# COLLAPSE RISK OF TALL STEEL MOMENT-RESISTING FRAMES IN DEEP SEDIMENTARY BASINS DURING LARGE MAGNITUDE SUBDUCTION EARTHQUAKES

by

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### Abstract

The Pacific Northwest has the potential to experience large-magnitude earthquakes generated by the Cascadia Subduction Zone, which is located approximately 100 km from the city of Seattle. Tall buildings in Seattle are particularly vulnerable to these earthquakes, because the city lies above a deep sedimentary basin, which can amplify the intensity of earthquake ground motions at long periods. Steel moment-resisting frames are important, because they are one of the most common structural system types in the existing tall building inventory of western US cities, and due to concerns regarding the potential for fracture-prone welded connections, which came to light following the 1994 Northridge earthquake. This thesis evaluates the response of an archetype 1970s 50-story steel moment-resisting frame office building in Seattle under 30 simulated scenarios of a magnitude-9 (M9) Cascadia Subduction Zone earthquake, which has a return period of approximately 500 years. The resulting probability of collapse, conditioned on the occurrence of the M9 scenarios considered, is 30%. The annualized collapse risk of the archetype building is also assessed considering all earthquake sources that contribute to the seismic hazard through a multiple stripe analysis. The results indicate a 50-year collapse risk of 6.9% when basin effects are neglected, and 10.5% when basin effects are considered. These results exceed by a factor of 10 the 1% in 50-year target implicit in modern seismic design standards. These high collapse risks are largely driven by: (i) deep sedimentary basin effects, which amplify long period shaking; and (ii) the expected brittle behavior of fracture-prone welded beam-to-column connections. The simulations of the performance of the building under the M9 scenarios outside of the basin or with ductile beam-to-column connections result in a negligible probability of collapse. In terms of economic impacts, the earthquake-induced repair costs of the archetype building conditioned on iii

the occurrence of the simulated Seattle M9 ground motions are estimated at 44% of building replacement cost, and the annualized losses are 0.19% of building replacement cost when basin effects are neglected versus 0.29% when basin effects are considered.

# Lay Summary

The Cascadia Subduction Zone is located approximately 100 km from the city of Seattle, WA and can generate large-magnitude earthquakes. Tall buildings in Seattle are particularly vulnerable to the ground motion characteristics of these earthquakes. Furthermore, the city lies above deep soft-soil deposits, i.e., a sedimentary basin, which can amplify the intensity of earthquake ground motions. Steel moment-resisting frames are one of the most common types of structures in the existing tall building inventory of western US cities, and include seismically vulnerable construction details, which came to light following the 1994 Northridge earthquake. This thesis aims to quantify seismic risk in existing tall steel moment-resisting frame buildings in Seattle, with a focus on life-safety (collapse risk) and economic impacts (earthquake-induced repair costs) and with explicit consideration of deep sedimentary basin effects.

# Preface

The Author was responsible for designing, implementing, and documenting the analyses presented in this thesis, with guidance from his supervisor. As part of this work, the author traveled to Seattle to meet with collaborators at the University of Washington, who provided access to the physicsbased ground motion simulations used in this study and to review existing tall building drawings at the Seattle Department of Construction and Inspection.

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# Dedication

To my partner, family and friends who stand by me everyday

## **Chapter 1: Introduction**

### 1.1 Background, Aims and Objectives

Since the 1906 San Francisco earthquake, structural engineers generally regarded steel momentresisting frame (MRF) systems as being among the most ductile and reliable seismic force-resisting systems for buildings (FEMA, 2000b). The common expectation was that, when subjected to earthquake shaking, MRFs would experience only localized damage due to ductile yielding of members and connections. This expectation led to widespread construction of this system, particularly in the high seismic regions of the western US from the 1960s through the 1990s (FEMA, 2000b; ATC, 2018). The 1994 Northridge earthquake dramatically changed perceptions of the performance of such frames, after which post-earthquake inspections revealed cracking in the beam-to-column joint welds in several dozens of low- and mid-rise, steel-frame buildings (FEMA, 2000b).

Since the late 1950s, Seattle's skyline has changed dramatically with the construction of tall buildings in the downtown area. Among the multi-faceted earthquake risks facing the city, the concentration of tall buildings and infrastructure in the densely populated downtown neighborhood raises questions about the risks to life, property, and recovery from large earthquakes. As a first step towards addressing these questions, this study develops an inventory of tall buildings in Seattle. The inventory classifies tall buildings in terms of height, age, use, and structural system characteristics. Although tall buildings are not the only structures at risk, they are of special concern due to their large size and occupancies, where earthquake damage to one tall building can

have disproportionate effects on its occupants, its neighbors, and the community at large (ATC, 2018).

The Seattle tall building inventory includes over 50 tall buildings constructed from the 1960s to the 1990s, many of which are believed to be steel MRFs. These buildings are of interest due to: (1) their prominence as one of the most common structural system types in the tall building inventory; (2) their design, which followed an equivalent lateral force procedure based on the first-mode translation response, without capacity design principles that protect against story mechanisms, and lower base-shear strengths than those specified in modern building codes; and (3) concerns regarding the potential for fracture-prone welded connections, which came to light following the 1994 Northridge earthquake.

Tall buildings are most susceptible to long-period ground motions, which are characteristic of distant, large magnitude subduction earthquakes. The vulnerability of these structures is compounded by sedimentary basins, which tend to increase the intensity of earthquake ground motions at long periods and the resulting damage in tall structures (Marafi et al., 2017). This issue is particularly important in Seattle, which lies above a deep sedimentary basin, and has the potential for large-magnitude Cascadia Subduction Zone (CSZ) earthquakes that can dominate the seismic hazard at long periods.

This thesis evaluates the impact of deep basins during large magnitude subduction earthquakes on the collapse risk of an archetypical pre-1994 tall steel MRF building in Seattle. Consistent with the tall building inventory data, a 1970s 50-story steel MRF archetype building was modeled. The archetype building was subjected to 30 simulated scenarios of a magnitude-9 (M9) CSZ interface earthquake, recently generated by Frankel et al. (2018) for an average shear-wave velocity, in the top 30-meters of soil, of 600 m/s. Marafi et al. (2019) modified these motions to account for effects for a local site with an average shear-wave velocity of 500 m/s in the upper 30-meters, which is typical of downtown Seattle.

To isolate deep basin effects, the response during the M9 simulations was also evaluated for the town of La Grande (73 km south of Seattle). This location is outside of the deep sedimentary basin, but has a similar distance to the fault-rupture plane as the Seattle site (Marafi et al., 2019). The impact of fracture-prone welded connections on the predicted building response was also quantified by re-evaluating the M9 CSZ simulations for Seattle: (i) without fracture-prone beam-to-column connections, (ii) without fracture-prone column splices, and (iii) with ductile beam-to-column and splice connections throughout the building.

To benchmark building performance under the M9 scenarios against probabilistic estimates of the seismic hazard, which include crustal, intraslab, and interface earthquake sources, building response was evaluated using a multiple stripe analysis (MSA) (Jalayer & Cornell, 2009). In MSA, each stripe consists of ground-motions with shaking intensities corresponding to return periods of 100, 475, 975, 2475, and 4975-years, which are used to evaluate the 50-year collapse risk of the structure (ASCE, 2016).

The MSA was carried out using two versions of the US National Seismic Hazard Model (NSHM): the 2014 NSHM, which neglects basin effects, and the 2018 NSHM, which explicitly considers

basin amplification. Furthermore, a hybrid seismic hazard model, which includes both empirical and simulated seismic hazard characterization, was also considered in the assessment. The hybrid hazard model, which is based on the 2018 NSHM, utilizes physics-based simulations to represent the large interface earthquake portion of the hazard and empirical relationships for all other earthquake sources (crustal and intraslab).

### **1.2 Thesis Structure**

This thesis addresses the seismic performance of older seismically deficient steel buildings in deep sedimentary basins, with a focus on collapse risk and earthquake-induced repair costs. Chapter 1 provides the background, aims and motivation of this work, as well as the overall organization of the thesis. In addition, Chapter 1 outlines the methodology adopted to address the research questions outlined in this work. These questions are addressed primarily through nonlinear dynamic analysis of an archetype steel MRF, which permits evaluating collapse risk and economic impacts.

Chapter 2 provides a brief literature review with a focus on welded steel MRFs, the existing tall building inventory in Seattle, WA, as well as the seismic hazard in the Pacific Northwest.

Chapter 3 describes ground motions used in the analysis, including the simulated M9 ground motions, as well as ground motion suites consistent with the US 2014 and 2018 NSHMs at different intensities of earthquake ground motion shaking.

Chapter 4 provides characteristics of the archetype building design such as geometry, structural detailing and other relevant parameters. Furthermore, this chapter outlines the numerical models used to obtain structural response parameters, as well as the building performance models used to estimate earthquake-induced repair costs.

Chapter 5 presents the analysis results of the building subjected to the simulated M9 ground motions and shaking at different intensity levels per the 2014 and 2018 NSHMs. The resulting annualized collapse risk and economic losses are presented in this chapter, as well as a sensitivity study on the impact of connection performance on the analysis results.

Chapter 6 provides the summary and conclusions of this work. It also describes the limitations of the present study and provides recommendations for future work.

