Seismic collapse probability prediction of an RC frame building using various ground motion selection methods

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ABSTRACT: Seismic collapse fragility of the ten-story Red Book building as a template of RC frame buildings designed to current New Zealand standards is investigated in this paper. To simulate structural collapse, a fiber-element model is used which enables prediction of the sidesway mode of collapse as well as the loss in vertical load carrying capacity of structural components. Response history analyses (RHA) are performed at discrete hazard levels from low to high intensities to estimate collapse probability at individual hazard levels. Collapse probabilities are then fitted to a lognormal distribution to obtain the collapse fragility curve. In order to investigate the variation of the collapse fragility using 4 different ground motion (GM) selection methods at each hazard level, two alternatives of uniform hazard spectrum (UHS); conditional mean spectrum (CMS); and generalised conditional mean spectrum (GCIM). The use of a hazard-based intensity is also proposed in the paper. It is shown that depending on the ground motion selection method used, the predicted collapse probability of the building varies to a considerable extent.

1 INTRODUCTION

As collapse of old buildings in the past earthquakes has been the primary source of casualties and injury, collapse prevention of buildings is considered as a key performance measure to decide the acceptability of buildings designed using code provisions. Advancements in structural models and analysis have made it possible to simulate most of the collapse modes of buildings under earthquake ground motions (GMs). Major improvements in seismic collapse quantification of buildings have been made since the work of Ibarra and Krawinkler (2005) who introduced a procedure to compute collapse fragility curve of buildings and combined it with the seismic hazard curve to compute mean annual frequency of structural collapse. Since the introduction of this model, the majority of research in structural modelling for collapse simulation has concentrated on the development and improvement of nonlinear lumped plasticity models that capture the sidesway collapse mode of buildings, in which the lateral strength and stiffness become insufficient to resist destabilizing P-Δ effects leading to large inter-story drifts (Haselton 2006; Zareian et al. 2010; FEMA 2012a; FEMA 2012b; Lignos 2012; Zimmerman et al. 2015).

Inherent in commonly used lumped plasticity nonlinear modelling, proposed structural models suffer from several drawbacks. Significant among them is that the loss of vertical load carrying capacity of one or more components in the structure, such as a loss in axial capacity of columns, is not generally modelled. Another limitation of traditional lumped plasticity models is their inability to capture the variation of moment capacity of components cross sections due to the inevitable variation of axial force during lateral cyclic excitation. To overcome these limitations application of a fiber element-based nonlinear model for collapse analysis has recently been proposed by the authors (Koopaee et al. 2015). The fiber model enables simulation of the sidesway mode of collapse due to destabilizing P-Δ effects and also of the loss in vertical load carrying capacity of buildings due to inability of concrete in columns to carry axial compressive stresses due to permanently opened cracks. The fiber model is used in this paper for an improved simulation of the seismic collapse of buildings failing by flexure.

Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell 2002) has been commonly used for
probabilistic seismic collapse assessment of buildings using one suite of GMs which are scaled starting from elastic structural response to structural collapse. Individual GMs in this method are assumed to represent various hazard intensities merely by scaling the GM to match the intensity corresponding to the required hazard level. As a single suite of GMs are selected and scaled, GMs are treated as dynamic loading protocols which may not realistically represent the hazard intensities at various levels. Recently, a more convincing approach has been used, which is sometimes referred to as Multiple Stripe Analysis (MSA) (Jalayer and Cornell 2009; NIST 2011; Baker 2013; Bradley 2013). In this approach, different suites of GMs are used at different hazard intensities to make the analysis results more predictive of building behaviour at that site. It requires site-specific motions to be selected for each analysis case. In this approach, collapse assessment requires more information about the site. The analysis results are not only dependent on the structural properties but also on the site and hazard in the area.

The wider goal of the research is to understand the seismic collapse risk of typical RC frame buildings designed based on current New Zealand standards. In particular, this study aims to investigate the influence of the GM selection methods on collapse safety prediction of New Zealand buildings. Collapse safety assessment of RC buildings designed according to American standards has been previously studied (Haselton and Deierlein 2007; Haselton et al. 2010; Liel et al. 2010). The work in this paper however offers several improvements over the past research including (i) application of a fiber-element model to simulate structural collapse, enabling the modelling of loss in vertical load carrying capacity of structural components; (ii) using multiple stripe analysis to obtain the collapse fragility rather than IDA; (iii) comparison of different GM selection approaches in terms of collapse prediction at various hazard intensities; and (iv) assessment of the role of the choice of structural period on the collapse fragility curve.

