STATEMENT OF ORIGINALITY

This work contains no material which has been accepted for the award of any other degree or diploma in any university or other tertiary institution and, to the best of my knowledge and belief, contains no material previously published or written by another person, except where due reference has been made in the text.

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Date: __________________________________________
The following publications were written based on the work presented in this thesis.

**Journal Papers**


**Conference Papers**


I would like to express my deep and sincere gratitude to Assoc. Prof. Michael Griffith and Dr. Togay Ozbakkaloglu, supervisors of this project, for their valuable guidance, generous support and encouragement throughout this study. Their friendly discussions and valuable comments have been very helpful in preparation of this thesis.

The Earthquake Engineering Research Scholarship provided by the Australian Earthquake Engineering Society (AEES) is gratefully acknowledged.

Sincere thanks are extended to the computing officer, Dr. Stephen Carr and the departmental administrative staff, Ms Diane Keable, Mrs Josie Peluso, Mrs Ann Smith and Mrs Julie Ligertwood, for their help.

A special thank you is owed to Mr. Sudhir Adettiwar for his support. It was great to have support from my friends Rupali, Sanyogita and Parag for helping me to get through difficult times. I also wish to thank my colleagues for their assistance and friendship.

Finally, I would like to dedicate this dissertation to my family and my husband, Mahesh Neelam for their unwavering support. I wish to express my heartfelt thanks for their understanding, encouragement and patience.
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. Non-ductile detailing practice employed in these structures make them prone to potential damage and failure during an earthquake. Furthermore, the use of infill walls is a divisive issue as on positive side dual wall-frame systems have beneficial effects related to strength, stiffness, and ductility. However, if not designed properly infill wall can also lead to undesirable structural failures of complete wall frame system. Although, there has been significant amount of international research in this area, it is worth noting that very little research exists for Australian frames.

This thesis presents the experimental and analytical research conducted at The University of Adelaide to gain some insight into the behaviour of typically detailed Australian reinforced concrete frames subjected to ground motions. The main objectives of this research were (1) to investigate the behaviour of non-seismically designed reinforced concrete frames under a 500-YRP earthquake; (2) to determine the different magnitudes of earthquake (YRP) that are likely to cause excessive drifts in or collapse of gravity-load-designed reinforced concrete frames and (3) to investigate the effect of infill walls on the moment-resisting frames subjected to seismic loads. The experimental program consisted of earthquake simulation tests on a 1/5 scale model of a 3-storey reinforced concrete frame and four ½-scale reinforced concrete brick infilled frame specimens subjected to quasi-static cyclic loading. The analytical study included static pushover and non-linear dynamic analyses of the 3-, 5- and 12-storey reinforced concrete frames.

From the overall performance of gravity-load-designed bare reinforced concrete frames considered in this study, it was concluded that the non-seismically designed frames appear to be capable of resisting a “design magnitude earthquake” (i.e., 500-YRP) in low earthquake hazard regions. However, their behaviour under more severe
earthquakes (e.g. a 2500-YRP earthquake) is questionable. 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. The
experimental research on reinforced concrete infilled frame indicated that the infill
wall does not adversely effect the in plane ultimate strength, stiffness, and ductility
of the bare reinforced concrete frame.
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a     Acceleration coefficient
C     Earthquake design coefficient
C_{eu}  Ultimate base shear coefficient
C_{max}  Maximum base shear coefficient
C_s     Ratio of maximum strength at failure
C_w     Code-prescribed strength
C_y     Actual structural yield strength
D_c    Smaller column dimension for rectangular column
D_b    Diameter of the smallest bar in the column
E_b    Young’s modulus of brick infill
E_c    Young’s modulus of concrete
E_s    Young’s modulus of steel
f_c    Concrete compressive strength
f_{sy} Yield strength of steel
g     Acceleration due to gravity
H_{total}  Total height of the building
h_{storey} Height of each of the storey of the building
I     Importance factor
k_p    Probability factor
L     Length
P     Annual probability of exceedance
R_f    Structural response modification factor
R_u    Ductility reduction factor
S     Site factor
S_a    Dynamic ground acceleration
T     thickness of infill
V     Base shear
W     Weight of building
Z     Earthquake hazard factor
\( \Delta E \)  Cumulative energy dissipated
\( \Delta_y \)  Displacement at yield
\( \Delta_{\text{max}} \)  Maximum displacement
\( \Delta_{\text{roof}} \)  Roof displacement
\( \Delta_{\text{storey}} \)  Storey displacement (interstorey)
\( \mu_s \)  Structural ductility factor
\( \Omega_s \)  Overstrength above code
\( \Omega_y \)  Overstrength at first yield
\( \Omega_{\text{total}} \)  Total overstrength
\( \theta \)  Angle of infill diagonal to the horizontal

ACI  American concrete institution
ARP  Annual return period
AS  Australian standard
ATC  Applied technology Council
CEB  Comité euro-international du béton
(Euro-international committee for concrete)
DE  Design earthquake
DME  Design magnitude earthquake
EPA  Effective peak acceleration
FEMA  Federal emergency management agency
GLD  Gravity load designed (frame)
IDI  Interstorey drift index
IMRF  Intermediate moment resisting frame
LVDT  Linear Voltage Displacement Transducer
MCE  Maximum considered earthquake
MRF  Moment resisting frame
NEHRP  National earthquake hazards reduction program (American)
OMRF  Ordinary moment resisting frame
PGA  Peak ground acceleration
RC  Reinforced concrete
SAA  Standards Association of Australia
SEAOC  Structural Engineers Association of California
SMRF  Special moment resisting frame
TFT  Taft earthquake
UBC  Uniform building code
URM  Unreinforced masonry
YRP  Year return period
Chapter 1

Introduction

1.1 Background and Research Needs

Many intra-tectonic plate regions are considered to have low to moderate seismic risk. However, devastating earthquakes (e.g., Charleston 1886 (Johnston 1996); Bhuj 2001 (Humar et al. 2001); Molise 2002 (Maffei et al. 2004)) can occur in these regions and result in high consequences in terms of casualties and damage. Over the last several decades extensive studies have been conducted on the seismic performance of multi-storey structures. However, little is known about the seismic behaviour of structures located in regions of low seismicity. Low to medium rise reinforced concrete (RC) structures built in the majority of these regions are designed primarily for combinations of gravity and wind loads. Therefore, during an unexpected seismic excitation, satisfactory response of such framed structures relies on their inherent ductility and overstrength (Kaung et al. 2005). Also, the lack of knowledge regarding site specific earthquake records in these regions makes it difficult to develop suitable design spectra for seismic analysis. Hence, evaluation of seismic behaviour of RC structures in low seismicity regions where design is dominated by gravity load considerations is of significant interest.

Although gravity-load-designed (GLD) RC frame buildings in Australia appear capable of surviving “design magnitude” earthquakes (i.e., earthquakes with 500 Year Return Periods (YRP)), there is much uncertainty on their response to longer return period earthquakes (Griffith 2003). Research by Pappin et al. (2000) concluded that for low hazard regions such as Australia, the collapse limit state
rather than the ultimate strength limit state is likely to govern the design of a building. For the collapse limit state, a structure must not collapse after experiencing an earthquake ground motion greater than 2000 YRP (Pappin et al. 2000).

![Graph showing relationship between PGA and annual probability of exceedance for different seismic regions](image)

**Figure 1.1—Relationship between PGA and annual probability of exceedance for different seismic regions (from Paulay et al. 1992)**

Figure 1.1 illustrates how the ratio of peak ground acceleration (PGA) for the 2000 YRP over the 500-YRP earthquakes is much higher for low seismicity regions (about 2.8) than for high seismicity region (about 1.2). This agrees with the research by Pappin et al. (2000) and implies that for low earthquake hazard regions, even if a structure is designed for the strength limit state (500 YRP) earthquake, it will not necessarily satisfy the collapse limit state. This raises serious questions on the performance of GLD frames in many low seismicity regions where they are presently designed only for the strength limit state earthquake, if at all.

Another issue which needs greater attention is the seismic performance of dual wall-frame systems. RC frames with clay brick masonry infill is a common structural system worldwide due to economic and traditional reasons. Unreinforced masonry infills mainly function as a building envelope or internal partitions. As demonstrated during many earthquakes (e.g., Izmit 1999 (Sezen et al. 2000); Bhuj 2001 (Humar et al. 2001)), the masonry infill panels can drastically alter the seismic performance of
the complete structural system due to their interaction with the bounding frame. The use of masonry infill can enhance the overall lateral load capacity of the structure. Moreover, the serviceability displacement limits can be fully achieved by infilled frames which bare structures cannot satisfy (Calvi et al. 2004). However, if not designed properly infill walls can also lead to undesirable structural failures of complete wall frame system. Hence, it is important to assess the seismic behaviour of infilled frames.

Research in Australia has been accelerated mainly after the tragedy of the 1989 Newcastle earthquake which revealed that a lethal earthquake can occur in Australia. Limited research has been done on the seismic performance of typically detailed Australian RC frames. In particular major concerns are related to the performance assessment of GLD RC frames, owing to the non-ductile detailing practice employed in these structures and divisive issue regarding the effect of infill walls on the seismic behaviour of moment resisting frames (MRF).

1.2 Objectives of the Research

The overall focus of the present research is to gain some insight into the seismic behaviour of RC frames. The main objectives of this research were (1) to investigate the behaviour of non-seismically designed RC frames under a 500-YRP earthquake, (2) to determine the different magnitudes of earthquake (YRP) that are likely to cause excessive drifts in (> 1.5%) or collapse of (> 2.5%) GLD RC frames and (3) to study the effect of infill wall on the performance of RC frames subjected to seismic loads.
1.3 Methodology and Scope of Work

The performance of non-seismically designed RC frames under the 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 motion 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 ground acceleration patterns for the analytical study were based on artificially generated site-specific earthquake data for Boston, USA covering a range of return periods from 500 to 2500 years. Behaviour of the frames was analysed in relation to the local and global response, drift limits, base shear, ductility and overstrength.

The effect of infill on the MRF was assessed by means of experimental study. The experimental programme included tests of four ½-scale frame specimens under quasi-static cyclic loading. One of the frames tested did not have brick infill and served as a reference for the three infilled frames with gap sizes of 5mm, 10mm and 15mm, respectively. Important parameters such as the level of ductility, stiffness and strength of a RC frame were considered.

1.4 Outline of Thesis

The research presented in this thesis has been divided into six chapters: Chapter 2 presents the conceptual review of topics essential for the seismic analysis of RC frames relevant to this study; Chapter 3 is the review of past research on GLD RC frames and RC infilled frames; Chapter 4 deals with the performance assessment of typically detailed Australian GLD RC frames under different earthquake magnitudes based on the results of experimental and analytical studies; Chapter 5 deals with the experimental study on the contribution of infill wall panels on the in-plane stiffness and strength of surrounding frames under seismic loads. Finally, the conclusions of this research and recommendations for future research are presented in Chapter 6.
Chapter 2

Conceptual Review

2.1 Introduction

One of the major challenges of present time is to reduce the loss of life and damage to property due to the failure of seismically hazardous structures. 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). Relevant information from the past analytical and experimental studies along with evaluations of the post-earthquake structural behaviour has contributed towards the establishment of appropriate seismic demand and seismic capacity of structures (discussed in chapter 3). Moreover, in the context of earthquake-resistant design, simplified engineering techniques based on different performance parameters are used to derive structural properties that comply with specified targets for a required level of safety. This chapter presents the conceptual review of the elements essential for the seismic analysis of RC frames relevant to the present study.
2.2 Seismic Design

2.2.1 Basic Concepts

The level of protection provided by the earthquake-resistant design of structures depends on economic constraints and seismic risk accepted by society. In practice, different limit states are considered for the seismic protection of structures. These limit states are related to the preservation of functionality, different degrees of efforts to minimize damage and the prevention of loss of life (Paulay et al. 1992). Although, it is possible to design a structure for severe earthquakes without damage, it is evident that such a design would be uneconomical. Seismic performance of buildings is mainly based on the concept of acceptable levels of damage under one or more events of specified intensity. However, the prime objective being “the possibility of damage is acceptable, but not the loss of life” (Park et al. 1975).

To design structures that can resist earthquakes requires an understanding of the seismic risk which is quantitative measure of earthquake hazard. For intraplate areas seismic design provisions are often based on the codes of practice from high seismic regions due to limited knowledge of geology and also, as the strong-motion accelerogram records have been predominantly recorded from small events often at large distances (Lam et al. 1996; Hutchinson et al. 2003). The quantification of seismic risk at a site can be assessed probabilistically in terms of average return period (recurrence interval), or probability of exceedance of certain ground acceleration accounting for the combining effects of frequent moderate earthquakes occurring close to the site and infrequent larger earthquakes occurring at greater distances (Greenhalgh et al. 1990; Paulay et al. 1992). Some of the probabilistic events or earthquake hazard levels have been listed in Table 2.1.

In the National Earthquake Hazards Reduction Program (NEHRP) provisions (FEMA 450 2003), the design earthquake (DE) ground motions have been defined as being two-thirds of the maximum considered earthquake (MCE) ground motions where, the MCE is stated as the “most severe earthquake effects”. Also, for regions of low and moderate seismicity the maximum considered earthquake ground motion is defined as the ground motion with a 2 percent probability of exceedance in 50
years. In the present study structures response to the earthquake ground motions with 10%, 5%, and 2% probability of exceedance in 50 years corresponding to mean return periods of about 500, 1000, and 2500 years, respectively have been studied and the design magnitude earthquake (DME) of 500 YRP as specified by AS1170.4 (2007) has been considered. Different levels of earthquake hazard are shown in Table 2.1.

