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SEISMIC COLLAPSE ASSESSMENT OF DUCTILE AND NON-DUCTILE REINFORCED CONCRETE BUILDINGS IN ALASKA

M. Raghunandan[1] and A. B. Liel[2]

ABSTRACT

In 1964, Alaska experienced the second largest earthquake (M_w 9.2) ever recorded, in the Aleutian-Alaska subduction zone. In future, Alaskan cities are at risk of ground shaking from both crustal and subduction earthquakes, although the relative hazard from each depends on the location within the state. Structural collapse capacities vary significantly depending on the source of the ground motions, largely because ground motions from subduction events are longer in duration and have higher energy associated with longer period spectral content. This study assesses the collapse capacities of older non-ductile buildings designed according to the 1967 UBC and modern ductile buildings designed according to the 2012 IBC for the three most populated cities of Alaska: Anchorage, Fairbanks and Juneau. The probability of collapse in 50 years is estimated to be 0.1 – 2% for modern buildings, and 0.9- 7% for older buildings. The analysis results also indicate reduction in collapse capacities of ductile and non-ductile buildings by 41% and 10% respectively on being subjected to subduction instead of crustal ground motions. In Anchorage, accounting for the increased fragility of buildings' response to subduction shaking results in a 137% increase in collapse risk.

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ABSTRACT

In 1964, Alaska experienced the second largest earthquake (M_w 9.2) ever recorded, in the Aleutian-Alaska subduction zone. In future, Alaskan cities are at risk of ground shaking from both crustal and subduction earthquakes, although the relative hazard from each depends on the location within the state. Structural collapse capacities vary significantly depending on the source of the ground motions, largely because ground motions from subduction events are longer in duration and have higher energy associated with longer period spectral content. This study assesses the collapse capacities of older non-ductile buildings designed according to the 1967 UBC and modern ductile buildings designed according to the 2012 IBC for the three most populated cities of Alaska: Anchorage, Fairbanks and Juneau. The probability of collapse in 50 years is estimated to be 0.1 – 2% for modern buildings, and 0.9- 7% for older buildings. The analysis results also indicate reduction in collapse capacities of ductile and non-ductile buildings by 41% and 10% respectively on being subjected to subduction instead of crustal ground motions. In Anchorage, accounting for the increased fragility of buildings' response to subduction shaking results in a 137% increase in collapse risk in the modern building.

Background and Motivation

Alaska is located in one of the most seismically active subduction zones in the world. In 1964, the Aleutian-Alaska subduction zone caused the world's second largest ever recorded earthquake (M_w 9.2) in the Prince William Sound region of Alaska [1]. The most populated city, Anchorage, experienced damage and destruction to about 30 blocks of commercial buildings and dwellings in the downtown area, including irreparable damage to the J.C. Penny Company building, and collapse of the Four Seasons apartment building and a new six story structure [1]. Ground shaking in Anchorage lasted around three minutes in duration. Since 1964, the population of Alaska has more than doubled and significant new building construction has occurred [2]. Therefore, should a similar large earthquake occur in the future, as expected by seismologists, it could cause a similar or even larger scale of damage. In fact, the Federal Emergency Management Agency (FEMA) has estimated that Alaska has the second highest average annualized earthquake-loss ratio (ratio of average losses to the infrastructure) in the U.S. with the present infrastructure and policies in place, being second only to California [3].

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Ground motions from subduction earthquakes are generally longer in duration and have higher energy content associated with longer periods as compared to those from crustal earthquakes. Previous studies indicate that longer duration ground motions can cause higher levels of accumulated damage in structures, as compared to shorter duration ground motions having the same intensity [4,5]. Although a number of studies have been conducted recently to assess the structural collapse risk from crustal earthquakes in seismic regions like California [6–8], there is relatively less work investigating the collapse risk of buildings at risk of subduction earthquakes in Alaska or elsewhere. Raghunandan and Liel [9] quantified the collapse risk of buildings in Seattle, Washington and Portland, Oregon, finding that, on average, the collapse capacities of representative ductile and non-ductile concrete moment frame buildings decrease by 36% and 12%, respectively, on being subjected to subduction ground motions as compared to crustal ground motions. Subduction earthquakes contribute to around 80% of the total annual seismic collapse risk of the buildings analyzed in Seattle and Portland [9]; the remainder of the risk comes from local crustal faults (*e.g.* the Seattle fault). Similarly, Alaska's communities are susceptible to both crustal and subduction earthquakes (Fig.1a).

