# Seismic Behaviour of Reinforced Concrete Frames in Australia

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## ABSTRACT

Many intra-tectonic plate regions are considered to have low to moderate seismic risk. However, devastating earthquakes can occur in these regions and result in high consequences in terms of casualties and damage. This paper presents an experimental and analytical investigation to understand the seismic capacity of typically detailed Australian reinforced concrete (RC) frames. The experimental programme included a series of progressively increasing earthquake simulator tests, using base motion with design spectrum similar to that for firm soil sites in Australian design code. The analytical study consisted of inelastic time-history analyses of 3-, 5- and 12-storey RC frames with ground acceleration patterns based on artificially generated earthquake data for Boston region (on the east coast of the US). The main objectives of this research were (1) to investigate the behaviour of non-seismically designed RC frames under a 500 year return period (YRP) earthquake and (2) to determine the different magnitudes of earthquake (YRP) that are likely to cause excessive structural and non-structural damage or collapse of gravity-load-designed (GLD) RC frames. The performance of the frames was analysed in relation to the drift limits, base shear, ductility and overstrength.

**Keywords:** reinforced concrete frames, earthquake, drift, ductility, overstrength, static push-over analysis, dynamic analysis, Australia.

# 1 INTRODUCTION

Earthquake-resistant design consists of determining the anticipated demands and providing the necessary capacity to meet these forces and/or deformations by satisfying prescribed safety and serviceability criteria or limit states (Naeim 2001). However, there is much uncertainty associated with the modelling of seismic hazard in regions of low seismicity such as Australia owing to the paucity of earthquake data in these regions (Hutchinson et al. 2003). Reinforced concrete (RC) frames built in the majority of these regions are designed primarily for combinations of gravity and wind loads. Non-ductile detailing practices employed in these structures make them prone to potential damage and failure during an earthquake. Moreover, the collapse limit state has been identified as the critical state for low seismicity regions (Pappin et al. 2000). However, structures in these regions have been assigned low seismic design intensities and there is growing evidence that this design approach could lead to severe damage or loss of life (Paulay et al. 1992).

Relevant information from the past analytical and experimental studies has contributed towards the establishment of appropriate seismic demand and capacity limits of structures. Therefore, it is essential to test realistically designed concrete frames to evaluate the complex interaction between beams, columns and joints during an earthquake event (Bechtoula et al. 2006). A few studies have been conducted to investigate the dynamic response of non-seismically designed RC frames against earthquake loading. In an Australian research by Corvetti et al. (1993), detailing was found to be the key aspect in achieving the required performance of three ½-scale exterior RC beam-column joints on the frames The seismic behaviour of RC frame with band beams, designed and detailed according to the Australian Standards was investigated in two sets of tests performed by Stehle (2001) and Abdouka (2003). The study revealed that the interior and exterior wide-beam-column specimens with no special provision for seismicity suffered undesirable torsional cracking together with the pullout of the bottom beam bars. In contrast, the specimens with improved detailing showed ductile behaviour. Furthermore, a series of time history analyses were conducted on a 4-storey frame designed for a region of low seismicity (Stehle 2001). The study revealed that the frame responded with less than a peak inter-storey drift ratio of 2% for low and moderate seismic events however, higher levels of seismicity may cause some undesirable torsional cracking in the structure.

In this study, the performance of non-seismically designed RC frames under different earthquake ground motion records was assessed through experimental and analytical work. The experimental programme consisted of shaking table tests of a 1/5-scale, 3-storey RC frame using design code compatible ground motions for firm soil sites consistent with the Australian earthquake design code. The analytical study included static pushover and non-linear dynamic analyses of 3-, 5- and 12-storey RC frames. The main objectives of this study were (1) to investigate the behaviour of non-seismically designed RC frames under a 500 year return period (YRP) earthquake and (2) to determine the different magnitudes of earthquake (YRP) that are likely to cause excessive structural and non-structural damage or collapse of gravity-load-designed (GLD) RC frames.

