Collapse Behaviour of Limited Ductile High-Strength RC Columns under Multidirectional Earthquake Actions

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Abstract

Collapse prevention is one of the main goals of performance-based seismic design of structures; hence, it is important to understand the collapse behaviour of critical building components such as columns. Earthquakes impose complex multidirectional loading on the columns in a building frame system. The loading comprises of bidirectional lateral actions in the two horizontal directions and variable axial load in the vertical direction of the column. However, most of the experimental testing in the literature focused on evaluating the unidirectional lateral response of the columns under constant axial load, particularly due to the complexity involved in simulating the multidirectional response under quasi-static conditions. Nevertheless, with the recent advancements in the large scale testing facilities, it is now possible to simulate complex multidirectional earthquake actions on the structural components under quasi-static conditions.

The primary aim of this research is to investigate the collapse behaviour of limited ductile high strength RC columns under multidirectional earthquake actions. Limited ductile high-strength RC columns are commonly constructed in mid to high rise buildings in Australia and other regions of low to moderate seismicity. Such columns possess a smaller displacement capacity compared to the ductile RC columns due to the wider spacing of the transverse reinforcement, which results in inadequate confinement of the concrete core, and thus may be prone to collapse when subjected to multidirectional earthquake actions in a rare or very rare earthquake event.

The collapse behaviour of limited ductile high-strength RC columns has been investigated in this research by conducting a comprehensive experimental testing program in which 14 limited ductile high-strength RC columns, representative of typical Australian construction practice, were experimentally tested till collapse. Six specimens each were tested under unidirectional and bidirectional lateral loading with constant axial load and the last two specimens were tested under bidirectional lateral loading with axial load variation. The variables of the testing program included axial load ratio, transverse reinforcement ratio, concrete compressive strength and the
loading protocols. The experimental behaviour of the specimens was thoroughly studied in terms of force-displacement behaviour, moment-curvature behaviour, axial displacement-lateral drift behaviour, stiffness degradation and energy dissipation. Detailed comparisons were also made between the unidirectional and bidirectional behaviour of the tested specimens.

The bidirectional lateral loading and axial load variation protocols used in the experimental study were developed by conducting a numerical study in which a case study RC building was subjected to a suite of 20 ground motions representative of low to moderate seismic regions. The resulting patterns of bidirectional lateral displacements and axial load variation in the columns were studied by rigorous statistical processing of the results and were subsequently generalised in the form of bidirectional loading protocols, namely octo-elliptical (1:1) and octo-elliptical (0.6:1) paths and axial load variation protocols, namely synchronous axial load variation and nonsynchronous axial load variation.

Finally, a model was proposed for predicting the force-displacement backbone envelope of limited to moderately ductile RC columns subjected to unidirectional and bidirectional lateral actions. The expressions for the post-peak displacement capacity were calibrated with a comprehensive database of experimental tests from the literature. The model predicted the experimental force-displacement backbone envelope of the specimens tested in this study with reasonable accuracy. The applications of the proposed model, particularly relating to the seismic performance assessment of limited to moderately ductile RC columns in regions of low to moderate seismicity, are also discussed in detail with the aid of a case study example. The thesis is concluded with a set of design recommendations that would improve the existing seismic design procedure of RC columns in Australia and other regions of low to moderate seismicity.
Declaration

I hereby declare that this thesis contains no material which has been accepted for the award to the candidate of any other degree or diploma, except where due reference is made in the text of the examinable outcome. I affirm that to the best of my knowledge, the thesis contains no material previously published or written by another person except where due reference is made in the text of the thesis.

7/16/2020

X SR

Saim Raza

Signed by: Saim Raza
List of Publications

Journal Articles (Published and Submitted Papers)

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Conference Papers


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Notations

\( A_c \) = Cross-sectional area of the section bounded by the centre-line of the outermost ties

\( A_g \) = Gross cross-sectional area of the concrete section

\( A_v \) = Cross-sectional area of one leg of transverse reinforcement

\( A_{ch} \) = Cross-sectional area of the section measured to the outside edges of the transverse reinforcement

\( A_{st} \) = Total cross-sectional area of longitudinal bars

\( A_{vt} \) = Total cross-sectional area of transverse reinforcement

\( a \) = Minor axis length of the ellipse

\( b \) = Width of the section/ major axis length of the ellipse (section 3.7)

\( b_c \) = Width of the core measured between center-lines of the outermost ties

\( b_h \) = Width of the core measured to outside hoops

\( D_C \) = Smaller column dimension

\( D_o \) = Minimum dimension of the concrete core

\( d \) = Effective depth of the section

\( d_c \) = Depth of the core measured between center-lines of the outermost ties

\( d_h \) = Depth of the core measured to outside hoops

\( d_b \) = Diameter of the longitudinal bar

\( E_c \) = Modulus of elasticity of concrete

\( E_d \) = Energy dissipation in each cycle

\( +F_i \) = Maximum cyclic force in the positive direction
\(-F_i\) = Maximum cyclic force in the negative directions

\(F_u\) = Ultimate lateral force capacity

\(F_y\) = Lateral force capacity at yield

\(F(y)\) = Lateral force in y-direction

\(F_{af}\) = Lateral force capacity at axial load failure

\(F_{cr}\) = Lateral force capacity at cracking

\(F_{lf}\) = Lateral force capacity at lateral load failure

\(f_{cm}\) = Actual mean cylinder strength of concrete

\(f_t\) = Flexure tensile strength of concrete = \(0.6\sqrt{f'_{c}}\)

\(f_{yhl}\) = Longitudinal reinforcement yield strength

\(f_{yhl}\) = Transverse reinforcement yield strength

\(f'_{c}\) = Characteristic compressive (cylinder) strength of concrete at 28 days

\(H\) = distance between points of contra-flexure and maximum bending moment

\(h\) = Total depth of the section

\(h_x\) = Maximum centre-to-centre spacing of longitudinal bars laterally supported by corners of crossties or hoop legs around the perimeter of the column

\(I_g\) = Gross moment of inertia

\(I_{eff}\) = Effective moment of inertia

\(K_g\) = Gross stiffness

\(K_i\) = Stiffness in each cycle

\(k\) = Constant parameter to calculate drift
\[ k_1 = \text{Constant factor dependent on displacement ductility} \]
\[ k_f = \text{Confinement effectiveness factor} \]
\[ k_n = \text{Concrete strength factor} \]
\[ L = \text{Shear span length} \]
\[ l_p = \text{Length of the plastic hinge region} \]
\[ M = \text{Factored moment at section} \]
\[ M_u = \text{Ultimate moment capacity} \]
\[ M_y = \text{Yield moment capacity} \]
\[ M_{cr} = \text{Cracking moment} \]
\[ m = \text{Constant parameter used in the calculation of transverse reinforcement} \]
\[ n = \text{Axial load ratio} = \frac{P}{A_g f_c'} \]
\[ n_b = \text{Balanced axial load ratio (Chapter 2)/ number of laterally restrained longitudinal bars (Chapter 3)} \]
\[ n_d = \text{Design axial load ratio limit} \]
\[ P = \text{Design axial load} \]
\[ P_o = \text{Nominal axial strength at zero eccentricity} \]
\[ P_u = \text{Factored axial force} \]
\[ R_f = \text{Force reduction factor (AS 1170.4)} \]
\[ s = \text{Spacing of ties} \]
\[ S_{a(T_x)}, S_{a(T_y)}, S_{a(T_z)} = \text{RSA in the X, Y and Z directions for the ground motion under consideration} \]
\[ S_p = \text{Structural performance factor} \]
\( V_n \) = Nominal shear capacity considering flexural ductility

\( V_o \) = Shear strength of column without modification for ductility

\( V_p \) = Shear force at the development of the flexural capacity

\( X \) = X displacement/drift in a vertical ellipse

\( X_r \) = X displacement/drift in a rotated ellipse

\( x \) = Distance to the neutral axis (Chapter 7)/drift/displacement in the X-direction (Chapter 3)

\( Y \) = Y displacement/drift in a vertical ellipse

\( Y_r \) = Y displacement/drift in a rotated ellipse

\( Y_{r,max} \) = Maximum Y displacement/drift in a given elliptical loop

\( y \) = Lateral displacement in y-direction

\( Z \) = Design axial load

\( Z_S \) = Synchronous axial load in a given elliptical loop

\( Z_{NS} \) = Nonsynchronous axial load in a given elliptical loop

\( \alpha_{1v}, \alpha_{2v}, \alpha_{3v} \) = Coefficients dependent on the structural system and configuration

\( \alpha \) = Ellipse size factor

\( \alpha_1 \) = Force reduction factor in direction 1

\( \alpha_2 \) = Force reduction factor in direction 2

\( \alpha_e \) = confinement effectiveness factor

\( \beta \) = Ratio of axial load ratio to the balanced axial load ratio (Chapter 2)/Biaxial interaction factor (Chapter 7)

\( \gamma_1 \) = Displacement reduction factor in direction 1
\[ \gamma_2 = \text{Displacement reduction factor in direction 2} \]
\[ \gamma_{cl} = 1.5 \text{ (parameter defining primary or secondary seismic elements)} \]
\[ \varepsilon_{sy.d} = \text{Yield strain of tensile steel} \]
\[ \nu = \text{Maximum shear stress} \]
\[ \nu_n = \text{Variation in axial load (in \%)} \]
\[ \rho_h = \text{Transverse reinforcement ratio by area} = \frac{A_{re}}{s_b} \]
\[ \rho_s = \text{Transverse reinforcement ratio by volume} = \frac{A_v(2b_c+2d_c)}{b_h d_h s} \]
\[ \rho_v = \text{Longitudinal reinforcement ratio} \]
\[ \rho_d = \text{steel ratio of diagonal reinforcement (if any), in each diagonal direction} \]
\[ \theta = \text{Angle} \]
\[ \theta_t = \text{Constant defining the type of column (i.e. cantilever or double curvature)} \]
\[ \theta_r = \text{Angle of rotation of ellipse} \]
\[ \theta' = \text{Angle of the arc} \]
\[ \phi = \text{Strength reduction factor} \]
\[ \phi_e = \text{Maximum elastic curvature} \]
\[ \phi_u = \text{Ultimate curvature} \]
\[ \phi_y = \text{Yield curvature} \]
\[ \varphi_f = \text{Factor of safety} \]
\[ \mu = \text{Effective coefficient of friction} \]
\[ \mu\phi = \text{Curvature ductility factor (Chapter 2)/ Structural ductility factor (Chapter 7)} \]
\[ \mu_{S_a}(T_x), \mu_{S_a}(T_y), \mu_{S_a}(T_z) \] = Mean RSA in the X, Y and Z directions of the suite of
the ground motions considered in the region of interest

\[ +\Delta_l = \] Maximum cyclic displacement in the positive direction

\[ -\Delta_l = \] Maximum cyclic displacement in the negative direction

\[ \delta_a = \] Ratio of the horizontal displacement due to the sliding between cracking
surfaces to the damaged length of the column

\[ \delta_u = \] Ultimate drift

\[ \delta_y = \] Yield drift

\[ \delta_{af} = \] Drift at axial load failure

\[ \delta_{cr} = \] Drift at cracking

\[ \delta_{lf} = \] Drift at lateral load failure

\[ \delta_{pl} = \] Plastic drift

\[ \lambda = \] Correction factor related to unit weight of concrete

\[ \omega'; \omega = \] mechanical reinforcement ratio of the compression and tension longitudinal
reinforcement, respectively
Abbreviations

CAL  Constant Axial Load
DBE  Design Basis Earthquake
HSRC High-Strength Reinforced Concrete Columns
LPOT Linear Potentiometers
LVDT Linear Variable Displacement Transducer
NSRC Normal-Strength Reinforced Concrete Columns
UHSRC Ultra High-Strength Reinforced Concrete Columns
MAST Multi-Axis Substructure Testing
MCE  Maximum Considered Earthquake
NS-VAL Nonsynchronous Variable Axial Load
OpenSees Open System for Earthquake Engineering Simulation
PGA  Peak Ground Acceleration
PGV  Peak Ground Velocity
RC   Reinforced Concrete
RMSE Root Mean Squared Error
RSA  Response Spectral Acceleration
SPOT String Potentiometer
SS-VAL Synchronous Variable Axial Load
VAL  Variable Axial Load
Chapter 1  Introduction

1.1 Background

Reinforced concrete (RC) structures have experienced partial or complete collapse in many past earthquakes globally (Izmit 1999, Wenchuan 2008, L'Aquila 2009 etc), resulting in loss of human lives along with huge losses to the economy. RC columns are the main elements that support an RC structure vertically, and as such, the collapse of an RC structure is conditioned on the collapse of its columns. The collapse of one or more columns can actually lead to the redistribution of internal forces in the structural system and thus may serve as a precursor to the progressive collapse of the whole structure.

High-strength RC (HSRC) columns are widely used in mid- to high-rise RC structures all over the world due to the numerous advantages they offer over normal-strength RC columns (NSRC), such as the reduction in column size and self-weight of the structure, creation of more floor space, material saving and improved durability. However, high-strength concrete is brittle in nature due to smaller concrete compressive strain [1]. The drawback of low ductility of the high-strength concrete is overcome by providing adequate confinement to the concrete core in regions of high seismicity, thereby resulting in a ductile column. However, limited ductile detailing requirements, which result in widely spaced transverse reinforcement are the construction standard in regions of low to moderate seismicity, such as Australia. Such columns are expected to possess a low collapse drift capacity due to the inadequate confinement provided to the brittle core of the high-strength concrete.

The limited ductile HSRC columns in mid to high rise RC buildings in Australia are generally not part of the lateral load resisting system and just act as gravity members, as RC rectangular walls and building cores form the primary lateral load resisting system in such buildings [2]; however, for vertical stability of the building, these columns still need to possess enough drift capacity to displace along with the floor
diaphragm without collapsing (i.e. losing vertical load-carrying capacity). If the displacement demand imposed by the floor diaphragm on these columns during a major earthquake event in Australia exceeds the collapse drift capacity of these columns then the vertical collapse of the column will become imminent, which may trigger the collapse of the entire structure. Many post-earthquake reconnaissance surveys [3] have also concluded that the main cause of collapse during an earthquake is the loss of vertical load-carrying capacity rather than lateral load-carrying capacity in the load-bearing elements (i.e. columns, walls) of the building.

Hence, it is essential to understand the collapse behaviour and capacity of limited ductile HSRC columns to ensure earthquake safety of the building stocks in Australia and globally in other regions of low to moderate seismicity. However, the collapse behaviour of limited ductile HSRC columns under earthquake actions has received considerably less attention compared to ductile HSRC columns. Moreover, most of the experimental testing on HSRC columns has been conducted under unidirectional cyclic actions and constant axial load (CAL).

On the other hand, earthquake ground shaking is multi-directional and imposes varying triaxial forces on a building column during an earthquake. These forces consist of bidirectional lateral actions in the two horizontal axes and variable axial load (VAL) in the vertical axis of the column. While modern design codes provide guidance on bidirectional strength design of an RC column where strength of the cross-section in each direction is assessed (i.e. use the simplified rectangular stress block analysis to calculate the capacity about the minor and major axis) and then combination factors are used to approximate the bidirectional response, there is, in contrast, a lack of guidance regarding the determination of collapse drift capacity of columns subjected to multidirectional earthquake loading.

A realistic assessment of the collapse capacity of limited ductile HSRC columns can be made by experimental testing under multidirectional earthquake actions. The choice of a realistic loading protocol for simulating the earthquake actions under quasi-static conditions is of paramount importance in this regard. Despite experimental testing of
RC columns under different bidirectional lateral loading protocols such as linear, diagonal, circular, rhombus, expanding square, square in each quadrant, elliptical and hexagonal orbital patterns, there is no widely accepted standard in the literature as to which of these protocols is a more realistic representation of the actual loading imposed on an RC column during an earthquake. Further to this, there is no quantitative study that investigated the bidirectional lateral displacement pattern and typical patterns of axial load variation in columns of RC buildings during earthquakes. Thus, there is a lack of guidance for realistically simulating multidirectional earthquake actions comprising of bidirectional lateral loading and axial load variation on RC columns under quasi-static conditions.

The above-mentioned issues, including the highlighted research gaps, have been addressed in this research by first developing loading protocols for quasi-static testing of RC columns under multidirectional earthquake actions. The proposed loading protocols are then employed in the experimental testing of limited ductile HSRC columns to develop a better understanding of their collapse behaviour. Force-displacement models that could reliably predict the post-peak (especially collapse) behaviour of RC columns under multidirectional earthquake actions are then proposed. The post-peak displacement models were developed by using an extensive database of RC columns from the literature that included NSRC as well as HSRC columns. It is noted that the previous models in literature were developed mostly for NSRC columns and in this study, an attempt has been made to develop unified post-peak drift models that could predict the displacement capacity of both NSRC and HSRC columns with reasonable accuracy. Finally, design guidelines for limited ductile HSRC columns are outlined to ensure collapse prevention of such columns in a rare or very rare earthquake event.

1.2 Aims and Objectives

The overall aim of this research is to develop a fundamental understanding of the collapse behaviour of limited ductile HSRC columns under multidirectional earthquake actions.
The specific objectives of this research are:

- Develop realistic multidirectional loading protocols comprising of bidirectional lateral loading and axial load variation protocols for quasi-static testing of RC columns
- Evaluate and compare the collapse behaviour of limited ductile HSRC columns subjected to different loading histories by experimental testing under
  - Unidirectional lateral loading and CAL
  - Bidirectional lateral loading and CAL
  - Bidirectional lateral loading and VAL
- Develop Force-Drift models and formulae that can reliably estimate the full-range elastic and inelastic response of RC columns under multi-directional earthquake actions
- Propose design recommendations to improve existing seismic design procedure of limited ductile HSRC columns from the perspective of collapse prevention in a major earthquake event

1.3 Research Significance

The research employs experimental, numerical and empirical methods to develop a fundamental understanding of the collapse behaviour of limited ductile HSRC columns subjected to multidirectional earthquake actions. The adequacy of the requirements of the concrete design standard in Australia is assessed and appropriate recommendations are made to ensure collapse prevention of RC columns constructed in Australia and other regions of low to moderate seismicity. The proposed force-drift models and formulae would enable structural design engineers and practitioners to make a reliable estimation of the inelastic displacement capacity of the RC columns at the design stage, which will then lead to the design of columns that are less prone to collapse in a major earthquake event. In this way, the research outcomes will have important implications for the earthquake safety of our building stocks in Australia and globally. The development of new quasi-static testing procedures in the form of the proposed bidirectional lateral loading and axial load variation protocols are the
technical innovations of the thesis that can revolutionize the experimental structural engineering, especially in the quasi-static testing environment. All of these contributions would ultimately lead to further development of the displacement-based design procedures.

1.4 Thesis Outline

This research work consists of five different studies conducted with the aim of improving the understanding of the collapse behaviour of limited ductile HSRC columns commonly constructed in mid to high rise buildings in regions of low to moderate seismicity. The research comprises of three experimental, one numerical and one empirical study.

The thesis is organized as follows:

Chapter 1 presents the background, aims, objectives and significance of the research.

Chapter 2 provides a comprehensive literature review on the displacement behaviour of RC columns and culminates with the identification of important research gaps that are addressed in this research.

Chapter 3 presents a broad overview of the experimental testing program conducted in this study. The design details of the test specimens, the test matrix, test setup and instrumentation are discussed in detail. The development of loading protocols for simulating bidirectional lateral actions and axial load variation in the quasi-static testing of RC columns is also presented.

Chapter 4 is dedicated to the experimental results of 6 limited ductile HSRC columns tested till collapse under unidirectional lateral actions and CAL. The effect of axial load ratio, transverse reinforcement ratio and concrete compressive strength on the collapse behaviour of the specimens is discussed in detail.

Chapter 5 presents the experimental results of 6 limited ductile HSRC columns tested till collapse under bidirectional lateral actions and CAL. The effect of the loading path on the collapse behaviour of the specimens was investigated by testing the specimens under three different bidirectional loading protocols.
Chapter 6 provides details of the numerical study conducted to investigate the range of axial load variation in columns of RC buildings constructed in regions of low to moderate seismicity. The governing factors influencing the axial load variation of columns were studied in detail. Experimental results of two limited ductile HSRC column specimens tested under bidirectional lateral actions and VAL are then discussed. The collapse behaviour of the specimens is compared with previously tested columns under unidirectional and bidirectional lateral actions with CAL.

Chapter 7 describes the details of the force-drift models and formulae proposed by the author to reliably estimate the full range behaviour of RC columns under unidirectional and bidirectional cyclic actions. The application of the proposed models is also discussed with the aid of a case study example.

Chapter 8 outlines the conclusions of the research and provides design recommendations to improve the existing seismic design procedure of limited ductile HSRC columns constructed in mid to high rise buildings in Australia and other regions of low to moderate seismicity. Recommendations for future research are also provided.
Chapter 2  Literature Review

2.1  Introduction

Columns are the vertical load-bearing elements that can either form a part of the lateral load resisting system of the structure i.e. in a moment-resisting frame or just serve as gravity members in an RC wall frame structure. In either case, collapse prevention of the RC columns is central to the global stability of an RC wall or frame structure in the event of an earthquake. This is because the collapse (loss of vertical load-carrying capacity) of one or more columns can lead to the redistribution of the vertical forces to the other load-bearing members, which can then result in the partial or complete collapse of the whole building structure. Therefore, it is important to understand the various modes of failure of an RC column and the common deficiencies that may trigger collapse.