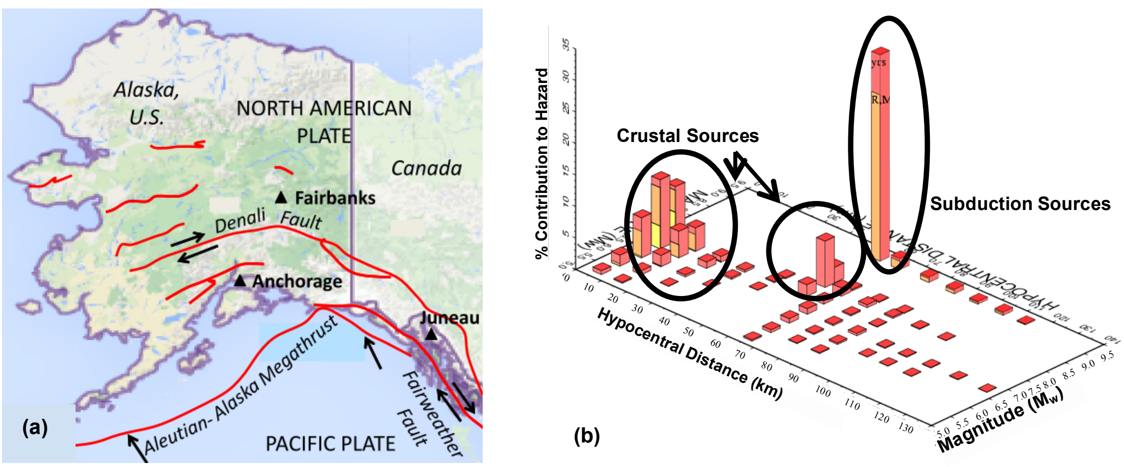

Figure 1. (a) Map of Alaska showing the approximate location seismic faults; (b) Deaggregation of probabilistic seismic hazard for Anchorage showing contribution from different sources[1]. (Results in (b) are for Sa(1s) and the 2% in 50 year hazard level.)

This paper evaluates the seismic collapse performance of modern ductile buildings and older non-ductile buildings in Alaska on being subjected to crustal and subduction ground motions. To this end, a set of 2D nonlinear multiple-degree-of-freedom building simulation models are created for 4-story reinforced concrete moment frame buildings designed and detailed according to (1) the 2012 IBC 2012 and (2) the 1967 UBC for the cities of Anchorage, Fairbanks and Juneau, Alaska (Fig. 1a). The older buildings represent non-ductile construction of the era immediately following the Great Alaska quake; the new buildings represent modern construction. These three cities are chosen because they are the three most populated cities in Alaska, with a total population of approximately 360,000 according to the 2010 U.S. Census. Moreover, the cities are in different seismic environments and, as such, are subject to different seismic design load levels according to present and past building codes. The collapse capacity of each of the six building models is calculated by conducting nonlinear dynamic analysis on the numerical models using sets of (1) subduction and (2) crustal ground motions. Separate sets of collapse fragility curves are generated for each ground motion set. The results of each are convolved with the seismic hazard for each site to evaluate seismic collapse risk of the buildings. A better understanding of the vulnerability of buildings and infrastructure in the high seismic risk regions of Alaska will assist in reducing seismic losses by identifying the most vulnerable buildings.