![Figure 1 - Plan and elevation view of the New Zealand Red Book building used as the case study](image)

2 CASE STUDY BUILDING

The ten-storey New Zealand Red Book building (Bull and Brunsdon 1998), which acts as a design example of the New Zealand Concrete Standards has been adopted for this study. Figure 1 shows the plan and elevation views of the building layout. The primary lateral load carrying system consists of four one-way perimeter moment resisting frames which are three bays long. Vertical loads are transferred primarily through interior columns with gravity beams supporting one-way floor units.
Further details of the structural properties and design details can be found in Bull and Brunsdon (1998). The building is assumed to be located in Christchurch, New Zealand and has been designed for soil class C based on the NZ loadings standard (NZS1170.5 2004) site classification.

3 STRUCTURAL MODEL

A two-dimensional nonlinear analysis frame model is created in OpenSEES structural analysis platform (OpenSees 2012). Modelling of the frame members consists of nonlinear fiber elements used in Koopaee et al. (2015) for building collapse analysis. Popovics (1973) concrete model with an ultimate strain of $\varepsilon_{cu} = 0.03$ is used for modelling of confined concrete. To mimic the behaviour of steel bars the generic model proposed by Kunnath et al. (2009) is utilized which accounts for the strength degradation due to low-cycle fatigue in cyclic loading as well as buckling of reinforcing bars in compression. The degrading behaviour of RC sections is affected by buckling of longitudinal steel bars in compression, which is modelled based on (Dhakal and Maekawa 2002a; Dhakal and Maekawa 2002b). Further details of the structural model used as well as verifications can be found in Koopaee et al. (2015).

The loss of the vertical load carrying capacity and sidesway mode of collapse were considered. Other modes of collapse, such as shear failure of structural components and punching shear failure in slab-column joints, were neglected. The contribution of the slab in structural component stiffness and strength and base flexibility were also neglected in the structural model. A fiber-based section analysis was performed to identify the cracked stiffness of structural sections. Based on this analysis stiffness reduction factors of 0.63 and 0.42 were used for beam and column sections respectively to account for cracking. Using the cracked stiffness of structural components the period of the first mode amounts to $T_1 = 1.5s$.

4 GROUND MOTION SELECTION AT INDIVIDUAL HAZARD LEVELS

Two alternatives of the uniform hazard spectrum (UHS), from the code and obtained by performing a probabilistic seismic hazard analysis (PSHA) are used in the study to select ground motions at various hazard levels. Of the many alternatives to the UHS for ground motion selection, conditional mean spectrum (CMS), as first presented by Baker and Cornell (2006) and further elaborated in Baker (2010), is also examined. To include other characteristics of ground motions related to the structural response, generalised conditional intensity measure (GCIM) proposed by (Bradley 2010; Bradley 2012) is also used, in which IMs other than spectral acceleration can also be used to select GMs.

The range of period within which the GMs are scaled to match with the target spectrum for CMS and GCIM methods is decided based on structural characteristics. The lower limit of the period range is computed as the highest horizontal modal period whose inclusion takes the mass participation beyond 90% of the total mass of the building.

The upper limit of the scaling period range corresponded to the maximum likely post-yielding inelastic response of the structure, and is calculated using the secant stiffness of the structure corresponding to the design ductility as in the following equation:

$$T_e = T_i \left( \frac{\mu}{1 + r(\mu - 1)} \right)^{0.5}$$

$$T_c = \frac{\mu}{\sqrt{1 + r(\mu - 1)}}$$

(1)

Where $T_c$ is the secant period, $T_i$ is the initial elastic period, $\mu_i$ is the ductility factor, and $r$ is the ratio of post-yield to elastic stiffness of the system assuming that the skeleton force-displacement response is represented by a bilinear approximation. Since the $r$ factor is small, Eq. (1) can be simplified to $T_e = T_i(\mu)^{0.5}$ and $T_c = T_i(\mu)^{0.5}$. The case study building here has been designed for a ductility of 4 ($\mu = 4$) and hence, the upper limit of the period range becomes twice the initial period.
4.1 Selection of ground motions based on UHS

A PSHA is conducted to obtain the UHS at the site to be used as the target spectrum to select GMs. According to the code soil classification, shear wave velocity of the soil is assumed to be \( V_{s30} = 300 \text{ m/sec} \). The resulting UHS for two intensity levels are illustrated in Figure 2. Superimposed in the figure are also NZS1170.5 (2004) design response spectra, for which the building has been designed. As an alternative to the spectra obtained by PSHA, NZS1170 design spectra are also used as the target spectra to select GMs. Throughout the paper where the UHS is obtained by a PSHA, the method is termed as “PSHA” for brevity. Likewise, when the UHS is obtained by the NZS1170 spectra as the target the method is termed “NZS1170”, nevertheless the ground motion selection approach is not similar to the method stipulated in NZS1170.5 (2004).

