safety Implementation Program [9] and the Los Angeles mandatory retrofit programs [10] include plans to transition from required structural assessments and retrofit during substantial building renovation to more proactive ordinances, including all existing tall steel buildings, that follows similar policies for soft-story and non-ductile concrete buildings. Just as with current programs for concrete buildings, developing and implementing an effective program for tall steel buildings requires guidelines for screening and assessing WSMFs, along with practical and cost-effective retrofit solutions that can be informed with the tools presented in this project.

METHODOLOGY

The risk assessment methodology is summarized in Figure 1. The methodology is based on nonlinear response history analyses (NLRHAs) that require: (1) ground motion seismograms representative of the seismic hazard at the building site, and (2) building models capable of simulating all the possible collapse mechanisms of the building. The building models are created in OpenSees using the fiber connection model, and the ground motions are carefully selected using a hazard-consistent approach that respects spectral shape [11–13]. Although duration was not explicitly considered for ground motion selection, the significant duration from 5% to 75% Arias intensity of the record sets falls within the expected range of 10 to 30s for downtown San Francisco.

The NLRHAs provide the demand parameters that summarize structural response (i.e., drift, floor accelerations, fractures, etc.). In this framework, the building is considered collapsed for a given ground motion when the peak drift demand at any story exceeds 10%. The collapse/non-collapse results are enough to measure safety but incomplete for measuring damage, repair cost, and functional recovery time. Therefore, the demand parameters are further processed using FEMA P58, as implemented in the NHERI SimCenter loss assessment package, PELICUN [14]. This package leverages the demand parameters of each building and estimates the damage and repair cost of each component. The inventory and location of the components within the building are described by the performance model. The damage to each component is estimated probabilistically using a set of fragility functions that provide the likelihood of the component reaching a specific damage state given the demand parameter. Similarly, the repair cost per damage state per component—adjusted to 2021 dollars assuming a 1.35 inflation factor from the 2011 values in FEMA P58—is described by a probability distribution. Both the fragility functions and the consequence functions are specified in FEMA P58 for all the components used in these analyses.

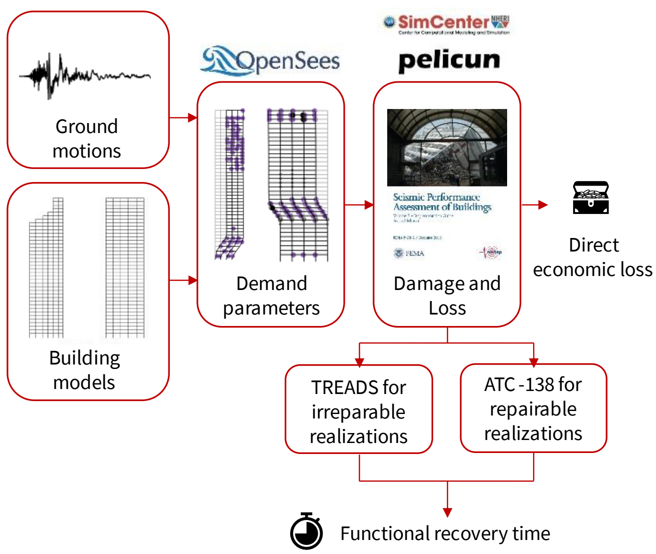
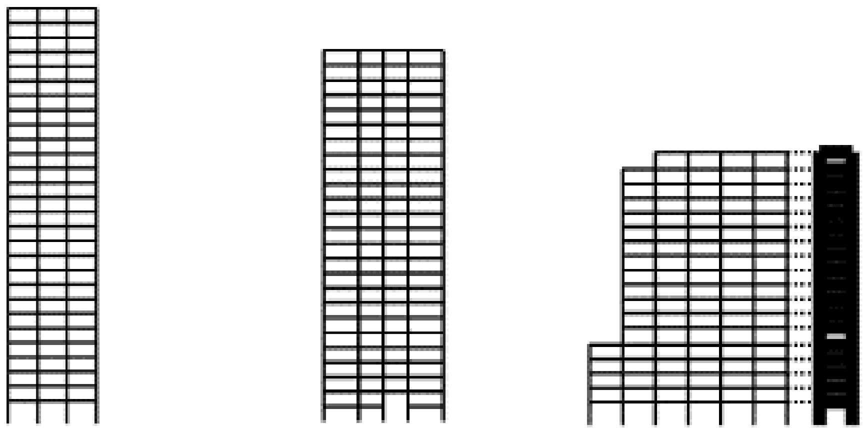


Figure 1. Outline of the risk and loss assessment methodology for each building of the portfolio.