This chapter first provides a basic understanding of different types of failure modes of the RC columns and gives an overview of the design deficiencies (particularly relating to the confinement reinforcement) that led to the collapse of RC columns in previous earthquakes, and then compares the confinement requirements for limited ductile, moderately ductile and fully ductile RC columns based on the specifications of different concrete design standards. This is followed by a summary of the previous studies conducted on the RC (particularly HSRC) columns under unidirectional and bidirectional lateral actions with CAL or VAL and the identification of the research gap about limited understanding of the collapse behaviour of limited ductile HSRC columns as opposed to moderately and fully ductile columns. A review of the existing drift models for predicting the failure drift capacity of the RC columns is also conducted using an extensive database of RC columns from the literature. Finally, a summary of the existing open problems that would be addressed in this research is presented.
2.2 Types of Failures of RC Columns

The failure of RC columns under lateral loading can be categorized into three types, which are as follows:

**Flexure Failure:** In flexure failure, the degradation in the lateral load-carrying capacity of the column commences after yielding of the longitudinal reinforcement. Flexural damages, such as concrete crushing, buckling of longitudinal reinforcement and spalling of concrete are primarily responsible for the degradation in the lateral capacity of the column in flexure failures [4]. According to the classification criteria of ASCE/SEI 41/06 [5], columns with shear capacity ratio \( \frac{V_p}{V_o} \leq 0.6 \) of less than 0.6 are expected to undergo flexure failures.

The shear capacity ratio is defined as the ratio of plastic shear demand \( V_p \) to the shear strength of the column, \( V_o \) (without modification for ductility). The plastic shear demand can be determined by dividing the maximum moment capacity of the column cross-section with the shear span length of the column, whereas the shear strength can be determined from available shear strength models, such as Sezen and Moehle [6]. Figure 2.1 schematically shows the conditions for various failure modes of RC columns under lateral loading.

**Shear Failure:** In shear failures, the degradation in the lateral load-carrying capacity of the column occurs before yielding of the longitudinal reinforcement. ASCE/SEI 41/06 [5] classifies columns with the shear capacity ratio of greater than 1 \( \frac{V_p}{V_o} > 0.6 \) as shear-critical columns. Shear failure is the most undesirable failure mode as it results in the lowest collapse drift capacity of the column. Moreover, a shear dominated column is unable to reach its maximum lateral load-carrying capacity. Shear failures are accompanied by extensive diagonal cracking. Columns with an aspect ratio of less than 2 \( \frac{L}{h} \leq 2 \) and low amount of transverse reinforcement are expected to experience shear failures. It is noted that aspect ratio is the ratio of the shear span length \( L \) to the depth \( h \) of the column.
**Flexure-Shear Failure:** In this type of failure, the degradation in the lateral load-carrying capacity of the column occurs after yielding of the longitudinal reinforcement, but results from the shear distress. According to the classification criteria of ASCE/SEI 41/06 [5], columns with shear capacity ratios between 0.6 and 1.0 \((0.6 < \frac{V_p}{V_o} \leq 1.0)\) are expected to experience flexure-shear failures.

![Diagram of shear strength and displacement ductility](image)

Figure 2.1. Column failure modes under lateral loading

### 2.3 Collapse of RC Columns during Earthquakes – Causes and Mechanism

A number of deficiencies may result in damage that can trigger the collapse of RC columns during an earthquake. These deficiencies mainly include i) inadequate transverse reinforcement and confinement to the concrete core; ii) inadequate lap-splice and bond anchorage; iii) poor quality concrete [7-8].

Inadequate amount of transverse reinforcement results in non-ductile or limited ductile RC columns and is the main cause of shear failure. Shear failure may also occur due to the small aspect ratio of the column, which would result in a short and rigid column that would attract more shear forces than columns with a higher aspect ratio. A diagonal fracture in the mid-length of the column is usually exhibited by shear critical columns as depicted in Figure 2.2 (a), where a damaged RC column from L’Aquila, Italy (2009) earthquake is shown.
Another cause for collapse can be the inadequate flexural capacity that results from high levels of axial load, poor quality concrete and inadequate longitudinal reinforcement in the plastic hinge region of the column [9]. It is noted that the plastic hinge region is the region over the length of the column where flexural moments exceed the yielding capacity. According to Varum [10], exterior columns in a building can be more prone to flexure deficient behaviour due to high levels of variable axial force during ground motion excitations. An RC column damaged due to the flexural deficiencies during Lorca, Spain (2011) earthquake is shown in Figure 2.2 (b).

The potential of the collapse of an RC column can also increase due to inadequate lap-spike and bond anchorage, which results in weakness of the beam-column joint area where failure might occur due to the bond-splitting. Figure 2.2 (c) shows a collapsed column during Izmit, Turkey (1999) earthquake due to the lap-spike failure.

Bidirectional earthquake actions are another important aspect which can increase the vulnerability of collapse of an RC column. Previous studies have shown that the displacement capacity of the RC columns which support high axial loads and are subjected to bidirectional cyclic actions is very small [11-12], thereby making such columns more prone to collapse.
Generally, the collapse of an RC column under lateral loading occurs in the following manner: The first signs of distress usually appear in the form of cracks in the plastic hinge region. The cracks may be flexural or shear cracks or both depending on the type of the column i.e. flexure-critical, shear-critical or flexure-shear critical. As the displacement demand on the column increases, the cracks increase in size and number, thereby leading to crushing and spalling of the concrete cover in the plastic hinge region. Cover spalling is then followed by the buckling of the longitudinal reinforcement and opening of stirrups [8], ultimately leading to the loss of axial load carrying capacity of the column, when there is a sufficient reduction in the cross-sectional area of the concrete in the plastic hinge region. Figure 2.3 shows the columns of a five-storey building that are near collapse in the Wenchuan (2008) earthquake.

![Columns near collapse](image)

Figure 2.3. Partial collapse of RC columns in Wenchuan (2008) earthquake: a) external column; b) internal column (Photos reproduced from Kafle et al. [3])

### 2.4 Confinement Requirements for Different Ductility Levels of RC Columns

The displacement capacity of an RC column is dependent on a number of factors such as aspect ratio of the column, amount of transverse/confine reinforcement, concrete compressive strength, transverse reinforcement yield strength and axial load ratio [13]. However, two parameters that largely influence the displacement capacity of an RC column are the amount of transverse/confine reinforcement and axial load ratio. The transverse reinforcement in an RC column, besides preventing buckling of longitudinal bars and avoiding shear failure, also provides confinement to the
concrete core, which in turn significantly affects deformability of the RC column. The lateral confinement of the concrete core, particularly in the plastic hinge region, enhances the deformation capacity of the RC column by increasing crushing strength and ultimate strain of the concrete core. Moreover, it also restrains the dilation or expansion of the concrete core by imposing lateral confining pressure. Therefore, the amount of confinement reinforcement is one of the major parameters affecting the ductility of an RC column, and it serves as the basis for classifying an RC column as limited ductile, moderately ductile or fully ductile.

Due to the perceived lower seismic risk in regions of low to moderate seismicity, confinement reinforcement is widely spaced, thereby resulting in a limited ductile RC column. On the other hand, ductile detailing prevalent in high seismic regions results in a confinement reinforcement that is very closely spaced. Table 2.1 provides a comparison of the confinement requirements for different ductility levels of NSRC and HSRC columns in accordance with the specifications of RC design standards. The limited ductile and ductile detailing requirements, specified by the design standards of Australia AS 3600 [14], Canada CSA A23.3 [15], Europe [16] (Eurocode 8), US ACI 318 [17] and New Zealand NZS 3101-1 [18] have been compared in the table.

A comparison of transverse reinforcement spacing requirements for different ductility levels of NSRC columns in Table 2.1 indicates that transverse reinforcement in moderately ductile columns is approximately 2 times more closely spaced than corresponding limited ductile columns. Likewise, transverse reinforcement in fully ductile columns is 3 to 4 times more closely spaced than limited ductile columns.

It is also noted that, while spacing requirements for HSRC columns are reduced relative to NSRC columns in limited ductile detailing, they remain the same in ductile detailing specifications. This may be in particular, because spacing requirements for ductile detailing are already so stringent that any further reduction may not be practically viable as it may lead to congestion and placement problems.

In addition to reducing the spacing requirements, design standards also specify an expression for determining a minimum transverse reinforcement ratio for moderately
to fully ductile detailing of RC columns. A notable thing that can be observed in these expressions is the inclusion of the axial load ratio \( \left( \frac{P}{f'cA_g} \right) \), in determining the minimum amount of transverse reinforcement for the confinement of concrete core in ductile columns. These expressions indicate that the amount of transverse reinforcement in ductile columns is proportional to the axial load ratio of the column. Thus, columns supporting higher axial load ratios are provided with more amount of transverse reinforcement in order to ensure ductile behaviour. ACI 318 [17] goes to a further extent in this regard and provides separate expressions based on the limiting values of axial load ratio and concrete compressive strength, thereby resulting in even more stringent confinement requirements for HSRC columns with higher axial load ratios >0.3. This is in particular due to the extremely brittle nature of high-strength concrete at high axial load ratios.

The difference between different ductility levels is illustrated in this section by designing a HSRC column in accordance with the requirements of different concrete design standards for axial load ratios \((n)\) of 0.2 and 0.4 in Table 2.2. It can be observed in Table 2.2 that the minimum amount of transverse reinforcement required for moderate ductility is around 3 times the amount required for limited ductile behaviour at an axial load ratio of 0.2. Furthermore, as the axial load ratio is doubled, the amount of transverse reinforcement required to ensure moderately ductile behaviour of RC column also increases more than twice.

On the other hand, a fully ductile column has approximately 5 times higher transverse reinforcement at low axial load ratio \((n = 0.2)\) and about 9 times higher transverse reinforcement at high axial load ratio \((n = 0.4)\), than a corresponding limited ductile column. This underscores the significance of axial load ratio in reducing the ductility level of an RC column as such that significantly higher transverse reinforcement is required to compensate for the loss in ductility due to higher axial load ratio. It is also evident from Tables 2.1 and 2.2 that limited ductile RC columns have generally the same amount of transverse reinforcement irrespective of the axial load ratio, which implies that seismic resilience of limited ductile columns supporting a higher axial load can be
at serious risk as their ductility which is already limited, is further reduced due to high axial load. It is worth noting that among all the design standards the limited ductile detailing of NZS 3101-1 is the most stringent.

Table 2.1. Confinement requirements of RC design standards for different ductility levels

<table>
<thead>
<tr>
<th>Standard</th>
<th>Spacing in critical region (ss)</th>
<th>Minimum Transverse Reinforcement Ratio by Area/volume ($\rho_h/\rho_s \geq$) for confinement in critical region</th>
</tr>
</thead>
<tbody>
<tr>
<td>AS 3600 [14] (Limited Ductile)</td>
<td>Dc, 15db, 300 mm</td>
<td>-</td>
</tr>
<tr>
<td>CSA A23.3 [15] (Limited Ductile)</td>
<td>Dc, 16dB, 48dt, 300 mm</td>
<td>-</td>
</tr>
<tr>
<td>EN 1998-1 [16] Eurocode 8 DCL (Limited Ductile)</td>
<td>0.6Dc, 12db, 240 mm</td>
<td>-</td>
</tr>
<tr>
<td>EN 1998-1 [16] Eurocode 8 DCM (Moderately Ductile)</td>
<td>0.5Dc, 8db, 175 mm</td>
<td>$\rho_s = \frac{30\mu\varepsilon_{sy,d}}{k_e} \frac{P}{f'cA_g Dc} - 0.035$</td>
</tr>
<tr>
<td>EN 1998-1 [16] Eurocode 8 DCH (Ductile)</td>
<td>0.33Dc, 6db, 125 mm</td>
<td>$\rho_s = \frac{30\mu\varepsilon_{sy,d}}{k_e} \frac{P}{f'cA_g Dc} - 0.035$</td>
</tr>
<tr>
<td>ACI 318 [17] (Limited Ductile)</td>
<td>16db, 48db or Dc</td>
<td>-</td>
</tr>
</tbody>
</table>
### ACI 318 [17] (Ductile)

\[
\rho_h = 0.3 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}} \quad \text{(a)}
\]

\[
\rho_h = 0.09 \frac{f'_c}{f_{yt}} \quad \text{(b)}
\]

\[
\rho_h = 0.2kfkA_p \frac{P}{f_{yt}A_{ch}} \quad \text{(c)}
\]

Greater of (a) and (b) for \( P \leq 0.3A_gf'_c \) and \( f'_c \leq 70 \)

Greater of (a), (b) and (c) for \( P > 0.3A_gf'_c \) or \( f'_c > 70 \)

### NZS 3101-1 [18] (limited Ductile)

\[
\rho_h = \frac{(1.0 - \rho_v m) A_g f'_c}{3.3} \frac{P}{f_{yt} \phi f'_c A_g} - 0.0065
\]

where \( m = \frac{f_{yt}}{0.85f'_c} \)

### NZS 3101-1 [18] (Ductile)

\[
\rho_h = \frac{(1.3 - \rho_v m) A_g f'_c}{3.3} \frac{P}{f_{yt} \phi f'_c A_g} - 0.006
\]

where \( m = \frac{f_{yt}}{0.85f'_c} \)

#### 2.5 Behaviour of HSRC Columns under Unidirectional Cyclic Actions

The force-displacement behaviour of HSRC columns under unidirectional lateral actions has been widely investigated over the past few decades. However, most of the studies focussed on HSRC columns with detailing representative of moderately to fully ductile HSRC columns constructed in regions of higher seismicity. The studies primarily concluded that the displacement capacity of the column reduces with the increase in the concrete compressive strength. Therefore, a relatively higher amount of transverse reinforcement is needed to produce the same ductility in HSRC columns as NSRC ones. It was also concluded that the axial load ratio has the most significant effect in reducing the drift capacity of the column. This section presents a summary of the studies conducted in this regard and their major findings.
Table 2.2. Minimum transverse reinforcement ratio for different ductility levels – A case study example

<table>
<thead>
<tr>
<th>Column Properties</th>
<th>Design Standard</th>
<th>Minimum Transverse Reinforcement Ratio by Area $\rho_h$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>n=0.2</td>
</tr>
<tr>
<td>Cross-Section=250×250 mm</td>
<td>AS 3600 [14] (Limited Ductile)</td>
<td>0.42</td>
</tr>
<tr>
<td>Concrete compressive strength=$f'_c=70$ MPa</td>
<td>CSA A23.3 [15] (Limited Ductile)</td>
<td>0.34</td>
</tr>
<tr>
<td>Longitudinal rebar=6N16</td>
<td>Eurocode 8 [16] DCL (Limited Ductile)</td>
<td>0.42</td>
</tr>
<tr>
<td>Transverse reinforcement rebar=N10</td>
<td>Eurocode 8 [16] DCM (Moderately Ductile)</td>
<td>1.1</td>
</tr>
<tr>
<td>Transverse reinforcement yield strength=$f_{yh}=500$ MPa</td>
<td>Eurocode 8 [16] DCH (Ductile)</td>
<td>1.89</td>
</tr>
<tr>
<td>Concrete cover=20 mm</td>
<td>ACI 318 [17] (Limited Ductile)</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>ACI 318 [17] (Ductile)</td>
<td>1.75</td>
</tr>
<tr>
<td></td>
<td>NZS 3101-1 [18] (Limited Ductile)</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>NZS 3101-1 [18] (Ductile)</td>
<td>1.0</td>
</tr>
</tbody>
</table>
The earliest studies on the behaviour of HSRC columns were conducted in Japan. The primary aim of these studies was to find out the amount of transverse reinforcement that would result in ductile behaviour of HSRC columns so that appropriate design provisions could be included in the design standards.

Sakai et al. [19] conducted an experimental study to investigate the flexural behaviour of short \((L/h = 2.0)\) HSRC columns with concrete compressive strength of about 100 MPa. The variables of the study were longitudinal reinforcement ratio, transverse reinforcement ratio and transverse reinforcement yield strength. All the columns were tested to the point of collapse. The study noted that the lateral confining stresses in hoops greatly affected the ductility of the columns. It was also observed that high-strength hoops improved the ductility of the HSRC columns.

Muguruma and Watanabe [20] tested eight HSRC column specimens with transverse reinforcement yield strength (330-790 MPa), concrete compressive strength (85-116 MPa) and axial load ratio (0.25-0.63) as the variables of the study. The volumetric amount of transverse reinforcement ratio was maintained constant at 1.6%. It was observed that axial loads larger than one half of the balance load had a very adverse effect on the limiting drift response of the specimens; however, the opposite effect was observed for smaller axial loads. The study showed that limiting drift increased with the increasing yield strength of the transverse reinforcement at high axial load ratios, whereas no significant improvement was observed at low axial load ratios, which was attributed to the presence of the excess amount of transverse reinforcement in the specimens.

Sugano et al. [21] conducted tests on columns with concrete compressive strengths of 39 MPa, 59 MPa and 78 MPa and transverse reinforcement yield strengths of 834 and 1370 MPa to develop design guidelines for HSRC columns with high-strength transverse reinforcement. The authors concluded that high-strength concrete with high-strength confinement reinforcement can result in improvement of both strength and ductility of RC members in high-rise buildings.
Hibi et al. [22] evaluated the impact of transverse reinforcement and axial load on the limiting drift of HSRC columns. The control parameters included confinement reinforcement ratio (0.53-0.89%), axial load ratio (0.3 and 0.45) and yield strength of lateral reinforcement (800-1250 MPa). Drift ratios exceeding 4% were obtained for specimens having an axial load ratio of 0.3. However, specimens with higher axial load ratios failed in shear with a drift ratio of 3.5%. Hibi concluded that a large amount of transverse reinforcement is needed in order to obtain ductile behaviour in HSRC columns subjected to loads greater than the balanced load.

Azizinamini et al. [23] assessed the ductility and flexural capacity of square HSRC columns and concluded that columns with axial load ratio below 0.2 and designed according to the specifications of ACI 318-89 exhibited sufficient ductility, whereas the provisions of ACI 318-89 overestimated the flexural capacity and should be revised accordingly.

Thomsen and Wallace [24] tested 12-quarter scale HSRC columns under simulated seismic loading. Spacing and configuration of the transverse reinforcement, yield strength of the reinforcement and axial load ratio were the primary variables of this study. It was noted that high-strength transverse reinforcement provides more effective confinement than corresponding normal-strength transverse reinforcement. The study also observed that there was no decrease in the flexural strength of the tested columns up to a lateral drift of 2.0%.

Sheikh et al. [25] reported that the increase in the amount of the lateral confinement results in an almost proportional increase in the ductility and energy dissipation of HSRC columns when the applied axial load is measured as a fraction of ultimate axial load capacity rather than as $P/f'_c A_g$. It was also found that for the same amount of confinement reinforcement, NSRC columns displayed better ductility than HSRC columns tested at similar axial load ratios.

Galeota et al. [26] investigated the seismic behaviour of HSRC columns under cyclic flexure. Axial load ratio and the amount of longitudinal and transverse reinforcement were the key variables in this study. It was found that an increase in volumetric
transverse reinforcement ratio from 1.22 to 3.66% significantly improved the strength, ductility and energy dissipation performance of the columns. Similarly, it was noticed that an increase in axial load ratio from 0.2 to 0.3 had little impact on ductility, strength and stiffness degradation of the column.

Bayrak and Sheikh [27] conducted an experimental study to evaluate the ductility and energy absorption of HSRC columns. The authors concluded that just like NSRC columns, an increase in the amount of confinement and reduction in the axial load also results in an increase in ductility and energy absorption of HSRC columns. It was reported that under high axial loads HSRC columns can be made to behave in a ductile manner if sufficient confinement is provided in an efficient configuration.

Bayrak and Sheikh [28] compared the seismic performance of HSRC and ultra-high-strength RC (UHSRC) columns with NSRC columns and made recommendations for the ductile design of HSRC and UHSRC columns. The authors concluded that a column with 102 MPa concrete compressive strength behaved in a ductile manner when it was provided with 70% more confinement reinforcement than the ACI code specifications (ACI 318-95). It was recommended to include axial load ratio in the design expressions for confinement reinforcement because with the increase in axial load ratio, the displacement capacity of the column reduced significantly and strength and stiffness degradation of the column were accelerated.