![Figure 2 - UHS obtained by NZS1170 (2004) and PSHA along with CMS for 500 and 2500 years return periods for Christchurch](image)

Figure 2 shows that the two forms of UHS, i.e. obtained by NZS 1170 and PSHA, are similar in terms of spectral shape. However, the code spectra predict remarkably larger spectral accelerations compared to the values obtained by PSHA. At each hazard level, GMs that best match the site characteristics are selected from the Pacific Earthquake Engineering Research (PEER) NGA database. Selected GMs fall within a magnitude range of \( 5 \leq M_w \leq 8 \) and source to site distance of \( 20 \text{ km} \leq R_{JB} \leq 150 \text{ km} \) to ignore any near fault effects. They were produced by strike-slip or reverse faults, and recorded on soils with shear wave velocities within a range of \( 150 \text{ m/s} \leq V_{s30} \leq 400 \text{ m/s} \). Selected ranges of magnitude and distance are based on the two major Canterbury faults and Alpine faults in the area which can produce GMs with different characteristics. Among the GMs with these characteristics, 20 GMs (each with two horizontal components) are selected which, once scaled, best match the target spectrum within the period. Scaling of the GMs is limited to a factor within a range between 0.3 and 2.5. Although wide ranges of magnitude and distance have been chosen to select GMs to include the different source characteristics in the vicinity of Christchurch, the scaling factor limit to match the target spectra automatically excludes unrepresentative records to a large extent.

4.2 Selection of ground motions based on GCIM and CMS

For the case study building herein, it is anticipated that the seismic response of the structure will generally be a function of the intensity, frequency content and duration of the incident GM. In order to account for GM intensity over a wide range of vibration periods, use is made of peak ground acceleration, PGA; (pseudo) elastic spectral accelerations, \( S_a \), at ten vibration periods of 0.05s, 0.1s, 0.3s, 0.5s, 0.75s, 1s, 2s, 3s, 5s and 10s. As the structural model takes the fatigue effects of reinforcing
bars into account in cyclic loading it is also prudent to account for other aspects of the GM. As such, consideration is given to both cumulative absolute velocity (CAV), which accounts for the amplitude, frequency content and duration of GM in a cumulative manner, and also 5–75% and 5–95% significant durations, $D_{575}$ and $D_{595}$. Weighting factors, which indicate the importance of each $IM_i$, were applied such that 70% of the weighting was evenly distributed among the amplitude-based $IM$s (i.e. PGA, $Sa$ and CAV) while 30% was evenly distributed among the other $IM$s. The median of the GCIM distributions is the CMS which is used as the target spectrum for selecting ground motions based on CMS.

5  GENERATION OF COLLAPSE FRAGILITY CURVES

Different GMs are selected and scaled at limited discrete hazard levels to generate collapse fragilities. At different hazard levels, the collapse probability (i.e. $P(C|IM)$) is calculated as the ratio of the number of GMs causing collapse to the total number of GMs used in the RHA at the hazard level. Then, the cumulative distribution function, assuming a lognormal distribution, of collapse probability values at discrete hazard levels defines the collapse fragility curve.

The analysis of the case study building is continued until the collapse probability is larger than 50%. The analysis is stopped at 50% probability of collapse as it was shown by previous studies that intensities less than the structure’s median collapse intensity typically have the largest contribution to the annual rate of building collapse and they are therefore more important in the generation of the collapse fragility of buildings (Bradley and Dhakal 2008; Eads et al. 2012).

6  COLLAPSE PROBABILITY OF THE CASE STUDY BUILDING

RHAs are performed for the 40 selected GMs (two horizontal components of 20 GMs) at each hazard level corresponding to various return periods until more than 50% of the GMs cause collapse. Each component of a GM is applied separately to the structure. Since GMs are selected at each individual hazard level, a hazard related IM such as return period can be defined. Figure 3 illustrates collapse probabilities of the building at various hazard levels by various GM selection approaches. Collapse probabilities are depicted in terms of return period as well as $Sa(T_1=1.5s)$ and $S_a(T_1=1.5s)$. Although the analyses could be stopped in UHS methods once the collapse probability reaches above 50%, they were continued for two more hazard levels to be able to achieve more collapse data points and following up the collapse probability trend with increasing intensity. It is noted that in addition to the NZS1170 method, PSHA was used to estimate the hazard at each intensity level. Therefore, Figure 3a could lead to a misinterpretation of the collapse assessments as NZS1170 suggests higher structural responses at a given hazard level compared to the PSHA. As expected, the UHS methods provide the highest response compared to other approaches. Notably in the figure, at some rare events the NZS1170 results are close to PHSA method despite having higher spectral accelerations at the same hazard level. These close results mostly stem from the increase of the error in matching the selected GMs to the target spectrum in NZS1170. Limitation of the scaling factor in matching the GMs with a target spectrum may lead to further error in GM selection. This indicates the fact that, by using RHAs at discrete hazard levels instead of IDA for collapse assessment, continuing the analyses until very rare events in order to achieve more information does not necessarily increase the accuracy of the method when real GMs are selected to match a target spectrum.