The results of FEMA P58 are further processed using the recently developed methodologies TREADS [15] and ATC-138 [5] to estimate realistic functional recovery times (Figure 1). The ATC-138 method is limited to recovery times for repairable realizations while TREADS provides a logic for estimating recovery times for repairable, irreparable, and collapse realizations. Since ATC-138 is expected to become Volume 8 of FEMA P58, we use this methodology for all the repairable realizations and TREADS for irreparable and collapse realizations.

SEISMIC PERFORMANCE RESULTS

The risk assessment methodology is applied to 40 buildings from the database. This section shows the results for three of those buildings that demonstrate the wide range of performance encountered within the portfolio. Figure 2 presents the elevation of a representative frame of each of the selected three buildings. ID488 is the most vulnerable building from the portfolio, ID258 is the least vulnerable, and ID109 is average. The seismic performance results are computed for three assumptions of the fracture toughness of the beam-to-column connection flanges measured by the Charpy-V-notch test (CVN): 8 ft-lb (lower bound), 12 ft-lb (expected), and 40 ft-lb (modern-code minimum requirement). These three values also demonstrate the importance of appropriately capturing the quality of the weld metal when assessing the performance of WSMFs.



(a) (b) (c)

Figure 2. Building elevation in the weakest direction. (a) ID488 (space frame), (b) ID109 (space frame), and (c) ID258 (perimeter frame).

Economic loss

Figure 3 depicts the expected economic loss associated to a 140-year earthquake as a fraction of the replacement cost of the building (*E[L|140-year]*). The replacement costs are estimated using RSmeans [16] data and the known construction area of each building. Similar to life safety, ID488 has the largest losses followed by ID109 and ID258 in that order. These bars show that the impact of CVN changes the expected loss significantly for all the buildings. For instance, ID488 increases the *E[L|140-year]* from 77% assuming 12ft-lb to 100% assuming 8ft-lb; on the contrary, the *E[L|140-year]* drops from 77% to 65% for CVN=40 ft-lb. This sensitivity of *E[L|140-year]* to CVN contrasts with the minor impact of CVN on *P[C|MCEr]* of ID488. This indicates that measuring CVN is important for accurately estimating economic loss especially for more frequent events that may be meaningful to stakeholders.

The darker portion of the bars in Figure 3 denotes the portion of the expected loss that corresponds to collapse simulations. This collapse portion dominates the economic loss for ID488 but is less important for ID109 and ID258. However, safe buildings with low *P[C|MCEr]* do not always have lower economic losses than buildings with higher *P[C|MCEr]* as depicted by ID109 CVN=40ft-lb compared to ID258 CVN=12ft-lb. The former has *P[C|MCEr]*=20% and *E[L|140-year]*=19%, while the latter has *P[C|MCEr]*=4% and *E[L|140-year]*=25%. This observation suggests that losses associated with irreparable and repairable damages increase for safer buildings and may even lead to higher overall losses. The increased ductility and/or flexibility of "safer buildings" increases the likelihood of high drifts and corresponding structural and nonstructural damage, while the reduced ductility of the more hazardous building may reduce damage in lower intensity events but increases likelihood of collapse and associated losses. A practical way to mitigate irreparable and reparable losses is with improvements to the non-structural elements and their supports. These nuanced insights are only uncovered by the high-fidelity modeling approach pursued in this study.