# 2 EXPERIMENTAL STUDY

An experimental research project was conducted at the University of Adelaide, which involved the shake-table testing of a 1/5-scale model of an Australian detailed RC frame. The model frame was designed in accordance with the Standards Association of Australia (SAA) Concrete Structures Code, AS 3600 (1988), as a 3-storey, single bay portal frame with a storey height of 0.7m and column spacing of 1.2m. For the model frame, 4 and 6 mm deformed wires with $f_{sy}$ = 615 MPa and $f_{su}$ = 650 MPa were heat treated to improve their ductility. After heat treatment the properties were $f_{sy}$ = 570 MPa and $f_{su}$ = 620 MPa. Compressive strength of the micro-concrete at the time testing was 64 MPa. Concrete cover of 5.5 - 8 mm was used for the scale model. For the scale model to be representative of typical RC structures in Australia, the reinforcement details similar to the details used in the beam-column joints constructed and tested in Melbourne (Corvetti et al. 1993). Figure 1 shows the cross-section and joint details of the 1/5-scale model structure tested.

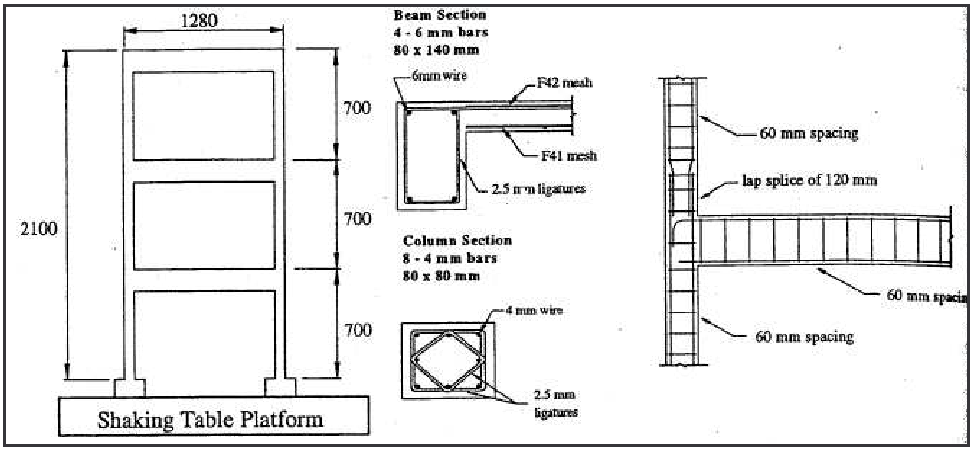


**Figure 1—Elevation and joint detail of 1/5-scale model test structure**

The model structure was subjected to simulated earthquakes with increasing magnitudes between 0.026g and 0.126g (Table 1), with free vibration tests conducted before and after each earthquake test, in order to measure the change in the natural frequencies and damping ratios of the model. However, for large values of EPA (exceeding 0.126g) the shake table began to rock. Hence, testing was stopped at this point. The √5 time scaled North/South component of the 1940 EI Centro, California strong ground motion record was used for the tests as its response spectrum shape closely matches the design spectrum for firm soils in the Australian Earthquake Code, AS 1170.4 (2007). Structure on soft soil sites would be more vulnerable, (Goldsworthy 2007) so these values are perhaps a lower-bound estimate of effects on frames for the same return period.

### *Test Results and Discussion*

**Table 1— Experimental results 1/5-scale model test structure**

| Earthquake test | EPA | Maximum Base Shear (V) | Maximum Roof Drift (%) | Natural Frequency | Damping Ratio |
|---|---|---|---|---|---|
| EQ2 | 0.026 g | 0.049W | 0.071 | 3.3 Hz | 3.7% |
| EQ3 | 0.031g | 0.078W | 0.11 | 3.2 Hz | 3.2% |
| EQ4 | 0.043g | 0.110W | 0.17 | 3.2 Hz | 3.4% |
| EQ5 | 0.047g | 0.146W | 0.22 | 2.9 Hz | 3.7% |
| EQ6 | 0.056g | 0.167W | 0.26 | 3.3 Hz | 4.5% |
| EQ7 | 0.069g | 0.197W | 0.34 | 3.2 Hz | 5.4% |
| EQ8 | 0.078g | 0.221W | 0.39 | 3.2 Hz | 5.8% |
| EQ9 | 0.093g | 0.237W | 0.47 | 3.1 Hz | 5.3% |
| EQ10 | 0.105g | 0.268W | 0.52 | 3.2 Hz | 5.5% |
| EQ11 | 0.126g | 0.311W | 0.84 | 2.9 Hz | 5.6% |

EPA = Effective peak shaking table acceleration, g = acceleration of gravity, W = total weight of model

The free-vibration tests conducted before the earthquake tests revealed that the first mode frequency and the damping ratio for the model frame were 3.2Hz and 3.2%, respectively, for the undamaged frame (Table 1). There was no significant change in the natural frequency of the model, although it did reduce slightly over the course of testing. This was due to the fact that the frequency was monitored using free-vibration tests that imposed only very small strains on the structure so that the decrease in frequency was primarily due to cracking in the concrete.