Ground Motion Database

Sets of *Crustal* and *Subduction* ground motions are compiled for the purpose of assessing the collapse capacities of the Alaskan buildings of interest. The *"Crustal"* database contains 35 far-field crustal earthquake ground motions from FEMA P-695 [6]. These ground motion recordings are from large magnitude shallow crustal earthquakes (M_w 6.5-7.6) that are recorded 7-26 km away from the rupture and have peak ground accelerations greater than 0.15g. The "*Subduction*" database consists of 42 ground motions from subduction events (M_w 6.8-9.0), including recordings from Tohuku, Japan (M_w 9.0, 2011) and Maule, Chile (M_w 8.8, 2010), which are compiled from different databases. The *Subduction* ground motions have peak ground accelerations greater than 0.01g (with 60% having PGA>0.15g) and are recorded at significantly larger distances from the rupture (27-390 km). The average durations of the *Crustal* and *Subduction* recordings are 13.9s and 44.3s, respectively. Most of the recordings are on rock or stiff soils. A pair of synthetic simulated recordings for the Great Alaska earthquake generated by Mavroeidis *et al*. [10] is used to test how the Alaskan building stock will perform during an earthquake similar to Great Alaskan earthquake. More detailed information about ground motion recordings, site conditions and processing can be found in [9].

Archetype Building Design and Analytical Models

Alaska's cities have varying levels of seismic hazard, due to their proximity to crustal and subduction faults (Fig. 1a). This study considers sites in three cities: Anchorage (61.2°N, 149.9°W), Fairbanks (64.85°N, 147.65°W) and Juneau (58.3°N, 134.4°W). These cities have varying levels of expected seismicity, and therefore different levels of seismic loads, with Anchorage being the highest, Fairbanks intermediate, and Juneau the lowest. For each of the cities, 4-story reinforced concrete moment frames (space frames) are designed and detailed according to both (1) the 2012 IBC [11] and (2) the 1967 UBC [12]. The two building codes are selected to represent modern and older construction in the U.S. Table 1 summarizes the seismic design parameters for the buildings examined.

Table 1. Seismic design parameters for the Alaska reinforced concrete frame buildings.

Design	Seismic Hazard used to Determine Design Base Shear[a]		
Code	Anchorage	Fairbanks	Juneau
1967 UBC	Zone 3	Zone 3	Zone 2
2012 IBC	S_s=1.5g; S_1=0.68g	S_s=0.99g; S_1=0.38g	S_s=0.53g; S_1=0.36g

[a] The seismic hazard information is provided in terms of seismic zones for the older buildings in the 1967 UBC. For the modern buildings, the seismic hazard is reported in terms of the risk-targeted Maximum Considered Earthquake (MCE_R) ground motion response acceleration value at T=0.2 s (S_s) and T=1 s (S_1) from the 2012 IBC.

The properties of the resulting building designs are provided in Table 2. The height of the first story is assumed to be 15 feet, whereas the upper stories are 13 feet high. Columns are spaced 20 feet apart. The buildings designed according to 2012 IBC have high strength and deformation capacity and belong to the category of so-called "special" moment resisting frames that satisfy a large number of requirements such as strong-column-weak-beam and shear capacity design. The non-ductile buildings designed according to 1967 UBC are not subject to the same level of detailing or design requirements. In particular, low transverse reinforcement detailing in columns and joints prevents components from deforming inelastically and dissipating energy during ground shaking, thereby making the buildings susceptible to brittle shear and axial failure in beam-columns and joints. Consequently, these older buildings have significantly lower strength and deformation capacity as compared to their modern ductile counterparts. Most of the sites in the Alaskan cities of interest are NEHRP site class B (*i.e.,* rock), C (*i.e.,* very dense soil and soft rock) or D (*i.e.,* stiff soil) [1]. For this study, the modern buildings are designed for the more conservative site class D. There is no soil class considered in computing the design base shear for the 1967 UBC buildings.

Table 2. Building design information for the Alaska reinforced concrete frame buildings.