We estimate the expected annual economic loss (EAL) to consider the effect of all the possible ground motion scenarios from a probabilistic seismic hazard analysis. EAL for CVN=12ft-lb is 1.2% of building replacement cost for ID488, 0.7% for ID109, and 0.5% for ID258. These EALs drop to 1.1%, 0.42%, and 0.27% when the connections are re-welded (i.e., CVN=40ft-lb). Notice that building ID258 with CVN=12ft-lb was already equivalent to modern buildings in terms of the *P[C|MCE$_r$]* but improving the connections cut by half the expected economic losses, suggesting that risk mitigations investments are still beneficial from the financial standpoint. Meanwhile, ID488, whose behavior was controlled primarily by weak columns, saw relatively little improvement from connection toughness due to the unresolved underlying vulnerability. An alternative retrofit scheme targeting the weak columns might yield more marked improvement.

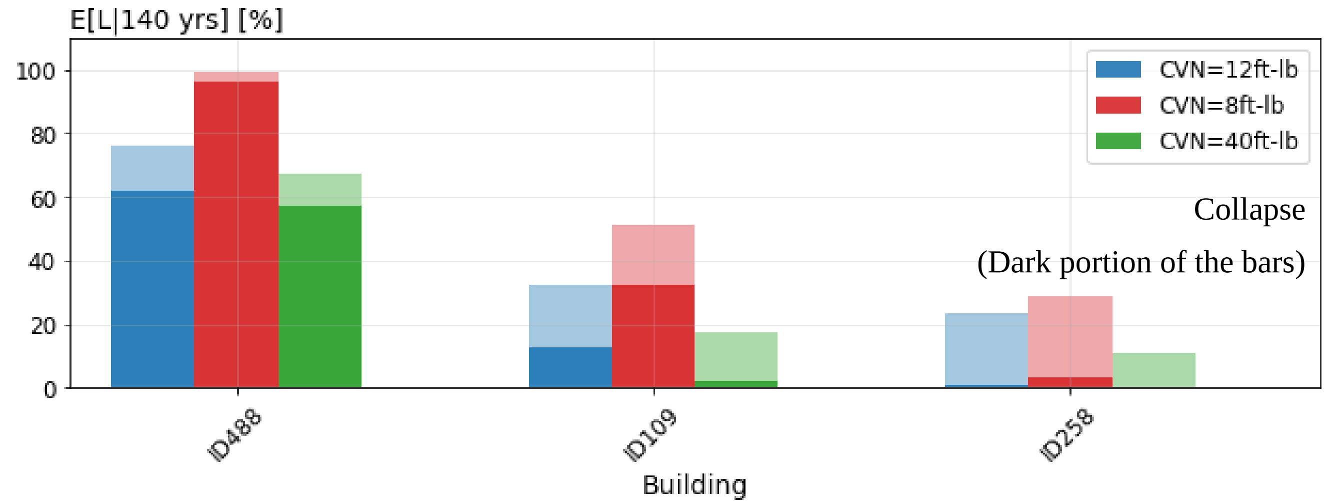


Figure 3. FEMA P58 expected loss after a 140-year event for buildings ID488, ID38, and ID258 assuming three types of connection models: fiber section with CVN=12ft-lb, fiber-section with CVN=8ft-lb, and fiber-section with CVN=40ft-lb.

Functional recovery time

Figure 4 presents the expected functional recovery time after a 140-year earthquake (*E[FRT|140-year]*) and compares the results with the 4-month recovery target for office buildings suggested by SPUR [17] specifically for San Francisco (dashed horizontal line). Evidently, only ID258 with re-welded connection meets the SPUR target. The *E[FRT|140-year]* shows a similar variation with CVN as the *E[L|140-year]* since both are influenced by non-structural damage. Modern tall buildings are expected to have *E[FRT|140-year]* lower than 4 months [15], meeting the SPUR recommendations and far below the results in Figure 7. Not shown in this figure but important to acknowledge is the potential large uncertainty surrounding functional recovery time calculations that require numerous assumptions on ground motions, structural response, component inventory, fragility functions, impeding factors, and repair scheduling.

