INFLUENCE OF ADVANCED STRUCTURAL MODELING AND SUBDUCTION MAINSHOCK-AFTERSHOCK SEQUENCES ON SEISMIC FRAGILITY OF RC STRUCTURES

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ABSTRACT

Current seismic codes and standards in practice aim at preventing collapse of structures but do not address serviceability and performance of the buildings after a major earthquake adequately. For instance, following a mainshock, ground motion activities, known as aftershocks, occur as a cluster, potentially causing incremental damage to structures whose seismic capacities may be reduced by the mainshock. This paper proposes an advanced structural modeling technique, which can simulate various features of cyclic deterioration and degradation in material and structural components using nonlinear fiber beam-column elements. The proposed model accounts for inelastic buckling and low-cycle fatigue degradation of longitudinal reinforcement, and can simulate the multiple failure modes of reinforced concrete structures under dynamic loading. Furthermore, a comprehensive ground motion selection accounting for subduction earthquakes, is implemented. Finally, a new set of fragility curves has been developed, which accounts for the structural modeling and aftershock effects. The proposed methodology significantly improves the accuracy of seismic risk and vulnerability assessment by reducing the uncertainties associated with structural modeling and variability of earthquake ground motions.

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ABSTRACT

Current seismic codes and standards in practice aim at preventing collapse of structures but do not address serviceability and performance of the buildings after a major earthquake adequately. For instance, following a mainshock, ground motion activities, known as aftershocks, occur as a cluster, potentially causing incremental damage to structures whose seismic capacities may be reduced by the mainshock. This paper proposes an advanced structural modeling technique, which can simulate various features of cyclic deterioration and degradation in material and structural components using nonlinear fiber beam-column elements. The proposed model accounts for inelastic buckling and low-cycle fatigue degradation of longitudinal reinforcement, and can simulate the multiple failure modes of reinforced concrete structures under dynamic loading. Furthermore, a comprehensive ground motion selection accounting for subduction earthquakes, is implemented. Finally, a new set of fragility curves has been developed, which accounts for the structural modeling and aftershock effects. The proposed methodology significantly improves the accuracy of seismic risk and vulnerability assessment by reducing the uncertainties associated with structural modeling and variability of earthquake ground motions.

Introduction

Over the last decades, several earthquakes occurred with damaging aftershocks and imposed enormous hazard and risk to several urban cities around the world. An example of major mainshock-aftershock (MSAS) events are the 4th September 2010, M 7.0 Darfield event in New Zealand, followed by the 22nd February 2011, M 6.2 Christchurch event, causing extensive damage to unrepaired structures [1], and the 11th March 2011, M 9.0 Tohoku earthquake in Japan, with numerous major aftershocks as large as M 7.9 [2]. These historical events indicate that neglecting aftershock hazard and risk may lead to a biased assessment of the seismic performance of structures, which jeopardizes the integrity of designed and constructed structures. Reliable seismic performance evaluation and quantifying the potential failure risk of existing RC structures require accurate modeling of nonlinear behavior of these structures under cyclic dynamic loading.

Numerous studies have been carried out to investigate the performance of structures by considering aftershock sequences [3-4]. Recently, the significance of aftershocks was investigated extensively by Goda and Taylor [5] using peak ductility demand of inelastic SDOF

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systems for real and artificial MSAS sequences. They concluded that aftershocks increase the peak ductility demand by 5% to 20%, depending on the vibration periods, hysteretic characteristics of structural systems, and seismic excitation levels.

Knowing that structures with softening behavior are more susceptible to additional damage due to aftershocks, the response of RC structures under multiple earthquakes has gained much attention among researchers in earthquake engineering community. Faisal et al. [6], Di Sarno [7], Goda and Tesfamariam [8] and Raghunandan et al. [9] studied the performance of RC buildings under MSAS sequences. Their studies confirmed that there is a lack of conservatism in the safety of conventionally designed structures subjected to multiple earthquakes.