Xiao at al. [29] assessed the seismic performance of six scaled HSRC columns using concrete strength (76-86 MPa), axial load ratio (0.1-0.2), longitudinal steel ratio (2.46-3.53%) and lateral reinforcement ratio (1.63-3.67%) as the variable parameters of the study under a shear dominant loading condition with reversed bending along the column height. The research concluded that for axial load ratio below 0.2 and longitudinal reinforcement ratio of 2.46-3.53%, HSRC columns can have sufficient ductility if transverse reinforcement is designed following provisions of ACI 318. However, HSRC columns having 50% of the transverse reinforcement required by ACI 318, had a limited ductility, particularly for high axial loads. Moreover, shear failure is expected for such HSRC columns.
Legeron and Paultre [30] evaluated the behaviour of six large scale HSRC columns under cyclic flexure and CAL. The variables included axial load ratio (0.15, 0.25 and 0.4) and the amount transverse reinforcement (1.96-4.26%). The study concluded that axial load level has a beneficial effect on the moment-resisting capacity of the HSRC columns while it has a negative impact on the inelastic drift behaviour of the column. For a transverse reinforcement of 4.26%, the displacement ductility dropped from 8.8 to 5.2% with the increase in axial load ratio from 14 to 39%. Similarly, for a transverse reinforcement of 1.96%, the displacement ductility dropped from 4.4 to 1.6% with the increase in axial load ratio from 14 to 39% respectively. It was observed that in order to have sufficient ductility for HSRC columns, approximately 50% of ACI code specified confinement reinforcement must be provided for axial load ratio less than or equal to 0.15. However, at higher axial load ratio, ACI code specified confinement was not enough to ensure ductile behaviour. Moreover, it was concluded that for higher axial load levels confinement reinforcement must be related to axial load level and should be calculated accordingly.

Paultre et al. [31] investigated the effect of concrete strength and transverse reinforcement yield strength on the ductility of HSRC columns. Eight large scale HSRC columns were tested for this purpose. Concrete strength was varied from 80 MPa to 120 MPa while transverse reinforcement was varied from 1.96% to 4.26%. Axial load ratio was kept constant at 0.4 and 0.52, respectively. Longitudinal reinforcement was also kept constant at 2.15%. It was observed that at constant volumetric transverse reinforcement and CAL, concrete strength significantly influences the flexure behaviour of columns. It was seen that displacement ductility dropped from 10.1 to 5.2 to 4.7% when concrete strength was increased from 78.7 to 98.2 to 109.2 MPa, respectively. It was concluded that the amount of transverse steel can be decreased provided yield strength of the confinement reinforcement is increased. However, it was noted that the high yield strength of the transverse reinforcement is not always effective, especially when columns are poorly confined. The authors also noted that the requirements of ACI and New Zealand codes need to be modified to use high-strength transverse reinforcement.
Matamoros et al. [32] performed experiments to investigate the behaviour of HSRC columns subjected to shear reversals into the nonlinear range of response. Loading history, axial load and concrete compressive strength (35 MPa and 70 MPa) were the main variables in this study while transverse reinforcement ratio was kept constant at 1% and shear span to depth ratio was 3.4. It was observed that the increase in axial load had an adverse effect on the limiting drift of columns. The study concluded that ACI 318-99 specifications resulted in sufficient ductility of HSRC columns.

Bayrak and Sheikh [33] evaluated the performance of high-strength transverse reinforcement in effectively confining HSRC columns. The study concluded that the use of high-strength steel can serve as an effective alternative to reduce the congestion problem in HSRC columns in the regions of high seismicity. It was noted that the length of the plastic hinge region is not affected by the strength of transverse reinforcement. It was also concluded that the use of high-strength transverse steel with a short yield plateau is preferable over high-strength steel with a long yield plateau.

Hwang and Yun [34] experimentally tested eight one third scale HSRC columns under constant axial load ratio of 0.3 and unidirectional cyclic loading. The variables of the study included volumetric transverse reinforcement ratio (1.58%, 2.25%), transverse reinforcement configuration and yield strength. It was reported that the flexural capacity of the specimens exceeded the capacity calculated by the provisions of ACI 318-02. Furthermore, the specimens provided with 42% higher transverse reinforcement than the requirements of the code exhibited ductile behaviour. The authors recommended to use a transverse reinforcement yield strength of 549 MPa or lower for specimens with an axial load ratio of 0.3 for preventing buckling of longitudinal bars. It was also reported that specimens with rectangular hoops result in more ductile behaviour compared to other configurations.

Woods et al. [35] studied the effect of the amount of transverse reinforcement and spacing on the ductility of HSRC columns. The study found out that there is a banded relationship between the transverse reinforcement amount and spacing and the ductility index of the columns, which implies that for a given tie spacing, a number of
transverse reinforcement values can influence the ductility of the column positively. Similarly, for a given content of transverse reinforcement, a number of values of reinforcement spacing can affect the ductility of the HSRC columns in a positive manner.

Ho et al [36] studied the effectiveness of adding confinement for improvement in ductility of HSRC columns. The authors suggested that for HSRC columns more confinement is needed to maintain flexural ductility at a minimum level of structural safety. From the results of the study, it was concluded that although the addition of transverse reinforcement is generally effective in improving the flexural ductility, its effectiveness rapidly reduces with the increase in concrete compressive strength and axial load level. Thus for heavily loaded HSRC columns to have same flexural ductility as that of their counterpart NSRC columns, exceptionally large amount of confining reinforcement may be needed. The study also developed formulas for direct evaluation of curvature ductility factors of the confined HSRC columns.

Barrera et al. [37] examined the ductility of forty slender RC columns under monotonic flexure and CAL. Variable parameters included concrete strength (30, 60, 90 MPa), shear span ratio (7.5, 10.5, 15), axial load ratio (0 to 0.45), transverse reinforcement (0.8-3.1%) and longitudinal reinforcement ratio (1.4-3.2%). The study concluded that the ductility ratio in displacements decreases with the longitudinal reinforcement ratio and the increasing strength of concrete, while it increases with the confinement level. The study also proposed an equation for determination of ultimate displacement taking into account the total elastic displacement and second-order effects.

Ho [38] proposed expressions for designing moderately ductile as opposed to fully ductile HSRC columns for construction in regions of low to moderate seismicity. The author tested the columns designed with the proposed confinement reinforcement and found out that they exhibited sufficient drift capacity under medium axial loads. On the other hand, it was noted that HSRC columns designed with lateral reinforcement based on ultimate shear demand failed at a very low drift.

Hwang et al. [39] conducted experimental testing on full-scale HSRC columns under quasi-static cyclic loading. The study concluded that with adequate confinement, HSRC
columns can sustain satisfactory drift level of 3.0%. It was also reported that the drift capacity of HSRC columns decreased with the increase of concrete compressive strength primarily due to the brittle nature of the high-strength concrete.

Bechtoula et al. [40] investigated the performance of high strength and UHSRC columns under severe cyclic loading. Twelve cantilever columns of varying sizes and compressive strengths (80, 130 and 180 MPa) and designed according to Japanese code were tested. The study found out that with the increase in the compressive strength of concrete, the drift corresponding to peak load and the ductility of the specimen decreased. It was observed that 180 MPa concrete column specimen showed a very brittle spalling of concrete, followed by a significant decrease in strength. An equation predicting the moment drift envelop curves based on geometrical and material characteristics of the columns was also suggested as part of the study.

Ou et al. [41] investigated the shear behaviour of RC columns with high-strength steel and concrete tested at axial load ratios of 0.1 and 0.2. All the columns were designed to fail in shear mode. It was noted that the increase in axial load increased the shear strength of the column and reduced its ductility. The study observed that the amount of transverse reinforcement did not significantly affect the concrete shear strength. However, an increase in transverse reinforcement delayed the shear failure to a higher drift. The study suggested modifications to the ACI shear-strength equations for HSRC columns.

Ou and Kurniawan [42] tested 8 shear critical HSRC columns at axial load ratios of 0.3 and 0.4 and compared the results with the previous study with an axial load ratio of 0.1 and 0.2. It was reported that with the increase in axial load ratio from 0.1 to 0.4, the cracks changed from flexure-shear cracks to web-shear cracks. The study recommended taking into account the axial load ratio for calculation of the minimum amount of shear reinforcement. It was also found that the shear strength of the concrete generally increased with the increase in axial load ratio but the rate of increase decreased at high axial load ratios.
Jin et al. [43] conducted an experimental study to investigate the seismic behaviour of seven shear-critical HSRC and one NSRC column. Aspect ratio, concrete compressive strength, axial load ratio and transverse reinforcement ratio were the key variables in this study. It was observed that the overall behaviour of HSRC columns was more brittle than NSRC columns under the same conditions. HSRC columns experienced greater strength and stiffness degradation beyond the peak load and showed low ductility and energy dissipation. Moreover, severe concrete spalling with sudden failure was observed for HSRC columns. It was concluded that high axial load ratio has a very damaging effect on the ductility of HSRC columns.

The literature statistics of flexure critical HSRC columns tested to collapse under unidirectional cyclic actions and CAL are shown in Figure 2.4. It can be seen in the figure that most of the previous studies were conducted on flexure critical HSRC columns with transverse reinforcement ratio > 0.5%, which mostly results in moderately to fully ductile HSRC columns. On the other hand, limited ductile HSRC columns designed according to Australian concrete standard can have transverse reinforcement ratio below 0.5% as shown in Table 2.2 of section 2.4. The lack of experimental testing on HSRC columns with transverse reinforcement ratio < 0.5% is because most of the previous experimental testing was focussed on understanding the behaviour in high seismic regions where moderately to fully ductile columns with high transverse reinforcement ratios are constructed, whereas limited ductile columns with low transverse reinforcement ratios are the standard practice in low to moderate seismic regions, such as Australia. In view of this, the collapse behaviour of limited ductile HSRC columns needs to be evaluated given that the amount of transverse reinforcement in such columns is small compared to the specimens tested in the literature.
Figure 2.4. Literature statistics of the transverse reinforcement ratio (by area) in flexure critical HSRC columns tested under unidirectional lateral actions.

2.6 Multidirectional Earthquake Actions

Earthquake ground motions comprise of three orthogonal components that act simultaneously and impose varying triaxial forces on a building column during an earthquake. The triaxial forces consist of bidirectional lateral actions in the two horizontal axes and VAL in the vertical axis of the column. The variation in axial load on RC columns can occur due to framing action in the building that resists the overturning moments from the lateral earthquake forces induced by the two concurrent and orthogonal horizontal components of the ground shaking. This type of variation is classified as *synchronous axial load variation* where any change in axial load is synchronous with and a function of the lateral loads, and therefore the extreme values (maximum or minimum) of both axial and lateral load occur at the same time. The second type of axial load variation is classified as *nonsynchronous axial load variation* where the variation of axial load and lateral loads are uncoupled and vary independently of each other. This kind of variation may arise due to the vertical component of the ground motion, which acts independently of its horizontal counterparts and usually has a relatively higher frequency content that results in different frequencies of axial load and lateral load variation. It is noted that the two types of axial load variation have also been referred to as proportional/in phase and
non-proportional/out of phase axial load variation in Abrams [44], Saadeghvaziri [45] and ElMandooh and Ghobarah [46].

The majority of the studies under multidirectional earthquake actions are conducted on the NSRC columns with only one study on HSRC columns. This section provides a summary of the main findings about the behaviour of RC columns subjected to bidirectional lateral actions under constant and VAL.

2.6.1 RC Columns under Bidirectional Lateral Actions and Constant Axial Load

The research on the bidirectional behaviour of NSRC columns commenced from the 1980s. The studies primarily investigated the effect of the type of loading path, transverse reinforcement ratio and axial load ratio on the ductility and energy dissipation of RC columns. Generally, the studies reported a significant reduction in the displacement capacity and an increase in the strength and stiffness degradation and energy dissipation of the column under bidirectional loading as opposed to unidirectional loading. The following summarizes the existing research about the influence of biaxial cyclic loading on the seismic behaviour of NSRC and HSRC columns:

Otani et al. [47] experimentally tested 8 RC columns under bidirectional cyclic loading. The amount of longitudinal and transverse reinforcement and concrete compressive strength were among the variables of the study. It was reported that loading in one direction reduced the stiffness in the other direction. Furthermore yielding under biaxial loading occurred at a lower load than the uniaxial loading due to the biaxial interaction.

Low and Moehle [48] tested three one quarter scale RC columns under uniaxial and biaxial lateral actions with CAL. It was reported that visual damage was more extensive in specimens tested under biaxial loading. Furthermore, the strength and stiffness were also smaller under biaxial loading compared to uniaxial loading.

Bousias et al. [49] investigated the seismic behaviour of flexure-dominated RC columns under 11 different biaxial load paths. The results indicated a strong coupling behaviour between the two directions of bending, which consequently resulted in increased
energy dissipation along with a significant reduction in strength and stiffness as compared to the uniaxial cyclic loading case.

Qiu et al. [50] noted that the plastic deformation ability of the RC column decreases significantly under biaxial cyclic loading. It was reported that the accumulative hysteretic energy dissipation is larger under biaxial loading as opposed to uniaxial loading.

Tsuno and Park [51] experimentally tested bridge piers under bidirectional lateral loading and concluded that the plastic hinge zone length is not affected by biaxial cyclic loading. It was also reported that the ultimate drift of the column under bidirectional cyclic loading is smaller than the unidirectional cyclic loading.

Bechtoula et al. [52] conducted experiments on RC columns with axial load ratio and lateral loading pattern as the variable parameters. The study found a significant impact of high axial load and bidirectional cyclic loading on the damage process of the column. No significant impact on the peak lateral load-carrying capacity of the column was observed under bidirectional cyclic loads.

Kawashima et al. [53] studied the biaxial excitation effects on the seismic behaviour of RC bridge columns using both cyclic and hybrid loading tests. Square, circular, diagonal and elliptical displacement paths were used in cyclic loading tests. Ground motions of Kobe and Northridge were used for hybrid loading tests. Under biaxial cyclic loading, the strength of columns was 14-23% and 15-35% smaller in NS and EW directions as compared to the strength under biaxial hybrid loading. Similarly, there was a significant deterioration in lateral resisting force under bidirectional cyclic loading. The failure of columns under cyclic loading was also more extensive than hybrid loading.

Chang [54] tested two RC bridge columns under pseudo-dynamic loads and one identical column under biaxial cyclic loading. The study noted that during unloading the biaxial hysteresis loops undergo greater stiffness degradation and pinching which implies that the damaged caused in one direction weakens the earthquake resistance of the other direction. It was also observed that the distinct characteristics of biaxial hysteretic loops are round corners and negative stiffness.
Osorio et al. [55] conducted uniaxial and biaxial cyclic testing on two identical columns with CAL. The study found out that the biaxial loading produces larger transversal strains due to the interaction of biaxial loads for the same load intensity and lower crack angles. These larger strains consequently reduce the ultimate axial strain of the concrete and hence may result in premature failure.

Rodrigues et al. [11] conducted an experimental investigation on seventeen RC columns under biaxial cyclic loading with CAL. The study observed that under biaxial cyclic loading column strength reduced approximately 8% in the strong direction and 20% in the weak direction compared to uniaxial cyclic loading. Similarly, biaxial loading reduced the ultimate ductility of columns by 50% to 75% in the weak direction and around 35% in the strong direction as opposed to uniaxial loading. It was also noted that the plastic hinge length and initial column stiffness in both directions are not affected by biaxial loading.

Pham and Li [56] investigated the seismic behaviour of seven lightly reinforced RC columns under reversed double curvature bending. Axial load ratio and direction of loading were the variables considered in this study. It was noted that the loading direction has a significant effect on axial load failure drift and the maximum energy dissipation of RC columns. However, drift ratio at maximum shear force and initial stiffness of the column was not significantly affected by the change in the loading direction.

Di Ludovico et al. [57] conducted experiments on four non-conforming RC columns under CAL and biaxial bending. The columns had plain reinforcing bars and were designed according to obsolete building codes. The columns were subjected to biaxial loading at an angle of 30° and 45°, respectively. It was found that in nonconforming columns the degradation in strength and rotational deformation capacity can be pronounced due to inadequate tie spacing. The results also showed that biaxial cyclic loading affects the rotational capacity of RC columns more than the strength capacity.

Del Zoppo et al. [58] presented the results of 10 RC columns tested under CAL and unidirectional and bidirectional lateral actions. The specimens were provided with
nonconforming transverse reinforcement. The study reported a significant effect of biaxial loading on the initial stiffness of RC columns as opposed to uniaxial loading. The angle of inclination of the loading protocol also notably affected the strength and ultimate displacement capacity of the column specimens.

2.6.2 RC columns under Unidirectional or Bidirectional Lateral Actions and Variable Axial Load

A survey of the literature shows that the majority of the experimental testing on RC columns has been conducted under synchronous axial load variation. The findings of the studies suggest that variation of axial load ratio has a significant effect on the flexural strength, symmetry of hysteretic behaviour and ductility of the column. As such, the hysteretic behaviour was found to be very unsymmetrical and ductility of the column reduced under large variations of axial load. The following provides a summary of the studies conducted on RC columns with axial load variation and bidirectional lateral actions.

Abrams [44] performed ten tests on RC columns under cyclic loading with VAL. Two types of axial load variation protocols were employed. In the first protocol, the axial load was varied as a function of bending moment and in the second one, the axial load was varied as a function of the lateral displacements. It was found that the shape of the hysteretic loop is significantly influenced by axial load variation. The study noted that flexure strength is independent of the sequence of axial force. It was also observed that flexural stiffness increased with the increase in axial compression.

Low and Moehle [48] tested two RC columns under biaxial lateral actions with VAL and compared the results with the specimens tested under biaxial lateral actions with CAL. It was reported that the state of damage generally worsened under axial load variations, although the maximum applied axial load was less than the balanced load.

Li et al. [59] tested five-quarter scale RC columns under varying axial load and bidirectional lateral load reversals. The axial load was varied proportional to the lateral forces on the column. A non-symmetrical hysteretic behaviour was observed under VAL. The study also noted a complicated interaction between biaxial lateral loads and
axial load variation. Moreover, the deformation capacity of the RC column was reported to be significantly reduced under large variations of axial loads.

Bousias et al. [49] tested one specimen under varying axial load and bidirectional lateral actions. The variation in axial load was proportional to the lateral force in the Y direction of the specimen. It was reported that columns experienced highly unsymmetrical hysteretic behaviour due to the VAL. A strong coupling between the three loading directions was also observed.

Esmaeily and Xiao [60] conducted an experimental study on RC columns with unidirectional lateral load and CAL and VAL. Synchronous and nonsynchronous axial load variations were employed in the test program. It was reported that under VAL, the flexural strength and displacement capacity of the column were different than under constant axial loads. The authors concluded that the pattern of VAL (synchronous or nonsynchronous) have a significant effect on the response of the columns and thus should be taken into consideration.

Rodrigues et al. [61] tested six RC columns under uniaxial and biaxial cyclic loading with VAL. The study found out that biaxial cyclic loading combined with variation in axial load can lead to a reduction of 60% of the drift where each damage state occurs as compared to uniaxial tests under variable load. It was also observed that biaxial cyclic loading under VAL results in a more abrupt decay of strength, with 30-40% more reduction of maximum strength in the weak direction and 12% more reduction of maximum strength in the strong direction as compared to uniaxial loading with CAL. Finally, it was found that axial load variation has a more pronounced effect in the reduction of columns ductility. Maximum column ductility was found to be reduced by 20% and 50% more in the weak and strong directions, respectively, as compared to uniaxial loading with CAL.

Hashemi et al. [62] conducted experimental testing on a limited ductile RC column under quasi-static loading protocol which consisted of biaxial cyclic loading with CAL. An identical specimen was then tested under hybrid simulation with more realistic boundary effects which included the variation of axial load and ratcheting of structural
lateral deformations. The study noted that variation of axial load results in pronounced in-cycle force degradation and a significant reduction of drift capacity. It was also observed that simulation of ratcheting behaviour allows for better estimation of force degradation between cycles than the symmetric cyclic loads in the quasi-static test.

Xu et al. [63] tested five RC columns under variable axial forces and rotations and reported that axial load variation results in asymmetric failure of RC columns and also reduces the lateral strength and ductility of the column. The study recommended accounting for the adverse effects of VAL and rotations in the design of RC columns.

All these studies were on the multidirectional behaviour of NSRC columns. On the other hand, HSRC columns have been tested under bidirectional lateral actions in only one study so far. The study was conducted by Kuramato et al. [64], in which three HSRC columns were tested under bidirectional cyclic loading with VAL and one specimen was tested under bidirectional cyclic loading and CAL. The loading path used was referred to as “four leaves” type. In the loading protocol, the column was first displaced in a rectangular path in the first and third quadrants and then in second and fourth quadrants and the axial load was varied as a function of the lateral forces in the two axes of the column. It was concluded that the ductility of the column is reduced when subjected to varying triaxial forces. It was also reported that the rate of compressive axial deformation was higher when the initial axial load was higher.

Figure 2.5 shows the literature statistics of NSRC and HSRC columns tested under bidirectional lateral actions with constant and VAL. The literature statistics are based on the review paper by Rodrigues et al. [65] and a further survey of the literature conducted by the author himself. The statistics show that only four HSRC columns have been tested under bidirectional lateral loading so far as opposed to around 92 NSRC columns that were tested under bidirectional lateral loading. Due to the brittle nature and low ductility of high-strength concrete, the understanding of the bidirectional behaviour of NSRC columns cannot be extended to HSRC columns, and the potential effects of bidirectional lateral loading are expected to be more severe for HSRC columns as opposed to NSRC columns. Therefore, further experimental testing is needed to
better understand the collapse performance of HSRC columns under bidirectional lateral loading.