Furthermore, from Table 1 it can be noted that all the maximum roof drift values were well within the code allowable drift limit of 1.5% specified by AS1170.4 (2007). The ½-scale test specimen in Corvetti et al.'s study (1993), having same detailing features as the 1/5-scale model frame in the present study, experienced premature failure and excessive cracking at a drift of 0.8%. However, no such brittle failure mechanism was observed during the experimental tests for the present study. This was attributed to the fact that small scale structures can exhibit better bond and confinement behaviour than their full-scale counterparts, although it could not be quantified. Hence, the model structure was considered to perform well for the design magnitude (500-YRP) earthquakes for Australia. As mentioned previously the model frame was not subjected to EPA exceeding 0.126g due to the onset of rocking of the shake table and hence, the question remained at the conclusion of testing: how close to collapse was the structure or, more generally, what magnitude of earthquake is likely to generate structural failure and/or collapse for GLD RC frames.

## 3 ANALYTICAL STUDY

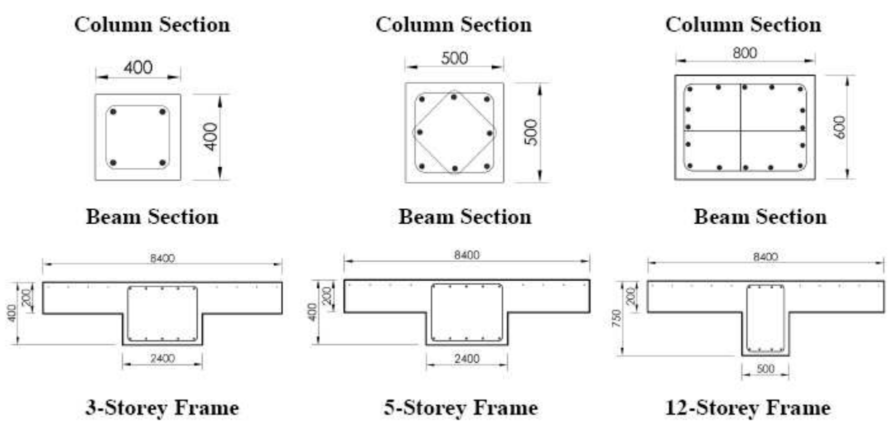


**Figure 2— Member details of frames**

**Table 2—Beam and column reinforcement details**

| **Beam Reinforcement** | | | |
|---|---|---|---|
| **Longitudinal Reinforcement** | | **3 and 5-Storey Frame** | **12-Storey Frame** |
| External beam section | Top reinforcement | 19 –20 mm bars | 11 – 24 mm bars |
| | Bottom reinforcement | 27 – 20 mm bars | 9 – 24 mm bars |
| Internal beam section (outer span) | Top reinforcement | 30 – 20 mm bars | 14 – 24 mm bars |
| | Bottom reinforcement | 27 – 20 mm bars | 9 – 24 mm bars |
| Internal beam section (inner span) | Top reinforcement | 30 – 20 mm bars | 14 – 24 mm bars |
| | Bottom reinforcement | 19 – 20 mm bars | 7 – 24 mm bars |
| **Column Reinforcement** | | | |
| **Main Bars** | **3-Storey Frame** | **5-Storey Frame** | **12-Storey Frame** |
| Column Section | 4 –32 mm bars | 8 –28 mm bars | 16 –28 mm bars |

In this part of the study three typical GLD Australian frames (full scale) (Chong et al. 2006) were analysed to determine the magnitude of earthquake (YRP) to cause drifts greater than 1.5% and greater than 2.5% in GLD RC frame structures. These values were taken from SEAOC Vision 2000 (1995) as being representative of the drifts when damage to structural and/or non-structural components would become excessive (>1.5%) or collapse would be approached (>2.5 %). It is recognised that these values may be conservative. The three frames considered were 3-, 5- and 12-storey RC frames with 3 bays of 10m each. The storey height was 4 metres for the 3- and 5-storey frames and 4.2 metres for the 12-storey frame. The 3- and 5-storey frames were designed using the band-beam system whereas standard beam design with flat concrete slabs was used in the 12-storey frame. The frames were designed in accordance with the SAA Concrete Structures code, AS 3600 (2001). Figure 2 shows the member details for all frames and Table 2 summarises the beam and column reinforcement arrangements used in the frames.