#### **1.3 Methodology**

The main purpose of this thesis is to gain insights into the seismic performance of pre-Northridge tall steel MRF building in Seattle, WA with explicit consideration of deep sedimentary basin amplification in both empirical and simulated seismic hazard characterizations. To this end, the methodology illustrated in Figure 1-1 is adopted. Two types of building-specific seismic performance assessments are conducted: a time- or risk-based assessment, and a scenario-based assessment. The risk-based assessments rely on seismic hazard data from the 2014 and 2018 NSHMs in the US. The scenario-based assessment relies on the 30 simulated scenarios of a magnitude-9 (M9) CSZ interface earthquake generated by Frankel et al. (2018). Risk-based assessments

under a range of ground motion intensity levels, five in this study, which are then combined with the ground motion hazard curve to provide the annual rates of exceedance of a performance measure (NEHRP, 2011), e.g. annualized collapse risk ( $\lambda_c$ ) or average annual losses (AAL). Scenario-based assessments permit estimating the probability of collapse (P<sub>c</sub>|M9) and earthquakeinduced repair costs (L|M9) conditioned on the occurrence of the M9 scenarios considered.

As a first step, a 1970s 50-story steel MRF archetype is identified as plausible design believed to be representative of the pre-Northridge tall steel MRF building stock in Seattle. A nonlinear model of this archetype is used to evaluate its anticipated seismic performance via nonlinear dynamic analysis. In order to isolate basin effects, the nonlinear model is subjected to ground motion suites representative of different earthquake shaking intensities per the 2014 NSHM, which neglects basin effects, and the 2018 NSHM, with explicitly considers basin amplification on ground motion shaking. The archetype is also subjected to the simulated M9 La Grande (outside of basin) and M9 Seattle (inside of basin) ground motions.

Nonlinear dynamic analysis results are used to develop collapse fragilities consistent with each of the seismic hazard models considered. Integration of these collapse fragilities with corresponding seismic hazard curves permits estimating annualized collapse risk. These collapse risk estimates are also assessed using a hybrid seismic hazard model, which includes both empirical and simulated seismic hazard characterizations. Similarly, the earthquake-induced repair costs of the archetype building conditioned on the occurrence of the M9 scenarios are evaluated, as well as the expected average annual losses. The results permit quantifying the impact of basin amplification on the archetype pre-Northridge steel MRF collapse risk and economic loss.

To study the impact of connection performance, four variations of the archetype building are subjected to the simulated M9 ground motions. The models consider fracture-critical connections in (a) beam-to-column and column splice connections ("base model" in Figure 1-1), (b) beam-to-column connections only ("beam fracture" in Figure 1-1) and (c) column splice connections only ("splice fracture" in Figure 1-1), as well as a model (d) without fracture-critical welded connections ("ductile model" in Figure 1-1). The results permit evaluating the impact of connection performance on the probability of collapse and earthquake-induced repair costs.



Figure 1-1. Methodology to evaluate (1) the impact of M9 Cascadia earthquakes and deep sedimentary basins on the collapse risk and economic loss of pre-Northridge steel moment frames, as well as (2) the sensitivity of results to connection modelling.

## **Chapter 2: Literature Review**

This chapter provides an overview of welded steel MRFs, the existing tall building inventory in Seattle, WA, as well as the seismic hazard in the Pacific Northwest.

### 2.1 Welded Steel Moment Resisting Frame

In the 1960s, engineers began to regard welded steel MRFs as being among the most ductile systems contained in the building code (FEMA 2000a). It was thought that when subject to earthquake shaking, these structural systems would have limited damage due to ductile yielding of members and connections. Engineers believed the potential collapse of such structures was near impossible. This perception led to widespread construction of many welded steel MRFs particularly in the western United States (FEMA 2000a). Following the 1994 Northridge earthquake, many steel MRFs experienced brittle fractures of beam-to-column connections, even in areas with moderate ground-motion shaking. Damage surveys indicated that brittle fractures initiated within the connections at low levels of plastic demand, or in some cases within the elastic range, inducing a significant loss in strength and stiffness (Youssef et al. 1995). Prior to Northridge, a few successes, or rather the lack of any drastic failures, established the reputation that welded steel MRFs had particularly good earthquake resistance characteristics (FEMA 2000b). Until Loma Prieta in 1989, only a handful of modern steel MRFs had been shaken by a major quake. Fewer than a dozen were closely inspected after the 1985 Mexico City, 1971 San Fernando and 1964 Prince William Sound earthquakes combined (FEMA 2000b).

Following the 1994 Northridge earthquake, the SAC Joint Venture, the American Institute of Steel Construction (AISC), the American Iron and Steel Institute (AISI) and the National Institute of Standards and Technology (NIST) convened an international workshop to coordinate efforts to investigate and solve this challenge (FEMA 2000c), which led to a massive research program funded by the US Federal Management Emergency Agency (FEMA). As a result, many studies have been conducted to assess the performance of pre- and post-Northridge welded steel MRFs (Gupta and Krawinkler 1999, Seung-Yul et al. 2002, Medina and Krawinkler 2005, and Maison and Bonowitz 1999 amongst others). These studies focused on structural performance alone, which provides valuable information for the structural engineering community, but fails to provide measures of risk associated with direct economic losses or other metrics used by stakeholders for decision making.

During the 1994 Northridge earthquake, tall buildings in downtown Los Angeles were not directly affected by the event. Following the earthquake, reconnaissance missions (EEFIT 1994, EERI 1994) were unaware of damage to welded steel MRFs. Subsequent inspections revealed that there had been cracking in the beam-to-column joint welds in low and medium rise structures, though none in the tall buildings in the downtown area due to considerably lower seismic demands. In 1995, the Los Angeles City Council passed ordinance number 170406, mandating connection inspections and repairs in welded steel MRF buildings. The final ordinance covered a specific geographic area that excluded some parts of the city, including the downtown area were where tall buildings were clustered (FEMA 2000b). Outside of Southern California, the Northridge damage prompted investigations of some welded steel MRF buildings in San Francisco that had been

subject to strong ground motions in the 1989 Loma Prieta earthquake. However, no inspections were mandated. To the author's knowledge, no inspections took place in Seattle, WA.

### 2.2 Tall Building Inventory in Seattle, WA

In order to characterize the seismic risk posed by Seattle's tall building stock, this study compiled an inventory of tall buildings in the city. For the purposes of this study, a 73 m (240 ft) threshold on building height was selected to define "tall" buildings, based on the city's requirement that buildings above this height, which do not adopt a dual system for lateral resistance, be evaluated following a performance-based seismic design approach (Director's Rule 5, 2015). These performance-based design procedures employ nonlinear analysis to evaluate expected building response to large earthquakes and, thereby, provide greater assurance that buildings meet the performance targets intended in current building codes to minimize life safety risks under extreme earthquakes.

While 73 m (240 ft) was a convenient threshold to define the initial scope of the inventory, this height is arbitrary as all buildings are vulnerable to earthquake damage and have implications on the seismic resilience of the city regardless of their height. From a structural response perspective, tall buildings have unique seismic response characteristics including: fundamental translation periods of vibration well in excess of 1 second, significant mass participation and lateral response in higher modes of vibration, as well as seismic resisting systems with slender aspect ratios (PEER, 2017). These characteristics result in distinct seismic behavior when compared to low- and midrise construction. However, the 73 m (240 ft) height does not have any empirical or scientific basis in how buildings respond to earthquakes (ATC, 2018). The compiled inventory here focused on

tall buildings constructed in the 1960s through the 1990s due to the prevalence of steel MRF construction in high seismic regions of the western US during this era (Molina Hutt et al., 2019).

Three primary data sources enabled the development of the tall building inventory: Emporis (Emporis, 2018), King County Open Data (King County, 2018), and the Seattle Department of Construction and Inspection (SDCI, 2019). Initially, a list of existing buildings, within the city of Seattle and greater than 73 m (240 ft) in height, was assembled. This list included building name, address, height, number of floors, year of construction and construction material. A total of 116 tall buildings were identified that had been constructed since 1904. Of these buildings, a total of 53 buildings were taller than 73 m (240 ft) in height and were constructed from the 1960s to the 1990s, as illustrated Figure 2-1. Approximately 64% of these buildings, 34 in total, use steel as its primary construction material. A similar study in San Francisco, identified 156 buildings over 73 m (240 ft) in height with approximately 50% adopting a steel MRF configuration, and almost 90% of those steel MRFs constructed in the 1960s, 1970s and 1980s (ATC, 2018).

Figure 2-1 highlights the predominance of steel-frame construction during the 1960s, 1970s and 1980s for buildings taller than 240 ft (73 m), particularly for buildings with 35 or more stories. In total, all 18 of the buildings with at least 35 stories were classified as having a steel lateral-force-resisting system. Eight of these buildings were confirmed to be steel MRFs, based on a review of construction permit documents at the Seattle Department of Construction and Inspection (SDCI, 2019). Typical details observed in the drawing sets indicated that the beam-to-column connections were consistent with those that fractured during the 1994 Northridge earthquake. Figure 2-1b suggests that steel buildings became less common in the 1990s.



Figure 2-1. Seattle tall buildings constructed from the 1960s to the 1990s by: (a) percentage of buildings per construction material, (b) breakdown of construction material per decade, and (c) distribution of construction material per number of stories. Data Sources: Emporis (2018), King County (2018) and SDCI (2019).

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Approximately 80% of the steel buildings, taller than 73m (240 ft) and constructed from the 1960s to the 1990s, are commercial, and just under 20% are residential or hotels. Whereas the structural system and construction date are relative indicators of expected seismic performance, the occupancy type provides insights into the services that tall buildings within the inventory provide to the community. Over 47% of the Seattle tall building inventory was constructed from the 1960s to the 1990s. As seen in Figure 2-1, tall building construction slowed down in the 1990s, but has resurged since then (Marafi et al., 2020). The number of tall buildings in Seattle constructed since the year 2000 has now surpassed the number of tall buildings constructed from the 1960s to the 1990s. Modern buildings are predominantly reinforced concrete shear walls and serve primarily a residential occupancy type.