A review of the literature has shown the improvement in structural modeling and ground motion record generation/selection. However, influence of advanced structural modeling, damage quantification, and ground motion characteristics on seismic performance assessments of structures considering MSAS is not investigated extensively and thus further research is warranted. In particular, considerations of axial-flexural force interaction between elements, cracking/crushing of cover and core concrete, longitudinal bar buckling and fracture due to fatigue are the most critical aspects of an advanced structural modeling to simulate damage accumulation in the structure realistically. A fiber beam-column model proposed in this study integrates all the above-mentioned sources of damage in simulation and provides significant improvements over conventional lumped plasticity models.

This paper quantifies the impact of MSAS sequences for a 2-story RC structure located in Seattle, US. Considerations of subduction (interface) earthquakes and advanced FE model that accounts for structural deterioration and degradation, such as longitudinal bar buckling and fatigue, distinguish this study from previous studies. Capturing different mechanisms of damage, and failure modes requires an advanced structural modeling which is one of the main motivations and novelties of this study.

**Ground Motion Selection Methodology**

The proposed building is located at a typical urban site condition in Seattle, Washington State, US (latitude/longitude = 47.609oN/122.333oW). Western Washington lies in the Cascadia subduction zone. An M 9 megathrust earthquake in the Cascadia subduction zone has the potential of extensive destruction and economic loss from Vancouver in Canada to Northern California in the US. The Cascadia rupture will cause strong ground shaking along the west coast, with shaking in excess of 1.0 g at many locations [10]. The greater Seattle area will experience 0.2 to 0.3 g accelerations due to such a subduction-zone earthquake. In addition to the offshore megathrust subduction zone, there are several faults in the upper crust that could cause significant earthquake damage due to proximity to the urban areas and shallow depth (5 to 20 km). Since the selected site is affected by a complex seismicity from various seismic sources (crustal, inslab, and interface), a comprehensive ground motion selection is essential for seismic performance assessment of buildings located in Seattle. In this paper, the results for subduction earthquake are presented.

**Site-Specific Seismic Hazard Information**

Using results of probabilistic seismic hazard analysis (PSHA) provided by the USGS (https://www.usgs.gov/), hazard curves that express the annual probability of exceeding various
values of intensity measures (IM) are extracted for the target site. Subsequently, a suite of ground motion records has been selected that is compatible with the selected hazard levels to be used in nonlinear dynamic analysis of the proposed structure. For the latter, earthquake scenarios based on seismic disaggregation can be used.

Although the influence of different IMs on nonlinear dynamic response of structures has been investigated by other researchers [11], selecting the most efficient and sufficient IM is still an open issue. The IM selected for this study is the 5% damped spectral acceleration response at the building’s estimated first mode period $Sa(T_1=0.5\,\text{s})$ where $T_1$ is the first natural vibration period of the structure. Due to the lack of locally recorded strong motion time-histories, ground motion data from the K-NET/KiK-net database for Japanese earthquakes [12] are integrated. The uniform hazard spectrum (UHS) and the 2% probability of exceedance in 50 years (MCE) design spectrum (IBC 2012 [13]) for the site of interest are shown in Fig 1(a). The UHS is constructed by enveloping the spectral amplitudes at all different periods that are exceeded with 2% probability in 50 years, as computed from PSHA. Knowing that the fundamental vibration period of the structure is 0.5 s and the annual frequency of exceedance of interest is 2% in 50 years, the $Sa(T_1)$ value is 1.381 g. Fig 1(b) shows the disaggregation distribution of magnitudes, distances, and their contributions to the site that will cause the occurrence of $Sa(T_1) = 1.381\,\text{g}$ at this site. The disaggregation information also shows three scenarios of events with different magnitudes and distances and their contributions to the specific site (Seattle). Therefore, when a single target response spectrum is constructed using the average values of magnitude and distance as suggested by the USGS, a significant bias may be caused in ground motion selection and consequently in seismic risk and performance assessment of the structure. To reduce the bias, a multi-scenario-based ground motion selection procedure is used by extending the single-scenario-based conditional mean spectrum (CMS) method proposed by Baker [14].