A non-linear computer program developed at the University of Kyushu was used for the analytical study (Kawano et al. 1998). In the analytical study the authors used the full frame spacing of 8.4m when modelling the width of the top flange of the band beams for the 3-storey and 5-storey frames. While studies by Stehle (2001) and Abdouka (2003) have shown that large drifts are required to fully yield the band beams, the fact that band-beam yielding could not be properly modelled was not felt to be a problem in these models as the plastic hinges formed in the columns rather than the beams. The program was validated using the experimental results of the 1/5-scale RC frame discussed in section 2 and a single RC column (Wu 2002) that were tested at the University of Adelaide. From the validation process it was established that Kawano's Program provided reasonably accurate predictions of the experimental results all the way to collapse. In all subsequent analyses, hinge rotations and column shear were monitored to ensure that premature local failures did not occur.

***Static Pushover Analysis***

Static pushover analysis was performed to quantify the ductility and overstrength of the model frames and compare them with results from earlier studies and code expressions. For this analysis, an equal lateral force was applied at each level of the frame simultaneously and the magnitude of these forces was increased gradually using the computer program detailed in the previous section.

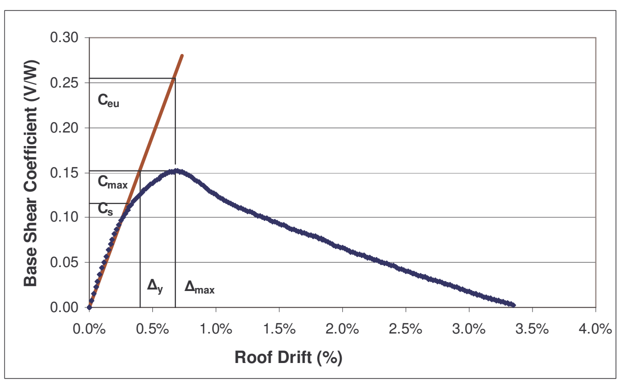


**Figure 3—Analytical static pushover curve for 3-storey frame**

The displacement in the structure was calculated at various lateral force levels and is plotted in Figure 3 in terms of the total lateral force (normalised by the total structural weight W) versus the drift at roof level. The straight line in Figure 3 shows the idealised linear elastic response of the building, which was drawn by extending the initial elastic portion of the response curve. This Figure also shows the points corresponding to the maximum base shear coefficient ($C_{max}$), the base shear coefficient at first significant yield ($C_s$), the maximum deflection ($\Delta_{max}$) corresponding to the peak strength, the deflection at yield ($\Delta_y$) and the base shear coefficient for elastic response ($C_{eu}$).

The results of the static pushover analysis for the three frames have been summarised in Table 3. The ductility and overstrength properties were computed by idealising the frame response (Figure 3). The total overstrength of a frame ($\Omega_{Total}$) is given by Eq. 1. Jain et al.'s study (1995) indicated that the overstrength was much higher for lower seismic regions in comparison to higher seismic regions. In the present study, the total overstrength was found to be in the range of 2 to 5. These values were much lower than the overstrength of the non-seismic frame tested by Lee et al. (2002) which was found to be 8.7 owing to the much different member and joint details used in the Korean study. Nevertheless, the code overstrength calculated in the present study was quite high and was determined to be in the range between 1.8 and 2.6, as shown in Table 3. Also, it was found that the structural ductility factor ($\mu_s$) of the 3-storey frame of this study was less than that reported by Lee et al. (2002) for the 3-storey frame they tested (2.4). However, the structural ductility of both the 3-storey frame and the 5-storey frame was found to be similar to the values specified in Abdouka's study (2003) for the tested non-ductile band beam system (1.29 - 1.65).