![Specimens Tested under Bidirectional Lateral Loading](image)

**Figure 2.5.** Literature statistics of the NSRC and HSRC columns tested under bidirectional lateral actions with CAL and VAL

### 2.7 Bidirectional Loading Protocols

Rodrigues et al. [65] summarized 7 typical bidirectional loading protocols used by various researchers in the quasi-static testing of RC columns. The typical bidirectional loading patterns included cruciform, diagonal cruciform, circular, rhombus, expanding square, square in each quadrant and elliptical loading path. ACI 374.2R-13 [66] has also proposed a hexagonal orbital pattern for bidirectional testing of RC columns. Figure 2.6 presents a summary of the typical bidirectional loading paths used in previous studies.

As indicated by Figure 2.6, there are numerous bidirectional loading protocols, but there is no guidance as to which of these protocols is a more realistic representation of the actual loading imposed on an RC column during an earthquake. Further to this, there is no study that quantitatively investigates the displacement pattern of RC columns during earthquakes. On the other hand, Ishida et al. [67] and Elkady and Lignos [68] proposed bidirectional loading protocols comprising of elliptical loops for rectangular hollow section and wide flange steel columns by processing the bidirectional drift response history of columns in multi-story steel buildings. Other researchers such as Clark et al. [69], Richard and Uang [70] and Krawinkler et al. [71] developed unidirectional loading protocols for beam-column connections, short links
in eccentrically braced frames and wood frame structures, respectively, using the Rainflow method [72]. It is noted that in the Rainflow method, the drift response history is processed in terms of number, range/amplitude and sequence of occurrence of drift cycles. However, this method is not applicable for processing the bidirectional drift response history of columns. Hence, there is a need to conduct a robust study for developing realistic bidirectional loading protocols.

Figure 2.6. Typical bidirectional loading paths used in quasi-static testing of RC columns (Rodrigues et al. [65])

2.8 Review of Predictive Models for Post-Peak Drift Capacity of RC Columns

In displacement-based seismic design, an estimate of the post-peak drift capacity of an RC column is of prime importance. The post-peak drift capacity of the column can be defined by two points, namely: lateral load failure and axial load failure drift on the force-displacement hysteresis of an RC column. Lateral load failure drift is taken as the conventional failure point in regions of higher seismicity and is defined as the drift capacity corresponding to 20% degradation in the lateral strength of the column. On the other hand, axial load failure drift is defined as the drift corresponding to the loss in the axial load carrying capacity of the column.
A substantial amount of work has been done to develop post-peak drift capacity models of lightly to moderately reinforced NSRC columns (Elwood and Moehle [73]; Ghannoum and Moehle [74]; Ousalem et al. [75]; Sasani [76]; Ho [77]; Tran and Li [78]; Wibowo et al. [12]; Wilson et al. [79]; Zhu et al. [4]). These models ignore the effect of concrete strength on the drift performance of reinforced-concrete (RC) columns. Therefore, it is important to evaluate the suitability of these models for predicting the drift capacity of HSRC column. This section presents a review of the existing lateral load and axial load failure drift models, based on a comparative study, conducted using a comprehensive database of experimental results from the literature.

2.8.1 Experimental Database

The experimental database considered in this comparative study comprises of 190 columns (79 HSRC and 111 NSRC columns) from 44 studies. Columns with a concrete compressive strength \( f'_c > 50 \text{ MPa} \) are considered as HSRC columns in this study. The database covers a very broad range of parameters and includes columns with flexure, flexure-shear and shear mode of failure. The database of the columns is presented in Appendix A of the thesis. The properties of the columns considered in this study are in the ranges given below. Also, the histogram plots showing the characteristics of the specimens in the database are shown in Figure 2.7.

- Aspect ratio: \( 1.0 \leq L/h \leq 10.5 \);
- Axial load ratio: \( 0.027 \leq n \leq 0.6 \);
- Concrete compressive strength: \( 12.1 \leq f'_c \leq 104 \text{ MPa} \);
- Longitudinal reinforcement ratio: \( 0.56\% \leq \rho_v \leq 4.7\% \);
- Transverse reinforcement ratio by area: \( 0.07 \leq \rho_h \leq 1.0\% \);
- Transverse reinforcement yield strength: \( 230 \leq f_{yh} \leq 1360 \text{ MPa} \);
Figure 2.7. Characteristics of the specimens in the experimental database
2.8.2 Lateral Load Failure Drift Models

Most of the existing lateral load failure drift models were developed for NSRC columns and are empirical in nature. This section discusses five such models and presents the results of a comparative study conducted to evaluate the performance of these models for both NSRC and HSRC columns. It should be noted that three of these models (Elwood and Moehle [73], Zhu et al. [4] and Ghannoum et al. [74]) were primarily intended for NSRC columns but for the sake of comparative study, these models have been extrapolated to give drift predictions for HSRC columns as well. The parameter ranges for which these models are applicable are given in Table 2.3.

Elwood and Moehle (2003)

Elwood and Moehle [73] proposed an empirical model to predict the lateral load failure drift capacity of shear-critical NSRC columns. The model was developed based on the observations from the experimental results of 50 columns. The key observation in the development of this model was that for a given transverse reinforcement ratio, the maximum shear stress degrades with increasing drifts at lateral load failure. The following empirical expression was proposed to estimate lateral load failure drift:

$$\delta_f = \frac{3}{100} + 4\rho_h - \frac{1}{41.5} \frac{v}{\sqrt{f'_c}} - \frac{1}{40} \frac{P}{A_g f'_c} \geq 0.01 \text{ (MPa units)}$$

(2.1)

The performance of this model is evaluated by comparing drift predictions with the experimental drifts of 82 shear-critical RC columns (58 NSRC and 24 HSRC) in the experimental database of Appendix A. The results of the comparison in Figure 2.8 (a) show that the model overestimates the drift capacity of most NSRC and HSRC columns of the current database. The coefficient of variation of this model for NSRC columns is 0.49.

Zhu et al. (2007)

Zhu et al. [4] developed empirical lateral load failure drift models for flexural-critical and shear-critical columns using a database of 125 columns from the literature. The authors found that transverse reinforcement ratio, aspect ratio, axial load ratio and
hoop spacing to depth ratio were the key parameters that influence the drift capacity of shear-critical RC columns at lateral load failure. However, for flexural-critical columns, longitudinal reinforcement ratio, transverse reinforcement ratio, concrete compressive strength, transverse reinforcement yield strength, axial load ratio and hoop spacing to depth ratio were found to be the influential parameters affecting the drift capacity at lateral load failure.

The following expressions were proposed to estimate the median drift at lateral load failure:

\[
(\delta_{lf})_{\text{shear}} = 2.02 \rho_h - 0.025 \frac{s}{d} + 0.013 \frac{L}{d} - 0.031 \frac{P}{A_g f'_c},
\]

\[
(\delta_{lf})_{\text{flexural}} = 0.049 + 0.716 \rho_v + 0.12 \frac{\rho_h f_{yh}}{f'_c} - 0.042 \frac{s}{d} - 0.07 \frac{P}{A_g f'_c}.
\]

The results of the comparative study including both flexural and shear-critical columns of the current database are presented in Figure 2.8 (b). The results show that while the model gives very reasonable predictions for the majority of NSRC columns, it is not suitable for HSRC columns. The coefficient of variation of this model for NSRC columns is 0.3.

Sasani (2007)

Sasani [76] developed a probabilistic deformation capacity model for predicting the drift at lateral load failure of shear-critical RC columns. The model is based on the notion that the drift capacity of RC column is dependent on the ultimate curvature of the column, which in turn depends on the maximum usable concrete strain, which is significantly affected by transverse reinforcement ratio. Moreover, the researchers argued that ultimate curvature is also dependent on the depth of the compressive zone, which is directly affected by the axial load ratio. The aspect ratio of the column was also included in the model. The following expression was proposed:

\[
\delta_f = \theta_t \rho_s^{0.74} \frac{n^{-0.18}}{h} \frac{L}{h}.
\]
where $\theta_t = 1.0$ for cantilever columns and $\theta_t = 0.85$ for double-ended or double curvature columns. The model does not have any limitation of concrete compressive strength. Hence, it can be used for shear-critical normal as well as HSRC columns. The comparison between model drift predictions and experimental drifts for the shear-critical columns of the current database are shown in Figure 2.8 (c). The model tends to give very good drift predictions for the majority of NSRC columns but overestimates the drift capacity of most HSRC columns. The coefficient of variation of the model for shear-critical NSRC and HSRC columns is 0.37.

Ghannoum and Moehle (2008)

Ghannoum et al. [74] proposed an empirical model to predict the drift at the initiation of shear failure of flexure-shear critical columns. The model was developed by performing regression analysis on a dataset of 56 column tests from the literature. The model related drift at shear failure with hoop spacing, axial load ratio and shear stress. The following expression was proposed to predict the lateral load failure drift at the initiation of the shear failure:

$$\delta_{lf} = 0.044 - 0.017 \frac{s}{d} - 0.021 \frac{P}{A_g f'_c} - 0.024 \frac{V}{\sqrt{f'_c}} \geq 0.009 \text{ (MPa units)}$$  \hspace{1cm} (2.5)

The performance of this model is evaluated using shear-critical NSRC and shear-critical HSRC columns of the experimental database. The results of the comparison of drift predictions and experimental results are shown in Figure 2.8 (d). It can be seen from the results that the model tends to overestimate the lateral load failure drift of the majority of the columns in the current database. As such, the model has a very high coefficient of variation of 0.62 for shear-critical NSRC columns.
Figure 2.8. Comparison between experimental and predicted drifts of existing lateral load failure drift models
Ho and Pam (2010)

Ho and Pam [77] model for predicting lateral load failure drift capacity of RC columns is based on the idealised curvature profile of the column, which depends on maximum elastic curvature, ultimate curvature and plastic hinge length of the column. Ho and Pam related the plastic hinge length and ultimate curvature of HSRC columns to structural parameters (axial load ratio, longitudinal and transverse reinforcement ratio and concrete compressive strength) using regression analysis. The following expressions were proposed for the calculation of plastic hinge length, ultimate curvature and lateral load failure drift of HSRC columns.

\[
\frac{l_p}{h} = 16.5 \left( \frac{P}{A_g f'_{c'}} \right)^{0.5} \left( \frac{f'_{c'}}{f_{y_h}} \right)^{1.5} \left( \frac{\rho_v}{\rho_s} \right)^{0.5} + 0.15 \quad (2.6)
\]

\[
\varnothing_u = 0.86 \left( \frac{P}{A_g f'_{c'}} \right)^{2} \left( \frac{f'_{c'}}{f_{y_h}} \right)^{2} \left( \frac{\rho_v}{\rho_s} \right) + 0.02 \quad (2.7)
\]

\[
\delta_f = \frac{1}{3} \varnothing_e H^2 + \left( \varnothing_u - \varnothing_e \right) \left( H - \frac{l_p}{h} \right) l_p \quad (2.8)
\]

Figure 2.8 (e) shows the results of the comparison of the model drift predictions with experimental drifts of the database. It can be seen that there is a lot of scatter in the results with the model giving reasonable approximations for only HSRC columns at low axial load ratio and NSRC columns at high axial load ratio. The results suggest that the model is not suitable for HSRC columns with high axial load ratio and NSRC columns with low axial load ratio.
Table 2.3. Parameter ranges for applicability of existing post-peak drift capacity models

<table>
<thead>
<tr>
<th>Models</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Lateral Load Failure</strong></td>
</tr>
<tr>
<td>Elwood and Moehle [73]</td>
<td>$13.8 \leq f'_c \leq 45$ MPa, $0.07% \leq \rho_h \leq 0.65%$ and $0.07 \leq n \leq 0.6$</td>
</tr>
<tr>
<td>Zhu et al. [4]</td>
<td>$16 \leq f'<em>c \leq 56.2$ MPa, $0.06% \leq \rho_h \leq 2.2%$, $0.0 \leq n \leq 0.8$, $249 \leq f</em>{yh} \leq 616$ MPa, $1.2% \leq \rho_v \leq 3.3%$, $1.2 \leq L/d \leq 7.0$ and $0.1 \leq s/d \leq 1.2$</td>
</tr>
<tr>
<td>Sasani [76]</td>
<td>$f'_c \leq 50$ MPa, $0.16% \leq \rho_s \leq 1.7%$, $n \geq 0.14$ and $1.0 \leq L/h \leq 4.0$</td>
</tr>
<tr>
<td>Ghannoun and Moehle [74]</td>
<td>$13 &lt; f'_c &lt; 45$ MPa, $0.0 &lt; n &lt; 0.6$ and $0.2 &lt; s/d &lt; 1.2$</td>
</tr>
<tr>
<td>Ho and Pam [77]</td>
<td>$f'_c &gt; 50$ MPa</td>
</tr>
<tr>
<td></td>
<td><strong>Axial Load Failure</strong></td>
</tr>
<tr>
<td>Elwood and Moehle [73]</td>
<td>$20.8 &lt; f'_c &lt; 33$ MPa</td>
</tr>
<tr>
<td>Ousalem et al. [75]</td>
<td>$13.5 &lt; f'<em>c &lt; 33.1$ MPa, $0.07% \leq \rho_h \leq 0.38%$, $0.073 \leq n \leq 0.4$ and $384 \leq f</em>{yh} \leq 587$ MPa</td>
</tr>
<tr>
<td>Zhu et al. (2007)</td>
<td>$13.5 &lt; f'<em>c &lt; 33.1$ MPa and $355 \leq f</em>{yh} \leq 587$ MPa</td>
</tr>
<tr>
<td>Tran and Li [78]</td>
<td>$13.0 &lt; f'<em>c &lt; 33.1$ MPa, $1.7% \leq \rho_v \leq 3.1%$, $355 \leq f</em>{yh} \leq 469$ MPa and $335 \leq f_{yl} \leq 547$ MPa</td>
</tr>
<tr>
<td>Wibowo et al. [12]</td>
<td>$13.5 &lt; f'_c &lt; 45.0$ MPa, $\rho_v \leq 2.0%$, $\rho_h \leq 0.4%$ and $0.1 \leq n \leq n_b$</td>
</tr>
<tr>
<td>ASCE 41-17 [121]</td>
<td>$n \leq 0.7$ and $0.05% \leq \rho_h \leq 0.75%$ (if ties are not adequately anchored) or $0.05% \leq \rho_h \leq 1.75%$ (if ties are properly anchored)</td>
</tr>
<tr>
<td>Eurocode 8 [128]</td>
<td>No range is specified</td>
</tr>
</tbody>
</table>
2.8.3 Axial Load Failure Drift Models

The axial load failure drift models predict the drift capacity corresponding to the loss of axial load carrying capacity of the column. Most of the existing axial load failure drift models were primarily developed for NSRC columns. Some of these models are empirical in nature while others are semi-empirical. This section discusses five such models and presents the results of a comparative study undertaken to evaluate the performance of these models for both NSRC and HSRC columns. It should be noted that the models were primarily intended for NSRC columns but the models have been extrapolated to give drift predictions for HSRC columns for the sake of comparison. The parameter ranges for which these models are applicable are given in Table 2.3.

*Elwood and Moehle (2003)*

Elwood and Moehle [73] proposed a semi-empirical model for predicting drift at axial load failure of shear-critical RC columns based on the shear friction model. The model relates the effective coefficient of friction with the drift ratio at axial load failure by an empirical equation that was derived by regression analysis of 12 full-scale pseudo-static column test results. The model uses the following expression to determine the drift at axial load failure:

\[
\delta_{af} = \frac{4}{100} \left( \frac{1 + (\tan \theta)^2}{\tan \theta + P \left( \frac{s}{A_w f_{yh} d_c \tan \theta} \right)} \right), \text{ where } \theta = 65^\circ
\]  

(2.9)

The performance of this model is evaluated using 82 columns (47 NSRC and 35 HSRC) in the current database that were tested to the point of collapse. The results of the comparison can be seen in Figure 2.9 (a). The model gives very reasonable drift predictions for the majority of NSRC and HSRC columns but it was observed that as the transverse reinforcement ratio of HSRC columns reduced, the model started overestimating the axial load failure drift of HSRC columns. The coefficient of variation of the model for NSRC columns is 0.45.

*Ousalem et al. (2004)*
Ousalem et al. [75] modified Elwood’s shear friction model to include the dowel action of longitudinal bars in the calculation of axial load failure drift of RC columns. The model employed regression analysis to express the effective coefficient of friction as a power function of axial load failure drift and other column parameters like axial load ratio, concrete compressive strength and transverse steel content. This model was developed using a dataset of 25 shear-critical columns tested to the point of collapse. The model proposed the following expressions:

$$\delta_{af} = \frac{1}{k} \left( \frac{\sqrt{0.03 + 1.33k}}{0.97 - 1.33k} \right)^{0.36}$$

and

$$k = \frac{\rho_h f_{yh}}{n f_c'}$$

where $k \leq 0.4$  \hspace{1cm} (2.10)

Model drift predictions are compared with experimental drifts of columns in the current database in Figure 2.9 (b). It can be seen that there is a lot of scattering in the results and the model does not give good estimations for either NSRC or HSRC columns. The model has a very high coefficient of variation of 1.0 for NSRC columns.

**Eurocode 1998-3 (2005)**

Eurocode 1998-3 [128] proposed expression to determine the total chord rotation or drift at the limit state of near collapse i.e. axial load failure. The expression takes into account both the elastic and inelastic deformation capacity of the column. The drift is related to design parameters, such as axial load ratio, transverse reinforcement ratio, confinement effectiveness, concrete compressive strength, transverse reinforcement yield strength and aspect ratio of the column. The expression is as follows:

$$\delta_{af} = \frac{1}{\gamma_{el}} 0.016 (0.03^n) \left[ \frac{\max(0.01; \omega')}{\min(9; \max(0.01; \omega))} f_c' \right]^{0.225} \left[ \min(9; \frac{L}{h}) \right]^{0.35} 25 \left( a_e \frac{f_{yh}}{f_c'} \right) 1.25^{100 \rho_d}$$

A comparison of the drift capacity predicted by the Eurocode model with the experimental drifts is shown in Figure 2.9 (c). The model does not show a very good correlation as evident by the scatter observed in the predictions for both NSRC and HSRC columns. The coefficient of variation of the model for columns in the database was found to be 0.37.
Zhu et al. (2007)

Zhu et al. [4] also modified Elwood’s shear friction model by expressing axial load failure drift as an exponential function of the effective coefficient of friction. The effective coefficient of friction was determined using classical shear-friction model. In the calculation of the effective coefficient of friction, the axial load carried by the longitudinal reinforcement was neglected. The model was developed using a database of 28 NSRC columns and uses the following expression to determine drift at axial load failure.

\[
(\delta_{af})_{\text{median}} = 0.184 \exp(-1.45\mu)
\]  \hspace{1cm} (2.12)

\[
\mu = \frac{P}{A_{vt} f_{yt}} \frac{d_c/s - 1}{\frac{1}{A_{vt} f_{yh}} \frac{d_c/s \tan\theta + \tan\theta}{\tan\theta}}
\]  \hspace{1cm} (2.13)

The results of the comparison between model drift predictions and experimental drifts of columns in the database are presented in Figure 2.9 (d). While the model gives reasonable approximations for the majority of NSRC columns, it largely overestimates the drift capacity of HSRC columns with low axial load ratios. The coefficient of variation of the model for NSRC columns is 0.53.

Tran and Li (2013)

Tran and Li [78] proposed a semi-empirical model to predict drift at axial load failure of lightly reinforced NSRC columns. The model is based on the energy analogy that external and internal work performed by the column at the point of axial failure are equal. The external work is performed by the applied axial load while internal work is done by the longitudinal reinforcement, transverse reinforcement and concrete. Using the energy principle the model determines the parameter \(\eta_{sl}\), which is the ratio of axial strength to the yield strength of longitudinal reinforcing bars at the point of axial failure. The model then empirically relates \(\eta_{sl}\) with the ratio of the horizontal displacement due to the sliding between cracking surfaces to the damaged length \((\delta_a^*)\) using a dataset of 47 columns tested to the point of axial failure. Finally, \(\delta_a^*\) is related to
the drift at axial failure. The following two independent equations are to be solved to determine displacement at axial failure using this model.

\[
\delta_{af} = 2 \delta_y + \delta_a^* \times h \tan \theta
\]  
(2.14)

\[
P = \rho_v bh \left( \frac{f_{yh}}{0.2874 \times \delta_a + 1} \right) \frac{1}{\sin \theta} + \frac{d_c f_{yh} A_{st}}{s} + k_1 \sqrt{f'_c} (0.8 A_g) \cot \theta
\]  
(2.15)

where \( \theta = 60^\circ \)

The model gave negative values of \( \delta_a^* \) for the majority of HSRC columns in the database. Therefore, the performance of this model is evaluated for 46 NSRC columns only. The results in Figure 2.9 (e) show that the model tends to give reasonable approximations for most NSRC columns in the current database except 3 columns that had low axial load ratio (\( n \leq 0.1 \)). For these columns, Tran and Li [78] model is giving a very high value of axial failure drift. Similarly, the model tends to largely underestimate the axial failure drift of NSRC columns with low longitudinal reinforcement ratio, \( \rho_v < 1.5\% \).