#### 2.3 Seismic Hazard in the Pacific Northwest

The Pacific Northwest has the potential to experience crustal, intraslab (deep) and interface earthquakes, such as large-magnitude earthquakes generated by the Cascadia Subduction Zone (CSZ), which is located approximately 100 km from the city of Seattle, WA. Furthermore, the city lies above a deep sedimentary basin, which can amplify the intensity of earthquake ground motions (Choi et al., 2005; Marafi et al., 2017; Morikawa and Fujiwara, 2013).

Past studies have shown that recorded motions have spectral accelerations that are larger in deep sedimentary basins than in surrounding locations (Bozorgnia et al., 2014; Choi et al., 2005; Marafi et al., 2017; Morikawa and Fujiwara, 2013). The effects of deep sedimentary basins on ground-motion characteristics have also been observed in physics-based simulations of earthquake ground motions (Aagaard et al., 2010; Frankel et al., 2018; Graves et al., 2011; Moschetti et al., 2017;

Wirth et al., 2018). Current seismic design provisions in the US, i.e. ASCE 7-16 (ASCE, 2016), were developed according to the 2014 National Seismic Hazard Model (NSHM) (Petersen et al., 2014), which neglects basin effects. In the western United States, deep basins underlie large metropolitan areas, including Seattle, Los Angeles, Salt Lake City, and the San Francisco Bay area. As a result, the most recent version of the national seismic hazard model, the 2018 NSHM, accounts for basin amplification on spectral accelerations in these areas and its adoption in future design standards would result in a considerable increase in seismic design spectral accelerations.

The 2018 NSHM accounts for basin effects on spectral acceleration for all earthquake sources using basin terms adapted from the crustal earthquake ground-motion models in the NGA West2 project (Bozorgnia et al., 2014). The proxy for basin depth is the depth from the surface to a layer with a shear-wave velocity of at least 1.0 km/s or 2.5 km/s, denoted as  $Z_{1.0}$  and  $Z_{2.5}$ , respectively. Compared to the other basins in the western United States, Seattle has the largest values of  $Z_{2.5}$ , which is equal to 6.9 km (Stephenson et al., 2017), as illustrated in Figure 2-2.



Figure 2-2. Map of Z<sub>2.5</sub> for the Puget Lowland Region. Figure Source: Marafi et al. (2019).

While the basin amplification terms used in the 2018 NSHM were developed by considering crustal earthquakes, interface earthquakes represent the largest contribution to the seismic hazard in Seattle, particularly at periods greater than 1 second. Frankel et al. (2018) generated 30 simulated scenarios of a magnitude-9 (M9) CSZ interface earthquake explicitly considering amplification of ground motion shaking due to the Seattle basin. Comparing response spectra developed per the 2018 NSHM and M9 simulations revealed that 2018 NSHM underestimated the basin effect on spectral acceleration at periods longer than 1 second in the Seattle area (Frankel et al., 2018; Marafi et al., 2019b). In addition, M9 simulated ground motions can be more damaging because of their longer duration and more damaging spectral shape compared to the ground motions selected to represent the 2018 NSHM particularly in the period range of 0.5-3.0 seconds (Marafi et al., 2020b).

### **Chapter 3: Seismic Hazard and Ground Motion Selection**

In order to evaluate the annualized collapse risk of the 1970s archetype building, a multiple stripe analysis (MSA) procedure (Baker, 2015) was adopted. In MSA, structural assessments are performed at a series of ground motion intensities (or "stripes") spanning from high to low probability of occurrence. The lower- and upper-bound intensity measure levels considered cover a range from negligible damage to complete loss. In this work, each stripe consists of ground-motions with shaking intensities corresponding to return periods of 100, 475, 975, 2475, and 4975-years, which are used to evaluate the 50-year collapse risk of the structure (ASCE, 2016). Nonlinear dynamic analyses were conducted with ground motion suites representative of each intensity level. The analysis results were then linked back to probabilistic seismic hazard data, which enables calculating a range of risk metrics, including the annualized collapse risk.

### 3.1 2014 and 2018 US National Seismic Hazard Model

In this study, the intensity measure used in the MSA was the spectral acceleration at 5.5 seconds, which corresponded to the calculated fundamental period ( $T_1$ ) of the archetype building. The MSA utilized median spectral acceleration over all orientations ( $SA_{RotD50}$ ) to permit integration with probabilistic seismic hazard data. The intensity stripes used in the MSA had return periods of 100, 475, 975, 2475, and 4975-years. The probability of collapse from each intensity level, and corresponding earthquake return period, were then integrated over the  $SA_{RotD50}(T_1)$  seismic hazard curve to estimate the probability of building collapse over a period of 50 years, i.e., the assumed design life of a building. This range of earthquake return periods accounted for low-intensity

shaking, where structural collapse is less likely, and high-intensity shaking where structural collapse is more likely.

A conditional mean spectrum (CMS) (Baker, 2011) was used to represent the expected ground motion response spectrum conditioned on the occurrence of a target spectral acceleration at the fundamental period of the archetype building. The CMS at the conditioning period was calculated as a weighted average of the CMS for each ground-motion model and particular seismic source, e.g., Seattle fault, according to its percentage contribution to the hazard, as obtained by seismic hazard deaggregation results. The spectral acceleration correlation functions used to calculate the CMS were assumed to be the same for crustal, intraslab, and subduction earthquakes (Jayaram et al., 2011; Baker & Jayaram, 2008). For each of the 15 combinations of return period and type of source mechanism, the CMS at the conditioning period was computed using the 2014 and 2018 NSHMs (USGS, 2019) for the downtown Seattle location.

While the 2014 NSHM does not explicitly account for basin effects, the 2018 NSHM does account for the effects of sedimentary basins in Seattle, Los Angeles, San Francisco Bay Area, and Salt Lake City regions. A common proxy for basin depth is the depth from the surface to a layer with a shear-wave velocity of 1.0 and 2.5 km/s, denoted as  $Z_{1.0}$  and  $Z_{2.5}$ . Basin effects on spectral acceleration are accounted for in the 2018 NSHM in all earthquake sources using basin terms (as a function of  $Z_{1.0}$  and  $Z_{2.5}$ ) adapted from the crustal earthquake ground-motion models in the NGA-West2 project (Petersen et al., 2019). Table 3-1 summarizes the spectral acceleration (at  $T_1$ =5.5s) and seismic hazard contribution per the 2014 and 2018 NSHMs of each source mechanism at each intensity level considered in the assessment. For a return period of 975 years, the spectral acceleration for the 2018 NSHM was approximately 80% higher than that for the 2014 NSHM. Figure 3-1 shows the 5.5-second period CMS for each source mechanism for earthquake return periods of 100, 475, 975, 2475, and 4975-years for the 2014 and 2018 NSHMs. As seen in Figure 3-1, even though the spectral acceleration at the conditioning period is the same for crustal, interface and intraslab sources for a given return period, the spectral shape associated with each source is unique. Furthermore, as the return period increases the corresponding spectral acceleration values also increase. For each return period considered, ground motions were selected with explicit consideration of the seismic hazard contribution of each of the three distinct types of source mechanisms: crustal, interface, and intraslab.

Table 3-1. Percentage contribution of each source mechanism (crustal, interface and intraslab) per the 2014 and 2018 National Seismic Hazard Models (NSHM) at each intensity level considered in the assessment (100, 475, 975, 2475 and 4975-year return periods).

	$SA_{RotD50}(T_1=5.5s)$		Source Mechanism Contribution (%)					
Return Period (years)	(g)		Crustal		Interface		Intraslab	
	2014 NSHM	2018 NSHM	2014 NSHM	2018 NSHM	2014 NSHM	2018 NSHM	2014 NSHM	2018 NSHM
100	0.01	0.01	45	36	19	29	36	35
475	0.03	0.05	38	29	51	60	11	11
975	0.05	0.09	33	25	61	70	6	5
2475	0.09	0.14	27	20	69	77	4	3
4975	0.12	0.20	25	17	71	80	4	3

To capture the inter-event uncertainty in the conditional mean spectra, motions were selected and scaled to match the target mean and variance Conditional Spectra (CS) (Jayaram et al., 2011). The CS was used to establish site- and structure-specific target spectra for selection and scaling of input ground motions for the nonlinear dynamic analyses. Figure 3-2 to Figure 3-6 show the response spectra for 100 motions selected to represent the three types of earthquake source mechanisms for all intensity levels considered in the assessment conditioned at a 5.5-second period. Motions were selected to have spectral ordinates that are within two standard deviations of the target conditional mean spectra whilst achieving the target mean spectral acceleration and target variance at each period. Figures 3-2 to 3-6 showcase this hazard-consistent ground motion selection procedure adopted, where crustal, interface and intraslab ground motion records are selected in proportion to the relative contribution of each seismic source to the hazard. These figures also permit comparing changes in the hazard contribution of each seismic source per the 2014 and 2018 NSHMs at each return period considered. For instance, the contribution of interface earthquakes at the 100-year return period intensity level per the 2014 NSHM is 19%, whereas this contribution increases to 29% per the 2018 NSHM.