<table>
<thead>
<tr>
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<th>Recurrence Interval (Return Year Period)</th>
<th>Probability of Exceedance</th>
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<tr>
<td>Frequent</td>
<td>43</td>
<td>50% in 30 years</td>
</tr>
<tr>
<td>Occasional</td>
<td>72</td>
<td>50% in 50 years</td>
</tr>
<tr>
<td>Rare</td>
<td>475</td>
<td>10% in 50 years</td>
</tr>
<tr>
<td>Very rare</td>
<td>970</td>
<td>5% in 50 years</td>
</tr>
<tr>
<td>Extremely rare</td>
<td>2475</td>
<td>2% in 50 years</td>
</tr>
</tbody>
</table>

2.2.2 Earthquake Ground Motions

The introduction of performance-based earthquake-resistant design for buildings has increased the use of simulated time histories of ground motion over the real ground motion records for which there may never be adequate suite of data in terms of tectonic structure, earthquake size, local geology, and near-fault conditions (Chen et al. 2002). The ground-motions are represented as time histories, response spectra or by other means as required for the structural analysis and design methods. Earthquake ground motions experienced at a site and the corresponding response spectrum are significantly influenced by the characteristics of the soil or rock at the site. Deep soft soils or loosely compacted fill will be more strongly shaken than sites with stiff soils, soft rock, or hard rock. Moreover, soft soils tend to amplify low frequency vibrations whilst stiff soils tend to amplify high frequency vibrations (ATC 48 1999; King 1998; Su et al. 2006). For Adelaide region which is considered for present study, stiff rather than soft soils will tend to amplify earthquake ground motions (Jensen 2000). As it is recognized that structure on soft soil sites would be more vulnerable, (Goldsworthy 2007) so the values estimated from the present study are perhaps a lower-bound estimate of effects on frames for the same return period.
The key parameters associated earthquake ground-motion which can influence the response of a structure are (Naeim 2001):

- Peak ground acceleration – mainly influences the vibration amplitudes.
- Duration – affects the severity of shaking.
- Frequency content – is related to the period of vibration of a structure.

A ground motion record with a moderate peak acceleration and long duration may cause more damage than a ground motion with larger acceleration and a shorter duration. Peak ground acceleration is widely used to scale the earthquake design spectra and acceleration time histories. However, it does not necessarily provide a complete representation of the severity of the earthquake, in terms of its potential to induce structural damage. Effective peak ground acceleration (EPA) has been proposed as an alternative to quantify the severity of the ground motion (Matheu et al. 2005). The EPA is considered to be a normalising factor for response spectra and is defined as the average value of the maximum ordinates of the 5\% damping acceleration spectra in the period range 0.1 to 0.5 seconds, divided by the standard (mean) value of 2.5 (Joint Committee of Structural Safety-Probabilistic Model Code 2002).

The acceleration response spectrum associated with a specific time-history recorded at a given location has its own unique shape (Figure 2.1). Hence, enveloped spectra known as design response spectra (Figure 2.2) are usually defined for design and evaluation, a procedure which encompasses a range of different possible motions assessed for a particular site (Booth 1994). Different methods are used for accelerogram scaling in which selected recorded time histories are adjusted to provide conformity to site conditions. Also, some spectrum matching techniques are available that generate time histories of ground motion whose response spectra match the design response spectrum (Chen et al. 2002).
Chapter 2 Conceptual Review

Figure 2.1—Time history and associated acceleration response spectrum

Figure 2.2—Envelope response spectrum for design

Force-based seismic design remains widely used method in codes. Seismic assessment and design based on displacement provides a conceptually appealing alternative to conventional force-based design as it places the focus of design directly on displacement demand, and hence damage. Various contributions have been made towards the development of displacement-based seismic design (Gulkan et al. 1974; Qi et al. 1991; Moehle 1992; Calvi et al. 1995; Kowalsky et al. 1995; Priestley et al. 1996; Bommer et al 1999; Panagiotakos et al. 1999; Tolis et al. 1999). Researchers have proposed deformation-based procedures using displacement spectra expressed as spectral ordinates versus effective period (period at maximum displacement). Seismic design displacement spectra include three-parameter demand representations, the three parameters being effective period, relative response displacement and equivalent damping ratio (Borzi et al. 2001).

In this study, the North/South component of the 1940 El Centro, California strong ground motion record was used for the shake table tests of 3-storey RC frame as its
response spectrum shape closely matches the design spectrum for firm soil sites in the Australian Earthquake Code, AS 1170.4 (2007) (discussed in section 4.2.2). For the analytical study, artificially generated earthquake ground motions were used. The acceleration response spectrum for the selected acceleration time-history records were generated and corresponding values of effective peak ground acceleration were scaled to obtain the acceleration coefficient of the considered site (Adelaide region) for different annual probability for exceedance (discussed in section 4.3.4.1). Australian Earthquake Code (AS 1170.4 2007, Section 3, Clause 3.1 and 3.2) specifies earthquake hazard factor (Z) (equivalent to an acceleration coefficient with annual probability of exceedance in 1/500) for different locations in Australia and probability factor (k_p) for the annual probability of exceedance, appropriate for the limit state under consideration (Table 2.2).

Table 2.2—Probability factor, k_p (AS 1170.4 2007)

<table>
<thead>
<tr>
<th>Probability factor, k_p (AS 1170.4 2007)</th>
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</thead>
</table>

2.2.3 Recommended Performance Levels

Conventional methods of seismic design address the objectives of life safety (strength and ductility) and damage control (serviceability drift limits). In the current code design procedures, there are uncertainties concerning many aspects such as
logical explanation of the rules on which decision making is based, the long-term risk, costs and benefits of earthquake protection, and the seismic demand and capacity of the structure (Ghobarah 2001; Zou et al. 2007; Krawinkler 2008). This indicates the need for improved seismic design approaches for performance enhancement. Several conceptual frameworks have been developed for performance-based earthquake engineering (SEAOC Vision 2000; FEMA 273 1997; FEMA 274 1997; ATC-40 1996).

Seismic performance is a measure of the degree of protection provided for the general public and the occupants of different structures against the potential hazards resulting from the effects of ground motions on building. Performance-based engineering yields structures with predictable performance within defined levels of risk and reliability (SEAOC Vision 2000 1995). Various documents such as SEAOC Vision 2000 (1995), FEMA 273 (1997), FEMA 274 (1997) and ATC 40 (1996) have specified different performance levels as listed below (Ghobarah 2001).

- Fully operational - Immediate occupancy
- Operational - Damage control, moderate
- Life safe – Damage state
- Near collapse – Limited safety
- Collapse

The importance level of the building is dependent not only on its occupancy and use but also on the economic, social and environmental consequences due to its failure. Performance objectives for different building categories as recommended by SEAOC Vision 2000 (1995) have been summarised in Figure 2.3. From Figure 2.3 it can be noted that the minimum acceptable performance objective for new structures include multiple goals such as life safety in the 475 year event and collapse prevention in the 970 year event. In the present study structural performance of RC frames has been assessed for different earthquake design levels.
2.3 Performance Measurement Considerations

Performance assessment of structures is related to many parameters such as displacement, ductility, strength and stiffness under seismic loading. These factors have been discussed below. Additional details for these parameters in regards to the previous studies on RC frames have been discussed in Chapter 3.

2.3.1 Drift

During an earthquake event, inelastic deformability of structural elements has significant effect in dissipating the seismic induced energy. Moreover, it has been specified that the use of lateral displacements as demand parameters rather than force parameters permits a more direct way to control the damage in the structure during the design process and that the importance of limiting storey drifts should be emphasized in Earthquake Engineering (Miranda et al. 2002; Otani 2004; FEMA 306 1998).

Drift can be computed individually for each storey (interstorey) or as an average over the entire height of the building. The global (overall or roof) drift ratio is defined as the lateral roof displacement divided by building height, whereas the interstorey drift
is defined as the storey deformation relative to the storey height (Ghobarah 2000). In the performance-based design, interstorey drift has become one of the main parameters in evaluating the system performance under different levels of earthquakes (SEAOC 1995; FEMA 274 1997). Table 2.3 shows quantitative measures of performance based on specified limits of transient and permanent drift. In the present study three typical GLD Australian frames 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 drift values were taken from SEAOC Vision 2000 (1995) (Table 2.3) as being representative of the drifts when damage to structural and non-structural components would become excessive (>1.5%) or collapse would be approached (>2.5 %). It is recognised that these values may be conservative.

Table 2.3—Performance levels (SEAOC Vision 2000 Committee, 1995)

<table>
<thead>
<tr>
<th>NOTE:</th>
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<tbody>
<tr>
<td>This table is included on page 13 of the print copy of the thesis held in the University of Adelaide Library.</td>
</tr>
</tbody>
</table>

2.3.2 Ductility

Weaknesses in the structural system and deficiencies in reinforcing steel detailing in RC buildings greatly affects the damage and modes of failure of components which directly has influence on structure’s ability to sustain the loads and remain stable (Ghobarah 2000). Ductility is described as the ability of structure to offer resistance in the inelastic domain of response without excessive strength or stiffness degradation. The ductility demands experienced at critical locations in a structure during formation of plastic mechanism under sustained inelastic loading have traditionally been regarded as a key measure of potential seismic structural damage (Chandler et al. 2000). The primary aim of detailing in composite structures as specified by Paulay et al. (1992) is to produce ductile members that can sustain large
inelastic deformation demands imposed by severe earthquakes and thereby, can minimize major damage and the survival of buildings with moderate resistance with respect to lateral forces.

Structural ductility factor (displacement ductility) \( (\mu_d) \) is expressed as the ratio of maximum displacement \( (\Delta_{\text{max}}) \) to the yield displacement \( (\Delta_y) \). Idealized structural response curve is shown in Figure 2.4. The straight line in the Figure 2.4 shows the idealised linear elastic response of the building, which was drawn by extending the initial elastic portion of the response curve. The initial elastic response was estimated visually.

As a result of ductility, the structure has a capacity to absorb energy by hysteretic behaviour and due to this energy dissipation capacity, the elastic design force can be reduced to a yield strength level by the ductility reduction factor \( (R_\mu) \) (Uang 1991).

\[
(R_\mu) = \left( \frac{C_{\text{eu}}}{C_{\text{max}}} \right)
\]

Where, \( C_{\text{eu}} \) is base shear coefficient for elastic response; \( C_{\text{max}} \) is the base shear coefficient corresponding to the idealized yield displacement \( \Delta_y \).

![Figure 2.4—Idealized structural response](image-url)

Although the ductility factor of an individual structural member can be determined experimentally, there is no general agreement of how the concept of ductility factor should be applied at the structural system level (Uang 1991). Park (1998) suggested
adapting a value of 3 for moderate seismicity for economic design (referred in Kuang et al. 2005). Structural ductility factors of 2 and 3 have been specified for ordinary moment-resisting concrete frames and intermediate moment-resisting concrete frames, respectively by AS1170.4 (2007).

2.3.3 Strength and Stiffness

The importance of overstrength in the survival of buildings during earthquakes has been recognized by many researchers. Overstrength is referred as the measure of the extra reserve strength of structure before reaching to its maximum strength. The overstrength factor is defined on global behaviour of a structure as a ratio of the actual structural yield level to the code-prescribed strength demand arising from the application of prescribed loads and forces (Lee at al. 2002). As shown in Eq. 2.2, the total overstrength of a frame ($\Omega_{\text{Total}}$) is the product of the code overstrength factor ($\Omega_s = C_s/C_w$) and the yield overstrength ($\Omega_y = C_{\max}/C_s$) (Lee et al. 2002).

$$\Omega_{\text{Total}} = \Omega_s \Omega_y = \frac{C_s}{C_w} \times \frac{C_{\max}}{C_s} = \frac{C_{\max}}{C_w} \quad (2.2)$$

Where, $C_s$ is the base shear coefficient corresponding to the first significant yield of the structure; $C_{\max}$ is the base shear coefficient corresponding to the idealized yield displacement $\Delta y$; $C_w$ is the code-prescribed design base shear coefficient.

Code overstrength is a measure of the extra strength of a frame above what it was intentionally designed for and is sometimes referred to as the Allowable Stress Factor whereas the structural overstrength results from internal force redistribution (redundancy), higher material strength than those specified in the design, strain hardening, member oversize, multiple loading combinations, effect of nonstructural elements, and so on (Uang 1991). The overstrength against lateral load is very significantly affected by the factored gravity or wind loads used in design of a structure. This results in higher overstrength factor for structures located in low seismic zones. Moreover, seismic overstrength is important for buildings with short periods because ductility is ineffective in reducing the required elastic strength in this period range (Uang 1991; Lee et al. 2002). In the NEHRP provisions (FEMA 450
2003) the range overstrength factor between 2 to 3 has been specified for ordinary and intermediate moment frames.

Stiffness relates loads or forces to the resulting structural deformations and the slope of the idealized linear elastic response is used to quantify stiffness (Paulay et al. 1992). An important aspect of RC structures subjected to seismic loading is the change in effective natural frequency related to the stiffness degradation caused by cracking and local spalling of concrete as well as slipping and reduction in effective modulus of the steel (Gulkan et al. 1974). Stiffness is a characteristic of RC frames that help resist lateral loads and therefore, it is important to estimate stiffness if deformations under the action of lateral forces are to be reliably quantified and subsequently controlled (Paulay et al. 1992).

2.4 Structural Response Against Earthquake Loading

Earthquakes cause considerable damage and can lead to collapse of structures. In particular, large magnitude earthquakes in urban areas can result in casualties, extensive property damage and business interruption due to the failure of seismically hazardous buildings. A structure may be subjected to local or global failure, the latter being significant. However, the initial local failure may extend from element to element and can result in progressive collapse. Collapse of structural system refers to its inability to sustain gravity loads when subjected to seismic excitation (Ibarra et al. 2005). Furthermore, different structural systems can lead diverse damage scenarios during earthquake impact. Wide-spread damage and collapse of structures is mainly caused due to following major issues (Folic 1991; Naeim et al. 2000; Ghobarah 2000):

- improper utilization of land and inadequate zoning regulations,
- improper construction on liquefiable soil,
- soft- and weak-story building configurations,
- strength and stiffness discontinuities,
- vertical, horizontal and mass irregularities,
- poor material quality and workmanship,
- poor structural detailing,
- unaccounted-for torsional response,
- weak-column/strong-floor configurations,
- lack of adequate load paths,
- unaccounted for influence of ‘non-lateral resisting components’ on building response,
- lack of adequate correlation between the building as analyzed and designed and the building as built,

Table 2.4—General damage descriptions by performance levels (SEAOC 1995)

NOTE:
This table is included on page 17 of the print copy of the thesis held in the University of Adelaide Library.

Thorough understanding of damage state is necessary as it can contribute towards the reduction of earthquake consequences and mitigation of risk. Permissible levels of
damage to the different systems and sub-systems in the building, for different performance levels are shown in Table 2.4.