![Figure 1. (a) Uniform hazard spectrum and MCE design spectrum for Seattle and Maximum considered earthquake response spectra (2% probability of exceedance in 50 years) for Los Angeles and Seattle and (b) PSHA disaggregation for Seattle with regards to magnitude, distance, and earthquake type](image)
Ground Motion Selection

Fig 2(a) shows the predicted median spectra from ground motion prediction equations (GMPEs), the average predicted median spectra and +1σ spectra, for interface earthquakes. After defining the event-based CMS for the specific location and structure of interest, ground motion sequences are selected for interface event. It is worth mentioning that even though the records are from Japanese earthquakes, influenced by seismotectonic and site characteristics in Japan, target CMS are defined based on the USGS PSHA information. Considering that the selected GMPEs together with weights are suitable for the Pacific Northwest, the selected records reflect regional seismicity and expected ground motions for Seattle and thus are adequate for use in incremental dynamic analysis (IDA, [15]). Once the initial selection of records is completed, records can be matched with the target CMS for each scenario. The period range from 0.2T1 to 2.0T1 (0.1 s to 1.0 s) is selected in this study. Fig 2(b) shows response spectra of the selected ground motion (geometric mean of two components) matched with the target Sa(T1).

Figure 2. (a) Uniform hazard spectrum for Seattle with average predicted median spectrum, +1.7σ spectra associated with M=9 and R=11.8 km from multiple interface GMPEs, (b) Ground motions selected after scaling spectra and matching with the event-based CMS

RC Frame Building Model Development

The RC building considered in this paper is a 2-story moment-resisting frame taken from the benchmark buildings developed by Haselton [16]. The proposed frame represents a class of ductile RC frames designed according to the seismic code provisions of the 2003 IBC and ASCE 7-02 to ensure the ductile behavior, which prevents or delays sudden collapse under seismic loading. The ductile frame was designed for a high seismic hazard site in California, corresponding to the NEHRP soil category D. The MCE for Los Angeles and the counterpart for Seattle are depicted in Fig 1(a). The black solid line shows the MCE at Los Angeles based on IBC-2003 which is close to the MCE at Seattle (IBC-2012). Because differences of the design spectra at two locations are not remarkable, in addition to having the same soil condition
(NEHRP soil class D), the indicated building layout for structural modeling can be used for Seattle. Fig 3(a, b) show the properties of the moment-resisting RC frame and the details of the FE modeling. The proposed structural model in this study employs the force-based nonlinear fiber beam-columns elements available in OpenSees [17]. Further details of the modeling technique and experimental validation are available in Kashani et al. [18 and 19]. The zero-length bond-slip elements are used at the level of the foundation to simulate the reinforcement slippage at column-foundation interface. The computed T1 of the proposed model in this paper is slightly less than the T1=0.6s reported by Haselton [16] using the lumped plasticity model. In order to simulate cyclic response of the reinforcing steel, a uniaxial material model known as Steel02 is which is available in OpenSees is used. The Steel02 material model accounts for the Bauschinger effect. However, it does not account for cyclic strength and stiffness degradation due to bar buckling and fatigue. In this study, Kashani et al. [20] phenomenological material model of reinforcing bars is employed and implemented in OpenSees using a uniaxial Hysteretic material model. This model simulates the inelastic buckling of reinforcing bars. The generic Fatigue material model available in OpenSees is used to simulate low-cycle fatigue degradation of reinforcing bars. Once the Fatigue material reaches a damage index of 1.0, the stress of the parent steel material becomes zero. In this paper, the material model developed by Park et al. [21] is used for concrete. This model is known as Concrete02 in OpenSees. Monotonic nonlinear pushover analyses are performed for the 2-story frame; the results are shown in Fig 3(c). The proposed fiber model predicts the maximum base shear up to 10% more than the value calculated using the lumped plasticity model [16]. This difference is due to considering axial-flexural interaction in nonlinear fiber beam-column element.