Design Code	Anchorage				Fairbanks				Juneau			
	DBS[a]	T_1[b]	μ[c]	Ω[d]	DBS	T_1	μ	Ω	DBS	T_1	μ	Ω
1967 UBC	0.068	1.11	3.5	2.9	0.068	1.11	3.5	2.9	0.034	1.29	2.9	4.3
2012 IBC	0.104	0.96	11.7	2.4	0.063	1.00	12.4	3.4	0.060	1.12	9.7	3.1

[a] Design base shear: Calculated as ratio of the design base shear to the building weight (V_{design}/W).
[b] First-mode fundamental period: Based on eigenvalue analysis, considering cracked concrete sections.
[c] Ductility capacity: Computed as the ratio of ultimate displacement to the effective yield displacement calculated from the nonlinear pushover analysis of the building [6].
[d] Overstrength: Calculated as the ratio of maximum base shear of the building from nonlinear static pushover analysis to the design base shear.

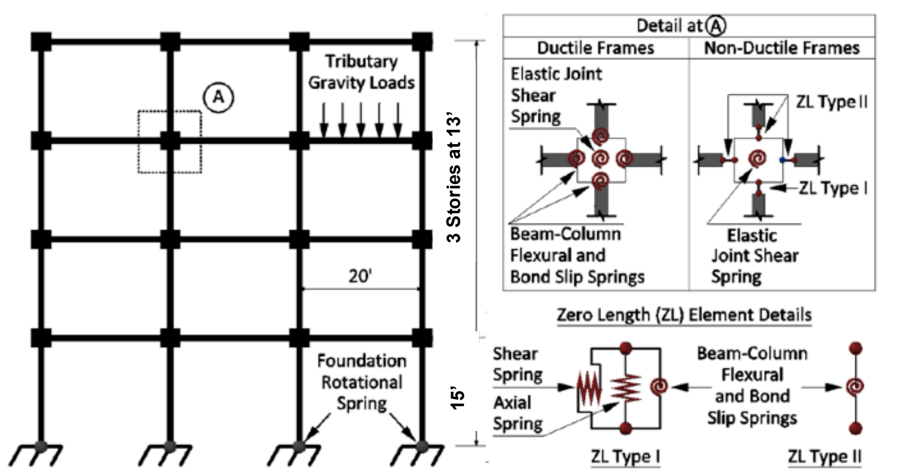

Figure 2. Graphical representation of the analytical building model along with nonlinear modeling parameters [10].

All buildings are modeled as 2D, three bay, space frames as shown in Fig. 2. The analytical models are implemented in the software *OpenSees* [13]. Flexible foundations are modeled by employing elastic, semi-rigid rotational springs at the base of ground floor columns. The nonlinear models must be capable of capturing different modes of strength and stiffness deterioration and component failure to successfully simulate structural collapse. Under earthquake loading, flexural failure due to large deformations in beams and columns leads to collapse of the ductile (modern) buildings. On the other hand, brittle axial and shear failure of columns at low deformations instigates collapse of the non-ductile (older) buildings.

Since different modes of failure lead to collapse in the ductile and non-ductile buildings, the modeling approach used to analytically represent the two eras of buildings also differs. The analytical models for the ductile buildings employ lumped plasticity beam-column elements to describe the flexural behavior of structural members. The plastic hinges are modeled using the hysteretic material developed by Ibarra *et al*. [14], which can simulate the strength and stiffness deterioration due to hysteresis under dynamic loading. The material properties for the plastic hinges are calculated based on empirical relationships obtained by calibrating the model to experimental test results for more than 250 reinforced concrete columns [7]. The modeled properties of the beam-column plastic hinges vary depending on the structural element size and reinforcement detailing. For the non-ductile buildings, nonlinear behavior of column is modeled by providing a zero-length element at the top of the columns consisting of lumped shear, axial and rotational springs. The response of the column is determined by the flexural response of the column until shear failure occurs. The axial and shear failure springs track the response of the associated beam-column element and detect axial and shear failure when the response reaches pre-defined shear and axial limit surfaces. The positions of the shear and axial limit state surfaces are determined from the properties of the columns. In the case of shear failure, the limit surface is defined in the small displacement range by the strength relationship proposed by Sezen and Moehle [15] and in the larger displacement range by the force-displacement relationship proposed by Elwood [16]. The axial limit surface is defined by the level of axial force and lateral drift in the column according to a relationship derived by Elwood [16]. Once the response reaches this surface, the properties of the respective shear and axial springs are updated to represent the expected negative slope of the element [17].