2.4.1 Beam Failure

As observed during past earthquakes several deficiencies in the beam design and detailing lead to substantial damage (Figure 2.5). Failures in beams are commonly connected to inadequate provision of stirrups which leads to steady deterioration of the shear strength and stiffness in the hinging zone. Furthermore, due to formation of hinging zone the shear demand could be significantly higher than the design shear strength in the beam resulting in shear failure may take place (FEMA 274 1997). The main design aspects for beams are related to the prevention of longitudinal bar buckling and providing sufficient transverse reinforcement for confinement and shear strength mainly in potential plastic hinge locations. Another important aspect is the proper anchorage of reinforcement in well-compacted concrete so that they can develop yield strength without associated deformations, such as slip or pullout becoming excessive (Paulay et al. 1992). The possibilities of a compression bar buckling and bar pullout in a beam were monitored in the present study (discussed in section 4.3.4.2).

Along with the conventional narrow beam construction, many other construction techniques have been developed such as wide band beam construction in which beams are very wide and shallow. This form of construction is very popular in Australia for multilevel structures as it offers reduced interstorey heights and hence, allowing reduced total height or additional floors for the same height of structure. Also, the ease of formwork construction and cost efficiency are other added advantages. The design for band beam construction is mostly gravity-load-dominated. Although, such construction is considered to possess sufficient strength and deformation capacity for resisting the design magnitude earthquake (500 YRP) in low seismicity regions however, deficient detailing feature such as poor beam anchorage could cause undesirable damage pattern from seismic design point of view (Stehle 2001; Abdouka 2003). Both the conventional and band-beam frames were considered in the present study.
2.4.2 Column Failure

During an earthquake, the design shear forces may be significantly lower than the potential shears acting on the column if design is based on gravity and/or wind loads or if the column was designed using factored loads. Elwood et al. (as referred in Ibarra et al. 2005) concluded that shear failure in columns does not necessarily lead to collapse of the system however, it was concluded that the shear failure is usually accompanied by a reduction of axial capacity. There are numerous examples of column failures during past earthquakes (Figure 2.6). Common causes of column failure are provision of lap splice just above the floor level, inadequate shear capacity due to lack of sufficient transverse reinforcement causing shear failures or buckling of longitudinal rebar. In addition, column-infill masonry wall interaction can also lead to severe damage and failure of columns (discussed in section 5.7). In addition to the concrete crushing failure, shear failure and compression bar buckling in the interior and exterior columns were monitored as the possible failure modes in this study. This is discussed in section 4.3.4.2.
2.4.3 Beam-Column Joint

Distress in the beam-column joints has been the cause of building collapse (Figure 2.7) in many recent earthquakes worldwide (e.g., the Izmit earthquake, 1999 (Sezen et al. 2000); the EI Asnam earthquake 1980 and the Mexico earthquake 1985 (Paulay et al. 1992). During seismic loading beam-column joint region is subjected to large shear forces which may result in significant loss of stiffness due to inadequate shear strength and anchorage capacity in the connection. (Paulay et al. 1992; FEMA 274 1997).

Commonly the joint failure in RC frames is related the following deficiencies:

- Insufficient or absence of confinement reinforcement in the beam-column joint region leads to inadequate joint shear resistance.

- Improper detailing of the longitudinal reinforcement anchored in or passing through the connection, which may cause bond slip and anchorage failure of the beam reinforcement due to the effect of the lateral loading.

These detailing deficiencies are mainly observed in the non-ductile frames and have also been incorporated the frames tested in this research to study their behaviour under seismic loading.
2.4.4 Infill Wall

The effect of infill panels on the structural response of frames is widely recognized. Dual wall-frame systems have beneficial effects related to strength, stiffness, and ductility. However, if they are not design properly, infill walls can also experience brittle shear failures of concrete columns and soft first storey collapse due to over strengthening of upper stories of a structure (Shing et al. 2002). Also, a common failure in RC frame due to “non-structural” infill walls is due to the stiffening effect of infill which attracts higher seismic forces to the infilled frame, leading to the shear failure of infill, followed by damage or failure to the columns (Paulay et al. 1992). In addition infill wall with improper height to wall thickness ratio may become prone to collapse when significantly damaged. Moreover the size and nature of the gap between infill wall and the surrounding frame members can greatly affect the seismic response of a wall-frame system. Figure 2.8 shows severe damage to wall-frame system during an earthquake event.
Once the separation between infill wall and RC frame has occurred and as the deformation increases, failure of the wall frame system can occur due to one of the following mechanism (CEB 1996; FEMA 306 1998).

- For infilled RC frames subjected to lateral loading, corners of compression diagonal of the infill experience high stress concentrations causing failure due to local crushing of the infill panel (Figure 2.9a). In case of weaker RC frames the damage is more extensive and extends to the frame itself.

- In case of strong frame, RC elements can transmit high forces to the compressed diagonal resulting in diagonal cracking (Figure 2.9b). These cracks initiate in the central region of the infill and run parallel to the compression diagonal and propagate to the frame corner.

- For masonry infill panel having low shear resistance along the bed joints and large length to height aspect ratio, failure can occur due to horizontal sliding along the bed joints (Figure 2.9c). This behaviour mode commonly occurs in conjunction with other failure mechanisms.

- When strong infill weak frame, premature failure of columns or of beam-column joints can lead to failure of the total wall-frame system (Figure 2.9d).

The behaviour of RC infilled frames subjected to seismic loading is discussed in chapter 5.
2.4.5 Soft/Weak-Story Mechanism

Possibly among all structural problems, the soft/weak story failures (Figure 2.10) have resulted in more deaths and destruction than any other (Naeim et al. 2000). A pure soft-story problem is related to the abrupt change in stiffness of adjacent floors in a structure. The soft story results in a localized excessive drift magnifying the P–Δ loads and causes heavy damage or collapse of the story during a severe earthquake. However, a pure weak story exists when the strength of the weak floor (most often the first floor) is substantially less than the strength of adjacent floors. This leads to plastic hinge formation or brittle failure at the weak floor. Pure cases of soft-story or weak-story failures are rare and generally the same floor is both soft and weak (Naeim et al. 2000).

Moment frames are often popular in building applications because they allow for maximum floor space utilization and access between bays, in particular utilizing the first storey for automobile parking and pedestrian walkways or not providing infill walls in a particular storey. However, these applications result in abrupt changes in storey stiffness and strength. These mechanisms are such that all columns or joints of a storey develop plastic hinges, possibly making the structure unstable for further lateral loading (Paulay et al. 1992; Dooley et al. 2001).
RC frames with weak column/strong beam characteristics also result in soft/weak first story. This problem is particularly pronounced in structures for which gravity load governs the design, resulting in beam flexural strengths exceeding by some margin the column flexural strengths. Consequently, inelastic deformations are concentrated at the soft story with response of the remaining structure being in elastic range, most likely causing the collapse of complete structure. Dooley et al. (2001) specified that while any increase in column-to-beam strength ratio is beneficial, it is more effective to increase the strength ratio without increasing the column-to-beam stiffness ratio than to increase both at the same time. This behavior is discussed for the tested RC frames in section 4.3.4.2.

Figure 2.10—Soft/weak story failure (Naeim et al. 2000)

2.5 Code Provisions for Non-Ductile Frames

Low to medium rise RC framed structures built in the majority of low-to-moderate seismicity regions such as Australia are designed primarily for combinations of gravity and wind loads and have less stringent design and detailing requirements. Current design provisions such as AS 3600 (2001) and ACI 318 (2002) specify three types of moment resisting frame systems: Ordinary Moment Resisting Frame (OMRF); Intermediate Moment Resisting Frame (IMRF), and Special Moment Resisting Frame (SMRF). In low-to-moderate seismicity regions, reinforced concrete frames are normally designed as OMRF which does not require any special provision for seismicity. Detailing requirements for these frames have been discussed below.

- **For beams**

  AS 3600 Clause 8.1.8.4 (c) specifies that not less than one quarter of the total positive reinforcement required at mid span shall continue past the
near face of the support and that not less than one-third of the total negative reinforcement required at a support shall be extended beyond the point of contra-flexure by distance equal to the overall depth of a cross-section in the plane of bending. Also, there is no requirement for beam bars to be anchored in the joint.

According to AS 3600 Clause 8.2.5, shear reinforcement is designed only for gravity load. Moreover if the design shear is less than the concrete shear strength no transverse reinforcement is needed. In addition there is no special provision for spacing at potential hinge regions and for band beams spacing can be as large as 300 mm.

- **For Columns**

  No requirement has been specified for anchorage of longitudinal reinforcement of column in the joint region. Also, there is no specification for splice location and the current practice in Australia is to provide column splices just above the slabs.

  AS 3600 Clause 10.7.3.3 (b) specifies the maximum spacing of column ties to be spaced Dc or 15db (where, Dc is the smaller column dimension for rectangular column, db is the diameter of the smallest bar in the column)

- **For beam-column joint**

  AS 3600 Clause 10.7.3.5, requires no lateral shear reinforcement when joint is restrained on all sides by a floor system of approximately equal depth.

  Also, there is no provision for the maximum column-to-beam flexural capacity ratio and hence, soft-storey mechanism is likely in the non-ductile RC frames.
2.6 Methodology for Performance Evaluation

For the performance assessment of a structure under seismic loading, estimation of critical response parameters under one or more ground motions is essential. These response parameters are measures of the structural behaviour which can be correlated with damage levels and hence, with the performance objectives (SEAOC 1995). Hence, well-defined methodology including choice of suitable ground records, appropriate testing method, and information on various response factors is of significance importance. This section discusses methodology for performance evaluation.

2.6.1 Analytical Techniques

Static pushover analysis is useful for analyzing the nonlinear response of a structure under predefined incremental lateral load pattern which is distributed along the building height. This method provides information on interstorey drifts which accounts for strength or stiffness discontinuities and P-Δ effects and sequential yielding of various structural elements with identification of critical regions and consequences of strength deterioration of elements, which can provide insight for design modification. However, pushover analysis procedure does not take into account all desired nonlinear response characteristics of the structure and its elements and thus, may not be applicable in some cases (SEAOC 1995; Ghobarah 2000).

Dynamic nonlinear time history analysis is another method used for investigating dynamic response of a structure for a selected ground motion and is considered to be the most sophisticated analysis technique currently available. This procedure provides very detailed information on the structural response, accounting for irregularities, nonlinear behaviour and modal interactions. However, it is considered to be complex and thus, needs more computational effort (SEAOC 1995).

In this research, analytical investigation of GLD RC frames against different magnitude earthquakes has been performed using both the static pushover and nonlinear time history analyses. This is discussed in detail in section 4.3.
2.6.2 Experimental Methods

Although powerful analytical techniques are available for evaluating seismic performance of RC structures, still experimental research remains a necessary tool and is used to predict behaviour of structural systems. Three types of testing methods may be used: quasi-static, pseudo-dynamic and shake table.

**Quasi-static tests** is a common form of structural testing which replaces the inertia forces generated during an earthquake event on a structure with equivalent static loads which are produced between a reaction frame and a specimen by hydraulic actuators. It is generally used for large-scale structural specimens. This testing method predicts the performance of a structure in terms of strength, stiffness, and ductility (Filiatrault 2002).

**Pseudo-dynamic tests** combine quasi-static tests with a numerical method to simulate the seismic response of a structure. It directly measures the inertial forces at specific points on the structure within a given time interval which are used to calculate the resulting displacements and can then be applied to the specimen using hydraulic actuator. This method is relatively simple however, results may not represent the true behaviour in case of materials which are sensitive to strain-rate effects (Filiatrault 2002).

**The shake-table test** is the only method applying direct simulation of inertia forces on structure with a distributed mass. Different ground motions can be easily reproduced by a shake table and hence, evaluation of the nonlinear response and the failure modes of the structures in a realistic seismic environment are possible. However, due to limited load bearing capacity of the testing platform, mostly small-scale testing is done (Filiatrault 2002).

For the present study experimental testing has been performed using both the quasi-static and shake table test for seismic assessment of RC frames. This included shake-table testing of a 1/5-scale model of typically detailed Australian RC frame (discussed in section 4.2) and four ½-scale frame specimens under quasi-static cyclic loading (discussed in chapter 5).
2.6.3 Computer Modelling

Modeling a structure and analyzing its response under dynamic loading using a computer program has significant importance in seismic performance evaluation. For code specified load factored linear elastic analysis, the results are reasonably acceptable. However, to achieve a realistic behaviour inelastic response should be considered. In recent years, significant research has been carried out to develop more refined models to overcome uncertainties related to terms such as dynamic loading, material nonlinearity and hysteresis (Chatzi et al. 2005).

Several computer programs have been developed for non-linear structural analysis of a structure to assess its seismic performance such as DRAIN-2D, IDARC and Plastique (Chatzi et al. 2005). Some of these programs had limitations regarding features such as, P-Δ effects, stiffness degradation and modal analysis. In the present study, the non-linear computer program developed at the University of Kyushu was used for the analytical study (Kawano et al. 1998). This program could deal with the above mentioned limitations and moreover, the source code of the program was available which enabled to have complete access and control of the input and output files.

2.7 Summary

In this chapter, important aspects of earthquake engineering relevant to present study were reviewed. Broad range of topics including basic concepts of seismology, different performance objectives, response of different structural elements, various performance parameters, and methodology for performance assessment were discussed.
Chapter 3

Previous Studies on Seismic Evaluation of Reinforced Concrete Frames

3.1 Research on Reinforced Concrete Gravity Load Designed Frames

3.1.1 Introduction

The behaviour of RC structures approaching their collapse state is considered to be rather complex due to the significant material and geometrical non-linearities present at this limit state. Structural damage caused during earthquakes consistently shows vulnerability of poorly executed structural details in the structure as the major source of distress, (Paulay et al. 1992; Humar et al. 2001; Han et al. 2004). 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). This section of the thesis presents the review of previous studies conducted to investigate the dynamic response of non-ductile structures against earthquake loading.
3.1.2 Previous Research

A number of experimental and analytical studies have been conducted to investigate the dynamic response of non-seismically designed RC frames against earthquake loading. Ductility, overstrength, drift, beam-column joint behaviour, and overall collapse mechanism are some of the important aspects of frame behaviour during an earthquake event, as discussed in Chapter 2. This section reviews the past studies categorised in terms of these key aspects which are relevant to the present study.