$$\Omega_{Total} = \Omega_s \Omega_y = \frac{C_s}{C_w} \times \frac{C_{\max}}{C_s} = \frac{C_{\max}}{C_w} \tag{1}$$

$$(R_\mu) = (C_{eu}/C_{\max}) \tag{2}$$

In order to have a direct comparison with the other international established research results, the formula derived by Uang (1991) (Eq. 3) was used for calculating the structural response modification factor ($R_f$). The values of $R_f$ calculated according to Eq. 3 for the three frames were compared with the structural response modification factor given in the AS 1170.4 (1993) which is 4.0 for concrete frame buildings with the joint and cross-section details used in the experimental study. In this equation, Uang (1991) assumed that the $R_f$ is a product of ductility ($R_u$) and overstrength ($\Omega_{Total}$). Also, the code prescribed base shear coefficient ($C_w$) was calculated for the three frames as per AS 1170.4 (1993) using Eq. 4 (the respective values are shown in Table 3). The results show that for the 3- and 5-storey buildings, the response modification factors calculated as per Uang's (1991) formula are in fairly good agreement with the code specified value of 4 (Table 2). However, for the 12-storey frame this $R_f$ factor is much higher (7.58) which suggests that the high rise frames are more resistant, in terms of base shear strength, than required by the code AS 1170.4 (1993).

**Table 3—Summary of static pushover results**

| Parameter | 3-Storey Frame | 5-Storey Frame | 12-Storey Frame |
|---|---|---|---|
| Overstrength above yield ($\Omega_y$) | 1.31 | 1.42 | 1.80 |
| Overstrength above AS 1170.4 ($\Omega_s$) | 1.84 | 2.11 | 2.58 |
| Total Overstrength ($\Omega_{Total}$) | 2.41 | 3.00 | 4.64 |
| Base Shear Coefficient at Peak Strength ($C_{max}$) | 0.15 | 0.16 | 0.14 |
| Code Base Shear Coefficient AS1170.4 ($C_w$) | 0.06 | 0.06 | 0.03 |
| Structural Ductility Factor ($\mu_s$) | 1.71 | 1.58 | 1.61 |
| Ductility Reduction Factor ($R_\mu$) | 1.69 | 1.60 | 1.63 |

$$R_f = R_\mu \Omega_s \Omega_y = \frac{C_{eu}}{C_{\max}} \frac{C_{\max}}{C_s} \frac{C_s}{C_w} = \frac{C_{eu}}{C_w} \tag{3}$$

$$C_w = \frac{V_w}{W} = I\left(\frac{CS}{R_f}\right) \tag{4}$$

where, I is importance factor; C is earthquake design coefficient; S is site factor; $V_w$ is base shear; W is gravity load; $R_f$ is structural response factor; and $CS \leq 2.5a$, where, a is acceleration coefficient.

***Non-Linear Time History Analysis***

Kawano's program was used to analyse the RC frames in terms of deflected shapes, hysteresis relationships, and base shear forces energy. Artificially generated site specific Boston earthquake ground motions (Somerville et al. 1997) were used for the dynamic analysis of the three different RC frames because their durations and frequency content were varied according to their respective magnitude with the 2500 YRP event having much longer duration and low frequency content. Comparison between these results and those using the earthquake ground motion model suggested by Wilson et al. (2003) is now underway. Unique 500-YRP and 2500-YRP earthquake ground motions were available for Boston with respective PGAs of 0.05g and 0.15g. From 20 acceleration patterns available, maximum acceleration values were selected for each return period earthquake. These earthquakes were then scaled by a factor to simulate the EPA expected in Australia for the corresponding 500 YRP and 2000 YRP event. The value of EPA for a given probability of exceedance was calculated using the probability factor (kp) from the AS 1170.4 (2007). Intermediate earthquakes that would cause a frame to show non-linear response were also investigated by further scaling.

***Global Response***

Table 4 summarises the dynamic analyses results for the three frames. The interstorey drift index (IDI) is defined as the percentage of relative displacement of a storey over its height. As shown in Table 3, the maximum IDI for the 3-storey frame under a 500-YRP earthquake was calculated as 0.89%, which was less than the life safety limit of 1.5% as per SEAOC Vision 2000 (1995). Hence, it was concluded that the 3-storey frame would maintain its stability for the design earthquake of 500-YRP. In contrast, the frame experienced interstorey drifts of 1.64% and 2.37% under the 800- and 2500-YRP earthquakes, respectively. This suggests that the 3-storey frame would suffer major structural damage under an 800-YRP earthquake as the maximum drift exceeded the life safety limit of 1.5% and that it was near the collapse state (2.5% drift) for the 2500-YRP earthquake as per SEAOC Vision 2000 (1995).