Building Collapse Simulation

To quantify building resistance to earthquake-induced collapse, incremental dynamic analysis (IDA) is carried out on each building model [18]. In IDA, the analytical model of the building is subjected to a ground motion with a particular intensity and the response of the structure is measured. The ground motion is then scaled and reapplied to the building model, recording the new structural response. This process of scaling the ground motion is continued until the structure collapses. The analysis is then repeated for all ground motions in the *Crustal* and *Subduction* sets.

In the nonlinear dynamic analysis, collapse occurs when (a) interstory drifts increase without bounds due to large flexural deformations in beams and columns (“sidesway” collapse), (b) the total story shear capacity becomes less than the residual story shear capacity at any story, or (c) the total gravity load (axial) demand on the columns in a story exceeds the total axial capacity of columns at that story. Sidesway collapse due to large interstory drifts is the expected collapse mode for ductile moment frames that are susceptible to large flexural deformations under lateral loading [7]. The non-ductile buildings are incapable of undergoing such large deformations, and instead experience brittle shear or axial in columns at small drift levels. Therefore, the global capacity-demand failure criteria (b) and (c) are used to identify when collapse occurs due to these failure modes. These criteria are similar to those proposed by Baradaran Shoraka and Elwood [17]. More detailed information about the calculations identifying when collapse is triggered can be found in Raghunandan and Liel [9].

There are several possible intensity measures or IMs that can be used to quantify the intensity of the ground motion in IDA. The conventionally used intensity measure, elastic spectral acceleration at a building's fundamental period, *Sa(T)*, does not capture the spectral shape of the ground motion, which can significantly influence nonlinear structural response [7,19]. Therefore, an alternate advanced intensity measure, inelastic spectral displacement at the fundamental period, S_{di}, is used instead to quantify ground motion intensity. S_{di} is defined as the maximum displacement of a single-degree-of-freedom oscillator with bilinear behavior [20]. The oscillator response is defined by the fundamental period of the building of interest, the yield displacement, d_y, post-yield hardening stiffness of 5% of the elastic stiffness, and 5% damping. In order to facilitate comparisons across buildings, collapse capacity is quantified in this study using S_{di} computed with a bilinear oscillator with fundamental period of 1 second and yield displacement of 2.9 inches. The properties of the oscillator to calculate S_{di} are based on average properties of the buildings considered in this study. The value of S_{di} reflects both the intensity and shape of the ground motion spectra, due to period elongation of the oscillator that makes it respond to different regions of the spectra. S_{di} provides a simple IM that captures the effects of spectral shape on structural response and ensures that structural response is not biased by the scale factor applied to the record in IDA [20].

Fig. 3(a) illustrates IDA results for a 4-story modern ductile building in Anchorage, Alaska building subjected to the *Crustal* ground motion set. The collapse capacity of the structure is quantified by the ground motion intensity at which collapse occurs for each of the different ground motions. For the highlighted ground motion in Fig. 3(a), collapse occurs at S_{di} = 18 in (shown by the IDA "flat-lining" at this value). These results are used to compute the median and dispersion of the collapse capacity, assuming a lognormal distribution, and where the median collapse capacity corresponds to the intensity of ground motion that has a 50% probability of causing collapse of the building and the uncertainty in the collapse capacity is due to record-to-record variability in structural response (quantified by the logarithmic standard deviation). For the 4-story modern Anchorage building subjected to crustal ground motion recordings, the median collapse capacity is S_{di} = 15.4in and the dispersion is 0.31. For reference, the median collapse capacity of this structure quantified in terms of $Sa(T_1)$ is 1.76g.