3.1.2.1 Ductility and Overstrength

Figure 3.1—Details of Bracci et al.’s (1995) model frame

Behaviour of a three-storey, 1:3-scale gravity-load-designed (GLD) model frame subjected to simulated earthquakes representing minor, moderate and severe seismic risk (Table 3.1) for low to moderate earthquake zones was studied by Bracci et al. (1995). As shown in Figure 3.1, some of the detailing inadequacies in the Bracci et al.’s model were: a) minimal transverse reinforcement in columns, particularly in potential hinge locations; b) little or no transverse reinforcement in beam-column joints; c) discontinuous positive beam flexural reinforcement in the beam-column joints. Table 3.1 summarises the results of Bracci et al.’s (1995) study. The study by
Bracci et al. (1995) demonstrated that the model frame could resist minor earthquake without major damage however, it suffered extensive side-sway deformation particularly under severe earthquake (Table 3.1).

Table 3.1—Results of Bracci et al.’s (1995) model frame

<table>
<thead>
<tr>
<th>Test</th>
<th>Remark</th>
<th>Maximum Interstorey Drift (%)</th>
<th>Maximum Base Shear Coefficient (V/W)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Taft N21E</td>
<td>Minor shaking</td>
<td>0.28</td>
<td>0.06</td>
</tr>
<tr>
<td>PGA 0.05g</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Taft N21E</td>
<td>Moderate shaking</td>
<td>1.33</td>
<td>0.15</td>
</tr>
<tr>
<td>PGA 0.20 g</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Taft N21E</td>
<td>Severe shaking</td>
<td>2.24</td>
<td>0.15</td>
</tr>
<tr>
<td>PGA 0.30 g</td>
<td></td>
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</tbody>
</table>

Figure 3.2—Details of Lee et al.’s (2002) model frame

A 1:5 scale, 2-bay 3-storey RC frame model designed according to the Korean practice of non-seismic detailing was tested by Lee et al. (2002) under different earthquake magnitudes (Table 3.2). As shown in Figure 3.2, the model frame had similar characteristics as of Bracci et al.’s (1995) frame except the special style of anchorage in the joints. That is, the lengths of tension and compression anchorage were 40 and 25 $d_b$ respectively, from the critical section, where $d_b$ is the nominal diameter of reinforcement. Moreover, the length of the tail in the hook is included in
this anchorage length and the tail of the anchorage of the bottom bars in beams is directed downward into the exterior columns. The study by Lee et al. (2002) revealed that the model structure showed fairly good resistance even against severe earthquake excitations (Table 3.2) with overall displacement ductility ratio of 2.4.

Moreover, the non-seismic frame tested by Lee et al. (2002) had the design base shear of 0.048W. However, during the experiment the model frame developed a maximum base shear of 0.18W under the design earthquake in Korea (500 YRP) which was about 3.8 times the design base shear. Furthermore, the model frame resisted the higher levels of the earthquake excitations with a very high lateral strength of up to 0.39W (Table 3.2). The overstrength of the frame tested by Lee et al. (2002) was found to be 8.7.

**Table 3.2—Results of Lee et al.’s (2002) model frame**

<table>
<thead>
<tr>
<th>Test</th>
<th>Remark</th>
<th>Maximum Interstorey Drift (%)</th>
<th>Maximum Base Shear Coefficient (V/W)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Taft N21E PGA 0.12 g</td>
<td>Design earthquake in Korea (500 YRP)</td>
<td>0.26</td>
<td>0.18</td>
</tr>
<tr>
<td>Taft N21E PGA 0.20 g</td>
<td>Max. earthquake in Korea (1000 YRP)</td>
<td>0.78</td>
<td>0.32</td>
</tr>
<tr>
<td>Taft N21E PGA 0.30 g</td>
<td>Max. considered earthquake in Korea (2000 YRP)</td>
<td>1.08</td>
<td>0.37</td>
</tr>
<tr>
<td>Taft N21E PGA 0.40 g</td>
<td>Severe earthquake in high seismic regions</td>
<td>1.68</td>
<td>0.39</td>
</tr>
</tbody>
</table>

From the above discussion, it is evident that there was a notable difference in the behaviour of the RC frames studied by Bracc et al. (1995) and Lee et al. (2002). These variations in the responses can be attributed to the different levels of ductility and overstrength reserve in each frame. Furthermore, based on their study related to the behaviour of multistorey RC frames, Jain et al. (1995) specified that the overstrength varies with seismic zone, number of storeys and with design gravity load. The results of Jain et al.’s study (1995) also indicated that the seismic zone has the strongest influence on overstrength and that the overstrength was much higher for
Chapter 3 Previous Studies on Seismic Evaluation of Reinforced Concrete Frames

lower seismic regions in comparison to higher seismic regions (as discussed section 2.3.3).

Hence, it can be noted that ductility and overstrength are important parameters in improving performance of structure under seismic loading. Moreover, Paulay et al. (1992) has specified that the cost of providing increased seismic resistance is significantly less when provided by improved detailing practice. It is worth noting that the typical Australian detailing practice used in the present study lacks good bottom reinforcing bar anchorage in beam-column joint regions as did the US study by Bracci et al. (1995).

3.1.2.2 Collapse mechanism

Soft storey failure mechanism at collapse is commonly observed in RC frames in low-moderate seismic regions. Several studies (Bracci et al. 1995; El-Attar 1997; Lee et al. 2002) have demonstrated that ultimate failure for non-seismically designed RC frames is due to the undesirable column sway mechanism. Many studies done in the past have shown that the response of RC frames under seismic loading is significantly affected by the column-to-beam strength ratio. Moreover, slab effects should be considered in the design while calculating the column-to-beam strength ratio to avoid the soft storey collapse in a structure (Aoyama 1985; El-Attar et al. 1997, Sadjadi et al. 2007). In addition, French et al. (1991) stated that the slab contribution can increase the actual beam strength by 50% (as referred in Dooley et al. 2001).

In the study conducted by Han et al. (2004), a 1/3-scale, 3-storey frame designed according to the minimum requirements of ACI 318 (1999) was tested under quasi-static cyclic lateral loading. Tests results showed a hybrid failure mechanism as the interior joints presented a weak column-strong beam mechanism whereas the exterior joints had strong column-weak beam mechanism. This type of hybrid failure was also observed for gravity load-designed RC members in the study by Aycardi et al. (1994). Furthermore, Dooley et al.’s (2001) study recommended that if column to beam strength beam ratio of 2.0 is adopted in the design, there could be significant probability of achieving the desired performance objective during a design-basis
seismic event. Also, as specified by Paulay et al. (1992), the overall ductility demand is readily achieved in beam sidesway mechanisms. Therefore, it is of particular interest to see the seismic performance of typically detailed Australian frames, which have no such requirements in design.

### 3.1.2.3 Beam-Column Joint

Inadequate detailing of a beam-column joint can lead to the collapse of RC structures during earthquakes (e.g., the Izmit earthquake, 1999 (Sezen et al. 2000); the El Asnam earthquake 1980 and the Mexico earthquake 1985 (Paulay et al. 1992)) as mentioned in section 2.4.3. Experimental data from studies discussed below was used to give realistic drift limits for the beam-column joints that the existing non-linear analysis packages are unable to model.

Beres et al. (1996) studied different damage mechanisms and the effect of detailing on strength and deformation by conducting test on 34 full-scale interior and exterior beam-column joints. Beres et al. (1996) demonstrated that under lateral loading, the shear capacity of internal joints was higher than that of exterior joints. In the study performed by Clyde et al. (2000), four half-scale, non-ductile RC exterior joints were tested under quasi-static cyclic loading to investigate their behavior in a shear-critical mode. From the test results it was observed that all specimens failed by the development of the limiting joint shear capacity.

In another study, Hakuto et al. (2000) investigated the behaviour of interior and exterior beam-column joints with substandard reinforcing details, typical of pre-1970’s designed moment resisting frames in New Zealand. The study demonstrated improvement in the performance of the joint with beam bars anchored in the joint. Pantelides et al. (2002) evaluated the seismic performance of non-ductile RC exterior joint satisfying the requirements of ACI Code (1963). Three different details of beam and beam-column joint reinforcement were used. Two types of failure modes namely, a bond-slip failure mode and a joint shear failure modes were identified for poorly detailed joints.
In an Australian research by Huang et al. (1993) a number of isolated exterior beam-column joints designed according to AS 3600 (1988) were subjected to cyclic displacement. The test results revealed that the major damage in the specimens was confined to the joint region and was due to shear action. Corvetti et al. (1993) tested three exterior beam-column joints with each joint corresponding to a particular level of ductility as specified by AS 1170.4 (1993). The study demonstrated that the normal frame representative of typical design for low rise RC frames suffered premature joint failure due to poor confinement in the joint leading to excessive early cracking and slip. However, the intermediate frame with additional confinement around the joint exhibited displacement ductility of 3.3, and was considered suitable for relatively low seismicity regions such as Australia. The special moment frame with additional joint confinement and improved beam bar anchorage was found to be the most efficient with a peak displacement ductility of 5.9.

Figure 3.3—Specimen 1- Stehle et al. (2001)
In low to moderate seismic regions, band beams are one of the popular forms of RC construction. Stehle et al.’s study (2001) tested two ½-scale RC wide beam specimens. The first specimen was detailed according to the Australian standards AS 1170.4 (1993) and AS 3600 (1994), as shown in Figure 3.3. The second specimen was detailed with unique detailing strategy of debonding beam bars to avoid torsion cracking of the portion of the wide beam transferring out of balance moment to the side faces of the joint core, as shown in Figure 3.4. The test results revealed that the interior wide-beam-column specimens with unique detailing strategy of bar debonding and modified bottom beam bar anchorage performed very well at maximum applied drift of 4%. In contrast, the specimen 1 for which no special provision was provided for seismicity, showed undesirable damage due to torsion and shear cracking in the beam at the sides of the joint core.

The research by Stehle (2001) was extended by Abdouka (2003) by experimentally investigating ½-scale exterior wide beam-column joint with no special provision for seismicity. The study concluded that the test specimen connection was sufficient to withstand code-specified design level earthquake (500-YRP) having a yield drift ratio of 1.4% (in positive direction). In addition, the specimen demonstrated limited ductility in positive direction with little strength reduction after the attainment of peak strength at a drift of 2.3%. However, in the negative direction the specimen behaviour was very deficient which was attributed to the inadequate development
length of the bottom beam bars. Also, the displacement ductility for the test model was calculated in the range of 1.29 to 1.65.

Goldsworthy (2007) studied the likely behavior of columns in non-ductile RC moment resisting frames subjected to a 2500 YRP earthquake. Displacement demand and capacity were estimated for different site conditions using experimental and analytical study. The study concluded that although for a regular RC frame, the columns are not expected to fail, for a frame with soft storey the likelihood of failure would depend largely on the soil conditions at the building site, with a strong likelihood of total collapse for stiff and soft soil sites due to large displacement demands.

From the above studies on beam-column joint it can be concluded that detailing is the key aspect in achieving the required performance of RC beam-column joint in the frames subjected to seismic loading. In particular, the detailing variations such as confinement at potential hinge locations and at joint and also, the bar development lengths can significantly affect the seismic performance of the overall frame.

3.1.2.4 Lateral Drift

Drift is the main cause of structural and non-structural damage in buildings during a seismic event. Several studies (Moehle 1992; Wallace 1994; Priestley 1996) have shown that present criteria for the seismic design of new structures and for the seismic evaluation of existing structures can be improved if they are based on the explicit consideration of lateral deformation demands as the main seismic design parameter rather than lateral forces as discussed in section 2.3.1.

From their study based on RC frames Han et al. (2004) specified the yield roof drift ratio of 0.5% and the maximum drift ratio (at 20% strength deterioration down to 80% of the maximum lateral strength) of 4%. In study by Clyde et al. (2000) of non-ductile RC exterior joints, yielding of the reinforcement was observed at a drift ratio of 0.5% to 1% and always initiated at the beam longitudinal bars. Research by Beres et al. (1996) on full scale beam-column joints revealed that the interior joints attained
peak strength values at a drift ratio of about 1.5 to 2% whereas for exterior joints it happened at drift value of 1.5 to 2.7%.

Moreover, the study (Corvetti 1994) demonstrated that the specimen with no provision of seismic design details suffered extensive cracking at a drift value of 0.8% with the joint failure occurring at 2%. In contrast, the intermediate moment frame specimen reached peak load at drift of 2.4% and collapsed at about 3.3%. However, the special moment frame was found to be most efficient with collapse occurring at drift of 4%. Furthermore, the non-ductile specimen tested by Stehle (2001) demonstrated significant damage due to torsion and shear cracking in the beam at the sides of the joint core with initial cracking and yielding occurring at drift values of 0.8% and 1.2%, respectively. Lam et al. (2004) introduced a displacement-based approach for the seismic design and performance assessment of structures in intraplate regions. A Component Attenuation Model (CAM) was presented for predicting the displacement demand of intraplate earthquakes as functions of the magnitude-distance (M-R) combinations along with numerous geophysical and geotechnical parameters. They used a generic expression for predicting the peak displacement demand or peak velocity demand, which jointly define the displacement response spectrum.

Recognising the limited ductility and drift capacity of GLD RC frames, the Australian earthquake code limits (AS 1170.4 2007) the interstorey drift at the ultimate limit state to 1.5% of the storey height. Based on the past studies conducted on RC frames (discussed above) interstorey drift limit range can be considered for various behaviour modes during different stages of loading. These limits have been summarised in Table 3.3.