Ground motions recorded from crustal, intraslab, and interface earthquakes were included in each ground motion set in proportion to their contribution to the overall seismic hazard at the conditioning period, as seen in Table 3-1. It should be noted that the archetype for each intensity stripe was only subjected to ground motions conditioned at the fundamental period of the structure (5.5 seconds). This approach was found to sufficiently capture the full range of modal response in the analyses by Lin and Baker (2013). Marafi et al. (2020) provides further details of the ground-motion selection and scaling process.


Figure 3-1. Conditional mean spectrum conditioned at a 5.5-second period for crustal, interface and intraslab earthquakes at the 100, 475, 975, 2475 and 4975-year hazard levels according to the (a) 2014 and (b) 2018 National Seismic Hazard Models.



Figure 3-2. Ground motions scaled to match the 5.5-second period conditional mean and variance spectra for crustal, interface and intraslab earthquakes at the 100-year hazard levels according to the (a) 2014 and (b) 2018 National Seismic Hazard Model.



Figure 3-3. Ground motions scaled to match the 5.5-second period conditional mean and variance spectra for crustal, interface and intraslab earthquakes at the 475-year hazard levels according to the (a) 2014 and (b) 2018 National Seismic Hazard Model.



Figure 3-4. Ground motions scaled to match the 5.5-second period conditional mean and variance spectra for crustal, interface and intraslab earthquakes at the 975-year hazard levels according to the (a) 2014 and (b) 2018 National Seismic Hazard Model.



Figure 3-5. Ground motions scaled to match the 5.5-second period conditional mean and variance spectra for crustal, interface and intraslab earthquakes at the 2475-year hazard levels according to the (a) 2014 and (b) 2018 National Seismic Hazard Model.



Figure 3-6. Ground motions scaled to match the 5.5-second period conditional mean and variance spectra for crustal, interface and intraslab earthquakes at the 4975-year hazard levels according to the (a) 2014 and (b) 2018 National Seismic Hazard Model.

### 3.2 Empirical and Simulated Seismic Hazard Characterization

Frankel et al. (2018) produced 30 sets of broadband (0-10 Hz) synthetic seismograms for M9 CSZ earthquakes by combining synthetic seismograms derived from 3D finite-difference simulations ( $\leq 1$  Hz) with finite-source stochastic synthetics ( $\geq 1$  Hz). These three-dimensional simulations, which considered a variety of rupture parameters to determine the range of expected ground motions, were used in this study. The resulting ground motion spectra for sites outside sedimentary basins, i.e., La Grande: 46.84°N, -122.32°W, were consistent with existing ground motion models (GMM) (Abrahamson et al., 2016).

The response spectra from the synthetics at sites within sedimentary basins, i.e. Seattle 47.60°N, -122.30°W, show amplification factors that ranged from 2 to 5 between periods of 1 to 10 seconds, as observed in Figure 3-7 by comparing the average response spectra for all 30 synthetics in Seattle vs. La Grande. A similar increase in spectral acceleration in deep basins were also observed in recorded motions during interface earthquake (Morikawa & Fujiwara, 2013 and Marafi et al., 2017).

As illustrated in Figure 3-7, for periods in excess of 1 second, the mean of the 30 M9 earthquake scenarios for Seattle fall within the 975 and 2475-year probabilistic estimates of the hazard when basin effects are considered, i.e. 2018 NSHM, and exceed the 2475-year probabilistic estimate of the hazard when basin effects are neglected, i.e. 2014 NSHM. These intensity levels were used to benchmark building performance under the M9 scenarios.



Figure 3-7. Average spectral acceleration (*SA*<sub>RotD50</sub>) response spectra of ground motion suites considered in the assessment, including magnitude-9 (M9) Cascadia Subduction Zone simulations in Seattle and La Grande, and probabilistic ground motion shaking consistent with a 975 and 2475-year return period according to the 2014 and 2018 National Seismic Hazard Models (NSHM).

The response of the 1970s archetype was evaluated under ground motion records grouped into six suites:

- (1) La Grande M9 CSZ simulations ("M9 La Grande")
- (2) Seattle M9 CSZ simulations ("M9 Seattle")
- (3) 975-year return period conditional spectra without basin effects ("975-year 2014 NSHM")
- (4) 975-year return period conditional spectra with basin effects ("975-year 2018 NSHM")
- (5) 2475-year return period conditional spectra without basin effects ("2475-year 2014 NSHM")
- (6) 2475-year return period conditional spectra with basin effects ("2475-year 2018 NSHM").

Figure 3-8 provides a comparison of key ground motion parameters for each suite, including arias intensity ( $I_A$ ), significant duration ( $Ds_{5-95}$ ), spectral acceleration at the fundamental period ( $SA_{RotD50}(T_1)$ ) and spectral acceleration at the second mode of vibration ( $SA_{RotD50}(T_2)$ ) of the archetype building. The average  $I_A$  of the Seattle motions were found to be 49.6% (3.42 m/s) higher than La Grande (2.06 m/s), whereas the median and standard deviation of  $Ds_{5-95}$  was not found to significantly change. Furthermore, the log-normal standard deviation of  $SA_{RotD50}(T_1)$  increased to 0.03 in Seattle from 0.01 in La Grande. Including basin effects in probabilistic seismic hazard calculations (i.e., 2018 vs. 2014 NSHM) for the 975-year and 2475-year ground motion suites increased  $SA_{RotD50}(T_1)$  and  $SA_{RotD50}(T_2)$  (median increased by 62% on average) but had no significant effect on  $I_A$  and  $Ds_{5.95}$ . Figure 3-9 and Figure 3-10 provide a comparison of median acceleration time histories over all orientations between the simulated M9 earthquake scenarios in La Grande and Seattle. Similarly, Figure 3-11 and Figure 3-12 provide a comparison of  $SA_{RotD50}$  response spectra between the simulated M9 earthquake scenarios in La Grande and Seattle.

Figures 3-9 and and 3-10 showcase the variability in the ground motion shaking associated with the thirty M9 earthquake scenarios considered. For instance, by comparing arrival times in the acceleration time history of Scenario 1 vs. Scenario 3, it can be inferred that Scenario 1 has a closer site-to-source distance than Scenario 3. Similarly, by comparing peak ground accelerations, e.g., Scenario 29 vs. Scenario 30, the impact of directivity, e.g., rupture towards or away from the sites of interest, and other effects on the the resulting ground motion records is observed. While some differences in the resulting acceleration time histories are seen as a result of the Seattle basin, Figures 3-11 and 3-12, which illustrate the resulting response spectra, truly showcase the impact of deep basins on spectral acceleration amplitudes, particularly at periods longer than 1 second.



Figure 3-8. Histogram plots of (a) arias intensity (IA), (b) significant duration (Ds5-95), (c) spectral acceleration at fundamental period (SA<sub>RotD50</sub>(T<sub>1</sub>)), and (d) spectral acceleration at second period (SA<sub>RotD50</sub>(T<sub>2</sub>)), for ground motion suites considered in the assessment, including simulated M9 (Seattle and La Grande), and ground motion shaking consistent with a 975 and 2475-year return periods with (2018 NSHM) and without (2014 NSHM) basin effects (y axes denote ground motion count).



Figure 3-9. Median acceleration time histories comparing M9 Seattle (black) to M9 La Grande (red) for scenarios 1-16.



Figure 3-10. Median acceleration time histories comparing M9 Seattle (black) to M9 La Grande (red) for scenarios 17-30.



Figure 3-11. Median spectral acceleration over all orientations comparing M9 Seattle (black) to M9 La Grande (red) for scenarios 1-16.



Figure 3-12. Median spectral acceleration over all orientations comparing M9 Seattle (black) to M9 La Grande (red) for scenarios 17-30.

# **Chapter 4: Archetype Building Design and Numerical Modelling**

### 4.1 Seismic Design Requirement

Changes to the seismic design of buildings over the past century offer important clues to interpreting the building database presented in the previous section. The Uniform Building Code (UBC) was first published in 1927 to help promote public safety and standardized construction. The code was updated every three years until 1997, which was the final version before the introduction of the International Building Code (IBC) in 2000. One significant change over the course of the UBC revisions was the inclusion of seismic design code provisions. From 1927 to 1961, the UBC did not include any mandatory seismic design requirements. Mandatory code provisions were eventually introduced via recommendations published in 1959 by the Structural Engineers Association of California (SEAOC) in "Recommended Lateral Force Requirements and Commentary" (SEAOC, 1973), informally referred to as the "Blue Book". The Blue Book established performance goals that still underlie today's building codes. The code's minimum design provisions were intended to provide earthquake safety. Most were based on fundamental engineering principles, but many, including height limits and height-related details, were set by consensus engineering judgment (ATC, 2018).

Analysis of the Seattle tall building inventory suggests that from the 1960s to the 1980s steel MRFs were a prevalent lateral-resisting system in the construction of buildings greater than 73 m (240 ft) in height. The goal of this study is to evaluate the impact of deep basins during large magnitude subduction earthquakes on the collapse risk of these pre-Northridge tall steel MRF buildings.

Collapse risk metrics of older seismically vulnerable tall buildings are benchmarked against modern design targets. The study focuses on the evaluation of an archetype 50-story steel MRF building designed following the requirements of the 1973 UBC (UBC, 1973), hereinafter referred to as the 1970s archetype. The 1970s archetype is intended to represent a plausible design from the mid-1970s to the mid-1980s.