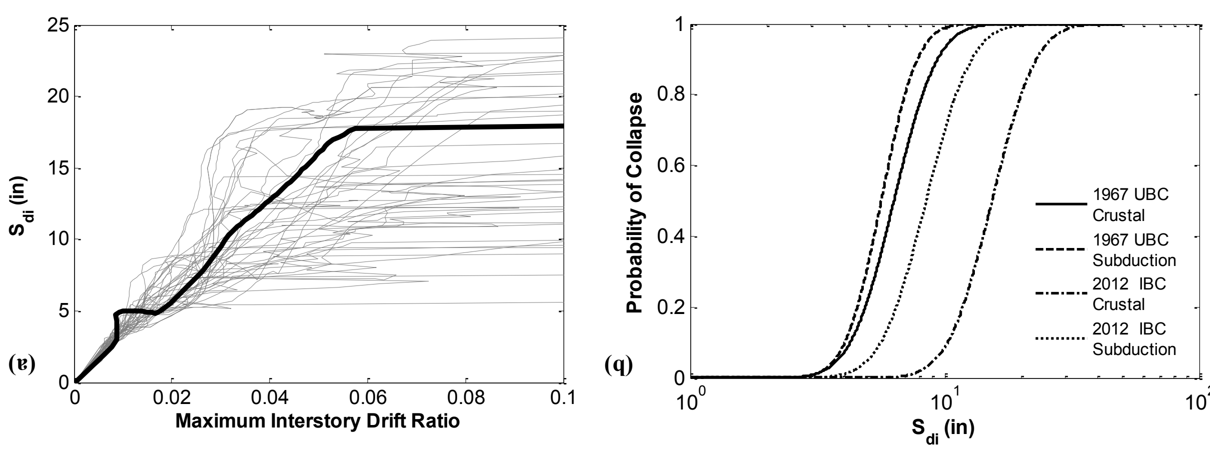

Figure 3. (a) IDA results for building in Anchorage designed according to the 2012 IBC and subjected to crustal ground motions; (b) Collapse fragility curves developed for buildings in Anchorage designed according to the 1967 UBC and 2012 IBC, illustrating differences in fragility when subjected to subduction and crustal motions.

Results

Collapse Fragilities for Archetype Alaskan Buildings

To quantify the collapse safety of a building on being subjected to crustal and subduction records, building collapse fragility curves for a given type of earthquake event are generated from the IDA results. Table 3 summarizes the collapse analysis results in terms of the median S_{di} collapse capacity (denoted x_m) and lognormal standard deviation of the fragility curve (denoted β) for all of the buildings considered in the study. Fig. 3(b) illustrates the collapse fragility curves calculated separately for each of the *Crustal* and *Subduction* sets for the Anchorage buildings.

Table 3. Summary of collapse fragility curve parameters for all archetypical buildings, as measured when subjected to *Crustal* and *Subduction* ground motion sets.

Building Code	City	Crustal (C)		Subduction (S)		Variation in x_m, S vs. C (%)
		x_m (S_{di}, in) [a]	β[b]	x_m (S_{di}, in)	β	
1967 UBC	Anchorage	6.3	0.31	5.6	0.26	-11%
	Fairbanks	6.3	0.31	5.6	0.26	-11%
	Juneau	5.7	0.32	5.2	0.32	-10%
2012 IBC	Anchorage	15.4	0.31	8.5	0.32	-45%
	Fairbanks	14.8	0.37	8.6	0.31	-42%
	Juneau	14.4	0.38	8.9	0.29	-38%

[a] Median collapse capacity (x_m) is quantified in terms of S_{di}.
[b] Logarithmic standard deviation of collapse capacity.