Wibowo et al. (2014)

Wibowo et al. [12] developed an empirical model for predicting axial load failure drift of NSRC columns. The study used a database of 46 NSRC column specimens tested to the point of collapse. The authors observed that axial load failure drift was significantly influenced by the longitudinal reinforcement ratio, transverse reinforcement ratio and axial load ratio. The following expression was proposed for predicting axial load failure drift:

\[
\delta_{af} = (1+\rho_v)^{-\left(\frac{1}{1-\beta}\right)} + 7\rho_h + \frac{1}{5n}
\]  
(2.16)

The evaluation results in Figure 2.9 (f) indicate that the model gives very good drift predictions for most NSRC columns. However, it tends to overestimate axial load failure drift of majority of the HSRC columns. This model has the coefficient of variation of 0.4 for NSRC columns.
ASCE 41-17 [121] proposed expressions that relate the drift capacity at axial load failure to axial load ratio, transverse reinforcement ratio, transverse reinforcement yield strength and concrete compressive strength. Separate expressions are provided based on the provision of adequate or inadequate development or splicing length. The proposed expressions for columns not controlled by inadequate development or splicing along the clear height are as follows:

\[
\delta_{af} = \delta_y + \frac{0.5}{5 + \frac{n f'_c}{0.8 \rho_h f_{yh}}} - 0.01 \quad \text{for } n \leq 0.5 \quad (2.17)
\]

\[
\delta_{af} = \delta_y \quad \text{for } n = 0.7 \quad (2.18)
\]

Linear interpolation should be performed for \(0.5 < n \leq 0.7\)

For columns controlled by inadequate development or splicing along the clear height:

\[
\delta_{af} = \delta_y + 0.012 - 0.085n + 12\rho_h \quad (2.19)
\]

The drift capacity for the NSRC and HSRC columns in the database has been computed using equations 2.16 and 2.17. The results shown in Figure 2.9 (g) indicate a good correlation compared to other models with a coefficient of variation of 0.31 for both NSRC and HSRC columns. However, still some scatter can be observed in the results wherein the model underpredicts or overpredicts the drift capacity of the specimens.
None of the existing lateral load failure drift models was able to predict the drift capacity of HSRC columns with reasonable accuracy. Similarly, among existing axial load failure drift models...
load failure drift models, only Elwood and Moehle [73] and ASCE 41-17 [121] models gave better estimations of axial load failure drift of HSRC columns. Overestimation of the drift capacity of HSRC columns by most of the existing models can be ascribed to the reduction in the confinement effectiveness of the transverse reinforcement with the increase in the concrete compressive strength that is not accounted for in the current models.

### 2.9 Research Gaps

Based on the extensive literature review conducted in this chapter the following research gaps have been identified:

1. The collapse behaviour of limited ductile HSRC columns that are commonly constructed in RC buildings in regions of low to moderate seismicity is not well understood as most of the previous studies have been conducted on moderately to fully ductile HSRC columns, whereas limited ductile columns have scarcely been investigated.

2. Although collapse behaviour of NSRC columns has been investigated under multidirectional earthquake actions, there has been only one study on HSRC columns under such loading scenarios. Further to this, no robust study has been conducted to develop realistic loading protocols for simulating multidirectional earthquake actions comprising of bidirectional lateral loading and varying axial load.

3. Whilst post-peak (inelastic) drift capacity models have been developed for NSRC columns, very limited work has been done in this area on HSRC columns. Thus, there is a need to develop unified drift models that may have the ability to predict the post-peak drift capacity of both NSRC as well as HSRC columns, as the existing models do not provide a very good estimate of drifts as shown in section 2.8.

The above-mentioned research gaps have been addressed in this study by conducting a comprehensive experimental testing program comprising of 14 limited ductile HSRC columns that were tested under unidirectional and multidirectional earthquake actions. Furthermore, loading protocols for simulating multidirectional earthquake
actions have been developed by conducting a robust numerical study. Finally, a piecewise force drift model has been proposed that can give the full-range elastic and inelastic response of NSRC and HSRC columns.
Chapter 3  Experimental Testing Program and Development of Loading Protocols

3.1 Introduction

Current detailing practices in regions of low to moderate seismicity such as Australia, result in limited to moderately ductile RC columns that are characterized by relatively low collapse drift capacities in contrast to ductile RC columns in regions of higher seismicity. Rare or very rare seismic events occurring in these intraplate regions may impose larger drift demands on such columns, thereby making them vulnerable to collapse.

The comprehensive literature review presented in chapter 2 of the thesis showed that the collapse behaviour of such limited ductile HSRC columns has scarcely been investigated under multidirectional earthquake actions. This chapter presents a description of the experimental program that has been carried out to address this research gap. The test matrix, specimen design, test setup, instrumentation and loading protocols have been described in detail in this chapter.

3.2 Test Matrix

The testing program comprised of 14 limited ductile HSRC columns (250×300 mm cross-section and 2550 mm height), representative of typical Australian construction practice, tested under double curvature bending using three loading histories in Multi-Axis Substructure Testing (MAST) system in smart structures laboratory of Swinburne University of Technology, Australia. Axial load ratio, transverse reinforcement ratio, concrete compressive strength and the type of loading history were the variables of the study. The first six specimens were tested under unidirectional lateral loading with CAL. The next six specimens were tested under bidirectional lateral loading with CAL and the last two specimens were tested under bidirectional lateral loading with VAL. One unidirectional and three bidirectional lateral loading protocols namely, linearised
circular (1:1), octo-elliptical (1:1) and octo-elliptical (0.6:1) path were employed in the testing. Two axial load variation protocols namely, synchronous variable axial load (SS-VAL) and nonsynchronous variable axial load (NS-VAL) were also used. Table 3.1 presents the detailed test matrix.

Table 3.1. Test matrix

<table>
<thead>
<tr>
<th>No.</th>
<th>Width × Depth × Height (mm)</th>
<th>Concrete Mean Strength $f_{cm}$ (MPa)</th>
<th>Longitudinal Reinforcement $\rho_v$ (%)</th>
<th>Ties (mm) $\rho_{hy}$ (%)</th>
<th>Axial Load Ratio $n$</th>
<th>Type of Loading Path</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>250×300 × 2550</td>
<td>75.0</td>
<td>6N16 (1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>0.15</td>
<td>Unidirectional + CAL</td>
</tr>
<tr>
<td>S2</td>
<td>250×300 × 2550</td>
<td>66.0</td>
<td>6N16 (1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>0.30</td>
<td>Unidirectional + CAL</td>
</tr>
<tr>
<td>S3</td>
<td>250×300 × 2550</td>
<td>87.0</td>
<td>6N16 (1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>0.45</td>
<td>Unidirectional + CAL</td>
</tr>
<tr>
<td>S4</td>
<td>250×300 × 2550</td>
<td>90.0</td>
<td>6N16 (1.6%)</td>
<td>N10@100 (0.52%)</td>
<td>0.45</td>
<td>Unidirectional + CAL</td>
</tr>
<tr>
<td>S5</td>
<td>250×300 × 2550</td>
<td>62.0</td>
<td>6N16 (1.6%)</td>
<td>N10@300 (0.18%)</td>
<td>0.30</td>
<td>Unidirectional + CAL</td>
</tr>
<tr>
<td>S6</td>
<td>250×300 × 2550</td>
<td>90.0</td>
<td>6N16 (1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>0.25</td>
<td>Unidirectional + CAL</td>
</tr>
<tr>
<td>S7</td>
<td>250×300 × 2550</td>
<td>86.0</td>
<td>6N16 (1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>0.15</td>
<td>Linearised Circular (1:1) + CAL</td>
</tr>
<tr>
<td>S8</td>
<td>250×300 × 2550</td>
<td>63.0</td>
<td>6N16 (1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>0.30</td>
<td>Linearised Circular (1:1) + CAL</td>
</tr>
<tr>
<td>Specimen</td>
<td>Section Size</td>
<td>Specimen Height</td>
<td>Longitudinal Reinforcement</td>
<td>Cross-sectional Area</td>
<td>Reinforcement Confinement</td>
<td></td>
</tr>
<tr>
<td>----------</td>
<td>--------------</td>
<td>-----------------</td>
<td>----------------------------</td>
<td>----------------------</td>
<td>---------------------------</td>
<td></td>
</tr>
<tr>
<td>S9</td>
<td>250×300 ×2550</td>
<td>90.0</td>
<td>6N16(1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>Octo-Elliptical (1:1) + CAL</td>
<td></td>
</tr>
<tr>
<td>S10</td>
<td>250×300 ×2550</td>
<td>83.0</td>
<td>6N16(1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>Octo-Elliptical (1:1) + CAL</td>
<td></td>
</tr>
<tr>
<td>S11</td>
<td>250×300 ×2550</td>
<td>105.0</td>
<td>6N16(1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>Octo-Elliptical (0.6:1) + CAL</td>
<td></td>
</tr>
<tr>
<td>S12</td>
<td>250×300 ×2550</td>
<td>74.0</td>
<td>6N16(1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>Octo-Elliptical (0.6:1) + CAL</td>
<td></td>
</tr>
<tr>
<td>S13</td>
<td>250×300 ×2550</td>
<td>87.0</td>
<td>6N16(1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>Octo-Elliptical (0.6:1) + SS-VAL</td>
<td></td>
</tr>
<tr>
<td>S14</td>
<td>250×300 ×2550</td>
<td>85.0</td>
<td>6N16(1.6%)</td>
<td>N10@150 (0.35%)</td>
<td>Octo-Elliptical (0.6:1) + NS-VAL</td>
<td></td>
</tr>
</tbody>
</table>

### 3.3 Specimen Design

Each column specimen had a cross-section of 250×300 mm and height of 2550 mm and was tested under double curvature. As such, the shear span ratio (i.e. \( \frac{M_u}{V_p h} \)) of the specimens was 4.25. The columns were cast with concrete mixes that had characteristic strengths of 65 and 100 MPa. The maximum compressive strength of each column was determined on the test day using 100 mm diameter and 200 mm high cylinder samples. Four axial load ratios were used in this study i.e. 0.15, 0.25, 0.3 and 0.45. Each specimen had a longitudinal reinforcement ratio of 1.6%, which consisted of six N16 bars; where, N16 denotes a 16 mm nominal diameter grade D500N reinforcing bar in accordance with AS/NZS 4671 [104]. Grade D500N denotes deformed normal-ductility reinforcement with a minimum characteristic yield stress, strain-hardening ratio and ultimate strain of 500 MPa, 1.08% and 5%, respectively.

The effect of confinement reinforcement on the collapse drift was studied by considering three transverse reinforcement configurations i.e. AS 3600 code-compliant [14], under-confined and over-confined under unidirectional lateral actions. On the other hand, only code-compliant transverse reinforcement was provided to the specimens tested under bidirectional lateral actions with CAL and VAL. It is noted that
the requirements of various concrete design standards in other low to moderate seismic regions result in somewhat similar detailing to AS 3600 as shown in chapter 2, so the results of this study can be considered representative of a broad range of low to moderate seismic regions.

According to the requirements of AS 3600, a minimum effective confining pressure of $0.01f'_{c}$ should be provided to the concrete core in the special confinement regions. Also, a minimum spacing of the lesser of $0.6D_c$ and 300 mm is stipulated for columns with $f'_{c}>50$ MPa. The spacing requirements outside the special confinement region are the lesser of $0.8D_c$ and 300 mm. The code-compliant RC columns considered in this study were designed with deem to conform core confinement requirements of AS 3600, according to which the requirement of the minimum effective confining pressure of $0.01f'_{c}$ is deemed to be satisfied if the spacing of the transverse reinforcement in the special confinement regions does not exceed the following for rectangular/square sections:

$$s = \frac{15n_b A_v f_{yh}}{f'_{c}\sqrt{A_c}}$$

(3.1)

The code-compliant grade 65 MPa HSRC columns were provided with a minimum transverse reinforcement of N10@150mm ($\rho_h=0.35\%$ in the stronger direction) in the special confinement region in accordance with the deem to conform core confinement requirement of AS 3600, whereas under-confined HSRC column section was designed below the minimum code requirements such that the transverse reinforcement was two times more widely spaced than the code-compliant specimen i.e. N10@300 mm ($\rho_h=0.18\%$). The over-confined column section, on the other hand, was provided with a transverse reinforcement of N10@100 mm ($\rho_h=0.53\%$). The effect of concrete compressive strength on the drift performance of HSRC column was studied by providing the same confinement reinforcement to grade 100 MPa specimen as that of code-compliant grade 65 MPa specimen. Figure 3.1 (a) presents the reinforcement detailing of the code-compliant specimen.
The specimens had a lap-splice length of 800 mm at the bottom of the specimen in the plastic hinge region. The length of the lap was calculated in accordance with the requirements of AS 3600, where the length is dependent on the bar size, yield strength, concrete strength and reinforcement configuration. It should be noted that the specifications of AS 3600 result in similar lap-splice length as that of ACI 318 [17]. All the specimens were provided with the same lap-splice length, and thus the effect of lap-splice on the behaviour of the specimens was not studied.

The material properties of the reinforcement were determined from material samples taken during construction. The yield stress of the N16 and N10 bars was 565 and 530 MPa, the ultimate stress was 715 and 671 MPa and the ultimate strain was 7.5% and 7.0%, respectively.

3.4 Test Setup

The testing program was conducted using MAST system in the Smart Structures Laboratory (SSL) at Swinburne University of Technology, Australia. The MAST system can apply six-degrees-of-freedom loading in switched-mode, mixed-mode, hybrid or a combination of these modes. The vertical load-carrying capacity of the MAST system is ±4 MN with four vertical ±1 MN actuators and horizontal load capacity is ±1 MN with two horizontal ±0.5 MN actuators in each direction. The details about the force and displacement capacities of the MAST system in the six degrees of freedom are shown in Table 3.2. The MAST system and its axes of loading are shown in Figure 3.2. Further details about the MAST system, including actuator/overall system setout/s can be seen in Menegon [105].
Figure 3.1. a) Detailing of the specimen (dimensions in mm); b) specimen bolted under MAST system

Table 3.2. Capacity of the MAST system in six degrees of freedom

<table>
<thead>
<tr>
<th>Degree of Freedom</th>
<th>Force Capacity</th>
<th>Displacement Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>$T_x$ - X-Axis Translation</td>
<td>±1000 kN</td>
<td>±250 mm</td>
</tr>
<tr>
<td>$T_y$ - Y-Axis Translation</td>
<td>±1000 kN</td>
<td>±250 mm</td>
</tr>
<tr>
<td>$T_z$ - Z-Axis Translation</td>
<td>±4000 kN</td>
<td>±250 mm</td>
</tr>
<tr>
<td>$R_x$ - X-Axis Rotation</td>
<td>±4500 kNm</td>
<td>±6.3°</td>
</tr>
<tr>
<td>$R_y$ - Y-Axis Rotation</td>
<td>±4500 kNm</td>
<td>±6.3°</td>
</tr>
<tr>
<td>$R_z$ - Z-Axis Rotation</td>
<td>±3500 kNm</td>
<td>±8.1°</td>
</tr>
</tbody>
</table>
The boundary conditions applied to the cross-head of the MAST system for testing the columns in double curvature configuration are summarized in Table 3.3. The top pedestal of the specimen was bolted to the cross-head of the MAST system, whereas the bottom pedestal was fixed to the strong ground floor of the laboratory as shown in Figure 3.1 (b). The crosshead of the MAST system maintained a zero rotation about the three axes for the duration of the test, resulting in a double curvature bending moment distribution in the specimens.

**Table 3.3. Boundary conditions in the MAST system**

<table>
<thead>
<tr>
<th>Degree of Freedom</th>
<th>Mode</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>(T_x) -X-Axis Translation</td>
<td>Displacement-controlled</td>
<td>Zero displacement or Incrementally increasing displacements</td>
</tr>
<tr>
<td>(T_y) -Y-Axis Translation</td>
<td>Displacement-controlled</td>
<td>Incrementally increasing displacements</td>
</tr>
<tr>
<td>(T_z) -Z-Axis Translation</td>
<td>Force-controlled</td>
<td>Constant or variable axial load</td>
</tr>
<tr>
<td>(R_x) - X-Axis Rotation</td>
<td>Displacement-controlled</td>
<td>Zero rotation</td>
</tr>
<tr>
<td>(R_y) - Y-Axis Rotation</td>
<td>Displacement-controlled</td>
<td>Zero rotation</td>
</tr>
<tr>
<td>(R_z) - Z-Axis Rotation</td>
<td>Displacement-controlled</td>
<td>Zero rotation</td>
</tr>
</tbody>
</table>
3.5 Instrumentation

During the tests, a total of 31 transducers, including 18 linear variable displacement transducers (LVDTs), 11 string potentiometers (SPOTs) and 2 linear potentiometers (LPOTs) were installed to measure the displacements of the column. Figure 3.3 shows the layout of the instrumentation.

Figure 3.3. Instrumentation layout (Dimensions in mm) a) SPOTs for lateral displacement b) LVDTs, SPOTs and LPOTs for measuring curvatures, vertical displacement and rocking of the specimens

The horizontal displacement profiles of the column were recorded using SPOTs (L20 to L25) attached to an independent steel post as shown in Figure 3.3 (a). The LVDTs (L1 to L18) were located on the tension/compression faces of the column and were used to calculate tension/compression strain and curvature distribution up the height of the
column as shown in Figure 3.3 (b). It is noted that the transducers L1 to L9 that are on the front face can be seen in the Figure, whereas L10 to L18 are on the back face of the column. Four vertical SPOTs (L26 to L29) were used for measuring axial displacement (shortening or elongation) of the column and to assess whether zero rotation in the cross-head was actually maintained for the duration of the test. The sliding of the bottom pedestal was also recorded using 1 SPOT (L19) and two LPOTs (L30 to L31) were used for recording any possible rocking of the bottom pedestal. The details of transducers measurements strokes are provided in Appendix B.

3.6 Unidirectional Lateral Loading Protocol

The specimens were tested under three different loading histories i.e. unidirectional lateral loading with CAL, bidirectional lateral loading with CAL and bidirectional lateral loading with VAL. The unidirectional lateral loading protocol used in this study is described herein.

The unidirectional lateral loading protocol comprised of incrementally increasing displacements in the stronger Y-direction of the specimen as summarized in Figure 3.4. In the loading protocol, each displacement excursion was repeated twice so that the strength degradation behaviour of the column can be captured properly. The displacement of the first and second loading cycles was equal to approximately 0.5 and 1.0 times the theoretical yield drift of the column, respectively, whereas, in accordance with the requirements of ACI ITG-5.1-07 [106], all the following cycles had a displacement increase between 5/4 and 3/2 times of the displacement excursion of the preceding cycle.
In order to develop realistic bidirectional lateral loading protocols, a numerical study was conducted on a case study RC frame-wall building to investigate the actual bidirectional lateral displacement path of the column during an earthquake. The building was subjected to a suite of ground motions that were scaled to design basis earthquake (DBE) and maximum considered earthquake (MCE) shaking levels of the Australian earthquake standard, and the resulting bidirectional displacement path of the columns was statistically processed to identify the typical patterns by using and refining the methodology proposed by Elkady and Lignos [68] for developing loading protocols for wide flange steel columns. The following provides the details of the numerical study conducted and the subsequent analysis.

### 3.7 Bidirectional Lateral Loading Protocols

In order to develop realistic bidirectional lateral loading protocols, a numerical study was conducted on a case study RC frame-wall building to investigate the actual bidirectional lateral displacement path of the column during an earthquake. The building was subjected to a suite of ground motions that were scaled to design basis earthquake (DBE) and maximum considered earthquake (MCE) shaking levels of the Australian earthquake standard, and the resulting bidirectional displacement path of the columns was statistically processed to identify the typical patterns by using and refining the methodology proposed by Elkady and Lignos [68] for developing loading protocols for wide flange steel columns. The following provides the details of the numerical study conducted and the subsequent analysis.

#### 3.7.1 Case Study Building

The case study building is a typical mid-rise frame-wall structure commonly constructed in regions of low to moderate seismicity (e.g. Australia). This case study building was identified during a reconnaissance survey performed by Menegon et al. [107] to identify typical RC construction in Australia. The building is 36.8 m high with 8 stories and has a total footprint area of 58.8 \( \times \) 27 m\(^2\), with 7 bays spanning in the East-West direction and 3 bays spanning in the North-South direction. Four core walls, including two lift cores, were provided in the building. The building was designed for
an imposed load of 4 kPa, a superimposed dead load of 1 kPa and a façade load of 0.5 kPa. The beams, columns and walls of the building were provided with limited ductile detailing in accordance with the recommendations of the Australian concrete standard, AS 3600-2009 [108]. The plan view of the building is shown in Figure 3.5.