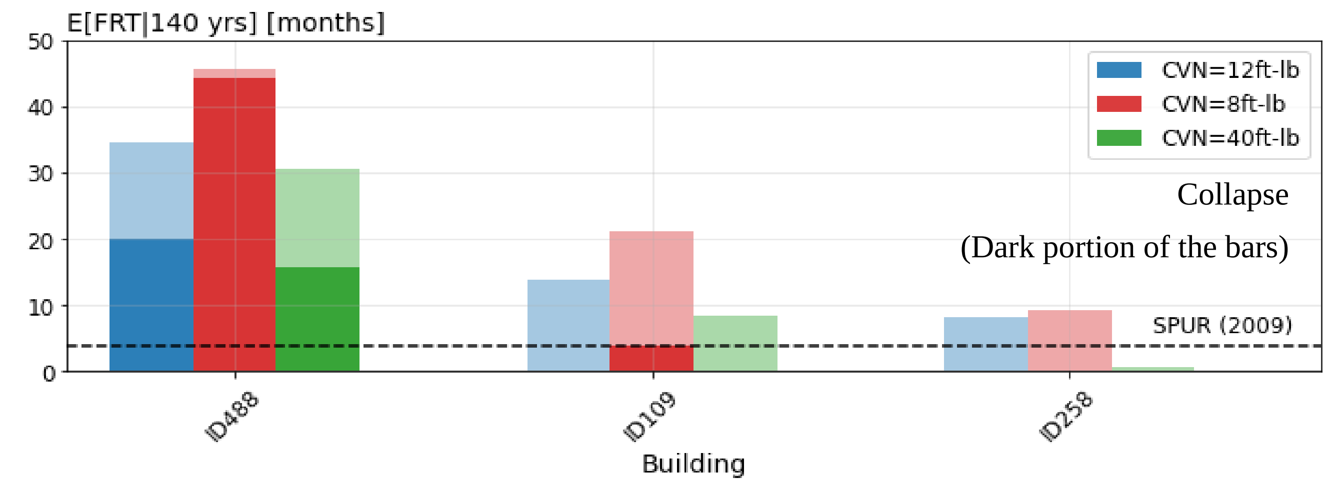


Figure 4. Expected functional recovery time after a 140-year event for buildings ID488, ID38, and ID258 assuming three types of connection models: fiber section with CVN=12ft-lb, fiber-section with CVN=8ft-lb, and fiber-section with CVN=40ft-lb.

CONCLUSIONS

Welded steel moment frame buildings (WSMF) constructed before 1995 pose a larger earthquake risk than originally expected due to their fracture-vulnerable connections. However, the performance of these buildings varies depending on their unique structural characteristics. Capturing structural uniqueness is especially important for existing tall buildings, which are prevalent in cities in California, Washington, and Oregon. This research is one of the most recent efforts explicitly addressing structural uniqueness in regional risk assessments using high-fidelity simulations of tall steel buildings and applying the full performance-based earthquake engineering framework by harnessing recent developments by the NEHRI SimCenter [18].

This paper presents the results of three WSMF buildings from a portfolio of 40 structures assembled for the city of San Francisco. The results show that not all the WSMFs are equally vulnerable despite having the same problematic connection, and that the particular building characteristics are crucial for more or less sensitivity to brittle connections. Furthermore, the economic loss and functional recovery calculations indicate that the repair costs and downtimes for these structures far exceed the expectations for modern buildings, raising questions about the ability of dense urban areas to thrive even after a relatively frequent earthquake (30% in 50 years likelihood).

The structural modeling and loss assessment methodology presented in this research constitute a set of practical tools that could facilitate the communication of structural engineers with key stakeholders such as policy makers, building owners, and the general public.

ACKNOWLEDGMENTS

This research was supported by the National Institute of Standards and Technology (NIST Award #70NANB17H245) and the Fulbright-Colciencias Scholarship for the doctoral studies of the first author. Wen-Yi Yen performed valuable preliminary work on this project. Anne Hulsey, Wen-Yi Yen, and the ATC Project 119—with support from the San Francisco Office of Resiliency and Capital Planning—collected a tall building inventory in San Francisco that inspired the archetypes studied in this paper. Also acknowledged are fruitful discussions with Jim Malley and Bob Pekelnicky from Degenkolb Engineers as well as David Ojala and Brian Shen from Thornton Tomasetti. The first author attends this conference thanks to the support from Thornton Tomasetti. The computational simulations for this project were performed on Stanford's Sherlock cluster. The results represent the findings and opinions of the authors and do not necessarily represent those of the research sponsors.