The building occupancy was assumed to be that of a commercial office building, with two levels for mechanical equipment, one at mid-height, and one at the top floor. The frames are made up of built-up box columns (denoted R in Figure 4-1), wide flange beams, and welded flange-bolted web beam-to-column connections, based on trends observed in drawings of existing tall steel MRF buildings. Figure 4-1 shows the elevation view of the archetype. Since the building is designed and located in Seattle, all the units shown in Figure 4-1 are imperial. For consistency the section sizes in this figure are based on American standards. In the 1970s, it was customary to have moment connections at all beam-to-column intersections, as illustrated in Figure 4-2. Typical story heights and beam spans are 3.8 m (12.5 ft) and 8.5 m (28 ft), respectively. At the lobby level the story height is 6.1 m (20 ft), hence resulting in a total height of the structure of 192.8 m (632.5 ft) above grade. The building enclosure is assumed to be composed of precast concrete panels and glass windows, a floor system composed of concrete slab 76.2 mm (3 in.) over metal deck 63.5 mm (2.5 in.) supported by steel beams of ASTM A36 [248 MPa (36 ksi)], and steel columns of ASTM A572 [345MPa (50 ksi)]. The configuration is consistent with that presented in (Molina Hutt et al., 2019) but designed for a Seattle site.



Figure 4-1. 1970s archetype steel moment-frame building, elevation. Adapted from Molina Hutt et al. (2019).



(a)



Figure 4-2. 1970s archetype steel moment-frame building (a) plan, and typical details (adapted from Molina Hutt et al., 2016) including (b) beam-to-column connection plan, (c) splice connection detail (d) beam-to-column connection elevation.

With regards to the seismic design requirements, the 1973 UBC followed an equivalent lateral force procedure, with minimum earthquake design forces (as specified in section 2314 of the 1973 UBC), as defined in Equations 4-1 and 4-2.

$$V = Z \cdot K \cdot C \cdot W \tag{4-1}$$

$$C = \frac{0.05}{T^{1/3}} \tag{4-2}$$

where V is the total lateral force, Z is a numerical coefficient based on the seismic zone, K is the horizontal force factor, prescribed by the code as a function of the lateral resisting system type, and C is a numerical coefficient for base shear determined as a function of the fundamental period of the structure (T). W is the effective seismic weight, defined as the total dead load.

Major cities in the west coast of the US, including Los Angeles, Seattle, San Francisco, San Diego and Oakland, all correspond to Seismic Zone 3 per the 1973 UBC, for which Z takes a value of 1. The value of K for buildings with a ductile space MRF system (as specified in Table 23-I of the 1973 UBC) takes a value of 0.67. The value of T in MRF systems used for computing the value of C, is equal to 0.1 times the number of stories, which corresponds to a period of 5 seconds for a 50story building. The resulting value of C is 0.029. Therefore, the resulting seismic lateral force for design is just under 2% of the effective seismic weight. It should be noted that the 1973 UBC seismic load calculations (discussed above) did not consider the effects of local site amplification or the effects of basins. The 1973 UBC does not specify drift limits, however, this paper follows drift limit recommendations from Appendix D of the Blue Book (SEAOC, 1973), which recommends that buildings taller than 13 stories satisfy a story drift ratio of 0.0025 for wind and 0.005 for seismic (based on elastic design drifts) (Molina Hutt, 2017). The design of the archetype building was controlled by the seismic drift requirements, resulting in relatively low strength capacity utilization ratios under the wind and seismic design forces. The resulting section sizes for a typical frame are shown in Figure 4-1. Additional details of the basis of design can be found in (Molina Hutt, 2017).

It is well established that changes in commonly used weld processes during the mid-1960s led to welds with low toughness, as evidenced by weld fractures observed in the 1994 Northridge earthquake (FEMA, 2000b). Therefore, it is assumed that fracture- prone pre-Northridge moment connections are present in the 1970s archetype. In addition, the 1970s archetype does not include consideration of panel zone flexibility or strong column-weak beam principles because the panel zone model proposed by Krawinkler was not developed until 1978 (PEER, 2010) and strong column-weak beam requirements were not introduced in the UBC provisions until 1988 (SAC, 2000). The archetype employs column splice connections that are typical of that era (Figure 4-2c), consisting of partial joint penetration welds of roughly half the thickness of the smaller connected section. These splices are typically located 1.2 m (4 ft) above the floor level at approximately every third floor.

As outlined in Molina Hutt et al. (2019), a review of 1973 UBC seismic design requirements against modern design standards highlights a number of design considerations not present in the 1970s, including: (1) use of response spectrum analysis method as opposed to equivalent lateral

force procedures; (2) consideration of lateral forces acting simultaneously in both building directions; (3) minimum base shear requirements; (4) consideration of p-delta effects; (5) consideration of accidental torsion and vertical and horizontal irregularities; (6) strong column weak beam requirements; (7) panel zone design checks; (8) capacity design principles; and (9) prequalified seismic connection details.

#### 4.2 Nonlinear Model Analysis

In order to conduct nonlinear dynamic analysis of the archetype building, finite element models were developed that were capable of capturing the response of all structural elements that significantly contribute to the strength and stiffness of the system (Molina Hutt et al., 2016, 2019). A two-dimensional numerical model of a representative frame was developed in LS-DYNA (LSTC, 2011). The dynamic properties of the two-dimensional model match closely with those of the three-dimensional, linear model developed to design the building, in which the first ( $T_1$ ), second ( $T_2$ ) and third ( $T_3$ ) modes are 5.5, 2.1 and 1.3 seconds, respectively.

Key structural elements considered in the nonlinear model include the beams, columns and panel zones. Component models representing the nonlinear response of these components were calibrated against available experimental test data for validation so as to capture the full range of deterioration in strength and stiffness, from the onset of nonlinearity up to the point sidesway instability. Beams were modelled as lumped-plasticity beam elements following recommendations in Lignos et al. (2011), which propose empirical relationships for modelling steel beams, based on a large database of experiments. These guidelines define the moment-rotation response as a function of the yield moment,  $M_y$ , pre-capping plastic rotation,  $\theta_p$ , post-capping plastic rotation,

 $\theta_{pc}$  and cumulative plastic rotation capacity,  $\Lambda$ , as seen in Figure 4-3a, all of which can be defined as a function of beam geometry and expected material properties. To account for fracture in the moment connections, a plastic rotation threshold,  $\theta_{fracture}$ , at which fracture is set to occur in the connections is introduced according to the ASCE 41 recommendations (ASCE, 2013), as seen in Figure 4-3b. The impact of introducing the plastic rotation threshold at fracture is observed by comparing the hysteretic response for a pre- (Figure 4-3b) versus post-Northridge (Figure 4-3a) sample moment connection.

Columns were modelled as lumped-plasticity beam elements with yield surfaces capable of capturing interactions between bending moment and axial force following the recommendations of Lignos & Krawinkler (2010). These models were calibrated based on experimental tests of tubular steel columns in Kurata et al. (2005), which account for varying rates of degradation in the moment-rotation response of columns as a function of axial load-to-capacity ratios. Column splices were modeled by inserting lumped plasticity hinges with strengths equal to the expected splice strength under tension and/or bending. These splices are capable of reaching their expected capacity followed by brittle failure, with the intent to capture the limited ductility observed in experimental tests on heavy steel section welded splices observed by Bruneau and Mahin (1990) (Bruneau, M. & Mahin, 1990). Full column capacity was assumed in compression since this is achieved by direct bearing. Panel zones were modeled using the Krawinkler model as outlined in PEER (2010), which incorporates an assembly of rigid links and rotational springs to represent the true dimensions of the panel zone. The nonlinear springs were calibrated to capture the trilinear shear force-deformation relation based on the geometric and material properties, assuming nondegrading panel zone shear behavior.







Figure 4-3. Hysteretic moment rotation response for (a) a sample beam-to-column connection showing analytical versus experimental (Al-Shawwa and Lignos, 2013) results and (b) an identical section with fracture prone pre-Northridge connection behavior, modeled per ASCE 41 (ASCE, 2013) recommendations. Figure adapted from Molina Hutt (2017)

Analytical models were subjected 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. Because the 1970s archetype adopts a space-frame configuration, the seismic mass (weight) stabilized by each frame is approximately equal to the tributary gravity load carried by the frame. A fixed base was assumed at ground level and 2.5% of critical viscous damping was assumed in the analysis (PEER, 2017). The damping model used in the analysis provides constant 2.5% viscous damping at a period range from 1 to 10 seconds. Additional details of the modelling approach can be found in Molina Hutt (2017).

### 4.3 Building Performance Model

Owners, insurers and financial institutions often use quantitative statements of probable building repair cost expressed as a percentage of building replacement value. In risk-based assessments, future repair costs can be converted to present dollars based on an assumed discount rate. Repair costs are expressed as a percentage of building replacement cost based on the gross square footage with an assumed unit cost of \$3,550/m<sup>2</sup> (\$330/ft<sup>2</sup>) (Molina Hutt et al., 2019). At each earthquake ground motion intensity level considered in the MSA, two thousand loss simulations are calculated. For each realization, the losses are calculated as follows: (i) engineering demand parameters, e.g. peak story drifts and accelerations, are estimated from the results of nonlinear dynamic analyses; (ii) fragility functions are used in conjunction with engineering demand parameters to determine the associated damage state for each component (structural and non-structural); (iii) consequence functions are then used to translate damage states into repair costs (FEMA, 2012). The direct economic losses for each realization are then estimated by conducting this calculation for every component at every story throughout the building.

Structural component quantities are based on the structural framing of the archetype building, as previously described in Section 4.1. Non-structural component quantities are estimated based on typical quantities found in buildings of similar occupancy using the FEMA P58 Normative Quantity Estimation Tool (FEMA, 2012). Non-structural components are assumed to be non-seismically rated as there was no consideration of seismic design of non-structural components in the 1970s building codes. A complete list of assumed structural and non-structural components can be found in Molina Hutt (2017).