![Figure 3.5. Plan view of the case study building (reproduced from Menegon et al. [107])]}

### 3.7.2 Numerical Modelling and Validation

The Open System for Earthquake Engineering Simulation (OpenSees - McKenna et al. [109]) was utilized as an analytical tool for nonlinear modelling and analysis of the case study building structure. The nonlinear uniaxial material model developed by Kent-Scott-Park (Scott et al. [110]) was used for defining material properties of concrete (Concrete01 in OpenSees), whereas steel reinforcement behaviour was based on the uniaxial material model proposed by Giuffre-Menegotto-Pinto ([111-112] - Steel02 in OpenSees). The distributed plasticity modelling comprising of fiber sections and nonlinear beam-column elements was used for defining the flexural behaviour of beam and column elements. A rigid diaphragm was assigned at each floor level to model the effects of the slab. The framing action of the slab was incorporated by providing beams
with effective slab width of 1.5m and a depth equal to the depth of the slab. The core walls were modelled as shell elements with elastic membrane plate sections of effective stiffness, calculated as a function of the axial load supported by the wall element. Wall elements were modelled elastic because the primary interest was in the behaviour of columns. The fundamental natural period (elastic period) of the case study building was found to be 1.48s.

In order to incorporate the shear behaviour and lateral strength degradation of the columns, a shear spring was used in series with the beam-column element. The behaviour of the shear spring was defined using the analytical element proposed by LeBorgne and Ghannoum [113]. In this model, lateral strength degradation is triggered in the column element when it exceeds the rotation or drift limit specified by the user. The nonlinear modelling for columns was validated with the results of specimen S2 tested in Chapter 4 of the thesis as shown in Figure 3.6, wherein a very good correlation is observed between the experimental results and the predictions by the numerical model.

Figure 3.6. Validation of the numerical model based on experimental results of column (specimen S2 in chapter 4).
3.7.3 Characteristics of Input Ground Motions and Scaling

The case study building was subjected to a suite of 30 scaled (15 DBE and 15 MCE) ground motions representative of low to moderate seismic regions to investigate the typical patterns of bidirectional lateral displacement and axial load variation in the columns. The two levels of shaking (i.e. DBE and MCE) were considered to see the effect of shaking level on the patterns of displacements and axial load variation if any. The ground motions were obtained from PEER ground motion database [114] and are presented in Table 3.4. The characteristics of the ground motions are as follows:

Moment magnitude, $M_w$: 5.5-6.5; Distance to rupture surface, $R_{rup}$: 10-40 km; Shear wave velocity averaged over the top 30 m, $V_{s30}$: 180-1500 m/s; Peak ground acceleration, PGA: 0.02–0.24g.

The ground motions were scaled to the DBE and MCE response spectrum levels of the Australian Earthquake Standard (AS 1170.4-[115]). DBE refers to an earthquake with a return period of 500 years, while MCE refers to an event with a return period of 2500 years. The scaling factor was calculated by dividing the AS 1170.4 DBE or MCE response spectrum acceleration (for a given soil site) corresponding to the period of the building in that particular direction with the spectral acceleration of the ground motion at the same time period. In this way, separate scaling factors were determined for the components of the ground motions in X, Y and Z directions for both DBE and MCE shaking levels. It is noted that X and Y refer to orthogonal horizontal directions, whereas Z refers to the vertical direction. The scaled DBE and MCE response spectrums of the selected ground motions along with DBE and MCE response spectrum of AS 1170.4 are presented in Figure 3.7.
Table 3.4. Characteristics of the input ground motions used in the numerical study

<table>
<thead>
<tr>
<th>No.</th>
<th>Earthquake Event</th>
<th>Magnitude $M_w$</th>
<th>Distance to Rupture $R_{rup}$ (km)</th>
<th>Average PGA (g)</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
</tr>
<tr>
<td>1</td>
<td>Griva, Greece (1990)</td>
<td>6.1</td>
<td>33.3</td>
<td>0.10</td>
<td>0.09</td>
</tr>
<tr>
<td>2</td>
<td>Georgia, USSR (1991)</td>
<td>6.2</td>
<td>31.5</td>
<td>0.10</td>
<td>0.09</td>
</tr>
<tr>
<td>3</td>
<td>Little Skull Mtn, Nevada, USA (1992)</td>
<td>5.7</td>
<td>16.1</td>
<td>0.18</td>
<td>0.12</td>
</tr>
<tr>
<td>4</td>
<td>Joshua Tree, California, USA (1992)</td>
<td>6.1</td>
<td>17.9</td>
<td>0.19</td>
<td>0.19</td>
</tr>
<tr>
<td>5</td>
<td>Big Bear-01, California, USA (1992)</td>
<td>6.5</td>
<td>26.5</td>
<td>0.15</td>
<td>0.10</td>
</tr>
<tr>
<td>6</td>
<td>Northridge-04, California, USA (1994)</td>
<td>5.9</td>
<td>25.2</td>
<td>0.11</td>
<td>0.10</td>
</tr>
<tr>
<td>7</td>
<td>Double Springs, Alabama, USA (1994)</td>
<td>5.9</td>
<td>12.8</td>
<td>0.08</td>
<td>0.06</td>
</tr>
<tr>
<td>8</td>
<td>Dinar, Turkey (1995)</td>
<td>6.4</td>
<td>36.9</td>
<td>0.04</td>
<td>0.03</td>
</tr>
<tr>
<td>9</td>
<td>Kozani, Greece-01 (1995)</td>
<td>6.4</td>
<td>19.5</td>
<td>0.17</td>
<td>0.13</td>
</tr>
<tr>
<td>10</td>
<td>Umbria marche, Italy (1997)</td>
<td>6.0</td>
<td>17.3</td>
<td>0.15</td>
<td>0.10</td>
</tr>
<tr>
<td>11</td>
<td>Northwest China-04 (1997)</td>
<td>5.9</td>
<td>27.9</td>
<td>0.24</td>
<td>0.23</td>
</tr>
<tr>
<td>No.</td>
<td>Location</td>
<td>$M$</td>
<td>$H$</td>
<td>$S$</td>
<td>$W$</td>
</tr>
<tr>
<td>-----</td>
<td>-----------------------------------------------</td>
<td>------</td>
<td>------</td>
<td>------</td>
<td>------</td>
</tr>
<tr>
<td>12</td>
<td>Chi Chi, Taiwan (1999)</td>
<td>6.2</td>
<td>24.0</td>
<td>0.13</td>
<td>0.13</td>
</tr>
<tr>
<td>13</td>
<td>Parkfield, California, USA (2004)</td>
<td>6.0</td>
<td>11.5</td>
<td>0.24</td>
<td>0.17</td>
</tr>
<tr>
<td>14</td>
<td>L’Aquila, Italy (2009)</td>
<td>6.3</td>
<td>15.8</td>
<td>0.06</td>
<td>0.04</td>
</tr>
<tr>
<td>15</td>
<td>Christchurch, New Zealand (2011)</td>
<td>6.2</td>
<td>25.5</td>
<td>0.18</td>
<td>0.19</td>
</tr>
</tbody>
</table>

Figure 3.7. Scaled response spectra of the selected ground motions (log scale): a) DBE-X; b) DBE-Y; c) DBE-Z; d) MCE-X; e) MCE-Y; f) MCE-Z

3.7.4 Bidirectional Drift Response History

The lateral drifts in the X and Y axes of the bottom (1st storey) and top (8th storey) storey corner columns (A1) were plotted against each other to visualize the displacement path.
of the column under each scaled (both DBE and MCE) ground motion. It was observed that generally, the displacement path of the column consisted of elliptical loops of four different orientations, namely, vertical (Y-direction), diagonal-1, horizontal (X-direction) and diagonal-2. Figure 3.8 shows the four observed orientations of the elliptical loops.

The drift plots indicated that columns, particularly the first storey ones, did not undergo large drifts, especially under DBE. This is because of the large stiffness of the building due to the presence of four core walls and also due to the modest nature of the scaled ground motions, as they are representative of shaking levels in low to moderate seismic regions. For the purpose of illustration, the displacement paths of the bottom and top storey corner column A1 under Joshua Tree (1992), Umbria Marche (1997) and Christchurch (2011) MCE ground motions are shown in Figure 3.9. The identified (representative) elliptical loops observed in the displacement path under these ground motions are highlighted in Figure 3.10. It is noted that Figure 3.10 does not highlight all the elliptical loops, but only the representative ones (i.e. those with the largest displacements). It can be observed in Figures 3.9 and 3.10 that under a particular ground motion, orientations of elliptical loops are generally similar for the bottom and top storey columns; however, amplitudes and aspect ratios of the loops are different. The figures also indicate that displacement path under Christchurch (2011) earthquake is primarily dominated by vertical elliptical loops, whereas Joshua Tree (1992) has more domination of diagonal loops and Umbria Marche (1997) has all the four orientations of the elliptical loops.
Figure 3.8. Orientations of the elliptical loops: a) vertical (-10°≤θ≤10°); b) diagonal-1 (10°<θ<80°); c) horizontal (80°≤θ≤100°); d) diagonal-2 (100°<θ<170°)

3.7.5 Statistical Processing of the Bidirectional Drift Response History

In order to develop a better understanding about the bidirectional displacement path of the column under ground motion excitations and also to arrive at a generalized displacement pattern comprising of elliptical loops, the drift response history of the bottom and top storey corner columns, A1 under DBE and MCE ground motions was statistically processed to evaluate the number of elliptical loops at different drifts and in each orientation. Figure 3.11 shows the parameters used to define an elliptical loop and its geometric properties. The definition of each parameter is provided herein.

\( X_{max} \): Maximum drift in the X-axis of the ellipse
$X_{min}$: Minimum drift in the X-axis of the ellipse

$X_{range}$: Drift range in the X-axis of the ellipse = $X_{max} - X_{min}$

$Y_{max}$: Maximum drift in the Y-axis of the ellipse

$Y_{min}$: Minimum drift in the Y-axis of the ellipse

$Y_{range}$: Drift range in the Y-axis of the ellipse = $Y_{max} - Y_{min}$

$\theta$: Angle between the elliptical loop and Y-axis = $\arctan\left(\frac{X_{range}}{Y_{range}}\right)$

$X_0$: X coordinate of the centre of ellipse = $\frac{X_{max} + X_{min}}{2}$

$Y_0$: Y coordinate of the centre of ellipse = $\frac{Y_{max} + Y_{min}}{2}$

$a$: length of the minor axis of ellipse

$b$: length of the major axis of ellipse

Tables 3.5 and 3.6 present the statistics of the elliptical displacement loops for drifts greater than 0.25%. It is noted that the statistics of bottom and top storey column have been combined herein because bottom storey column mostly experienced small drift loops (<0.25%), especially under DBE, and as such did not have many elliptical loops in the range of interest (i.e. > 0.25%). The data has been summarized in terms of the number of elliptical loops corresponding to different drifts in the Y-direction of the column, angle $\theta$ defining the orientation of each elliptical loop and aspect ratio of the ellipses. The aspect ratio of an ellipse is defined as the ratio of minor to major axis ($a/b$) length of an ellipse. The aspect ratio of each ellipse has been calculated using coordinates $(x, y)$ at any three points on the ellipse to solve equation (1) for unknowns $a$ and $b$. The equations for the first two points are subtracted from each other to get $b$ in terms of $a$, which is then substituted in the equation for the third point to solve for $a$.

$$\frac{(x - x_o) \cos \theta + (y - y_o) \sin \theta)^2}{a^2} + \frac{(x - x_o) \sin \theta - (y - y_o) \cos \theta)^2}{b^2} = 1$$  \hspace{1cm} (3.2)
As it would be expected, elliptical loops with much larger drifts were observed for MCE ground motions as opposed to DBE ground motions. The results indicate that the number of vertical elliptical loops are greatest in number and horizontal elliptical loops are least in number, whereas elliptical loops with diagonal-1 and diagonal-2 orientations are in the intermediate range. This is because ground shaking was relatively stronger in the Y-direction of the building compared to the X-direction as indicated by the response spectrum of the ground motions in Figure 3.7. The average angle $\theta$ and average aspect ratio ($a/b$) of all the elliptical loops for a given drift and orientation were determined and are presented in Table 3.5 and 3.6 for DBE and MCE shaking levels, respectively. For convenience, the average angle for vertical loop and horizontal loops are taken as 0° and 90°, respectively, if they are within ±10° offset range. On the other hand, the average angle of diagonal-1 elliptical loops was found to be in the range of 29-31° for DBE shaking and 27-37° for MCE shaking, respectively, whereas the average angle of orientation of diagonal-2 elliptical loops was in the range of 144-145° for DBE shaking and 136-152° for MCE shaking, respectively. The average aspect ratio of the elliptical loops, on the other hand, was 0.35 and 0.26 for DBE and MCE shaking, respectively. Finally, the average ratio of the overall maximum displacement in the X to Y-axis of the column was found to be 0.64 and 0.65 for DBE and MCE shaking, respectively.
Table 3.5. Statistical analysis of bottom storey and top storey corner column (A1) drift data under DBE

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Ellipse Orientation</th>
<th>Drift (%) in the Y-Direction</th>
<th>Average Ratio of Maximum Drift in X to Y-Axis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td>0.5</td>
</tr>
<tr>
<td>No of occurrences</td>
<td>-10°≤θ≤10°</td>
<td>20</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td>10°&lt;θ&lt;80°</td>
<td>8</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>80°≤θ≤100°</td>
<td>5</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>100°&lt;θ&lt;170°</td>
<td>9</td>
<td>6</td>
</tr>
<tr>
<td>Average angle to vertical axis</td>
<td>-10°≤θ≤10°</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>10°&lt;θ&lt;80°</td>
<td>31</td>
<td>29</td>
</tr>
<tr>
<td></td>
<td>80°≤θ≤100°</td>
<td>90</td>
<td>90</td>
</tr>
<tr>
<td></td>
<td>100°&lt;θ&lt;170°</td>
<td>145</td>
<td>144</td>
</tr>
<tr>
<td>Average aspect ratio of ellipses (a/b)</td>
<td>-10°≤θ≤10°</td>
<td>0.41</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>10°&lt;θ&lt;80°</td>
<td>0.32</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>80°≤θ≤100°</td>
<td>0.52</td>
<td>0.68</td>
</tr>
<tr>
<td></td>
<td>100°&lt;θ&lt;170°</td>
<td>0.36</td>
<td>0.25</td>
</tr>
</tbody>
</table>
Table 3.6. Statistical analysis of bottom storey and top storey corner column (A1) drift data under MCE

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Ellipse Orientation</th>
<th>Drift (%) in the Y-Direction</th>
<th>Average Ratio of Maximum Drift in X to Y-Axis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>0.25</td>
<td>0.5</td>
</tr>
<tr>
<td>No of occurrences -10°≤θ≤10°</td>
<td>25</td>
<td>23</td>
<td>15</td>
</tr>
<tr>
<td>10°&lt;θ&lt;80°</td>
<td>9</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td>80°≤θ≤100°</td>
<td>20</td>
<td>4</td>
<td>1</td>
</tr>
<tr>
<td>100°&lt;θ&lt;170°</td>
<td>17</td>
<td>6</td>
<td>11</td>
</tr>
<tr>
<td>Average angle to vertical axis -10°≤θ≤10°</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10°&lt;θ&lt;80°</td>
<td>37</td>
<td>36</td>
<td>34</td>
</tr>
<tr>
<td>80°≤θ≤100°</td>
<td>90</td>
<td>90</td>
<td>90</td>
</tr>
<tr>
<td>100°&lt;θ&lt;170°</td>
<td>143</td>
<td>136</td>
<td>147</td>
</tr>
<tr>
<td>Average aspect ratio of ellipses (a/b) -10°≤θ≤10°</td>
<td>0.32</td>
<td>0.35</td>
<td>0.28</td>
</tr>
<tr>
<td>10°&lt;θ&lt;80°</td>
<td>0.35</td>
<td>0.28</td>
<td>0.19</td>
</tr>
<tr>
<td>80°≤θ≤100°</td>
<td>0.37</td>
<td>0.35</td>
<td>0.17</td>
</tr>
<tr>
<td>100°&lt;θ&lt;170°</td>
<td>0.37</td>
<td>0.36</td>
<td>0.3</td>
</tr>
</tbody>
</table>

3.7.6 Mechanism Leading to the Formation of Elliptical Loops of Various Orientations

The displacement of the column in the form of elliptical loops is because the motion in the two axes of the column is in the form of sine and cosine waves of unequal amplitudes, whereas different orientations of the ellipses result from the phase shift between the displacements in the X and Y direction of the column. For instance, vertical elliptical loops are formed when X displacements are leading Y with a phase of 90 or 270 degrees. Figure 3.12 (a) shows the X and Y displacements of a vertical loop, where X displacements are leading Y with a phase of 90 degrees. Conversely, horizontal loops
are formed when Y displacements are leading X by 90 or 270 degrees, as shown in Figure 3.12 (b), in which Y displacements lead X by 270 degrees and result in a horizontal loop. On the other hand, if Y displacements lead X with a phase between 0 to 90 or 90 to 270 degrees then a diagonal-1 loop is formed, whereas if X displacements lead Y with a phase between 0 to 90 or 90 to 270 degrees then a diagonal-2 elliptical loop is formed. This is depicted in Figures 3.12 (c) and 3.12 (d) where Y displacements lead X by 216 degrees and X displacements lead Y by 36 degrees, respectively, and result in diagonal-1 (45° orientation) and diagonal-2 (135° orientation) loops of Figures 3.8 (b) and 3.8 (d), respectively.

The phase shift between X and Y displacements of a building column depends on the dynamic characteristics of the building and also on the characteristics of the ground motion. Therefore, the number of elliptical loops in a particular orientation would vary from columns of one building to another, and similarly, from one ground motion to another. However, the displacement path of the column can generally be expected to comprise of elliptical loops, as the displacements in the two axes of the column are generally in the form sine and cosine waves of unequal amplitudes, which result in the formation of elliptical loops. In view of this, the displacement pattern observed for the case-study building columns can be considered as representative of the general displacement path of the columns of any building, in the sense that displacement pattern of the columns will generally be dominated by elliptical loops of various orientations, although the number of loops in a particular orientation may vary. Thus, the results of the displacement path of the case study building column may be considered sufficient for developing a generalized and simplified loading protocol representative of the actual displacement path of the column during an earthquake.
Figure 3.12. Phase shift between X and Y displacements of the column: (a) vertical elliptical loop (X leads Y by 90 degrees); (b) horizontal elliptical loop (Y leads X by 270 degrees); (c) diagonal-1 elliptical loop (Y leads X by 216 degrees; (d) diagonal-2 elliptical loop (X leads Y by 36 degrees)

3.7.7 Proposed Bidirectional Loading Protocols

The statistical processing of the bidirectional drift response history of the columns of the case study building showed that the bidirectional displacement path of the column under earthquake excitations comprised of elliptical loops of four different orientations, where each elliptical loop had a different number of occurrences. Further investigation revealed that the formation of elliptical loops of various orientations was a consequence of the phase shift between X and Y displacements of the building, which in turn, is dependent on the dynamic properties of the building and the ground motion. As such we can always expect the displacement path of any building column under any earthquake to consist of elliptical loops of different orientations as phase shift will
always exist between the X and Y displacements of the column due to the different dynamic characteristics of the building and the ground motions in the two directions.

Accordingly, two bidirectional loading protocols comprising of elliptical loops that generalize the displacement path of an RC column during earthquake actions are proposed. The protocols are referred to as Octo-elliptical (0.6:1) path and Octo-Elliptical (1:1) path, respectively. A third protocol referred to as Linearised Circular (1:1) path that is a modified version of the loading protocol used by researchers (Hogan et al. [116]) in the testing of RC walls is also employed in the experimental testing to compare the results of the proposed protocols with a conventional loading protocol. It should be noted that 1:1 and 0.6:1 refer to the ratio of the overall enveloped magnitude of displacements in the X to the Y axis of the column. The enveloped magnitude of displacements for each loading protocol are shown by the dotted lines in Figure 3.13, which shows that for linearised circular (1:1) and octo-elliptical (1:1) paths, the overall envelope of displacements is circular, whereas the envelope for octo-elliptical (0.6:1) path is elliptical.

3.7.7.1 Octo-Elliptical (0.6:1) Path

The octo-elliptical (0.6:1) loading protocol consists of a total of eight elliptical loops rotated in four different directions. The first four loops displace the column in the counter clockwise direction, whereas the last four displace it in the clockwise direction. The loops are classified as vertical, diagonal-1, horizontal and diagonal-2 elliptical loops. The loading path begins with a vertical ellipse ($\theta = 0^\circ$) that displaces the column in the Y-direction (strong-axis) from the origin. This is followed by diagonal displacement via diagonal-1 ($\theta = 31^\circ$) ellipse, and then horizontal displacement in the X-direction (weak-axis) through the horizontal ellipse ($\theta = 90^\circ$). Finally, the diagonal-2 ellipse ($\theta = 149^\circ$) displaces the column diagonally again before bringing it back to the origin. It can be seen that the angles of orientations of the ellipses in the proposed loading protocol are quite similar to those observed during the statistical processing of the results in tables 3.5 and 3.6. After finishing one complete cycle of displacements in the counter clockwise direction, the four ellipses are repeated again but this time in the
clockwise direction. The transition from one elliptical loop to another is provided using smooth arcs. The two small semi-circles visible around the origin are due to these arcs. The upper semicircle is formed when the ellipses are displacing in the counter clockwise direction, whereas lower semicircle is formed during the displacement in the clockwise direction.