REFERENCES

[1] Fuller T, Singhvi A, Williams J. San Francisco's big seismic gamble. The New York Times 2018.
[2] Fuller T. At Risk in a Big Quake: 39 of San Francisco's Top High Rises. The New York Times 2018.
[3] Cornell CA, Krawinkler H. Progress and Challenges in Seismic Performance Assessment. PEER Center News 2000; 3(2): 1–4.
[4] Moehle J, Deierlein GG. Framework Methodology for Performance-Based Earthquake Engineering. 13th World Conference on Earthquake Engineering 2004(February).
[5] Cook DT, Liel AB, Haselton CB, Koliou M. A framework for operationalizing the assessment of post-earthquake functional recovery of buildings. Earthquake Spectra 2022: 875529302210815. DOI: 10.1177/87552930221081538.
[6] Molina Hutt C, Vahanvaty T, Kourehpaz P. An analytical framework to assess earthquake-induced downtime and model recovery of buildings. Earthquake Spectra 2021; (Accepted)(October): 1–45.
[7] City of Santa Monica. Seismic Retrofit Program 2017. https://www.smgov.net/Departments/PCD/Programs/Seismic-Retrofit/ [accessed November 10, 2021].
[8] City of West Hollywood. WEHO Seismic Retrofit Program 2017. https://www.weho.org/city-government/city-departments/planning-and-development-services/building-and-safety/seismic-retrofit [accessed November 10, 2021].
[9] Kornfield L. CAPPS Earthquake Safety Implementation Program: Workplan 2012-2042, City and County of SF 2011. http://sfgov.org/esip/sites/default/files/FileCenter/Documents/9765-esipplan.pdf [accessed November 10, 2021].
[10] LADBS. Non-ductile Concrete Retrofit Program 2021. https://www.ladbs.org/services/core-services/plan-check-permit/plan-check-permit-special-assistance/mandatory-retrofit-programs/non-ductile-concrete-retrofit-program [accessed February 15, 2021].
[11] Lin T, Haselton CB, Baker JW. Conditional spectrum-based ground motion selection. Part I: Hazard consistency for risk-based assessments. Earthquake Engineering & Structural Dynamics 2013; 42(12): 1847–1865. DOI: 10.1002/eqe.2301.
[12] Lin T, Haselton CB, Baker JW. Conditional spectrum-based ground motion selection. Part II: Intensity-based assessments and evaluation of alternative target spectra. Earthquake Engineering & Structural Dynamics 2013; 42(12): 1867–1884. DOI: 10.1002/eqe.2303.
[13] Baker JW, Lee C. An Improved Algorithm for Selecting Ground Motions to Match a Conditional Spectrum. Journal of Earthquake Engineering 2018; 22(4): 708–723. DOI: 10.1080/13632469.2016.1264334.

[14] Zsarnóczay A, Deierlein GG. a Computational Framework for Estimating Damage , Loss and Community Resilience 2020: 0–12.
[15] Molina Hutt C, Hulsey AM, Kakoty P, Deierlein GG, Monfared AE, Wen-Yi Y, et al. Toward functional recovery performance in the seismic design of modern tall buildings. Earthquake Spectra 2021; 1(27). DOI: 10.1177/87552930211033620.
[16] RS Means Company. RSMeans Building Construction Cost Data 2021. 2021.
[17] SPUR. The resilient city: Defining what San Francisco needs from its seismic mitigation policies. San Francisco, California: 2009.
[18] Deierlein GG, Mckenna F, Zsarnóczay A, Kijewski-Correa TL, Kareem A, Lowes L, et al. A Cloud-enabled Application Framework for Simulating Regional-scale Impacts of Natural Hazards on the Built Environment. Frontiers in Built Environment 2020. DOI: 10.3389/fbuil.2020.558706.