Performance objectives for buildings of different types have been suggested by the Vision 2000 committee (1995) of the Structural Engineers Association of California (SEAOC). These performance objectives are related to the acceptable level of damage that a building might experience during an earthquake event (as discussed in section 2.3.1). Of significance to this study is the value of 2.5% drift assigned to the point of impending collapse. This value was also used in this study to classify the RC frame analysis results.
### Table 3.3—Interstorey drift at different behaviour levels

<table>
<thead>
<tr>
<th>Type of Frame</th>
<th>Drift at Initial Cracking</th>
<th>Drift at Yield</th>
<th>Drift at Peak Load</th>
<th>Drift at Collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional Beam-Column Joints</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bracci et al. (2005)</td>
<td>0.5</td>
<td>-</td>
<td>1.2</td>
<td>2.1</td>
</tr>
<tr>
<td>Lee et al. (2002)</td>
<td>1.0</td>
<td>1.7</td>
<td>-</td>
<td>3.7</td>
</tr>
<tr>
<td>Beres et al. (1996)</td>
<td>0.3 - 0.6</td>
<td>1.0</td>
<td>1.5 - 2.7</td>
<td>3.0 - 3.5</td>
</tr>
<tr>
<td>Corvetti et al. (1993)</td>
<td>0.1</td>
<td>-</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>Wide Beam-Column Joints</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stehle et al. (2001)</td>
<td>0.8</td>
<td>1.2</td>
<td>3.65</td>
<td>-</td>
</tr>
<tr>
<td>Abdouka (2003)</td>
<td>0.8 - 1.0</td>
<td>1.5</td>
<td>2.0 - 2.3</td>
<td>4.0</td>
</tr>
<tr>
<td>SEAOC Vision 2000 (1995)</td>
<td>&lt; 0.2</td>
<td>&lt; 0.5</td>
<td>&lt; 1.5</td>
<td>&gt; 2.5</td>
</tr>
</tbody>
</table>

### 3.2 Research on Reinforced Concrete Infilled Frame

#### 3.2.1 Introduction

Designers and researchers have noted the, sometimes good, sometimes bad, effect that infill has on RC frame (as discussed in section 2.4.4). Bendimerad F., Director of the Earthquakes and Megacities Initiative stated during 13th World Conference on Earthquake Engineering that “approximately 80% of the people at risk of death or injury in earthquakes in the world today are the occupants of reinforced concrete frame infill-masonry buildings” (Langenbach et al. 2006).

Although, significant contribution of infills in global behaviour of structure has been identified, infill panels are often considered to be as non-structural elements. This is partly due to the complex interaction mechanism between the infill and the surrounding frame and large number of variable parameters affecting infill frame behaviour and also due to the lack of widely accepted design theory for such structures. Therefore, this topic still attracts many research efforts.
3.2.2 Previous Studies

Infilled frames have been used in civil construction for more than 200 years (FEMA 306 1998). The pioneering work on behaviour of masonry infilled frame was introduced by Polyakov (1957). Among the early research work on frame-to-infill interface behaviour were those performed by Wood (1958), Benjamin et al. (1958), Sanchanski (1960), Holmes (1961), Stafford Smith (1962, 1966, 1967b), Mallick et al. (1968), and Mainstone (1971). These were followed by many other experimental and analytical investigations performed by a number of researchers. Many of these studies included masonry-infilled steel frames. However, the behaviour of masonry infilled RC frames is considered to be more complex than that of steel infilled frames (Mehrabi et al. 1996).

Various studies on the performance of masonry-infilled RC frames have demonstrated that infill panels highly influence the bounding frame. Furthermore, frame-to-infill interaction greatly affects the lateral load capacity of the structure and also determines the failure mechanism (Calvi et al. 2004). Some of the recent experimental studies have been discussed below.

3.2.2.1 Experimental Research

Many researchers have investigated the performance of masonry-infilled RC frames. An experimental study was conducted by Lee et al. (2001) on brick infilled RC frame designed according to Korean practice of non-seismic detailing. The study concluded that the masonry infills contribution to the global capacity of the structure is about 80% in strength and 85% in stiffness. Moreover, the displacement ductility of the infilled frame (4.23) was found to be about twice that of bare frame (2.36). Another study by Dolce et al. (2005) on RC plane frames designed according to Eurocode 8 for low ductility structures demonstrated that the infilled frame collapsed at PGA which was twice that resulted failure for the bare frame. Moreover, the results suggested that properly realized and uniformly distributed infill panels can substantially increase the seismic resistance of the framed structures. Recently a shake table experiment was conducted by Hashemi et al. (2006) on RC infilled frame designed according to ACI 318 (2002) and NEHRP recommendations in seismic
regions. The study demonstrated that the infill plays a significant role in strength of
the structure and also makes it stiffer by a factor of 3.8. All these studies have
revealed that infill panels highly influence the bounding frame behaviour under
seismic loading. Furthermore, frame-to-infill interaction greatly affects the lateral
load capacity of the structure and also determines the failure mechanism (Calvi et al.
2004). Also, in order to evaluate the structural response of infilled frames, the
variability of the frame-to-infill interface behaviour depending on the presence of
gaps and connectors should be considered (CEB 1996).

Some common trends can be observed in the results of cyclic tests on the infilled
frames tested by various researchers (CEB 1996). In the hysteresis loop envelopes
(lateral load vs. horizontal displacement) of cyclic tests, there is always an initial
linear part corresponding to the stage at which infilled frames behave as a composite
system. However, while separation between infill and frame occurs and as gradually
damage extends to the infill and to the frame members, the stiffness of the wall-
frame system decreases gradually until the force-response reaches its maximum
value. This is followed by a falling branch which may be steep or shallow depending
on the failure mode of the infilled frame (CEB 1996). Furthermore, from the load-
displacement data of the wall-frame system peak base shear can be noted. The point
of peak base shear takes place at the time when the wall is still able to sustain large
portion of the load at its incipient failure (Hashemi et al. 2006). Most of the
experimental studies on infilled frames show various behaviour modes during
different stages of loading as shown in Table 3.4. It will be important to note these
stages in the present study.

Table 3.4 — Behaviour modes at different inter-storey drift levels (FEMA 307
(1998))

<table>
<thead>
<tr>
<th>Inter-storey Drift Level</th>
<th>Behaviour Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5%</td>
<td>Composite</td>
</tr>
<tr>
<td>1.5%</td>
<td>Frame</td>
</tr>
<tr>
<td>2.5%</td>
<td>Infill</td>
</tr>
</tbody>
</table>

NOTE:
This table is included on page 41 of the print copy of
the thesis held in the University of Adelaide Library.
Presence of vertical loading is another factor that can influence the performance of infilled frames. In multi-storey infilled frames, the gravity load can be due to the dead weight of the panel above it or additionally from roof or floor joists supported by it. Stafford Smith (1968) (referred in CEB 1996) investigated the effect of vertical loading on upper beam of a single-storey, single-bay steel frame, on the lateral stiffness and resistance of the infilled frame. Considerable increase in the lateral resistance of the infilled frame was noticed depending on the aspect ratio of the infills. However, for values of vertical load exceeding 40-60% of the vertical compressive strength of the infill the lateral strength decreases. Viliasis and Stylianidis (1989) (referred in CEB 1996) studied the effect of compressive axial load in the columns of the RC infilled frames. Significant increase in the lateral resistance of the frames was reported. In the present study, the test setup did not include vertical forces to simulate gravity loads.

### 3.3 Summary

In this chapter past studies on RC frames with and without infill were reviewed. It can be noted that the RC frames in low seismicity regions are more susceptible to earthquake effects mostly due to the gravity load dominated construction. The typical deficiencies of the non-seismically designed frame which make them vulnerable to failure as observed from different earthquakes are: a) inadequate anchorage of positive beam flexural reinforcement in the beam-column joints b) little or no confinement reinforcements in beam–column joints c) Splice location just above the joint. Although, there has been significant amount of international research in this area, only few studies (Stehle et al. 2000; Lam et al. 2004 and Goldsworthy 2007) have been conducted to investigate the dynamic response of non-seismically designed Australian RC frames against earthquake loading. Hence, further research work is needed to quantify the magnitude of earthquake likely to generate structural failure and/or collapse of typically detailed Australian RC frames.

Furthermore, despite of the effect of infill panels on the structural response of frames being widely recognized, the use of infill walls is a divisive issue because of having both the beneficial and undesirable effects on RC frame, as mentioned earlier.
In addition, most of the current design codes do not provide rules for the design of infilled frames. In particular, the effect of construction gap has got little attention by researchers. It is also worth mentioning that most of the studies till date have included different infill and frame materials and limited research has been conducted on unreinforced brick infilled RC frames. Hence, the evaluation of seismic behaviour of brick infilled reinforced concrete structures with gap effect is of significant interest.
Chapter 4

Performance of Reinforced Concrete Gravity Load Designed Frames

4.1 Introduction

This chapter presents the performance assessment of GLD RC frames under different earthquake magnitudes based on the results of experimental and analytical studies conducted at the University of Adelaide. The main objectives were (1) to investigate the behaviour of non-seismically designed RC frames under 500 YRP earthquake and (2) to determine the different magnitudes of earthquake (YRP) that are likely to cause excessive drifts in (> 1.5%) or collapse (>2.5%) of GLD structures.

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, USA. The performance of the frames was analysed and conclusions were drawn in relation to building displacements, base shear, ductility and overstrength.
4.2 Experimental Program

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 (Griffith et al. 1994).

4.2.1 Test Model

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 4.1 shows the cross-section and joint details of the 1/5-scale model structure tested.

![Figure 4.1—Elevation and joint detail of 1/5-scale model test structure](image)

4.2.2 Methodology

The model structure was subjected to simulated earthquakes with increasing magnitudes, with free vibration tests conducted before and after each earthquake test,
in order to monitor the change in the natural frequencies and damping of the models. The \sqrt{5} time-scaled North/South component of the 1940 El Centro, California strong ground motion record was used for the tests as its response spectrum shape closely matches the design spectrum for firm soil sites in the Australian Earthquake Code, AS 1170.4 (2007). The model was subjected to a series of El Centro earthquake tests where the magnitude of the effective peak shaking table acceleration (EPA) input varied between 0.026g and 0.126g (Table 4.1). The effective peak ground acceleration (EPA) is considered to be a normalising factor for response spectra and was defined in this study as the average value of the maximum ordinates of the 5% damping acceleration spectra in the period range 0.1 to 0.5 seconds, divided by the standard (mean) value of 2.5 (Joint Committee of Structural Safety-Probabilistic Model Code 2002). Hence, values for EPA ranging from 0.026g to 0.126g covers the full range of values for the effective peak ground acceleration coefficient given in the Australian earthquake code for the entire continent.

As mentioned previously in this section the model was subjected to increasing earthquake input. However, for large values of EPA (exceeding 0.126g) the shake table began to rock. Hence, testing was stopped at this point.

4.2.3 Test Results and discussion

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 4.1). From Table 4.1 it can be noted that all the maximum roof drift values were well within the code allowable drift limit of 1.5% specified by AS 1170.4 (2007). If EPA is plotted versus maximum drift (Figure 4.2) no significant departure from linear response is detectable until the last (0.126g) test which probably corresponds to the onset of shake table rocking.

However, if maximum base shear is plotted versus maximum drift (Figure 4.2) there is a distinct departure from the initially linear data at about 0.22W, or about 0.4% drift. This suggests that the model may have experienced some significant cracking and possibly the onset of reinforcing steel yield at that point. 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.

Table 4.1—Experimental results 1/5-scale model test structure

<table>
<thead>
<tr>
<th>Earthquake test</th>
<th>EPA</th>
<th>Maximum Base Shear (V)</th>
<th>Maximum Roof Drift (%)</th>
<th>Natural Frequency</th>
<th>Damping Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>EQ2</td>
<td>0.026</td>
<td>0.049W</td>
<td>0.071</td>
<td>3.3 Hz</td>
<td>3.7%</td>
</tr>
<tr>
<td>EQ3</td>
<td>0.031g</td>
<td>0.078W</td>
<td>0.11</td>
<td>3.2 Hz</td>
<td>3.2%</td>
</tr>
<tr>
<td>EQ4</td>
<td>0.043g</td>
<td>0.110W</td>
<td>0.17</td>
<td>3.2 Hz</td>
<td>3.4%</td>
</tr>
<tr>
<td>EQ5</td>
<td>0.047g</td>
<td>0.146W</td>
<td>0.22</td>
<td>2.9 Hz</td>
<td>3.7%</td>
</tr>
<tr>
<td>EQ6</td>
<td>0.056g</td>
<td>0.167W</td>
<td>0.26</td>
<td>3.3 Hz</td>
<td>4.5%</td>
</tr>
<tr>
<td>EQ7</td>
<td>0.069g</td>
<td>0.197W</td>
<td>0.34</td>
<td>3.2 Hz</td>
<td>5.4%</td>
</tr>
<tr>
<td>EQ8</td>
<td>0.080g</td>
<td>0.221W</td>
<td>0.39</td>
<td>3.2 Hz</td>
<td>5.8%</td>
</tr>
<tr>
<td>EQ9</td>
<td>0.093g</td>
<td>0.221W</td>
<td>0.47</td>
<td>3.1 Hz</td>
<td>5.3%</td>
</tr>
<tr>
<td>EQ10</td>
<td>0.105g</td>
<td>0.237W</td>
<td>0.52</td>
<td>3.2 Hz</td>
<td>5.5%</td>
</tr>
<tr>
<td>EQ11</td>
<td>0.126g</td>
<td>0.311W</td>
<td>0.84</td>
<td>2.9 Hz</td>
<td>5.6%</td>
</tr>
</tbody>
</table>

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

On the other hand, the trend in damping ratio is more interesting. While the overall trend sees an increase in damping with increasing earthquake load, and hence drifts, the model appears to have gone through a distinct transition between the EQ5 and EQ7 tests. The damping ratio was essentially constant for tests EQ2 – EQ5 (~ 3.5%) and essentially constant again for tests EQ7 – EQ11 (~ 5.5%). The rather rapid change in damping occurred over the tests where the maximum drift went from 0.22% to 0.34% and the EPA increased from 0.047g to 0.069g. It was also just after the EQ7 test that deviation in the linearity of the base shear versus drift plot was observed. Hence, it was assumed that at about 0.25% drift the model began to experience some significant cracking and possibly the first onset of steel reinforcement yield. However, this is a bit earlier than was suggested by the base shear versus drift plot and lower again than the value of 0.5% drift which Han et al. (2004) reported for the onset of yielding.
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 in Section 4.2.2, 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. Although, there has been significant amount of international research in this area, it is worth noting that very little research exists for Australian frames.
4.3 Analytical Study

In this part of the study three typical GLD Australian frames (full scale) were analysed to determine the magnitude of earthquake (YRP) to cause excessive drift (> 1.5%) or collapse (> 2.5% drift) of GLD RC frame structures (Chong et al. 2006).