Similarly, in accordance with the results of the statistical processing of bidirectional drift response history of the case study building presented in section 3.7.5 (where average aspect ratio was found to be 0.35 and 0.26 for DBE and MCE shaking levels, respectively and the average of maximum displacements in the X to Y axis was 0.64 and 0.65 under DBE and MCE shakings, respectively), the aspect ratio (a:b) of the individual ellipses (i.e. loops) in the proposed protocol is kept 0.3:1 and the ratio of the overall enveloped weak to strong axis (X:Y) displacement is 0.6:1. This means that in octo-elliptical (0.6:1) path, the column is subjected to asymmetric displacement in the strong and weak directions, with the overall enveloped displacement in the weak direction being 60% of the displacement in the strong direction. As such, all the individual ellipses are circumscribed within an internal elliptical path as shown in Figure 3.13 (a).

3.7.7.2 Octo-Elliptical (1:1) Path

The octo-elliptical (1:1) path is similar to octo-elliptical (0.6:1) path except that the ratio of X to Y-axis displacements is equal in this protocol. As a result, the individual ellipses in octo-elliptical (1:1) path are circumscribed in a circular path instead of an elliptical one as shown in Figure 3.13 (b). The angles of the vertical, diagonal-1, horizontal and diagonal-2 ellipses in octo-elliptical (1:1) path are 0°, 45°, 90° and 135°, respectively.

The octo-elliptical (1:1) path can be employed if a more conservative assessment of the column’s capacity is required, especially for the situation where strong bidirectional actions are expected in the both axes of the column, whereas octo-elliptical (0.6:1) loading protocol is proposed for a more realistic assessment of the column’s capacity, as the overall enveloped displacement (X/Y) in this path (i.e. 0.6/1) is similar to what was observed in the numerical study (0.65/1).

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3.7.7.3 Linearised Circular (1:1) Path

The linearised circular path is a modified version of the loading protocol used by researchers in the testing of RC walls [116] and can be considered as a kind of conventional loading protocol that has been used in this study for the comparison of results with the proposed bidirectional loading protocols. The linearised circular (1:1) path consists of a quarter of a circle in each quadrant. Each quarter-circle starts and finishes at the origin. The column is first displaced in the first quadrant, followed by displacement in the third, second and fourth quadrants, respectively, before finishing back at the origin. After completion of one cycle of quarter circles, the specimen is subjected to the second cycle of quarter circles in all quadrants for capturing the strength and stiffness degradation under repeated displacement excursions, thereby resulting in a total of 8 quarter circles of loading per displacement increment. The linearised circular (1:1) path also imposes equal displacements in the two axes of the column and is shown in Figure 3.13 (c).

Figure 3.13. Proposed bidirectional loading protocols: a) octo-elliptical (0.6:1); b) octo-elliptical (1:1); c) linearised circular (1:1)

Figures 3.14 (a), 3.14 (b) and 3.14 (c) show the waveforms of X and Y displacements in octo-elliptical (0.6:1), octo-elliptical (1:1) and linearised circular (1:1) loading protocols, respectively. The Figures 3.14 (a) and 3.14 (b) clearly show the phase differences between X and Y displacements in the waveforms of octo-elliptical (0.6:1)
and octo-elliptical (1:1) paths, which result in four different orientations of the elliptical loops. On the other hand, there is a constant phase difference of 90 degrees in the X and Y displacements of the linearised circular (1:1) path.

![Waveforms of X and Y displacements for one complete counterclockwise cycle of loading protocol: a) octo-elliptical (0.6:1); b) octo-elliptical (1:1); c) linearised circular (1:1)](image)

**3.7.7.4 Mathematical formulation of Proposed Octo-Elliptical Loading Protocols**

The known parameters required for geometrically developing the proposed loading protocols for any given cycle are:

i) Drift in the y-direction for that particular cycle = \( y \)

ii) Aspect ratio (\( a/b \)) of the ellipses = 0.3

The octo-elliptical loading protocols begin with horizontal displacement from the origin to the starting point of the vertical ellipse (shown as a horizontal blue line in Figure 3.13 (a) and 3.13 (b)). The two known parameters i.e. \( y \) and \( a/b \) can be used to determine the x-coordinate of this displacement using \( x = 0.3y \).

Subsequently, X and Y coordinates of the vertical elliptical loop can be determined using equations (3.3) and (3.4) given below:

\[
X = xcos\theta \quad (3.3)
\]

\[
Y = ysin\theta \quad (3.4)
\]
The vertical ellipse is formulated using 60 points, with an angle step/increment of 6 degrees until one complete revolution of 360 degrees. The diagonal-1, horizontal and diagonal-2 ellipses are then obtained by rotating the vertical ellipse using equations (3.5) and (3.6) to determine the coordinates \((X_r, Y_r)\) of the other three orientations of the ellipse.

\[
X_r = X \cos \theta_r - a Y \sin \theta_r
\]  
\[
Y_r = X \sin \theta_r + a Y \cos \theta_r
\] (3.5) (3.6)

The values of \(a\) and \(\theta_r\) for the octo-elliptical (0.6:1) and octo-elliptical (1:1) loading protocols are provided in Table 3.7. It can be observed in Table 3.7 that the angle of rotation of ellipses falls in the same range as the results of the numerical study presented in Tables 3.5 and 3.6.

In the proposed loading protocols, a total of four transition arcs are provided for transition from one ellipse to another in any counter clockwise or clockwise cycle of ellipses. The coordinates \((X_t, Y_t)\) of each transition curve can be determined using equations (3.7) and (3.8) as follows:

\[
X_t = x \cos \theta'
\]  
\[
Y_t = x \sin \theta'
\] (3.7) (3.8)

Table 3.7. Parameters for mathematical formulation of the octo-elliptical (0.6:1) and octo-elliptical (1:1) loading protocols

<table>
<thead>
<tr>
<th>Loading Protocol</th>
<th>Ellipses</th>
<th>Ellipse Size Factor ((a))</th>
<th>Angle of Rotation ((\theta_r))</th>
<th>Angle of the Arc ((\theta'))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Octo-Elliptical (0.6:1)</td>
<td>Vertical</td>
<td>1</td>
<td>0</td>
<td>0-31 (in 5 steps)</td>
</tr>
<tr>
<td></td>
<td>Diagonal-1</td>
<td>0.8</td>
<td>31</td>
<td>31-90 (in 10 steps)</td>
</tr>
<tr>
<td></td>
<td>Horizontal</td>
<td>0.6</td>
<td>90</td>
<td>90-149 (in 10 steps)</td>
</tr>
<tr>
<td></td>
<td>Diagonal-2</td>
<td>0.8</td>
<td>149</td>
<td>149-180 (in 5 steps)</td>
</tr>
<tr>
<td>Octo-Elliptical (1:1)</td>
<td>Vertical</td>
<td>1</td>
<td>0</td>
<td>0-45 (in 8 steps)</td>
</tr>
<tr>
<td></td>
<td>Diagonal-1</td>
<td>1</td>
<td>45</td>
<td>45-90 (in 8 steps)</td>
</tr>
<tr>
<td></td>
<td>Horizontal</td>
<td>1</td>
<td>90</td>
<td>90-135 (in 8 steps)</td>
</tr>
<tr>
<td></td>
<td>Diagonal-2</td>
<td>1</td>
<td>135</td>
<td>135-180 (in 8 steps)</td>
</tr>
</tbody>
</table>
After determining the coordinates of ellipses and arcs of the counter clockwise drift cycle, the coordinates for the clockwise drift cycle can then be obtained by simply multiplying the y-coordinate of each ellipse and arc with -1.

3.7.8 Pattern of Axial Load Variation in Columns

The patterns of axial load variation in the corner perimeter and internal non-perimeter columns of the case study building were studied when subjected to ground motions in the three-axis in order to propose realistic axial load variation protocols. The results for the two type of columns are discussed herein.

Corner Perimeter Columns

The axial load variation in the corner perimeter columns of the case study building was studied for all DBE and MCE ground motions. The response history of axial load variation in the bottom storey column A1 for Christchurch ground motion (DBE) is shown in Figure 3.15, wherein the history of axial load variation is plotted with the history of drifts in the X and Y directions to understand the relation between the two. However, due to the large time range in the plot, the relation between axial load variation and lateral drifts is not very clear. The relationship was studied more closely by reducing the time range in the plots such that the regions of maximum variation in axial load were included only. The response history results for axial load variation and drifts in the X and Y-axis of the column for the reduced time range under Christchurch ground motion are presented in Figures 3.16 (a) and 3.16 (b). It can be observed that axial load variation is quite synchronous with the pattern of lateral drifts in the Y-direction, whereas there is a slight phase difference with lateral drifts in the X-direction. The synchronization with the pattern of lateral drifts is because axial load variation in the corner perimeter columns generally results from the push-pull effects due to the lateral forces induced by the horizontal components of the ground motions.

Similar behaviour was observed under other ground motions. The response histories for Dinar (1995) and Double Springs (1994) ground motions (DBE) for the time range with maximum variation in axial load are shown in Figures 3.16 (c), 3.16 (d), 3.16 (e) and 3.16 (f) for the purpose of illustration. The axial load variation under Dinar (1995)
ground motion was also found to be quite synchronous with the lateral drift history in the Y-direction and was slightly out of phase with the lateral drifts in the X-direction. However, under Double Springs (1994) ground motion, axial load variation was more synchronous to lateral drift history in the X-direction compared to the Y-direction.

The underlying reason behind this phenomenon is the energy content of displacements at their dominant frequency. If the energy content of X displacements at its dominant frequency is higher than the Y displacement, then axial load variation in the corner perimeter column will be more synchronous to lateral drifts in the X-direction. Otherwise, if the energy content of Y displacements at its dominant frequency is higher, then axial load variation will follow the sequence of drifts in the Y direction. This behaviour is explained in Figure 3.17 where the power spectral density of X and Y displacements is plotted with the frequency. It is noted that the power spectral density function (PSD) shows the energy content of a waveform at different frequencies.

Figure 3.15. Response history of axial load variation with lateral drifts in the ground storey corner perimeter column A1 for Christchurch (2011) ground motion (DBE): a) drift X; b) drift Y.
Figure 3.16. Response history of axial load variation with lateral drifts in the X and Y directions of bottom storey corner column A1 for DBE ground motions for the time range with maximum variation: a) & b) Christchurch (2011); c) & d) Dinar (1995); e) & f) Double Springs (1994).
Figure 3.17 shows that for Christchurch (2011) and Dinar (1995) ground motions, the power spectral density of Y displacements is more dominant than X displacements. This is why axial load variation follows the sequence of displacements in the Y-direction. On the other hand, for Double Springs (1994) ground motion, the power spectral density of X displacements is higher than Y and as a result, the axial load variation follows the pattern of displacements in the X-direction of the column. It is noted that there are two concentrations of energy in the Y displacements in Figure 3.17 (a), which are due to the higher mode effect, as the frequency of the building’s second mode in the y-direction is 2.85 Hz, and at this same frequency, the Y component of Christchurch (2011) ground motion has the maximum energy content as shown in Figure 3.18 (a), where the normalized power spectral density (normalized with respect to maximum energy) of the three components of Christchurch (2011) earthquake is shown. The synchronization of the two frequencies (i.e. the building’s second mode frequency and the ground motions dominant frequency) perhaps lead to the higher mode effect and that is why two different frequencies can be seen in the response history of drift-Y under Christchurch (2011) earthquake as indicated by Figure 3.15 (b). The lower frequency of the response history can be attributed to the first mode, whereas the higher frequency is to due to the participation of the second mode.

![Power Spectral Density](image)

Figure 3.17. Power spectral density of the corner column (A1) X and Y displacements: a) Christchurch (2011); b) Dinar (1995); c) Double Springs (1994)
Figure 3.18. Normalized power spectral density of ground motion accelerations: a) Christchurch (2011); b) Dinar (1995); c) Double Springs (1994)

**Internal Non-Perimeter Columns**

The relationship between axial load variation and lateral drifts of the internal non-perimeter column B3 for Christchurch (2011), Dinar (1995) and Double Springs (1994) ground motions (DBE) is shown in Figure 3.19 for the time range with maximum variation in axial load. It can be seen that for all the three ground motions, the axial load variation is totally nonsynchronous with the lateral drifts in the X and Y directions. This is because axial load variation in the internal non-perimeter columns is typically controlled by the vertical component of the ground motion as the effect of push-pull forces induced by the horizontal components of the ground motion is negligible on these columns. Figure 3.19 also shows that there are more cycles of variation in axial load compared to the cycles of lateral drifts, which is because the vertical component of the ground motion has a higher frequency content than the horizontal components, and therefore it results in more cycles of axial load variation in contrast with the number of cycles of lateral drifts. This is demonstrated in Figure 3.18, where it can be seen that the frequency content of the vertical ground motions is far higher than the horizontal components.
Figure 3.19. Response history of axial load variation with lateral drifts in the X and Y directions of the bottom storey internal non-perimeter column B3 for DBE ground motions for the time range with maximum variation: a) & b) Christchurch (2011); c) & d) Dinar (1995); e) & f) Double Springs (1994)
3.7.9 Proposed Axial Load Variation Protocols

The results of the previous section suggest that axial load variation in corner perimeter columns is typically synchronous to the lateral displacement of the building, whereas axial load variation in the internal non-perimeter columns is nonsynchronous to the lateral displacement and has a higher frequency, which is dependent on the frequency of the vertical component of the ground motion. Two loading protocols namely, synchronous and nonsynchronous axial load variation protocols are proposed accordingly. The details of the proposed protocols are presented herein.

3.7.9.1 Synchronous Axial Load Variation Protocol

The variation in axial load is synchronous with the variation of lateral displacement in the strong direction of the column in the proposed synchronous axial load variation protocol. The synchronous axial load variation pattern can be generated by normalizing the strong direction (Y-direction for the case presented here) displacement in each ellipse of the octo-elliptical (0.6:1) or octo-elliptical (1:1) path with the maximum displacement in that particular loop, and subsequently, taking its product with the design axial load and factor $v_n$ that accounts for percentage variation in axial load. For a given displacement history, the synchronous axial load variation pattern, $Z_s$ can be obtained using the following expression:

$$Z_s = Z + \left( \frac{Y_r}{Y_{r_{max}}} \right) \times \frac{v_n}{100} \times Z$$  \hspace{1cm} (3.9)

Figure 3.20 shows a sample response history for one complete cycle of the synchronous axial load variation protocol for specimen S13 in Table 3.1. The pattern generated herein is for the design axial load of $Z = 980$ kN, with the percentage variation in the axial load of $v_n = 30\%$. As such the axial load can be seen to be oscillating between the maximum and minimum values of 1270 kN and 690 kN, respectively. The displacements in Figure 3.20 are from the octo-elliptical (0.6:1) path, with the maximum values of ±3.9 and ±6.5 mm in the X and Y directions, respectively. Figure 3.20 (a) shows that the axial load is maximum (1270 kN) when the lateral displacement is maximum in the positive Y-direction (6.5 mm) and minimum (690 kN) when the
lateral displacement is maximum in the negative Y-direction (-6.5 mm). On the other hand, due to the phase shift in the X and Y displacements, the axial load variation is slightly nonsynchronous with the maximum and minimum displacements in the X-direction as shown in Figure 3.20 (b).

Figure 3.20. Response history of proposed synchronous axial load variation protocol: a) Y-direction; b) X-direction.

3.7.9.2 Nonsynchronous Axial Load Variation Protocol

In the proposed nonsynchronous axial load variation protocol, the variation is nonsynchronous to the lateral displacement of the building and has a higher frequency i.e. two cycles of axial load variation per cycle of lateral displacement. The following expression can be used to obtain the nonsynchronous axial load variation protocol:

$$Z_{NS} = Z + \left(2 \times \left(\frac{Y_f}{Y_{max}}\right)^2 - 1\right) \times \frac{v_n}{100} \times Z$$  \hspace{1cm} (3.10)

Figure 3.21 shows a sample response history of nonsynchronous axial load variation for the specimen S14 in Table 3.1. The design axial load of the specimen is $Z = 956 \text{ kN}$, with the percentage variation in the axial load of $v_n = 30\%$. As such the axial load can be seen to be oscillating between the maximum and minimum values of 1243 kN and 669 kN, respectively, whereas the displacements have again the same maximum values of ±3.9 and ±6.5 mm in the X and Y-directions, respectively. It can be seen in Figure 3.21
(a) that under nonsynchronous loading protocol, the axial load reaches its maximum value (1243 kN) whenever the column is pushed to its maximum amplitude of displacement (6.5 mm) either in the positive or negative Y-direction and minimum value (669 kN) when the column is at the origin (i.e. stationary). This is in contrast with the synchronous loading protocol in which the column was subjected to maximum axial load when maximum amplitude of displacement was attained in the negative Y direction and minimum axial load when the amplitude of displacement was maximum in the positive Y direction. It is noted that in the X direction, under nonsynchronous loading protocol, the axial load ratio is mostly at its minimum value when the displacement is maximum in either direction and is mostly maximum when the column is at the origin as indicated by Figure 3.21 (b).

Figure 3.21. Response history of proposed nonsynchronous axial load variation protocol: a) Y-direction; b) X-direction.

### 3.8 Summary and Conclusions

This chapter presented the details of a comprehensive experimental testing program comprising of 14 full-scale column specimens designed to investigate the collapse behaviour of limited ductile high-strength RC columns under multidirectional earthquake actions. The test matrix, specimen design, test setup and instrumentation have been described in detail. The chapter also provided details of the bidirectional
loading protocols and axial load variation protocols that were developed through robust statistical processing of the displacement path of the columns observed in the numerical study conducted on a case study RC wall-frame building subjected to ground motions representative of low to moderate seismic regions. The bidirectional loading protocols developed are referred to as Octo-Elliptical (0.6:1) path and Octo-Elliptical (1:1) path and the axial load variation protocols developed are synchronous axial load variation and nonsynchronous axial load variation protocols. The contents of this chapter lay the basis for the presentation of the results of the experimental testing program in the upcoming chapters.
Chapter 4 Experimental Testing under Unidirectional Lateral Actions and Constant Axial Load

4.1 Introduction

In the previous chapter, a detailed description of the experimental testing program, including the test matrix, specimen design, test setup, instrumentation and loading protocols was provided. This chapter presents the results of the first part of the testing program in which tests were conducted on limited ductile HSRC columns under unidirectional lateral loading and CAL. A detailed description of the failure mechanisms, force-displacement behaviour, moment-curvature behaviour and axial displacement-lateral drift behaviour of the specimens is provided in this chapter. Energy dissipation and stiffness degradation of the specimens is also discussed and compared. The chapter concludes with a summary of the main findings from experimental testing under unidirectional lateral actions and CAL.

4.2 Test Specimens

As discussed in the previous chapter, six specimens were tested under unidirectional lateral loading and CAL. The variables considered were axial load ratio, transverse reinforcement ratio and concrete compressive strength. The specimens were tested under four different axial load ratios i.e. $n = \frac{P}{f'_c A_g} = 0.15, 0.25, 0.3$ and $0.45$ to see the effect of the axial load ratio on the collapse behaviour. The effect of the transverse reinforcement was studied by considering three transverse reinforcement configurations i.e. AS 3600 code compliant, under-confined and over-confined. Four of the specimens were provided with code-compliant transverse reinforcement, whereas one specimen was under-confined, with ties two times as widely spaced as the code-compliant case. The over-confined specimen, on the other hand, had transverse
reinforcement 1.5 times as closely spaced as the code-compliant case. The specimens were cast with two concrete grades i.e. 65 MPa and 100 MPa to evaluate the effect of the concrete compressive strength on the behaviour of the specimens. Table 4.1 presents the design details of the specimens. The reinforcement detailing of the specimen can be seen in Figure 3.1 (a) of the previous chapter. The specimens were tested under unidirectional lateral loading protocol in the Y-direction described in section 3.6 and shown in Figure 3.4.

Table 4.1. Design details of the column specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Width × Depth × Height (mm)</th>
<th>Concrete Strength (MPa)</th>
<th>Longitudinal Reinforcement $\rho_v$ (%)</th>
<th>Ties (mm) $\rho_h$ (%)</th>
<th>Axial Load Ratio $n_P$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1 (Compliant)</td>
<td>250×300 ×2550</td>
<td>65</td>
<td>75</td>
<td>6-N16 (1.6%)</td>
<td>N10@150 (0.35%)</td>
</tr>
<tr>
<td>S2 (Compliant)</td>
<td>250×300 ×2550</td>
<td>65</td>
<td>66</td>
<td>6-N16 (1.6%)</td>
<td>N10@150 (0.35%)</td>
</tr>
<tr>
<td>S3 (Compliant)</td>
<td>250×300 ×2550</td>
<td>65</td>
<td>87</td>
<td>6-N16 (1.6%)</td>
<td>N10@150 (0.35%)</td>
</tr>
<tr>
<td>S4 (Over-confined)</td>
<td>250×300 ×2550</td>
<td>65</td>
<td>90</td>
<td>6-N16 (1.6%)</td>
<td>N10@100 (0.52%)</td>
</tr>
<tr>
<td>S5 (Under-confined)</td>
<td>250×300 ×2550</td>
<td>65</td>
<td>62</td>
<td>6-N16 (1.6%)</td>
<td>N10@300 (0.18%)</td>
</tr>
<tr>
<td>S6 (Compliant)</td>
<td>250×300 ×2550</td>
<td>100</td>
<td>90</td>
<td>6-N16 (1.6%)</td>
<td>N10@150 (0.35%)</td>
</tr>
</tbody>
</table>
4.3 Experimental Test Results and Discussion

This section provides a detailed description and analysis of the results of the aforementioned specimens tested under unidirectional lateral loading and CAL. The results are discussed in terms of general response and failure modes, force-displacement behaviour, moment-curvature behaviour, axial displacement-lateral drift behaviour, energy dissipation and stiffness degradation.