Collapse fragility curves based on four different GM selection methods are shown in Figure 4. Table 1 summarizes the results of the collapse performance assessment of the building reading from the constructed collapse fragility curves. Results obtained by conducting IDAs in literature have shown record-to-record variability values ranging from 0.35g to 0.45g (Haselton 2006; Liel et al. 2009; Zareian et al. 2010). Fragiadakis and Vamvatsikos (2010) reported values ranging from 0.30 to 0.4, while FEMA (2009) proposes a value of 0.4. Record-to-record variability values given in Table 1 are based on a more rigorous approach adopted in this study; i.e. conducting RHAs at discrete hazard levels. The adopted approach results in larger variability due to record randomness compared to the values in literature, such that only NZS1170 method resulted in the variability less than 0.4. These
results suggest that record-to-record randomness in design documents may require revision in future performance-based guidelines. It is however noted that the large record-to-record randomness in this study may be reduced by selecting larger number of GMs at each hazard level.

Figure 3 - Collapse probability of the building at various hazard levels assuming cracked period of the structure

Figure 4 - Collapse fragility curve of the case study building using four different GM selection methods
Table 1. Collapse Probability (P(C|S_a)) of the Case Study Building at 500 and 2500 Years Return Period Events

<table>
<thead>
<tr>
<th>GM selection method</th>
<th>Median Collapse Capacity $S_e (T_1 = 1.5S)$</th>
<th>Record-to-record randomness</th>
<th>500 Years Return Period - Neglecting modelling uncertainty</th>
<th>Total Uncertainty</th>
<th>2500 Years Return Period - Considering modelling uncertainty</th>
</tr>
</thead>
<tbody>
<tr>
<td>UHS (via PSHA)</td>
<td>0.60g</td>
<td>0.46</td>
<td>12%</td>
<td>0.68</td>
<td>21%</td>
</tr>
<tr>
<td>UHS (via NZS1170)</td>
<td>0.57g</td>
<td>0.38</td>
<td>11%</td>
<td>0.6</td>
<td>23%</td>
</tr>
<tr>
<td>CMS</td>
<td>0.85g</td>
<td>0.41</td>
<td>2%</td>
<td>0.65</td>
<td>9%</td>
</tr>
<tr>
<td>GCIM</td>
<td>0.81g</td>
<td>0.58</td>
<td>8%</td>
<td>0.77</td>
<td>14%</td>
</tr>
</tbody>
</table>

In order to compare the results with collapse potential of ductile buildings reported in Haselton et al. (2011), a modeling uncertainty of $\sigma_{\text{ln,modelling}}=0.5$ is assumed. Haselton et al. (2011) have reported an average $P(C|S_{2%/50})$ of 11% ranging from 3 to 20% for ductile code-conforming buildings, whereas values in Table 1 range from 9 to 23% with an average of 17%. The increase in collapse probability may have stemmed from inclusion of loss due to vertical load carrying capacity and/or using RHAs at discrete hazard levels. It is however noted that Haselton et al. (2011) have looked at a range of buildings using the same GM selection while this work looks at a single building with different techniques.

7 SUMMARY AND CONCLUSIONS

Collapse safety of a ten-storey RC moment resisting frame as a template of medium-rise buildings designed based on the current NZ standards was investigated in this paper. RHAs using different ensemble of GMs at discrete hazard levels were conducted to estimate the collapse probability at each hazard level and generate collapse fragility curve of the building. Four different GM selection methods were examined to select GM records at individual hazard levels. In order GMs to match with a target spectrum at each hazard level, a period range to match with target spectrum was proposed which varies depending on structural characteristics. Collapse probability assessment of the case study building using this method showed a larger record-to-record randomness compared to that stated in literature based on IDA results. Among the four GM selection methods considered, GCIM method resulted in the largest record to record variability of 0.58. Inclusion of the loss of vertical load carrying capacity in the structural model resulted in larger collapse probability in comparison with average values reported in literature for a range of ductile structures. For 2500 years return period event, the collapse probability of the case study building is found to vary between 9% and 23% depending on the GM selection method adopted. It was observed that conventional GM selection method based on a UHS leads generally to conservative prediction of collapse probability of the building.

REFERENCES


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