The results summarized in Table 3 show that the median collapse capacities of the modern (ductile) buildings designed according to 2012 IBC for Anchorage, Fairbanks and Juneau are reduced by 38% - 45% when building models are subjected to *Subduction* ground motions instead of *Crustal* ground motions. This result implies that, for a given level of ground motion intensity, the probability of collapse for these 4-story buildings is lower if the ground shaking comes from a crustal earthquake than if it comes from a subduction earthquake. A decrease in median collapse capacity is also observed for the older (non-ductile) 4-story buildings designed according to 1967 UBC; specifically, the *Subduction* motions reduce the collapse capacity of non-ductile buildings in Anchorage, Fairbanks and Juneau by 11%, 11% and 10%, respectively. Record-to-record variability, *β,* is approximately 0.30 for all of the buildings.

The earthquake source affects building collapse capacity due to the longer duration of *Subduction* ground motions. Recall that the average duration of the *Crustal* ground motions in the database is 69% smaller than the average duration of *Subduction* ground motions. As shown by Raghunandan and Liel [5], the longer the duration of shaking, the lower the intensity at which collapse occurs, due to the higher energy demands longer duration motions require structures to dissipate. However, the percentage reduction in median collapse capacity associated with subduction ground motions for the non-ductile buildings is much less than the reduction in collapse capacities observed for the ductile buildings (Fig. 3b). The ductile buildings dissipate large amounts energy over repeated loading cycles in long duration ground motions. Therefore, longer duration ground motions become more damaging to the structure as compared to shorter with lesser cycles of loading. On the other hand, non-ductile buildings are brittle and have low energy dissipation capacities, which are exhausted over small number of cycles of loading and are less sensitive to variability in duration [5]. There are also significant differences in frequency content of ground motions from *Subduction* and *Crustal* sets. These effects are already accounted for through the selection of S_{di} as the ground motion intensity measure.

Results in Table 3 also show variation in the median collapse capacity of the buildings located in Anchorage, Fairbanks and Juneau and across design eras due to differences in design seismic forces and design detailing requirements. The median collapse capacity from *Crustal* set of ground motions is S_{di}= 6.3in for Anchorage and Fairbanks and 5.7in for Juneau, indicating the smaller capacity of the building in Juneau because it was designed for lower seismic forces due to its location in a less severe seismic zone. The 2012 IBC buildings have greater strength and deformation capacity as compared to 1967 UBC buildings, resulting in significantly higher median collapse capacities. The collapse capacities of the modern buildings, quantified in terms of S_{di} are approximately 2.5 (*Crustal*) to 1.5 (*Subduction*) times greater than the collapse capacities of the older buildings. Similar differences were observed in comparing the collapse capacities of older and modern buildings in Oregon and Washington [9].

Seismic Collapse Risk of Alaskan Buildings

USGS [1] provides seismic hazard curves for every location in the U.S., defining ground motion intensity in terms of *Sa(T)* for pre-defined values of T. Since seismic hazard is not available in terms of S_{di}, this study calculates seismic collapse risk of buildings using collapse fragility curves recalculated for the IM *Sa(T=1s)* for all the buildings. A spectral period of 1s second is employed for all structures because it allows for consistent comparison among the buildings with different T_1. We note, however, that the use of *Sa(T=1s)* as an IM does not permit consideration of spectral shape effects, which can be important for modern buildings.

The probability of collapse in 50 years for each building is quantified using Eqs. 1 and 2, assuming a Poisson distribution of earthquake occurrences:

$$\lambda[Collapse] = \int_0^\infty \lambda[SA]\ f(c)dc \tag{1}$$

$$P[Collapse\ in\ 50\ years] = 1 - exp(-\lambda[Collapse] * 50\ ) \tag{2}$$

$\lambda[Collapse]$ is the average annual collapse frequency. $\lambda[SA]$ is the annual frequency of exceedance of the spectral acceleration demand (*i.e.* seismic hazard curve), and $f(c)$ is the lognormal probability distribution of the collapse capacity, *i.e.* the derivative of the collapse fragility curve. Both the capacity (*c*) and the demand (*SA*) are quantified in terms of *Sa(T=1s)*. For consistency with previous studies, the standard deviation of natural logarithm of collapse capacity, β, is assumed to be equal to 0.6, and considers uncertainty in design, modeling and record-to-record variability [21]. In this study, the calculations in Eqns. (1) and (2) are carried out separately for crustal and subduction events for each building by substituting the event-type-specific site hazard curve and collapse fragility curve. Separate hazard curves were obtained from the USGS. The crustal and subduction collapse frequencies are then added together to calculate the total annual collapse frequency at a site, and the total 50-year collapse probability.