The damage and repair costs are calculated using the software, SP3 (SP3, 2020), which employs the FEMA P58 methodology and component damage and loss functions. The building performance models are consistent with those utilized in Molina Hutt et al. (2019). Excessive residual story drifts are considered through a fragility function with a median value of 1% and a dispersion of 0.3 (SP3, 2020) to account for cases where the building is assumed to be damaged beyond repair. Similarly, total loss due to building collapse is considered through the collapse fragility determined from the nonlinear dynamic analysis results, as discussed later in Section 5.3.

## **Chapter 5: Analysis Results**

In this study, only sidesway collapse was considered, in which structural collapse is assumed to occur when the lateral displacement of a story or number of stories, due to p-delta effects and component deterioration, causes dynamic instability. This dynamic instability occurs when the lateral displacement of the structure increases without bounds.

### 5.1 Conditional Collapse Risk

The simulated M9 CSZ earthquake scenarios with basin effects, i.e. Seattle, resulted in a 30% chance of collapse. Response under probabilistic estimates of the hazard, with consideration of basin effects, indicated a 33% chance of collapse under ground motion shaking intensities with a 975-year return period, and 80% collapse probability under a 2475-year return period. In contrast, the simulated M9 CSZ earthquake scenarios without basin effects, i.e. La Grande, resulted in negligible collapse risk. Response under probabilistic estimates of the hazard, without consideration of basin effects, indicated a 25% chance of collapse under ground motion shaking intensities intensities with a 975-year return period, and 35% collapse probability under a 2475-year return period.

The results suggest the M9 ground motion simulations with basin effects are consistent with ground motion shaking intensities with a 975-year return period, when basin effects are considered and 2475-year return period, when basin effects are neglected. Comparison of M9 simulation results inside and outside the basin highlight a drastic impact of basin amplification on seismic

risk to existing tall steel MRF buildings. Contrasting probabilistic estimates of the hazard with and without consideration of basin effects also highlight how neglecting basin effects can significantly underestimate collapse risk by a factor of 1.3 at shaking with a 975-year return period, and a factor of 2.3 at a 2475-year return period. These trends are consistent with those of Marafi et al. (2020).

Figure 5-1a and Figure 5-1b illustrate the peak story drift profile and peak floor acceleration of the median of all ground motion simulations in each suite considered. A large concentration of deformation in the top stories is observed, with the remainder of the structure showing significantly lower levels of drift demand. Similar concentrations of deformation in a small number of stories were observed in simulations other than the median. While these concentrations of deformation were also observed at different locations, e.g. near the base or mid-height, depending on the ground motion input, their concentration atop the building is attributed to the inadequate consideration of higher mode effects in the design of the 1970s building, which follows an equivalent lateral force procedure, as previously discussed. Plots representing peak story drift and peak floor acceleration of non-collapsed ground motions in each suite can be found in Appendix A.

The results under 2475-year shaking with basin effects are not seen in Figure 5-1a because median runs resulted in collapse. Collapse probabilities and median response parameters including peak story drifts, floor accelerations, beam plastic rotation, column plastic rotation, splice plastic rotations, panel zone total rotations and normalized base shear for representative ground motion suites are summarized in Table 5-1 and Table 5-2.



Figure 5-1. Median peak (a) story drift and (b) floor acceleration of the archetype building subjected to ground motion suites including magnitude-9 (M9) simulations in Seattle and La Grande, and ground motion shaking with (2018 NSHM) and without (2014 NSHM) basin effects consistent with a 975 and 2475-year return periods.

Table 5-1. Conditional collapse probabilities and median simulation results including peak interstory drift, peak floor acceleration, peak rotations in beams, column splices and panel zones, as well as normalized base shear demands for ground motion suites consistent with the 2014 NSHM.

Ground Motion Suite	Pc (%)	IDR (%)	PFA (g)	Beam PR (% rad)	Column PR (% rad)	Splice TR (% rad)	Panel Zone TR (% rad)	NBS (-)
100-year	4	0.30	0.16	0.00	0.00	0.02	0.06	0.01
475-year	11	0.80	0.28	0.00	0.00	0.05	0.13	0.04
975-year	25	1.26	0.45	0.00	0.00	0.07	0.19	0.07
2475-year	35	2.07	0.63	0.67	0.05	0.13	0.27	0.10
4975-year	65	СР	СР	СР	СР	СР	СР	СР

Notation:  $P_c$  denotes probability of collapse; IDR denotes interstory drift ratio; PFA denotes peak floor acceleration; PR denotes plastic rotation; TR denotes total rotation; NBS denotes normalized base shear; CP indicates that 50% or more of the ground motions in the suite caused the structure to collapse.

Table 5-2. Conditional collapse probabilities and median simulation results including peak interstory drift, peak floor acceleration, peak rotations in beams, column splices and panel zones, as well as normalized base shear demands for ground motion suites consistent with the 2018 NSHM.

Ground Motion Suite	Pc (%)	IDR (%)	PFA (g)	Beam PR (% rad)	Column PR (% rad)	Splice TR (% rad)	Panel Zone TR (% rad)	NBS (-)
100-year	5	0.49	0.22	0.00	0.00	0.03	0.09	0.02
475-year	18	1.30	0.47	0.00	0.00	0.07	0.20	0.07
975-year	33	2.18	0.48	0.78	0.10	0.14	0.28	0.10
2475-year	80	СР	СР	СР	СР	СР	СР	СР
4975-year	92	СР	СР	СР	СР	СР	СР	СР

Notation:  $P_c$  denotes probability of collapse; IDR denotes interstory drift ratio; PFA denotes peak floor acceleration; PR denotes plastic rotation; TR denotes total rotation; NBS denotes normalized base shear; CP indicates that 50% or more of the ground motions in the suite caused the structure to collapse.

### 5.2 Annualized Collapse Risk

### 5.2.1 2014 and 2018 US National Seismic Hazard Model

While not as intuitive as the results of an intensity-based assessment, annualized collapse metrics are useful for risk management and recovery planning. ASCE 7-16 (ASCE, 2016) has a maximum risk target of 1% probability of collapse in 50 years for determining design spectral values. To compare the performance of the 1970s archetype building against this target, the annual rate of collapse,  $\lambda_c$ , was computed considering the full range of expected shaking intensities from all earthquake sources that contribute to the seismic hazard. Table 5-1 and Table 5-2 illustrate the probability of collapse with respect to the spectral acceleration at the fundamental period of the structure according to the 2014 and 2018 NSHMs. The lognormal distributions shown in Figure 5-2 are fitted to the probability of collapse at each earthquake return period considered in the MSA using a maximum likelihood estimation procedure, as outlined in Baker (2015). Figure 5-2 highlights the considerable increase in spectral acceleration amplitudes for a given return period when basin effects are considered in the probabilistic seismic hazard estimates. For example, the 2475-year spectral amplitude increases over 50%, from 0.09g in the 2014 NSHM to 0.14g in the 2018 NSHM.

The mean annual frequency of collapse,  $\lambda_c$ , was obtained by combining the collapse fragility with seismic hazard data, which describes the mean annual frequency of exceeding the ground motion intensity, as follows:

$$\lambda_c = \int_X P(C \mid IM = x) \cdot |d\lambda_{IM}(x)|$$
(5-1)

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Results from the set of 100 motions corresponding to IM=x is used to compute P(C|IM=x), which is the probability of collapse with respect to the intensity measure (IM) of interest.  $\lambda_{IM}(x)$  is the mean annual frequency of exceedance of IM=x, which is an output of the probabilistic seismic hazard assessment implemented in the 2014 and 2018 NSHMs. The integral was evaluated through numerical integration.



Figure 5-2. Collapse fragility the 1970s archetype building per the (a) 2014 and (b) 2018 National Seismic Hazard Models.

Following this calculation,  $\lambda_c$  of the 1970s archetype building was estimated to be  $14 \times 10^{-4}$  and  $22 \times 10^{-4}$ , using the 2014 and 2018 NSHMs, respectively, as indicated in Table 5-3. The collapse risk in 50 years for each archetype was also computed using  $\lambda_c$  by assuming a Poisson distribution. The 50-year collapse risk of the archetype building using the 2014 and 2018 NSHMs was estimated as 6.9% and 10.5%, respectively. This difference in collapse risk between the 2014 and 2018 NSHM versions indicates that the inclusion of basin effects is critically significant, and results in a 52% increase in the 50-year collapse risk. These collapse risks are much larger than the value of 1% in 50 years targeted by current codes.

#### 5.2.2 Hybrid Seismic Hazard Model

These collapse risk estimates were also assessed using a hybrid seismic hazard model, which includes both empirical and simulated seismic hazard characterization. The hybrid hazard model was based on the 2018 NSHM, but utilized physics-based simulations to represent the large interface earthquake portion of the hazard and empirical relationships for all other earthquake sources (crustal and intraslab). The portion of the collapse risk attributable to interface earthquakes was recomputed using the thirty simulated M9 CSZ scenarios. The assumptions embedded in these scenarios were varied to be consistent with the source variability used in the NSHM logic trees for a full rupture of the CSZ (Frankel et al., 2018). The annual rate of collapse from the suite of simulated M9 earthquakes,  $\lambda_{e-M9}$  was computed as the product of the annual rate for an M9 CSZ earthquake (i.e. reciprocal of the earthquake return period) and the probability of collapse of the archetype building conditioned on the occurrence of the M9 earthquake scenarios considered. The annualized collapse risk considering the simulated M9 earthquake scenarios,  $\lambda_{e-Hybrid}$  was computed as follows:

$$\lambda_{\rm c-Hybrid} = \lambda_{\rm c} - \lambda_{\rm c-Interface} + \lambda_{\rm c-M9}$$
(5-2)

where  $\lambda_c$  and  $\lambda_{c-M9}$  are as previously defined,  $\lambda_{c-Interface}$  is the deaggregated portion of the total collapse risk associated with interface earthquakes. Note that in reality, the NSHM assumes that the magnitude of a large interface earthquake could vary between M8.6 to M9.3 (USGS, 2018), whereas the simulations (Frankel et al., 2018) only considered variations of an M9 event. Equation 5-2 assumes that the M9 simulations represent the full range of large-magnitude events.