4.3.1 Test Specimen

The three frames considered were 3-, 5- and 12-storey RC frames with 3 bays of 10 metres each. The storey height was 4 metres for the 3- and 5-storey frames and 4.2 metres for the 12-storey frame. The compressive strength of concrete (fc) and yield strength of steel reinforcement (fy) used in the frames were 50 MPa and 500 MPa, respectively.

![Member details of frames](image)

Figure 4.3—Member details of frames

The 3- and 5-storey frames were designed using the band-beam system as this is a commonly used construction practice in Australia. Importantly, experimental data from Stehle et al. (2001) and Abdouka (2003) was used to give realistic drift limits for the wide beam-column joints that the existing non-linear analysis packages are unable to model. Standard beam design with flat concrete slabs was used in the 12-storey frame for which overseas data and the experimental results of Corvetti et al. (1993) were used to provide realistic drift limits for the joints. The frames were designed in accordance with the Standards Association of Australia (SAA) Concrete Structures code, AS 3600 (2001). Figure 4.3 shows the member details for the 3-, 5- and 12-storey frames and Table 4.2 summarises the beam and column reinforcement arrangements used in the frames. In the present study, 12mm stirrups at 400mm spacing were used in the beam and column members of the three frames.
### Table 4.2—Beam and column reinforcement details

<table>
<thead>
<tr>
<th>Beam Reinforcement</th>
<th>Longitudinal Reinforcement</th>
<th>3- and 5-Storey</th>
<th>12-Storey</th>
</tr>
</thead>
<tbody>
<tr>
<td>External beam section</td>
<td>Top reinforcement</td>
<td>19 – 20 mm bars</td>
<td>11 – 24 mm bars</td>
</tr>
<tr>
<td></td>
<td>Bottom reinforcement</td>
<td>27 – 20 mm bars</td>
<td>9 – 24 mm bars</td>
</tr>
<tr>
<td>Internal beam section (outer span)</td>
<td>Top reinforcement</td>
<td>30 – 20 mm bars</td>
<td>14 – 24 mm bars</td>
</tr>
<tr>
<td></td>
<td>Bottom reinforcement</td>
<td>27 – 20 mm bars</td>
<td>9 – 24 mm bars</td>
</tr>
<tr>
<td>Internal beam section (inner span)</td>
<td>Top reinforcement</td>
<td>30 – 20 mm bars</td>
<td>14 – 24 mm bars</td>
</tr>
<tr>
<td></td>
<td>Bottom reinforcement</td>
<td>19 – 20 mm bars</td>
<td>7 – 24 mm bars</td>
</tr>
</tbody>
</table>

### Column Reinforcement

<table>
<thead>
<tr>
<th>Main Bars</th>
<th>3-Storey Frame</th>
<th>5-Storey</th>
<th>12-Storey</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column Section</td>
<td>4 – 32 mm bars</td>
<td>8 – 28 mm bars</td>
<td>16 – 28 mm bars</td>
</tr>
</tbody>
</table>

#### 4.3.2 Methodology

A non-linear computer program developed at the University of Kyushu was used for the analytical study (Kawano et al. 1998). This program is capable of performing both static and dynamic analysis on structural models of plane frames consisting of beam-column elements. Moreover, the material and geometric non-linearity are fully formulated in the program. In addition, it also considers the stiffness degradation, P-Δ effects, shear deformation and creep and shrinkage effects. Details of the program are included in Appendix A.

The program was validated using the experimental results of the 1/5-scale RC frame discussed in section 4.2 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. The only limitation being that it could not automatically predict premature joint failure due to poor anchorage details. This was simply accounted for
by recognising the maximum drift limits given in the literature (Corvetti 1994; Stehle 2001; Abdouka 2003) for each framing system.

4.3.2.1 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. Ductility and overstrength are important parameters for analysing the seismic behaviour of structure as discussed in section 2.3.2 and 2.3.3, respectively. These parameters are considered in the structural design through a force reduction factor or a response modification factor ($R_f$). This factor represents the ratio of the forces that would develop under the specified ground motion if the structure were to behave elastically to the prescribed design forces at the strength limit state (Kim et al. 2005).

![Analytical static pushover curve for 3-storey frame](image)

**Figure 4.4—Analytical static pushover curve for 3-storey frame**

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. Input files for the static pushover analysis of three frames can be seen in Appendix A. The displacement in the structure was calculated at various lateral force level and is plotted in Figure 4.4 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 4.4 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 (\(C_{\text{max}}\)), the base shear coefficient at first significant yield (\(C_s\)), the maximum deflection (\(\Delta_{\text{max}}\)), the deflection at yield (\(\Delta_y\)) and the base shear coefficient for elastic response (\(C_{eu}\)).

Table 4.3—Summary of static pushover results

<table>
<thead>
<tr>
<th>Parameter</th>
<th>3-Storey Frame</th>
<th>5-Storey Frame</th>
<th>12-Storey Frame</th>
</tr>
</thead>
<tbody>
<tr>
<td>Overstrength above yield ((\Omega_y))</td>
<td>1.31</td>
<td>1.42</td>
<td>1.80</td>
</tr>
<tr>
<td>Overstrength above AS 1170.4 ((\Omega_s))</td>
<td>1.84</td>
<td>2.11</td>
<td>2.58</td>
</tr>
<tr>
<td>Total Overstrength ((\Omega_{\text{Total}}))</td>
<td>2.41</td>
<td>3.00</td>
<td>4.64</td>
</tr>
<tr>
<td>Base Shear Coefficient at Peak Strength ((C_{\text{max}}))</td>
<td>0.15</td>
<td>0.16</td>
<td>0.14</td>
</tr>
<tr>
<td>Code Base Shear Coefficient AS1170.4 ((C_w))</td>
<td>0.063</td>
<td>0.055</td>
<td>0.029</td>
</tr>
<tr>
<td>Structural Ductility Factor ((\mu_s))</td>
<td>1.71</td>
<td>1.58</td>
<td>1.61</td>
</tr>
<tr>
<td>Ductility Reduction Factor ((R_{\mu}))</td>
<td>1.69</td>
<td>1.60</td>
<td>1.63</td>
</tr>
</tbody>
</table>

The results of the static pushover analysis for the three frames have been summarised in Table 4.3. The ductility and overstrength properties were computed by idealising the frame response (Figure 4.4). The total overstrength of a frame (\(\Omega_{\text{Total}}\)) is given by Eq. 4.1 (defined in section 2.3.3).

\[
\Omega_{\text{Total}} = \Omega_y \times \Omega_s = \frac{C_s}{C_w} \times \frac{C_{\text{max}}}{C_s} = \frac{C_{\text{max}}}{C_w}
\]  

(4.1)

Based on their study on the behaviour of multistorey RC frames, Jain et al. (1995) specified that the overstrength varies with seismic zone, number of storeys and with design gravity load. The results of Jain et al.’s study (1995) also indicated that the
overstrength was much higher for lower seismic regions in comparison to higher seismic regions. This is largely due to the fact that the code required base shear strength is so low in low earthquake hazard regions. In the present study, the total overstrength was found to be in the range of 2 to 5. These values were within the range specified by FEMA 450 (2003) except for the 12-storey frame which was higher. However, 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 4.3.

The response of the frames was also analysed in terms of structural ductility factor ($\mu_s$) (defined in section 2.3.2). It is worth noting that the ductility factor of the 3-storey frame of this study (1.71) 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). In addition, the ductility reduction factor ($R_{\mu}$) was calculated as defined in Eq. 4.2 and was found to be fairly constant in the range of 1.6 – 1.7 for all three frames.

\[ (R_{\mu}) = \left( \frac{C_{eu}}{C_{max}} \right) \]  \hspace{1cm} (4.2)

In order to have a direct comparison with the other international established research results, the formula derived by Uang (1991) (as shown in Eq. 4.3) was used for calculating the structural response modification factor ($R_f$). The values of $R_f$ calculated according to Eq. 4.3 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 ($\Omega_{Total}$).

\[ R_f = R_u \Omega_s \Omega_y = \frac{C_{eu}}{C_{max}} \frac{C_s}{C_y} \frac{C_u}{C_w} \]  \hspace{1cm} (4.3)
Chapter 4 Performance of Reinforced Concrete Gravity Load Designed Frames

Also, the code prescribed base shear coefficient \( C_w \) was calculated for the three frames as per AS 1170.4 (1993) using Eq. 4.4 (the respective values are shown in Table 4.3)

\[
C_w = V_w = I \left( \frac{CS}{R_f} \right)
\]

(4.4)

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.

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 4.3). However, for the 12-storey frames 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).

4.3.2.2 Non-Linear Time History Analysis

As mentioned in section 4.3.2, a non-linear computer program (Kawano et al. 1998) was used to perform the dynamic analysis of the three RC frames in terms of deflected shapes, storey shear, hysteresis relationships, base shear forces, and energy dissipation.

I. Description of Earthquake Data Used in Analysis

As noted earlier, artificially generated site specific Boston earthquake ground motion data (Somerville et al. 1997) was used for the dynamic analysis of the three different RC frames, as the Eastern US and Australia were considered to be regions of roughly similar seismicity. This was preferred to the alternative of simply scaling up the 500-YRP code-compatible ground motion used in the experimental study since duration and frequency content aspects for varying return period earthquakes were better accounted for with the “Boston” records. A 500-YRP and a 2500-YRP earthquake data 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. 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. It is recognized that structure on soft soil sites would be more vulnerable, (Goldsworthy 2007). Hence, the values estimated from the present study using artificially generated Boston earthquake ground motion conforming to the stiff soil site condition are perhaps a lower-bound estimate of effects on frames for the same return period.

To understand the variation of the structural response of a frame with respect to the change in fundamental period, response spectra were generated for the 500-YRP and 2500-YRP Boston earthquake ground motions as shown in Figure 4.5. This was done using a FORTRAN program, where the Duhamel integral was numerically calculated assuming a damping coefficient of 2%. From the generated response spectra, it was observed that the peak spectral acceleration generally decreased in magnitude with increasing building period for the range of periods covering the 3-, 5- and 12-storey frames (Figure 4.5). This suggested that the lower-rise frames would be more adversely affected than the 12-storey frame for the given earthquake.

![Boston Earthquake Spectra](image)

**Figure 4.5—The response spectra generated from 500-YRP and 2500-YRP earthquakes**
II. Dynamic analysis Using Boston Earthquake

Global Response

Table 4.4 summarises the dynamic analyses results for the three frames. Input files for the dynamic analysis of three frames can be seen in Appendix A. Figure 4.6 shows the hysteresis performance of the 3-storey frame. This gave the estimation of peak base shear and drift experienced at the roof level. Also, the displacement profile and the interstorey drift index (IDI) (Figure 4.7) of the 3-storey frame under the 500, 800 and 2500 YRP earthquakes were considered. The interstorey drift index (IDI) is defined as the percentage of relative displacement of a storey over its height. As shown in Table 4.4 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) (Table 2.3). Hence, it was concluded that the 3-storey frame would maintain its stability for the design earthquake of 500-YRP. Moreover, for the 500-YRP earthquake, the maximum base shear coefficient was 0.14 (Table 4.4). This value exceeded the coefficient at yield (0.115) but was still less than the ultimate shear coefficient (0.151), suggesting that the frame had yielded already but did not reach the ultimate strength.

![3 Bay 3 Storey RC Frame - Base Shear vs Roof Drift](image)

Figure 4.6—Base shear vs. roof drift of 3-storey frame
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 2.3 and Table 2.4). In addition, the maximum base shear coefficient experienced for the 3-storey frame for the 800-YRP and 2500-YRP earthquakes were 0.16 and 0.18, respectively. These values were larger than the base shear coefficient at peak strength (0.151) indicating that the structure was near collapse. Also, this is in agreement with the study conducted by Abdouka (2003) on wide beam-column specimen, where initial cracking was observed at 0.8% with yield and peak drift ratios being 1.5% and 2.3%, respectively. It should also be noted that the maximum base shear coefficients of 0.16 and 0.18 for the 3-storey frame subjected to 800-YRP and 2500-YRP earthquakes, respectively give an indication of overstrength.

**Figure 4.8**—Base shear vs. roof drift for 5-storey RC frame

(a) Displacement profile  (b) Interstorey drift index

**Figure 4.9** — Displacement profile and IDI for 5-storey frame
The global response of the 5-storey frame is shown in Figure 4.8 and Figure 4.9 shows the displacement profile and IDI of the 5-storey frame. Table 4.4 shows the maximum IDI for the 5-storey frame under the 500-YRP and 1000-YRP earthquakes was lower than 1.5% which indicates that the frame response would be within the strength limit state for these earthquakes. This also matches with the static pushover analysis results according to which the base shear coefficient for peak strength was 0.163 which is higher than the base shear coefficients obtained from the non-linear time history analysis for the 500-YRP and 1000-YRP earthquakes (Table 4.4). 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 (Table 2.3 and Table 2.4). Also, this can be noted from the static pushover analysis where the base shear coefficient for the 2500-YRP earthquake (0.17) was found to be greater than the ultimate base shear coefficient (0.163).

![Figure 4.10—Base shear vs. roof drift for 12-storey RC frame](image)

Figure 4.10—Base shear vs. roof drift for 12-storey RC frame
Figure 4.11—Displacement profile and IDI for 12-storey frame

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. Figure 4.10 shows the global hysteresis performance of the 12-storey frame and Figure 4.11 shows the displacement profile and IDI of the 12-storey frame. As shown in Table 4.4 the 12-storey frame experienced a 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 (refer Table 2.3 and Table 2.4). In contrast, the base shear coefficient at peak strength was found as 0.135 from the static pushover which suggests that the 12-storey frame had passed the peak strength for both the 500-YRP and -2500-YRP earthquakes. However, as drifts were not very large, the frame could resist the collapse provided that it had enough ductility.