Collapse risk metrics summarized in Table 4 indicate that the collapse risk for older buildings is much higher as compared to (around 3-16 times) for all three cities as compared to their modern counterparts, indicating the higher vulnerability of older non-ductile buildings for collapse. In addition, results show that the collapse risk for buildings (both older and modern) in Anchorage is around 4 and 7 times higher than collapse risk for buildings in Fairbanks and Juneau. The results also demonstrate the importance of subduction sources in Anchorage, where subduction motions contribute 72 -80% of the collapse risk. However, in Fairbanks and Juneau, where crustal faults dominate the seismic hazard, the situation is reversed, with only 10-30% of the collapse risk associated with subduction earthquakes. Due to the importance of the subduction sources, we observe also that if the collapse risk were computed in Anchorage the standard way -- combining the total seismic hazard curve with a crustal-dervied fragility curve -- the collapse risk would be underestimated by approximately 12%. This discrepancy indicates that it is necessary to consider separate crustal and subduction building fragility curves to improve estimates of the collapse risk for regions that are susceptible to significant seismic hazard from subduction sources.

It is tempting to compare the collapse probabilities in Table 4 to the target 1% probability of collapse in 50 years targeted by modern building codes [21]. The values computed for Anchorage are higher than the 1% target for the modern building. However, due to the assumptions used in the calculations (excluding the so-called spectral shape factor and using β = 0.8) biases our values to be larger than those used to develop the 1% value, such that the comparison is somewhat unfair.

Table 4. Collapse risk, quantified by *P[Collapse in 50 years],* for Alaska archetypical buildings.

Design Building Code	Anchorage			Fairbanks			Juneau		
	Crust.[a]	Subd.[b]	Total[c]	Crust.	Subd.	Total	Crust.	Subd.	Total
1967 UBC	1.9%	5.1%	7.0%	0.8%	0.1%	0.9%	0.9%	0.2%	1.1%
2012 IBC	0.2%	1.8%	2.0%	0.1%	0.0%	0.1%	0.1%	0.0%	0.1%

[a] "Crust" considers only the crustal hazard and fragility.
[b] "Subd" considers only the subduction hazard and fragility.
[c] In total, the subduction and crustal components are computed separately and added.

Response to Simulated Ground Motions for 1964 Alaska Earthquake

The building models were also subjected to ground motions simulated by Mavroeidis et al. [10]. These simulations represent the ground motions from the 1964 Alaska Earthquake in Anchorage, for which no recordings are available. Nonlinear time history analyses showed that the ground motion time histories must be scaled by 2.4 times to collapse the older Anchorage 4-story building and 3.8 times to collapse the modern Anchorage 4-story building. The relatively low scale factor required to collapse the older buildings is consistent with patterns of building damage in that event, in which some of the non-ductile concrete buildings collapsed, likely because of irregularities or deficiencies not present in the archetype buildings considered here.

Conclusions

Alaska is seismically active and exposed to seismic hazard from both subduction and crustal earthquake events. Most of the studies conducted until now have quantified structural safety and response due to crustal ground shaking, which has different characteristics from subduction ones. This study indicates that the longer duration of the subduction ground motions significantly reduces collapse capacity of modern and older mid-rise buildings in Alaska. The most vulnerable buildings of those studied are the non-ductile (older) buildings in Anchorage, which have a substantially higher risk of earthquake-induced collapse than modern buildings. In addition, Anchorage's proximity to the Alaska-Aleutian megathrust subduction zone amplifies the risk there as compared to Fairbanks and Juneau, and indicates the importance of separate consideration of the hazard from subduction earthquakes.

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