The resulting annualized collapse risk,  $\lambda_{c-Hybrid}$ , was estimated to be  $17 \times 10^{-4}$ , equivalent to a 50year collapse risk of 8.1%. Marafi et al. (2020) determined that, when M9 physics-based simulations were used to represent the subduction interface portion of the seismic hazard, the annualized collapse risk in modern 4- to 24- story reinforced concrete shear wall buildings increased by 33% on average. The impact of the M9 ground motions was significant for those structures because of (i) damaging spectral shapes, and (ii) long duration of shaking. However, whereas the M9 spectral shapes might be particularly damaging for stiffer structural systems (e.g., reinforced concrete shear walls) with fundamental periods of vibration in the range of 1-3 seconds, spectral shapes are less damaging for very flexible structures (e.g., tall steel MRFs) with fundamental periods of vibration in excess of 5 seconds.

The differences in spectral shapes can be inferred from Figure 3-7, where M9 (Seattle) spectral amplitudes for buildings in the 1-3 second period range are consistent with the 2475-year probabilistic estimates of the hazard (2018 NSHM) and would result in increased seismic demands as the structure is damaged and its period elongates. These effects are less pronounced for longer

period structures. For example, at approximately 5 seconds, the M9 spectral amplitudes are closer to the 975-year probabilistic estimates of the hazard, and period elongation does not result in an increase in seismic demands. Similarly, while duration effects have been found to have a considerable impact on the collapse risk of modern ductile buildings, long-duration shaking is less significant in structural systems of limited ductility (Raghunandan, 2013), such as the pre-Northridge MRF considered in this study (Molina Hutt et al, 2018).

Table 5-3. 50-year collapse risk of the archetype building using the 2014 NSHM, 2018NSHM and 2018 NSHM with M9 earthquake scenarios considered.

Seismic Hazard Model	Annual Rate of Collapse	50-year Collapse Risk
2014 NSHM	14×10 <sup>-4</sup>	6.9%
2018 NSHM	22×10 <sup>-4</sup>	10.5%
2018 NSHM with M9	17×10 <sup>-4</sup>	8.1%

### 5.3 Earthquake-induced Repair Costs

Vulnerability functions, which are commonly used in catastrophe modelling, describe the expected consequence ratio (expected loss over total building cost) as a function of a ground motion intensity measure, typically the spectral acceleration at the fundamental period of the structure. Similar to the annual collapse risk, which is obtained by integrating the collapse fragility with the corresponding seismic hazard curve, vulnerability functions can also be integrated with the seismic hazard curve to estimate the average annual loss (AAL). AAL can be regarded as a proxy for earthquake insurance premiums.

At each earthquake return period considered in the MSA, earthquake-induced repair costs are estimated considering contributions from repairable damage, irreparable damage and collapse. The resulting vulnerability functions, computed using the 2014 and 2018 NSHMs are illustrated in Figure 5-3 and Figure 5-4, respectively. The square markers in each figure are representative of each "stripe" considered in the MSA (SA and corresponding loss ratio).

The resulting AALs associated with each seismic hazard model are also illustrated in Figure 5-3 and Figure 5-4. The pie charts show the relative contributions of collapse, repairable damage and irreparable damage to the total loss. The total AAL per the 2014 and 2018 NSHM are 0.19% and 0.29%, respectively. The results suggest that consideration of basin effects in empirical seismic hazard models, results in a 52% increase in annualized losses.

As seen in the Figure 5-3 and Figure 5-4, collapse risk is the greatest contributor to the expected losses with 79% for 2014 NSHM and 74% for 2018 NSHM. The loss contributions from irreparable damage from excessive residual drifts are the least contributors to the loss with 2% and 4% for the 2014 and 2018 NSHMs, respectively. These results are consistent with past studies that evaluated AALs in nonductile buildings, which also resulted in collapse as the greatest loss contributor in contrast to modern buildings, where irreparable damage from excessive residual drifts are the greatest contributor to the loss.


Figure 5-3. Loss vulnerability function and associated AAL including contributions from collapse, irreparable damage and repairable damage per 2014 NSHM.



Figure 5-4. Loss vulnerability function and associated AAL including contributions from collapse, irreparable damage and repairable damage per 2018 NSHM.

#### 5.4 Variations in Connection Performance

#### 5.4.1 Impacts on M9 CSZ Collapse Risk

To investigate the impact of fracture-prone welded connections on overall building response, three additional structural models were developed to benchmark against the baseline case, which includes fracture-pone welded beam-to-column connections as well as fracture-prone column splices. Performance under the M9 ground motions in Seattle was re-assessed assuming (i) fracture-prone beam-to-column connections, but ductile column splices (the "beam fracture" model), (ii) fracture-prone column splice connections, but ductile beam-to-column connections (the "splice fracture" model), and (iii) ductile beam-to-column and ductile splice connections throughout the building (the "ductile" model). For the suite of 30 M9 motions, Table 5-4 illustrates the sensitivity of the key performance characteristics to key model assumptions. Figure 5-5 illustrates the story drift profile of these models against the baseline case.

The results indicate that the presence of fracture-prone welded connections drives the collapse risk. As previously noted, the probability of collapse when brittle connections are present in both beam-to-column and splice connections under M9 simulations in Seattle was 30%. Retrofitting splice connections to have a ductile response would decrease the collapse risk to 23%, whereas retrofitting beam-to-column connections to have a ductile response would reduce the collapse risk under the M9 scenarios considered to a negligible amount. The analysis results assuming retrofitted splice and beam-to-column connections with a ductile response coincide with those of only retrofitting beam-to-column connections, which suggests that the effect of brittle splices is only significant when coupled with fracture-prone connections.



Figure 5-5. Median peak (a) story drifts and (b) floor accelerations of the M9 Seattle simulations with varying connection performance.

## Table 5-4. Conditional collapse probabilities and median simulation results including peak interstory drift, peak floor acceleration, peak rotations in beams, column splices and panel zones, as well as normalized base shear demands for the M9 Seattle simulations with varying connection performance.

Model	Pc (%)	IDR (%)	PFA (g)	Beam PR (% rad)	Column PR (% rad)	Splice TR (% rad)	Panel Zone TR (% rad)	NBS (-)
Baseline	30	3.43	0.61	2.61	0.00	0.29	0.28	0.10
Beam Fracture	23	3.02	0.61	1.88	0.00	0.21	0.28	0.10
Splice Fracture	0	2.43	0.54	1.10	0.00	0.17	0.32	0.10
Ductile	0	2.43	0.54	1.10	0.00	0.17	0.32	0.10

Notation:  $P_c$  denotes probability of collapse; IDR denotes interstory drift ratio; PFA denotes peak floor acceleration; PR denotes plastic rotation; TR denotes total rotation; NBS denotes normalized base shear; Beam Fracture denotes only beams are modelled with limited ductility; Splice Fracture denotes only column splices are modelled with limited ductility; and Ductile denotes all connections are modelled to have a ductile response.

#### 5.4.2 Impacts on M9 CSZ Scenario Losses

The impact of variations in connection performance on the expected M9 CSZ scenario losses were also computed as outlined in Equation 5-3. The results are summarized in Table 5-5.

$$L = P_C \times L|P_C + P_{NC,Irr} \times L|P_{NC,Irr} + P_{NC,Rep} \times L|P_{NC,Rep}$$
(5-3)

Where  $P_{c}$  denotes probability of collapse;  $P_{NC}$  denotes probability of non-collapse;  $P_{NC,Rep}$  denotes probability non-collapse, repairable damage;  $P_{NC,Irr}$  denotes probability of non-collapse, irreparable damage;  $L \mid P_{c}$  denotes loss conditioned on the occurrence of collapse;  $L \mid P_{NC,Rep}$ denotes loss conditioned on the occurrence of non-collapse, repairable damage;  $L \mid P_{NC,Irr}$  denotes loss conditioned on the occurrence of non-collapse, irreparable damage.

 Table 5-5. Conditional loss probabilities of the M9 Seattle simulations with varying connection performance.

Model	<b>Р</b> С	<b>P</b> <sub>NC</sub>	<b>P</b> NC, Rep	<b>P</b> NC, Irr	<i>L</i>   <i>P</i> <sub>C</sub>	<i>L</i>   <i>P</i> NC, Rep	$L \mid P_{\rm NC,  Irr}$	Total
	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)
Baseline	30	70	97.3	2.7	100	18.3	100	44
Beam Fracture	23	77	97.9	2.1	100	18.9	100	39
Splice Fracture	0	100	87.3	12.7	100	20.6	100	31
Ductile	0	100	89.5	10.5	100	20	100	28

The results in Table 5-5 suggest that connection performance has limited impact on anticipated economic losses conditioned on the occurrence of the M9 scenarios. For instance, the baseline model which includes fracture-critical connections in beams and column splices results in losses in the order of 44% of building replacement cost. An equivalent building with ductile beam and column connections results in a 36% reduction in loss down to 28% of building replacement cost. These results are in stark contrast to the reduction in collapse risk associated with the improved connection model from a 30% conditional probability of collapse, in the baseline case, down to a negligible probability of collapse for steel MRFs with ductile beam and column connections.