Furthermore, from static pushover analysis results the structural ductility for the 12-storey frame was calculated as 3.25. In Corvetti’s study (1994) the normal moment frame designed according to AS 3600 suffered premature failures however, the intermediate frame yielded at about drift limit of 0.9% and had displacement ductility of 3.3 with softening behaviour consideration. Therefore, it can be concluded that the 12-storey frame could suffer yielding under the 2500-YRP earthquake but it might be able to resist the collapse.
In Stehle et al.’s study (2000), the strong column-weak beam hierarchy was observed for lower levels of drift, and this was only reversed when the drifts became large and this behaviour was modeled analytically. However, the model used in this study did not have this level of sophistication. In this study the frame failure mechanism was predicted for each frame by manually determining the moment capacity for the beams and columns at each storey. The slab contribution was taken into account in calculating the beam capacity as it can be a crucial factor in determining the collapse mechanism of the structure (Aoyama 1985; El-Attar et al. 1997, Dooley et al. 2001; Sadjadi et al. 2007). With these values, the sum of the column moment capacities was compared with that of the beams for each storey level. For the 3-storey and 5-storey frames, the moment capacity of the beams was found to be greater than the columns. Therefore, a soft storey failure mechanism was expected in these frames when the collapse limit state was reached. This can clearly be seen for the 3-storey frame in its IDI plot (Figure 4.7b) for the 2500-YRP earthquake which shows the largest drift at the ground floor. However, the 12-storey frame was expected to exhibit a weak-beam strong-column failure mechanism as the moment capacity of the columns was found to be higher than the total moment capacity of the beams. This was reflected in the more uniform distribution of storey drift up the building (Figure 4.11b).

![Figure 4.12—The cumulative energy dissipation in each storey for the 3-storey frame](image)

The cumulative energy dissipation plots of the frames showed similar characteristics with more energy dissipation in lower storeys. The cumulative energy dissipation of
the 3-storey building for 2500-YRP earthquake is shown in Figure 4.12. From the figure it is clear that the hysteresis energy absorption was much larger for lower stories where drift was higher as seen in the IDI of the frame (Figure 4.7b).

**Local Response**

![3 Bay 3 Storey RC Frame - Ground Floor Shear vs Ground Floor Drift](image)

**Figure 4.13—Ground storey response of the 3 storey frame**

The local behaviour for each frame was analysed by plotting the hysteresis response for the storey that experienced the maximum interstorey drift, as shown in Figure 4.13 for the 1st storey of the 3-storey frame. The 3-storey frame experienced drift of 0.8% for the 500-YRP earthquake. Based on their tests on wide beam-column joint, Stehle (2001) and Abdouka (2003) reported some initial cracking to occur at a drift of about 0.8%. This implied that the 3-storey frame in this study might undergo minor cracking under the 500-YRP earthquake. Also, linear response of the frame under 500-YRP was indicated by the relatively straight and narrow hysteresis loops. Beyond 1.1% drift, the hysteresis loops began to widen indicating that the structure had started to yield. This started for the 800-YRP with much wider and irregular loops for the 2500-YRP earthquake indicating a non-linear behaviour. Moreover, the maximum drift exceeded the limit of 1.5% specified by AS 1170.4 (2007) for the 800-YRP and 2500-YRP earthquakes in the ground level, indicating onset development of a soft-column sidesway mechanism at this level.
Figure 4.14—3rd storey response of the 5-storey frame

The local hysteresis performance was plotted for the 3rd storey of the 5-storey frame (Figure 4.14) and 11th storey for the 12-storey frame (Figure 4.15) where the maximum storey drifts occurred. In the 5-storey frame the maximum drift was about 0.7% for the 500-YRP earthquake, implying the possibility of minor cracking (Abdouka 2003 and Stehle 2001). However, under the 1000-YRP and 2500-YRP earthquakes, the 5-storey frame started to exhibit significant non-linearity, with drifts larger than 1.5%. This implies that the 5-storey frame would suffer severe damage under both the 1000-YRP and 2500-YRP earthquakes. This is also suggested by the irregularity and widening of hysteresis loops.

Figure 4.15—11th storey response of the 12-storey frame
The 12-storey frame remained elastic under the 500-YRP and 2500-YRP earthquakes as indicated by the relatively straight and narrow hysteresis loops. Also, the observed drift demand was relatively low in all storeys, and hence, no major structural damage was expected according to drift levels specified in Table 4.4.

Table 4.5—Summary of local failures for frames

<table>
<thead>
<tr>
<th>Type of Earthquake</th>
<th>Shear Failure for 1st Floor Column</th>
<th>Bar Pullout in Ext. Joint of Critical Floor Beam</th>
<th>Compression Bar Buckling</th>
</tr>
</thead>
<tbody>
<tr>
<td>3-Storey Frame</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>500 YRP</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>800 YRP</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>2500 YRP</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>5-Storey Frame</td>
<td></td>
<td></td>
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<tr>
<td>500 YRP</td>
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<td>No</td>
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<td>1000 YRP</td>
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<tr>
<td>2500 YRP</td>
<td>Yes</td>
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<td>No</td>
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<tr>
<td>12-Storey Frame</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>500 YRP</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>2500 YRP</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
</tr>
</tbody>
</table>

Localised failures such as bar pullout and buckling of a compression bar in a beam were monitored at the critical floor level (i.e., the storey of maximum drift). The detailed calculations can be seen in Appendix B. In addition, as the maximum shear and axial loads were expected at the ground floor, the exterior and interior columns at this floor were analysed for shear failure and column compression bar buckling.
The possibility of a bar pullout was determined by comparing the moment at which bar pullout will occur to the applied moment in the beam by the different earthquake magnitudes. Compression bar buckling in a beam and column were monitored by calculating the moment at which this would occur and then comparing it to the applied moment in the corresponding member from the earthquakes output. The shear capacity of both beams and columns of the frame were determined according to Australian Standards (AS 3600 2001). Both the concrete and stirrup contribution were accounted for as well as any additional axial load applied to the member. The likelihood of shear failure to occur was determined by comparing the shear capacities to the shear forces exerted on the frame by the earthquake. Table 4.5 summarises the local failures for the three frames.

Localised failures were not detected for the 3-storey frame except for buckling of a compression bar in the ground-storey column, as shown in Table 4.5. For the 5-storey frame both shear failure and compression bar buckling were detected at the ground floor columns, whereas for the 12-storey frame shear failure was observed under the 2500 YRP earthquake in the interior first floor column. However, failure of beam by localised failures such as bar pullout and compression bar buckling did not occur in any of the frames for all earthquakes.

### 4.4 Implications

The experimental results revealed that the low rise 1/5-scale model structure exhibited predominately elastic behaviour for the 500 YRP design acceleration coefficients (≤ 0.10g) covering most cities in Australia. Furthermore, storey displacements of the frames were also observed to be well within the 1.5% drift limits. Results from the analytical study indicated that a 1000-YRP earthquake of would cause significant non-linear behaviour (approaching collapse) in both the 3- and 5-storey frames. In contrast, the total overstrength of the 12-storey frame was found to be larger than the other two frames and sufficient to prevent the collapse of this frame even under a 2500-YRP earthquake.
Hence, from the results experimental and analytical studies it can be concluded that the typically detailed Australian RC frames will be able to resist the “design magnitude earthquake” (500 YRP with EPA≈ 0.1g) However, their behaviour under more severe earthquakes (e.g. a 2500 YRP earthquake) is questionable.
5.1 Introduction

This chapter details the experimental study conducted at the University of Adelaide on RC frame with brick infill wall panels. The study investigated the seismic response of an intermediate moment resisting brick infilled frame with minimum seismic detailing requirements. This section deals with the effect of infill wall panels on in-plane stiffness and strength of surrounding frames.

The experimental programme included tests of four ½-scale frame specimens under quasi-static cyclic loading. One of the frames tested did not have brick infill and served as a reference for the three infilled frames with gap between the columns and the infill panel as 5mm, 10mm and 15mm respectively. The test specimens were detailed in accordance with the minimum seismic detailing requirements of the Australian Standard for Concrete Structures for an intermediate moment-resisting frame (IMRF). The performance of the infilled frames was assessed in terms of some important parameters such as the level of ductility, the strength and stiffness of frame.
5.2 Test Specimen Design

In this study, ½-scale RC frames were detailed in accordance with the minimum seismic detailing requirements for an intermediate moment resisting frame (AS 3600 1988). Four frames were tested covering a range of three different gap sizes. The first test frame was a bare frame (control frame) and provided benchmark data for the remaining three frames which were built with brick infill. The three brick infilled frames had the gap size between the columns and the brickwork (side gap) as 15mm, 10mm and 5 mm, respectively (Alaia et al. 1997).

![Initial frame layout (dimensions in mm)](image)

**Figure 5.1—Initial frame layout (dimensions in mm)**

<table>
<thead>
<tr>
<th>Member</th>
<th>Layout</th>
<th>Dimensions (mm)</th>
<th>Reinforcement</th>
<th>Cover (mm)</th>
<th>% Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column</td>
<td><img src="image" alt="Column Layout" /></td>
<td>200 × 200</td>
<td>8 Y16 &amp; W6 ligatures</td>
<td>20</td>
<td>4.0</td>
</tr>
<tr>
<td>Beam</td>
<td><img src="image" alt="Beam Layout" /></td>
<td>400 × 200</td>
<td>8 Y16 &amp; W6 ligatures</td>
<td>20</td>
<td>2.0</td>
</tr>
<tr>
<td>Foundation</td>
<td><img src="image" alt="Foundation Layout" /></td>
<td>500 × 200</td>
<td>4 Y24 &amp; W6 ligatures</td>
<td>20</td>
<td>1.3</td>
</tr>
</tbody>
</table>

The size of the test specimens were scaled so that their ultimate strength was well within the capacity of the testing facility. The frames were approximately 1.8m tall and 5m long. A foundation base was also added to the frame. This provided a flat level base for the construction of the infilled wall and offered a strong floor for
anchoring the frame. Figure 5.1 shows the half-scale frame dimensions. The member cross-section details are shown in Table 5.1.

A preliminary investigation was done to determine the mode of failure for the infilled wall. In order to study the effect of different frame dimensions on infill failure pattern, Equation 5.1 from Page’s research (1985) was used to calculate the $\lambda H$ values. Page’s research (1985) indicated that for values of $\lambda H$ between 1 and 5, diagonal shear failure of the infill wall would be expected. However, corner crushing would be anticipated for $\lambda H$ values greater than 6. For the test frame, $\lambda H$ value of 1.8 was calculated and hence, diagonal shear failure was the expected mode of failure.

$$\lambda H = h \left( \frac{E_c t \sin 2\theta}{4E_b I h} \right)^{1/4} \quad (5.1)$$

Where, $h = \text{height of infill (mm)}$;
$E_c = \text{young’s modulus of concrete (MPa)}$;
$E_b = \text{young’s modulus of brick infill (MPa)}$;
$I = \text{second moment of area of concrete section (mm}^4)$;
$\theta = \angle \text{of infill diagonal to the horizontal}$;
$t = \text{thickness of infill (mm)}$

![Modified frame layout (dimensions in mm)](image)

Initially for the control frame, two jacks were used alternatively to load the frame, firstly in the West direction and then in the East direction. After the first test, the test specimen was modified (Figure 5.2) so that the load could be applied at one point. To
minimise the influence of the loading point on the joint behaviour, a short beam stub was extended beyond the beam-column joint at one end of the frame so that hydraulic jack could be mounted such that the frame could be subjected to tension as well as compression.

5.3 Specimen Construction

The frame was built lying on its side as shown in Figure 5.3. This enabled the base foundation, columns and top beam to be poured simultaneously and reduce construction time. The steel fixing was completed in the lab with the assistance of lab technicians. The formwork was squared up and fastened to the strong floor using tie-down rods before pouring.

![Construction of concrete frame](image)

Figure 5.3—Construction of concrete frame

After curing process of 7 days the formwork was removed and additional 7 days curing was done before the frame was lifted into position and fastened to the strong floor. Grout was placed between the base of frame and floor, in order to provide better bond between the two surfaces. Once the frame was up righted, the infill panels were then built by a professional brick layer. Finally the instrumentation was assembled for the whole infill-frame system. The details of instrumentation are given in section 5.4.

5.4 Material Properties

Material properties for the test specimens have been summarized in Table 5.2 and are detailed in following sections (5.4.1 to 5.4.3).
Table 5.2—Material properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Value (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel yield stress ( f_{sy} )</td>
<td>430</td>
</tr>
<tr>
<td>Concrete compressive strength ( f'_c )</td>
<td>32</td>
</tr>
<tr>
<td>Young’s modulus of steel ( E_s )</td>
<td>200,000</td>
</tr>
<tr>
<td>Young’s modulus of concrete ( E_c )</td>
<td>28,500</td>
</tr>
<tr>
<td>Young’s modulus of brick infill ( E_b )</td>
<td>2000</td>
</tr>
</tbody>
</table>

5.4.1 Concrete

The concrete was supplied by CSR Ready-mix and Boral, with specified properties:

- 32MPa
- 20mm aggregate (10mm as well)
- 80 - 100mm slump
- 1.6m3

Concrete was poured via a skip and extensively vibrated. Six cylinders and six cubes were cast for each pour. Using these sample specimens 28 day strength and test day strength tests were carried out. It was found that the test day strength was about 8 MPa higher than the 28 day strength.

5.4.2 Steel

The steel reinforcement was typical deformed bar using regular commercially available products, such as Y16, Y20 and W6. Testing was carried out for Y16 for determining the actual properties of reinforcing bars used in the experimental program. This gave yield stress value of 430 MPa for steel.

5.4.3 Masonry

The brick infill consisted of clay PGH pavers cut to size and placed on edge. As the experimental program included half-scale frame specimens, consequently half-scale brickwork was required. In order to model the equivalent full-scale brickwork, 50 mm thick paving bricks were cut to have length-to-height ratio of 230:76 as of
normal bricks. A typical cement:lime:sand ratio of 1:1:6 was used for the mortar and mixing was done by shovel load. The bricks were laid by a professional brick layer.

Connection between the masonry wall and the concrete frame was done using Abey Alligator expansion ties as shown in Figure 5.4. These were placed at a spacing of 200mm along the columns and 300mm along the top beam. Ramset PT327 24mm drive-pin (zinc plated) screw and plugs were used to fasten the anchor plate to the concrete.

Figure 5.4—Abey Alligator expansion ties

5.5 Instrumentation

The instrumentation for the control frame varied from that of the other three infilled frame specimens. For the control frame test, the instrumentation was limited to load versus deformation data. However, the infilled frames were extensively instrumented in order to monitor the interaction between the concrete frame and the infill wall. The instrumentation for both control and infilled frame is detailed in following sections (5.5.1 and 5.5.2).