**Table 4— Summary of maximum drift and base shear for frames**

| **3 Storey Frame** | | | | |
|---|---|---|---|---|
| **Type of Earthquake** | **Maximum Roof Drift (%)** | **Maximum IDI** | | **Max. Base Shear Coefficient (V/W)** |
| | | **Drift (%)** | **Storey** | |
| 500 YRP | 0.65 | 0.89 | $2^{nd}$ | 0.14 |
| 800 YRP | 1.2 | 1.64 | $2^{nd}$ | 0.16 |
| 2500 YRP | 1.7 | 2.37 | $1^{st}$ | 0.18 |
| **5 Storey Frame** | | | | |
| **Type of Earthquake** | **Maximum Roof Drift (%)** | **Maximum IDI** | | **Max. Base Shear Coefficient (V/W)** |
| | | **Drift (%)** | **Storey** | |
| 500 YRP | 0.4 | 0.72 | 4th | 0.14 |
| 1000 YRP | 0.7 | 1.36 | 3rd | 0.16 |
| 2500 YRP | 1.0 | 1.78 | 3rd | 0.17 |
| **12 Storey Frame** | | | | |
| **Type of Earthquake** | **Maximum Roof Drift (%)** | **Maximum IDI** | | **Max. Base Shear Coefficient (V/W)** |
| | | **Drift (%)** | **Storey** | |
| 500 YRP | 0.15 | 0.25 | 11th | 0.14 |
| 2500 YRP | 0.35 | 0.88 | 8 th | 0.17 |

Table 4 shows the maximum IDI for the 5-storey frame under the 500-YRP and 1000-YRP earthquakes were lower than 1.5% which indicates that the frame response would be within the strength limit state for these earthquakes. Under the 2500-YRP earthquake, the 5-storey frame experienced the maximum IDI of 1.8 % indicating that the frame would likely suffer some significant structural damage exceeding the life safety limit but was still below the collapse limit state of 2.5% lateral drift. The 12-storey frame did not exceed 1.5% drift under either the 500-YRP or the 2500-YRP earthquake, and hence, the analysis for an intermediate YRP earthquake was not performed. As shown in Table 4 the 12-storey frame experienced maximum IDI of 0.25% and 0.88% under the 500-YRP and 2500-YRP earthquakes, respectively. This suggest that the frame would remain operational for the design earthquake of 500-YRP and suffer only moderate damage under a 2500-YRP earthquake. Therefore, it was concluded that the 12-storey frame could suffer yielding under the 2500-YRP earthquake but it might be able to resist the collapse.

## 4 SUMMARY AND CONCLUSIONS

From the response of the frames analysed in this study, following conclusions were drawn:

The experimental results and analytical studies indicated that for the 3-storey RC frame was able to resist the "design magnitude earthquake" (500-YRP with EPẠ≈ 0.1g) safely, with storey drifts well below 1.5%.

The response modification factor of the 12-storey frame calculated by Uang's (1991) formula was much higher than the code value. This suggests that the high rise frames are more resistant, in terms of base shear strength, than required by the Australian Code.

Dynamic analysis results showed that all the frames were able to survive the 500-YRP earthquake (EPẠ≈ 0.1g) with minimal structural damage and also their responses were within the life safety limit (1.5% maximum lateral drift) as per SEAOC Vision 2000 (1995). However, for the 2500-YRP earthquake the 3-storey and 5-storey frames were at or near the collapse limit state. This suggests that simply satisfying the strength limit state for the 500-YRP earthquake will not prevent collapse and significant loss of life in a bigger than expected (e.g., 2500-YRP) earthquake.

The 3-storey and 5-storey frames were expected to have soft ground storey failure mechanism at collapse due the fact that they had a weak-column strong beam design due to their use of band-beams with large moment capacities. However, the 12-storey frame was expected to exhibit a weak-beam strong-column failure mechanism.

From the overall performance of RC frames considered in this study, it is concluded that the GLD RC structures appear to be capable of resisting a "design magnitude earthquake" (i.e., 500-YRP) in low earthquake hazard regions. However, these frames are likely to approach collapse under more severe earthquakes (e.g. a 2500-YRP earthquake). Perhaps the earthquake design requirements should consider as an alternative the 'collapse prevention' limit state for longer return period earthquakes, of the order of 2000 – 2500 YRP.

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