Development of Fragility Functions

Developing fragility functions using nonlinear dynamic analysis of a structure is an important step in seismic vulnerability assessment procedures. A common approach is incremental dynamic analysis (IDA) which involves performing nonlinear dynamic analyses of a structural model using a set of ground motion records, each record scaled to multiple IM levels [15]. An advantage of IDA is that it covers a wide range of IM, thus allowing simulation of structural behavior from initial elastic response to complete collapse. In this study, to incorporate the aftershock effects in the assessment, analytical fragility is developed based on IDA of the structural model with and without considering longitudinal reinforcement. Fig 4 shows the median IDA results with corresponding fragility curves for the structural model with and without considering longitudinal reinforcement buckling and fatigue degradation.

Influence of Structural Modeling Technique

Inelastic buckling and low-cycle fatigue degradation of longitudinal reinforcement in RC structures significantly influence the nonlinear structural response to earthquake loading. Fig 4 shows the median IDA results with corresponding fragility curves for the structural model with and without considering longitudinal reinforcement buckling and fatigue degradation.
Figure 3. (a) Geometrical and RC details and (b) FE model description using nonlinear fiber beam-column elements (c) Static nonlinear pushover curves for the fiber model (this study) and lumped plasticity model by Haselton [16]

Results and Discussion

Influence of Structural modeling Technique

Results are presented for MSAS sequences of interface event. The maximum inter-story drift ratios of 0.005, 0.01, 0.03 and 0.08 which correspond to slight, moderate, extensive, and complete damage, respectively, are also shown on each graph. The results show that the fiber model including buckling and fatigue degradation increases the probability of collapse significantly. However, no significant change is observed in slight, moderate and extensive damage limit. Fig 5 presents the normalized concrete and steel strain (normalized with respect to crushing strain for concrete in compression and ultimate strain for steel in tension) in left-column at ground level for the proposed model (including bar buckling and fatigue) and the model with Steel02 (no buckling and no fatigue). It is clear that the maximum normalized strain response is higher considering buckling at drift levels of 0.03 for this specific column in comparison with Steel02.
Figure 4. (a) IDA and (b) fragility curves for the Steel02 and buckling-fatigue models by considering the interface MSAS records.

Figure 5. Normalized (a) steel and (b) concrete strain (left column at ground level in Figure 3a) for the buckling-fatigue model and the Steel02 model by considering the interface MSAS records.

Aftershock impact

Fig 6 shows the median IDA and corresponding fragility curves for MS and MSAS for the model including the bar buckling and fatigue effects for interface earthquake. The analyses results show that considering aftershocks does not have a significant impact on the structural damage at slight and moderate limit states. Fig 7 shows the maximum normalized strain of core concrete and reinforcement of the left column at ground level. The damage limit states based on inter-story drift ratios are defined for the global response of the structure and do not account for the impact of bar buckling and fatigue degradation at component level. As shown in Fig 7, aftershocks do not increase the structural damage at the slight and moderate damage limit states (it does not have any significant impact in increasing the maximum strain in materials in steel and concrete).
For interface events (with longer duration on average) considering aftershocks does not significantly increase the maximum strain, but the maximum strain in materials occur at lower drift ratios. This is due to the characteristics of ground motions, such as effective duration, energy therefore, consideration of fatigue in material significantly increases the accuracy of structural modeling.

Figure 6. (a) IDA and (b) fragility curves for the buckling-fatigue model by considering the interface MS and MSAS records

Figure 7. Normalized (a) concrete and (b) steel strain (left column at ground level) by considering the interface MS and MSAS records

**Conclusions**

In this paper, an advanced modeling technique using nonlinear fiber beam-column element is developed for RC frames. The proposed model is used for seismic performance assessment of a prototype 2-story RC framed structure considering subduction MSAS sequences. The main
conclusions of this paper can be summarized as follows:

1) Considering rebar buckling and low-cycle fatigue in a fiber-type model increases the probability of collapse and consequently the accuracy of risk and vulnerability assessments of RC buildings.
2) For interface records, damage is increased by about 5%. Slight and moderate damages are not affected by major aftershocks.
4) Analysis results show that considering global damage measures, i.e. drift, is not sufficient for fragility analysis of RC structures. An important area for future research is to develop damage indices for local components as well as global structural system that incorporate the effects of ground motion duration, the number of cycles, and cyclic degradation on structural performance.

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