5.5.1 Control Frame

The data channels for the control frame are shown in Figure 5.5. Frame load and deformation data were recorded continuously throughout the test. Two inclinometers were used, one above each beam-column joint in order to measure joint rotation. The applied loads (P1 and P2) were measured using two 225 KN force transducers, one positioned on the end of each jack (Figure 5.5). Horizontal deflections were
measured using a Linear Voltage Displacement Transducer (LVDT) having a stroke of ± 80mm.

![Diagram of Instrumentation Layout](image)

**Figure 5.5—Instrumentation layout for control frame**

### 5.5.2 Infilled Frame

<table>
<thead>
<tr>
<th>Instrumentation</th>
<th>Channel No.</th>
<th>Data source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Cell</td>
<td>0</td>
<td>Applied load from Jack</td>
</tr>
<tr>
<td>Exterior LVDTs</td>
<td>1-6</td>
<td>Column Displacement</td>
</tr>
<tr>
<td>Interior LVDTs</td>
<td>7-25</td>
<td>Gap Displacement</td>
</tr>
<tr>
<td>Strain Gauges</td>
<td>26-49</td>
<td>Strains in Steel Reinforcement</td>
</tr>
<tr>
<td>Inclinometers</td>
<td>50-53</td>
<td>Column Joint Rotation</td>
</tr>
</tbody>
</table>

For the three infilled frame specimens, a data acquisition PC running the Visual Designer software was used for recording a total of 54 channels of data. The breakdown of channels is shown in Table 5.3. The software package facilitated triggered snapshot readings to be taken at 10 KN load increment. The instrumental layout for the infilled frames is shown in Figure 5.6.

The applied load was measured by the 445 KN load cell. The exterior LVDTs (Channels 1-6) were positioned on the outside face of the column and measured the column deflection and column profile. They were mounted horizontally off a vertical free-standing frame, at 300mm spacings. The interior LVDTs (Channels 7-25) were fastened to the infill wall and measured the relative change in distance between the column and the infill wall. The horizontal LVDTs were spaced at 240mm spacings up the column although reduced 150mm spacing was used for the top three LVDTs. The vertically mounted LVDTs were spaced at 150mm along the beam with one at the midpoint. The strain gauges were typically 2.5mm gauges and were positioned on
Chapter 5 Performance of Reinforced Concrete Infilled Frames

the steel reinforcement and cast inside the concrete. Four readings were taken at each of the positions shown. At least two extra gauges were placed at each location to provide back-up in the event that any of the strain gauges were damaged during the concrete pour. The strain gauges were used to indicate when the steel reinforcement yielded, and to provide a measure of the ductility of the structure.

![LVDT Strain Gauges](image)

**Figure 5.6—Instrumentation layout infilled frame**

5.6 Experimental Procedure

The experimental setup for the test frame is shown in Figure 5.7. During testing the frames were anchored to a strong floor and were subjected to cyclic horizontal loading using a 1000 KN hydraulic jack. Each frame was subjected to a series of progressively increasing quasi-static cyclic loads until ultimate failure was reached. Figure 5.8 shows the load sequence for the control frame. For the control frame, two jacks were used alternatively to load the frame, firstly in the West direction and then in the East direction, keeping the top beam in compression, until specimen was no longer able to resist more load. For the infilled frames loading was done using only one jack. The cyclic loading was achieved by using alternate compression and tension loads. The load sequence for infill frame with 15mm gap size was identical to the control frame, but it resisted a larger load. The test procedure was slightly modified for the remaining two frames with gap size of 10mm and 5mm, respectively. For these frames, each load increment was applied three times in each direction as shown in Figure 5.9. This provided means of evaluating the stiffness degradation over a number of constant cycles.
Figure 5.7—Test setup

Figure 5.8—Load sequence for control frame

Figure 5.9—Load sequence for infilled Frame-10mm gap
5.7 Experimental Results and Discussion

5.7.1 Force-Displacement Data

The force-displacement plots for all frame specimens are shown in Figure 5.11 and Figure 5.12. Table 5.4 gives the average values of the yield load and maximum load resisted by the frame in the positive and negative loading directions. For determining the values of load and drift at yielding, a point at 75% of the maximum load was located on the ascending branch of the load-displacement curve in each direction and a line connecting the origin and the previously located point was extended until it intersected a horizontal line drawn at the maximum load. The abscissa of the intersection point gives the yield drift while the ordinate of point at 75% of the maximum load gives the yield load (Figure 5.10).

Table 5.4—Test results summary

<table>
<thead>
<tr>
<th>Frame</th>
<th>Side Gap (mm)</th>
<th>Yield Load (kN)</th>
<th>Maximum Load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Frame 1</td>
<td>No Infill</td>
<td>70</td>
<td>94</td>
</tr>
<tr>
<td>Test Frame 2</td>
<td>15</td>
<td>102</td>
<td>136</td>
</tr>
<tr>
<td>Test Frame 3</td>
<td>10</td>
<td>108</td>
<td>144</td>
</tr>
<tr>
<td>Test Frame 4</td>
<td>5</td>
<td>108</td>
<td>144</td>
</tr>
</tbody>
</table>

Figure 5.10—Load-deflection envelope
Chapter 5 Performance of Reinforced Concrete Infilled Frames

Figure 5.11 — Load vs. deflection graphs

Figure 5.12 — Load-deflection envelopes
It can be noted that each frame demonstrated reasonably stable post yield behaviour. Also, the infilled frames yielded at similar load levels (in the range of 100-110kN) and the yield drift values were in the range of 0.9% to 1.0%. These drift values are in agreement with those specified by FEMA 307 (1998) (Table 3.4). The maximum loads were first attained by the test frames (including the bare frame) at about 1.5% drift except for the frame with 15mm gap in which the drift was slightly higher (Figure 5.12). The maximum load resisted by the infilled frames were in the range of 135-145kN (Table 5.4). It was noted that for the infilled frames the drifts at the maximum loads were within the allowable drift limit of 1.5% as specified by AS1170.4 (2007) and FEMA 306 (1998). Therefore, it was concluded that the infill wall did not have any adverse effect on the in plane strength of the RC frame.

5.7.2 Failure Modes

All the test specimens failed with the formation of plastic hinges at top and bottom of the column. The infill walls suffered shear failure with most of the cracks along its horizontal beds and only some of these cracks developed into diagonal cracks further along the wall (Figure 5.13). This unexpected failure pattern can be attributed to the absence of vertical compression due to the gap between the beam and top of the wall and the length to height ratio of wall being greater than 3. The different failure mechanisms for infill wall have been discussed in section 2.4.4.

![Figure 5.13—Failure pattern in brickwork](image)

As the failure was along horizontal planes (Figure 5.14), brickwork fractured into horizontal blocks and acted as individual units. With the transfer of load from the column to the wall, each unit slipped along its bed until it came to bear against the opposite column. This caused the wall to exhibit the step appearance. Moreover, for
the frame with smallest gap size extensive shear cracking was noted in the columns as shown in Figure 5.15.

![Figure 5.14—Horizontal planes in infill wall](image1)

Also, the exterior joint performed poorly as compared to the interior joint. The premature failure of the exterior joint (Figure 5.16) was observed in all test specimens except for the test frame with 5mm gap. This failure was attributed to the provision of inadequate standard details for the exterior joint subjected to earthquake loading.

![Figure 5.15—Shear cracks in column](image2)
5.7.3 Seismic Ductility

Ductility of the frame-wall system was also estimated. For this the lateral displacement for which the frame could sustain at least 3 cycles of load without having loss of more than 30% in the stiffness was divided by the lateral displacement at yield point in the frame. As seen from Figure 5.17 the three values of load were plotted for each value of deflection. For a given deflection, the first loading cycle gives the largest value of load at that particular amplitude whereas the smallest value of load corresponds to the third loading cycle.

![Load-deflection plot](image)

**Figure 5.17—Load-deflection plot of 3 cycle maximum displacements for test 3**

In this study, with the aim of having acceptable response in terms of seismic ductility, it was assumed that the system stiffness during third loading cycle should
be more than 70% of the initial cycle’s stiffness. Applying this “70% rule” to the test data, the seismic ductility for the test specimens 3 and 4 was calculated as 2.1 and 2.5, respectively. Frames 1 and 2 were not subjected to the three loading cycles at each displacement and so, static ductility for frames 1 & 2 was estimated to be about 2.0. This was calculated by dividing the displacement at the ultimate load to the yield displacement. According to the Australian Standard (AS 1170.4 2007), the structural ductility factor ($\mu$) of 3 has been specified for intermediate moment resisting concrete frames. Hence, the code specified value is higher than that calculated from experimental results. Also, the displacement ductility was lower than that specified in Lee et al.’s (2001) study (as discussed in section 3.2.2.1).

### 5.8 Implications

An experimental study was conducted to investigate the seismic performance of RC frames with brick infill wall panels subjected to cyclic loading. For the test frames (including the bare frame) the drift values corresponding to the maximum loads were found to be within the allowable limit of 1.5%. From the measured response of the frames it was concluded that infill wall does not adversely affect the in plane ultimate strength, stiffness, and ductility of the bare RC frame. However, it was noted that the ductility of the frame-wall system was lower than the code specified value for IMRF. Furthermore, as compared to the interior joint, the exterior joint failed prematurely. This was believed to be due to poor detailing, suggesting improvement in exterior joint detailing subjected to earthquake loading.
Chapter 6

Summary and Conclusions

6.1 Summary

Researchers have repeatedly highlighted the importance of well designed and detailed RC structures in relation to their seismic behaviour. Moreover, the collapse limit state has been identified as critical state for low seismicity regions. 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). RC frames built in the majority of these regions are designed primarily for combinations of gravity and wind loads. Non-ductile detailing practice employed in these structures make them prone to potential damage and failure during an earthquake. Furthermore, the use of infill walls is a divisive issue as on positive side dual wall-frame systems have beneficial effects related to strength, stiffness, and ductility. However, if not designed properly infill wall can also lead to undesirable structural failures of complete wall frame system. Hence, performance assessment of RC structures in low seismicity regions is of significant interest. Although, from literature review presented in Chapter 3, it can be seen that there has been significant amount of international research in this area, it is worth noting that very little research exists for Australian frames.

In this research project, behaviour of typically designed GLD RC frames was assessed under different earthquake magnitudes using experimental and analytical methods, as presented in Chapter 4. Also, an experimental study was conducted to
investigate the seismic performance of RC frames with brick infill wall panels subjected to cyclic loading, as detailed in Chapter 5. The following section summarizes the conclusions of this research.

### 6.2 Conclusions for Gravity-Load-Designed Reinforced Concrete Frames

Following conclusions were drawn from the present study:

- The results of the experimental and analytical studies indicated that the non-seismically designed typical Australian 3-storey RC frame was able to resist the “design magnitude earthquake” (500-YRP with EPA ≈ 0.1g) safely, with storey drifts well within the code allowable drift limit of 1.5% specified by AS1170.4 (2007).

- 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. Alternatively, the inherent strength required for other loading conditions gives the taller, and longer period structures, proportionally more than the required strength than occurs for low-rise construction.

- Dynamic analysis results showed that all the frames were able to survive the 500-YRP earthquake (EPA ≈ 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, soft storey collapse was unlikely for the 12-storey frame which consisted of conventional beams and a flat slab floor system and columns at the ground storey being much larger due to the increased axial loads they had to carry.

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, their behaviour under more severe earthquakes (e.g. a 2500-YRP earthquake) is questionable. 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.

6.3 Conclusions for Reinforced Concrete Infilled Frames

From the measured response of the RC infilled frames, following conclusions were drawn.

- The research indicated that drift values corresponding to the maximum load were within the allowable limit of 1.5%. Hence, it can be concluded that the full height infill wall has no adverse effect on the ultimate strength, the stiffness and the displacement ductility of the bare RC frame.

- In practice, the gap size is usually based on the concrete creep and shrinkage, brick growth and thermal effects. If the contribution of the infill wall is to be ignored, then the gap size design should take into account the realistic storey drifts and the expected seismic behaviour of the structure along with the possible failure mechanisms. This is important as the insufficient gap size can lead to frame-wall interaction causing unexpected damage to the structure. Therefore, more research is needed on how the gap effect can alter the response of RC infilled frame.
• Although, the test frames were detailed in accordance with the minimum seismic detailing requirements of the Australian Standard for Concrete Structures for an intermediate moment-resisting frame (IMRF), the exterior joint failed prematurely during the experiment. This was attributed to the provision of inadequate standard details for the exterior joint subjected to earthquake loads.

6.4 Recommendations for Future Work

There are various areas of future work that should be investigated in regards to this study:

It has been concluded in this study that the behaviour of non-seismically designed RC frames in low to moderate seismicity regions under more severe earthquakes (e.g. a 2500-YRP earthquake) is questionable. Thus, the analytical study should be extended to conduct a non-linear time history analyses for these RC frames with different configurations and with some minor improvements in detailing strategies under different moderate to severe ground motions. Detailing enhancements include better confinement and reinforcing bar details at potential hinge location and beam-column joints. A non-linear computer program developed at the University of Kyushu was used for the present analytical study (Kawano et al. 1998). However, it could not automatically predict premature joint failure due to poor anchorage details. These effects should be incorporated into the program. Certainly, accounting for local failures would yield more realistic results.

Also, more research is needed to focus on the displacement-based approach, in order to establish the relation between the lateral drift and the reinforcement requirement for RC structures and hence, improving the detailing practices for the better performance of structures against earthquakes.

Experimental study reported herein on RC infilled frame investigated the behaviour of wall-frame system subjected to seismic loads. However, additional research should be conducted to cover the various variables such as different frame aspect ratio, geometry and strength of infills, geometry and location of openings. To achieve
greater understanding, the loading setup used in this study should be modified to include vertical forces which simulate gravity loads. Other configurations obtained by varying the number of bays and storeys should be investigated.
References


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ACI Committee 318 (1999). Building Code Requirements for Reinforced Concrete. ACI 318-99 and "Commentary" ACI 318R-99, American Concrete Institute, Detroit, MI.

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APPENDIX A

KAWANO’S PROGRAM BASIC MANUAL AND INPUT FILES OF THE STATIC PUSHOVER AND DYNAMIC ANALYSIS

NOTE:
This appendix is included on pages 101-194 of the print copy of the thesis held in the University of Adelaide Library.
APPENDIX B

LOCAL FAILURE CALCULATIONS

NOTE:
This appendix is included on pages 196-212 of the print copy of the thesis held in the University of Adelaide Library.