#### 5.4.3 Connection Modelling and Calibration

Given the sensitivity of connection performance on collapse risk, this section explores the use of an improved connection model that considers the toughness of the material and initial flaw in the weld material. As outlined in NIST (2017), one of the major factors that resulted in the failure of beam-column connection is flaws in the structure related to the method and quality of fabrication, particularly that of the welds. Flaws are generally expected at both the top and bottom flange of welded joint connections. Flaws are introduced by the backing bar at the weld root as illustrated in Figure 5-6. The response of welded-flange bolted-beam connection tests revealed that in many cases, welded connections are expected to have fractures prior to yielding (NIST, 2017).



Figure 5-6. Flaw in beam-column connection. Source: NIST (2017).

Based on  $a_0$  (the length of the flaw protruding into the flange (inches)) and  $K_{IC}$  (the critical stress intensity factor of the weld material (ksi-in) which is calculated using fracture toughness of the material), critical stresses can be calculated at both the top and bottom flange as outlined in Equation 5-4 through 5-7 (NIST, 2017). Equation 5-5 corresponds to bottom flange and equation 5-6 corresponds to top flange.

$$\sigma_{cr} = \begin{bmatrix} K_{IC} / F(a_0) \end{bmatrix}$$
(5-4)

$$F(a_0) = 1.2 + 2a_0 \tag{5-5}$$

$$F(a_0) = 0.5 + 2a_0 \tag{5-6}$$

$$K_{IC,dynamic} \approx \sqrt{\alpha} \cdot CVN \cdot E \tag{5-7}$$

Where CVN is the temperature-fracture toughness at the heat affected zone,  $\alpha$  is the empirical calibration coefficient and E is the modulus of steel.

The moment-frame connection subassembly in Figure 5-7 comprises both elastic and inelastic elements. Beam and column are modeled using elastic elements. The beam-column connection is modeled using an integrated beam with fiber sections intended to capture the key elements of the connection. In this case, the green fiber represents the top flange weld material; the blue fiber represents the bottom flange weld material and the red fibers represent the bolts. White fibers are empty fibers with zero strength and stiffness. The thick lines connecting the fiber section to the column centreline are indicative of rigid links. In this case, the beam is a deep W-section. The assumed initial flaw in the top and bottom flanges is 3.8 mm (0.15 in). Given a material toughness of 15 N-m (11 ft-lb), the resulting critical stresses are 226 MPa (32 ksi) and 365 MPa (53 ksi) for the top and bottom flange respectively. After reaching this critical stress, the tensile strength of the fiber drops to zero, while the compressive strength is retained because the forces can be transferred by direct bearing.

To evaluate the model, the connection is subjected to the testing protocol shown in Figure 5-8 as implemented by Kim et al. (2003) in their Specimen EC03. The results are shown in Figure 5-9 and indicate this modelling approach could enable a more accurate characterization of the expected beam-to-column connection performance with explicit consideration of weld performance. Further work is needed to characterize the response of these connections with a range of material toughness and weld flaw size, which may permit yielding of the connection prior to fracture occurring in the welds.



Figure 5-7. Graphical representation of fiber section model of fracture-critical welded beam-to-column connection.



Figure 5-8. Rotation protocol used to test fiber connection model.



Figure 5-9. Hysteretic moment rotation response for fracture-critical beam-to-column connection showing analytical versus experimental (Kim et al., 2003) results.

## **Chapter 6: Conclusions**

#### 6.1 Summary of Findings and Conclusions

This study evaluates the response of a representative 1970s 50-story steel MRF office building in Seattle under 30 simulated scenarios of an M9 CSZ earthquake. The 1970s archetype is assumed to have fracture-prone welded connections, consistent with those identified following the 1994 Northridge earthquake.

The resulting probability of collapse, conditioned on an M9 CSZ earthquake in Seattle is 30%. This collapse probability is consistent with the 975-year return period probabilistic estimate of the hazard, when basin effects are considered, and the 2475-year return period probabilistic estimate of the hazard, when basin effects are neglected. These estimated collapse probabilities far exceeds the 10% target under extreme ground motions (~2475-year return period) typically considered by modern seismic codes for new design.

The annualized collapse risk of the 1970s archetype building was also assessed through MSA. The results indicate a 50-year collapse risk of 6.9% when basin effects are not considered in the probabilistic estimates of the hazard (2014 NSHM). Including basin effects (2018 NSHM) increased the 50-year collapse risk to 10.5%, a tenfold increase on the 1% in 50 year collapse risk target implicit in modern seismic design standards. The portion of the collapse risk attributable to interface earthquakes was recomputed using the thirty simulated M9 scenarios, resulting in a 50-year collapse risk of 8.1%. The collapse risk estimates of the hybrid hazard model is lower than

the 2018 NSHM because the spectral shapes and amplitudes for the M9 ground motions at long periods are less significant that the interface earthquakes derived from empirical ground-motion models.

These high collapse risks in the 1970s 50-story steel MRF are largely driven by: (i) deep sedimentary basin effects, which amplify long period shaking; and (ii) the expected brittle behavior of fracture-prone welded beam-to-column connections. Increasing the ductility of beam-to-column connections improved the performance of the archetype and resulted in a negligible probability of collapse.

#### **6.2 Future Studies**

While this study is limited to the evaluation of a representative archetype, it provides insights into the collapse risk of pre-1994 tall steel MRFs in Seattle, which are believed to be a predominant type of construction among buildings taller than 240 ft (73 m) constructed in the 1960s, 1970s and 1980s. Additional work is needed to fully characterize the risk of this taxonomy of buildings and more importantly, future studies should evaluate cost-effective strategies to mitigate the risk through retrofit policies or other measures.

In order to better characterize the impact of connection performance on overall building response, further work is needed to develop high-fidelity models such as those explored within this work using fiber sections, which explicitly simulate the weld material in the connections. In this work, modelling of connections that fail prior to beam yielding were successful. However, more work is needed to develop models where fracture occurs after beam yielding. While many experimental programs after the 1994 Norridge earthquake suggest that beam fractures could occur prior to yielding, certain combinations of weld material toughness and initial flaw size could result in improved connection performance. Accurate characterizing of these connections is essential to evaluate the performance of existing pre-Northridge tall steel MRFs.

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# Appendices

## Appendix A

Appendix A provides peak story drift and floor acceleration results of the archetype building subjected to individual ground motions for all suites considered in the MSA according to the 2014 and 2018 NSHMs. These plots represent the results of non-collapse runs only.



Figure A-1. Peak (a) story drift and (b) floor acceleration results of the archetype building subjected to a ground motion suite representative of earthquake shaking with a 100-year return period per 2014 NSHM.



Figure A-2. Peak (a) story drift and (b) floor acceleration results of the archetype building subjected to a ground motion suite representative of earthquake shaking with a 475-year return period per 2014 NSHM.



Figure A-3. Peak (a) story drift and (b) floor acceleration results of the archetype building subjected to a ground motion suite representative of earthquake shaking with a 975-year return period per 2014 NSHM.



Figure A-4. Peak (a) story drift and (b) floor acceleration results of the archetype building subjected to a ground motion suite representative of earthquake shaking with a 2475-year return period per 2014 NSHM.



Figure A-5. Peak (a) story drift and (b) floor acceleration results of the archetype building subjected to a ground motion suite representative of earthquake shaking with a 4975-year return period per 2014 NSHM.



Figure A-6. Peak (a) story drift and (b) floor acceleration results of the archetype building subjected to a ground motion suite representative of earthquake shaking with a 100-year return period per 2018 NSHM.



Figure A-7. Peak (a) story drift and (b) floor acceleration results of the archetype building subjected to a ground motion suite representative of earthquake shaking with a 475-year return period per 2018 NSHM.



Figure A-8. Peak (a) story drift and (b) floor acceleration results of the archetype building subjected to a ground motion suite representative of earthquake shaking with a 975-year return period per 2018 NSHM.



Figure A-9. Peak (a) story drift and (b) floor acceleration results of the archetype building subjected to a ground motion suite representative of earthquake shaking with a 2475-year return period per 2018 NSHM.



Figure A-10. Peak (a) story drift and (b) floor acceleration results of the archetype building subjected to a ground motion suite representative of earthquake shaking with a 4975-year return period per 2018 NSHM.



Figure A-11. Peak (a) story drift and (b) floor acceleration results of the archetype building (Baseline Model) subjected to the suite of simulated M9 CSZ earthquake scenarios in La Grande.



Figure A-12. Peak (a) story drift and (b) floor acceleration results of the archetype building (Baseline Model) subjected to the suite of simulated M9 CSZ earthquake scenarios in Seattle.



Figure A-13. Peak (a) story drift and (b) floor acceleration results of the archetype building (Beam Fracture Model) subjected to the suite of simulated M9 CSZ earthquake scenarios in Seattle.



Figure A-14. Peak (a) story drift and (b) floor acceleration results of the archetype building (Splice Fracture Model) subjected to the suite of simulated M9 CSZ earthquake scenarios in Seattle.



Figure A-15. Peak (a) story drift and (b) floor acceleration results of the archetype building (Ductile Model) subjected to the suite of simulated M9 CSZ earthquake scenarios in Seattle.