4.3.1 General Response and Failure Mechanisms

All specimens failed in a predominantly flexure mode and were able to reach their theoretical flexural moment capacity before strength degradation and overall failure. Formation of horizontal flexure cracks was initiated at small displacement excursions followed by the development of vertical splitting cracks at large displacements, which subsequently led to spalling of the concrete cover at the edges of the column in the plastic hinge region. Collins et al. [117] also noted that vertical splitting cracks led to spalling of the concrete cover and subsequent reduction in the lateral load-carrying capacity of the HSRC columns. The extensive spalling of the concrete cover was followed by the buckling and fracture of the longitudinal and transverse reinforcement, respectively, ultimately resulting in axial load failure of the column by the formation of a diagonal plane. The failure zone for all the six specimens was observed to be the top plastic hinge region. However, damage and significant plastic deformation were also observed at the bottom plastic hinge region.

ASCE/SEI 41-13 [5] defines the predominant failure mode of the column on the basis of the ratio of plastic shear demand to the shear capacity (i.e. $V_p/V_o$) of the column. According to this criteria, a column is flexure-critical if $V_p/V_o \leq 0.6$, flexure-shear critical if $0.6 < V_p/V_o \leq 1.0$ and shear-critical if $V_p/V_o > 1.0$. The plastic shear demand can be determined by dividing the maximum moment capacity of the column cross-section with the shear span length of the column, whereas the shear strength can be determined from available shear strength models, such as Sezen and Moehle [6] given by equation 4.1 below.

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\[ V_n = k_1 V_o = k_1 \left[ \frac{A_{wt} f_{yh} d}{s} + \lambda \left( 0.5 \sqrt{f'_c} \left( 1 + \frac{P}{0.5 \sqrt{f'_c A_g}} \right) \right) 0.8 A_g \right] \] (4.1)

The expected failure modes of the tested specimens according to the classification criteria of ASCE/SEI 41-13 are provided in Table 4.2, which indicates that all specimens except S5 are flexure-critical. Specimen S5 has been classified as flexure-shear critical by ASCE/SEI 41-13 because of the wide spacing of the transverse reinforcement. However, it is worth mentioning that in the actual experimental test, this specimen like other specimens did not exhibit significant diagonal shear cracking and showed a predominantly flexural failure mode, perhaps due to the high aspect ratio of the specimen and also because according to ASCE/SEI 41-13, their classification approach is conservative particularly for columns with \(0.6 < V_p / V_n < 0.7\). Such columns, in reality, may exhibit flexure failures without shear degradation but may have been classified as flexure-shear critical for the sake of conservatism.

Table 4.2. Predicted failure mode according to the classification of ASCE/SEI 41-13

<table>
<thead>
<tr>
<th>Specimen</th>
<th>(V_p / V_n)</th>
<th>Type of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>0.45</td>
<td>Flexure</td>
</tr>
<tr>
<td>S2</td>
<td>0.46</td>
<td>Flexure</td>
</tr>
<tr>
<td>S3</td>
<td>0.43</td>
<td>Flexure</td>
</tr>
<tr>
<td>S4</td>
<td>0.38</td>
<td>Flexure</td>
</tr>
<tr>
<td>S5</td>
<td>0.65</td>
<td>Flexure-Shear</td>
</tr>
<tr>
<td>S6</td>
<td>0.55</td>
<td>Flexure</td>
</tr>
</tbody>
</table>

The vertical splitting cracks were found to be greater in number and size for the specimens with higher axial load ratio, less lateral confinement, and higher concrete mean cylinder strength. For instance, specimens S3 and S4 (i.e. \(n = 0.45\)) developed a larger number of vertical splitting cracks at very small displacement excursions and collapsed very suddenly without any noticeable concrete spalling. The under-confined specimen S5 (i.e. \(\rho_h = 0.18\%\)), also showed extensive vertical splitting before losing its axial load carrying capacity. Due to the high concrete compressive strength, specimen S6 (i.e. \(f_{cm} = 90\) MPa) exhibited more vertical splitting cracks than S2 despite being
tested at a slightly lower axial load ratio. The damage pattern of all the specimens at collapse is shown in Figure 4.1. It can be observed that most of the damage was concentrated in the top and bottom plastic hinge regions of the specimens. The drift limits marking the initiation of visible cracking, concrete spalling, lateral load failure (20% degradation in lateral strength) and collapse (axial load failure) are summarised in Table 4.3. It is noted that transverse bar fracture and longitudinal bar buckling occurred almost simultaneously with axial load failure (collapse) of the specimens. It can be observed in Table 4.3 that despite reasonable core confinement (particularly for specimen S4), concrete cover spalling and vertical bar buckling of specimens S3 and S4 occurred simultaneously, which is because these columns were supporting a very high axial load of 3000 kN corresponding to an axial load ratio of 0.45.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Visible Cracking (%)</th>
<th>Concrete spalling (%)</th>
<th>Lateral Load Failure (%)</th>
<th>Collapse (Axial Load Failure)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>0.9</td>
<td>2.4</td>
<td>3.0</td>
<td>4.8</td>
</tr>
<tr>
<td>S2</td>
<td>0.7</td>
<td>1.9</td>
<td>2.2</td>
<td>3.1</td>
</tr>
<tr>
<td>S3</td>
<td>0.5</td>
<td>1.1</td>
<td>1.0</td>
<td>1.1</td>
</tr>
<tr>
<td>S4</td>
<td>0.6</td>
<td>1.4</td>
<td>1.1</td>
<td>1.4</td>
</tr>
<tr>
<td>S5</td>
<td>0.5</td>
<td>1.4</td>
<td>1.8</td>
<td>1.9</td>
</tr>
<tr>
<td>S6</td>
<td>0.8</td>
<td>1.5</td>
<td>1.9</td>
<td>2.7</td>
</tr>
</tbody>
</table>

4.3.2 Force-Displacement Behaviour

The force-displacement hysteresis plots of the six specimens are shown in Figure 4.2. It is noted that the P-Delta effects have been incorporated in the hysteretic plots. Lateral load failure and axial load failure points are also marked on the hysteresis. Lateral load failure is the point corresponding to 20% reduction in the lateral strength of the column, whereas axial load failure is the point when the column is no longer able to support the initially applied axial load and the drift at this point is referred to as axial
load failure drift or the collapse drift. The effects of axial load ratio, amount of lateral confinement and concrete compressive strength on the hysteretic behaviour of the tested specimens are discussed below.

4.3.2.1 Effect of Axial Load Ratio

A comparison of the hysteretic curves of the specimens S1, S2 and S3 indicate a remarkable decline in the drift capacity of the column with the increase in axial load ratio. The lateral load failure and axial load failure drifts of the column reduced by around 35% with a corresponding increase in the axial load ratio by two times i.e. from \( n = 0.15 \) to \( n = 0.3 \). Likewise, a substantial decrement of around 75% is observed in the failure drifts of the column with the increase in axial load ratio by three times i.e. from \( n = 0.15 \) to \( n = 0.45 \). It is also noted that the collapse drift capacity of the code-compliant HSRC columns with \( n = 0.45 \) is very low i.e. in the order of 1.0%. This implies that columns supporting axial load ratios of this level could be quite prone to collapse in a rare or very rare earthquake event in low to moderate seismic regions. Moreover, the degradation in the lateral strength of the column is more rapid at higher axial load ratios, especially for \( n = 0.45 \). It can be observed in Figure 4.3 that the difference, as well as the ratio between the axial load failure and lateral load failure drifts, reduces with the increase in the axial load ratio of the column. Moreover, the two failures can be seen to be occurring almost simultaneously at \( n = 0.45 \). This implies that the collapse of a heavily loaded column occurs suddenly after the commencement of the degradation in the lateral strength.

The drift capacity at the limit states of the lateral load failure and axial load failure of the specimens in the positive and the negative directions are summarised in Table 4.4. It can be seen that the drift capacity is symmetric in the positive and the negative directions for all the specimens with the exception of specimen S2, which was able to sustain one cycle of 3.8% drift in the positive direction and collapsed midway while going to the same drift in the negative direction. This resulted in an unsymmetrical hysteresis of the specimen.
The lateral force capacity of the specimens S1 and S3 in Figures 4.2 (a) and 4.2 (c) is unsymmetrical in the positive and negative directions. This is because of a large magnitude positive moment produced at the beginning of the test due to the eccentricity of the axial load while the specimen was being loaded in the vertical direction.

![Images of specimens S1 to S6 with damage at axial load failure (collapse)]

Figure 4.1. Damage at axial load failure (collapse) - specimens S1 – S6
Figure 4.2. Force-drift response in the Y-direction of the specimens S1 – S6
Table 4.4. Drifts at lateral load failure and axial load failure

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Lateral Load Failure Drift (%)</th>
<th>Axial Load Failure/Collapse Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Positive</td>
<td>Negative</td>
</tr>
<tr>
<td>S1</td>
<td>2.8</td>
<td>-3.1</td>
</tr>
<tr>
<td>S2</td>
<td>2.3</td>
<td>-2.0</td>
</tr>
<tr>
<td>S3</td>
<td>0.8</td>
<td>-1.1</td>
</tr>
<tr>
<td>S4</td>
<td>1.1</td>
<td>-1.1</td>
</tr>
<tr>
<td>S5</td>
<td>1.8</td>
<td>-1.7</td>
</tr>
<tr>
<td>S6</td>
<td>2.0</td>
<td>-1.7</td>
</tr>
</tbody>
</table>

4.3.2.2 Effect of Lateral Confinement

The lateral load and axial load failure drift capacities of the code-compliant specimen S2 were more than the corresponding drift capacities of the under-confined specimen S5 by 16% and 40%, respectively, at the same axial load ratio \( n = 0.3 \). It can be seen in Figures 4.2 (e) and 4.3 (b) that the lateral load and axial load failure of the specimen S5 occurred almost simultaneously with a sharp degradation of the lateral strength. This implies that specimens with an inadequate amount of transverse reinforcement lost their axial load carrying capacity at around 20% degradation of the lateral strength. The comparison of the hysteretic curves of the code-compliant specimen S3 and the over-confined specimen S4 (with \( n = 0.45 \), as shown in Figures 4.2 (c), 4.2 (d) and 4.3 (c), indicates that the drift capacity of the over-confined specimen only increased marginally from 1.1% to 1.4% despite the increase in the amount of transverse reinforcement by around 50%. The relatively less increase in the drift capacity is due to the reduction in the effectiveness of the transverse reinforcement in confining the concrete core at high axial load ratios. A rapid strength degradation similar to the one observed for the code-compliant specimen can also be seen for the over-confined specimen in Figure 4.2 (d).
4.3.2.3 Effect of Concrete Compressive Strength

The hysteretic behaviour of the code-compliant specimens, S2 and S6 in Figure 4.2 (b) and Figure 4.2 (f) can be compared to investigate the effect of concrete compressive strength on the drift capacity of the RC columns. Specimen S6 had a lower axial load failure drift capacity (2.7%) than S2 (3.8%) in the positive direction despite being tested at a slightly lower axial load ratio i.e. \( n = 0.25 \) as compared to \( n = 0.3 \) at which specimen S2 is tested. It is noted, however, that the specimens have almost identical drift capacities in the negative direction. The smaller positive drift capacity of the specimen S6 is due to the higher concrete compressive strength, i.e. 90 MPa, in contrast to 66 MPa of the specimen S2. This reduction in the drift capacity with the increasing concrete strength can be visually observed in Figure 4.3 (d). The brittle behaviour of
the concrete at higher strength is also evident from the sharp degradation in the lateral strength of the specimen S6 in Figure 4.2 (f), which is in contrast with the relatively gradual strength degradation observed for specimen S2 in Figure 4.2 (b).

### 4.3.3 Moment Curvature Behaviour

The moment-curvature hysteretic behaviour at the top of the column specimens is shown in Figure 4.4 for the X-axis. The moment herein is the combination of the moment due to the lateral loading and secondary moment resulting from the P-Delta effects and the curvature is determined by dividing the difference of the strains obtained from the pair of the LVDTs located in the top plastic hinge region with the horizontal distance between the LVDTs. The displacement measurements from the LVDTs are used to determine strains by dividing these measurements with the gauge length of the LVDT. Due to excessive spalling, the LVDTs were removed before the axial load failure/collapse of the specimens.

The moment-curvature hysteretic curves indicate a remarkable decline in the ductility and energy absorption of the column with the increase in the axial load ratio. Also, a significant reduction in the curvature of the column can be noticed with the increase in the axial load ratio. The curvature of the column corresponding to ultimate (maximum) moment reduced by around 50% with the increase in axial load ratio from $n = 0.15$ to 0.3. Similarly, a reduction of about 75% is observed with the increase in axial load ratio from $n = 0.15$ to 0.45. Not much difference was observed in the curvature (at ultimate moment) of the over-confined specimen S4 and the code-compliant specimen S3 tested at the same axial load ratios ($n = 0.45$). Similarly, the curvatures of the under-confined and the code-compliant specimens at ultimate moment were also the same at $n = 0.3$, implying that the increase or decrease in the amount of confinement does not have much effect on the curvature at ultimate moment. It is also noted that despite being tested at a slightly lower axial load ratio, the curvature of specimen S6 ($n = 0.25$) at the ultimate moment is equal to that of S2 ($n = 0.3$), which is due to the higher concrete strength of S6 that nullified the effect of the lower axial load ratio, thereby resulting in a similar curvature to S2. As is the case with the force-displacement hysteresis, the
strength and stiffness degradation in the moment-curvature hysteresis is also different in the positive and negative directions for specimen S3 because of the positive moment produced from the eccentricity of the axial load at the beginning of the test.

Figure 4.4. Moment-curvature response in the X-axis of the specimens S1 – S6

4.3.4 Axial Displacement Lateral Drift Behaviour

The axial displacement of the test specimens was recorded using SPOTs attached at each of the four corners of the top pedestal. The axial displacement was taken as the average of the displacements of the four SPOTs. The relationship between axial displacement and lateral drift of the specimens under cyclic actions is shown in Figure 4.5. The points of ultimate capacity and lateral load failure of the specimens are marked in Figure 4.5 for all specimens except specimen S5, as transducers of this specimen, were removed before lateral load failure due to excessive spalling and damage to the specimen. Compared to other specimens, a large amount of axial shortening occurred.
at the very beginning of the test for specimens S3 and S4 that were tested at higher axial load ratio (n = 0.45). Figures 4.5 (c) and 4.5 (d) show that these two specimens underwent an axial displacement of around 5 mm during ramping of the axial load and were able to sustain a further axial displacement of 2 mm only when subjected to cyclic lateral actions. It is also worth noting that specimen S1 due to its low axial load ratio (n = 0.15) exhibited both axial shortening and axial elongation behaviour, whilst the rest of the specimens experienced axial shortening only.

Figure 4.5 indicates that for low to medium axial load ratios, the axial displacement of the specimens (S1, S2, S5 and S6) did not increase much with increasing lateral displacements before ultimate strength of the column was reached. However, axial displacement started to increase at a rapid rate with the increasing cyclic displacements after degradation in the lateral strength of the column commenced and
reached the point of lateral load failure. On the other hand, the axial displacement of the heavily loaded columns (S3 and S4) began to increase rapidly with increasing cyclic displacements before the ultimate strength of the column was reached.

4.3.5 Energy Dissipation

Energy dissipation was determined by calculating the area enclosed by the force-displacement hysteresis. The energy dissipation in each individual cycle can be given by:

$$ E_a = \int F(y)dy $$

The increase in the individual cycle energy with the increasing lateral drifts is shown in Figure 4.6 (a). Here the energy dissipation has been plotted at the end of the second cycle of each displacement increment. It can be observed that the hysteretic energy dissipation increased with increasing lateral drifts as a general trend. Therefore, specimens with a relatively higher drift capacity were able to dissipate more energy as compared to the ones with a lower drift capacity. It is also noted that the energy dissipation during the second repetitive cycle of the displacement excursion was less as compared to the first cycle due to the strength degradation and reduction in the hysteretic area. The decrease in the dissipated energy during the second cycle of a displacement increment was observed to be a function of the applied axial load ratio i.e. the higher the axial load ratio, the greater the strength degradation and consequently lesser energy dissipation due to the reduction in the hysteretic area. Figure 4.6 (a) also shows that columns with lower axial load ratios have a tendency to dissipate the same amount of energy as heavily loaded columns at larger drift levels than the latter. Conversely, heavily loaded columns dissipate the same amount of energy as lightly loaded columns at smaller drifts.

The total hysteretic energy dissipation of the specimens, which is essentially the sum of the energy dissipation of all individual cycles until collapse has been compared in Figure 4.6 (b). The total energy dissipation of the specimens reduced significantly with the increase in axial load ratio. For instance, the energy dissipation of specimen S1 ($n =$
was approximately 1.3 and 6 times of the energy dissipation of the specimens S2 \((n = 0.3)\) and S3 \((n = 0.45)\), respectively. Likewise, lateral confinement also tends to influence the energy dissipation of the specimens. The energy dissipation of the code-compliant specimen S2 at collapse was around 2.6 times of the corresponding energy dissipation of the under-confined specimen S5 at the same axial load ratio \((n = 0.3)\). However, lateral confinement does not seem to increase the energy dissipation much at higher axial load ratios i.e. the energy dissipation of the over-confined specimen S4 was just 1.6 times the energy dissipation of the code-compliant specimen S3 at \(n = 0.45\).

Lastly, high concrete strength also seems to reduce the energy dissipation capacity of an RC column. This can be concluded by comparing the total energy dissipation of the specimens S2 and S6. Specimen S6 despite being tested at a slightly lower axial load ratio has less total energy dissipation than the specimen S2.

![Image](image.png)

**Figure 4.6.** Energy dissipation (a) individual cycle hysteretic energy dissipation with lateral drifts (b) total hysteretic energy dissipation

### 4.3.6 Effective Moment of Inertia and Stiffness Degradation

In this study, cyclic stiffness \((K_i)\) of the column specimen is defined as the average stiffness in the positive and negative directions during a cycle and is computed as follows:

\[
K_i = \frac{|F_i| + |F_i|}{|\Delta_i| + |\Delta_i|}
\]

(4.3)
The effective moment of inertia of the specimens was determined from the experimental stiffness of the column. The effective moment of inertia \( I_{eff} \) for each specimen was calculated based on the gradient of a line taken from the origin through the drift corresponding to the development of the theoretical yield capacity. The theoretical yield capacity was taken as the lateral force corresponding to the development of a theoretical yield moment, which was taken as the moment corresponding to the development of a compressive strain corresponding to the maximum unconfined compressive strength (calculated using the Karthik and Mander model [118]) being reached in the extreme compressive fibre of the concrete or tensile yielding of the reinforcement, whichever occurred first. The theoretical yield capacity was calculated using the actual material properties (i.e. for steel and concrete) for the respective specimens.

The ratio of effective moment of inertia \( I_{eff} \) to the gross moment of inertia \( I_g \) of the column specimen was then compared with the predictions of the existing models such as Paulay and Priestley [119], Elwood and Eberhard [120], ACI 318 [17], AS 3600 [14] and ASCE/SEI 41-17 [121] in Table 4.5. The \( I_{eff}/I_g \) expressions of these respective models are as follows:

Paulay and Priestley (1992):

\[
\frac{I_{eff}}{I_g} = \frac{100}{f_{yl}} + n
\]  
(4.4a)

Elwood and Eberhard (2009):

\[
\frac{I_{eff}}{I_g} = \left( \frac{0.45 + 2.5n}{1 + 110 \frac{d_b}{L}} + n \right) \leq 1 \text{ and } \geq 0.2
\]  
(4.4b)

ACI 318:

\[
\frac{I_{eff}}{I_g} = \left( 0.8 + \frac{25A_{st}}{A_g} \right) \left( 1 - \frac{M}{P_u h} - \frac{0.5P_k}{P_o} \right) \leq 0.875 \text{ and } \geq 0.35
\]  
(4.4c)
AS 3600:

\[
\frac{I_{\text{eff}}}{I_g} = 0.8 \text{ for } n \geq 0.5
\]  
(4.4d)

\[
\frac{I_{\text{eff}}}{I_g} = 0.5 \text{ for } n = 0.2
\]  
(4.4e)

\[
\frac{I_{\text{eff}}}{I_g} = 0.3 \text{ for } n = 0
\]  
(4.4f)

For axial load ratios in-between, the value of \( I_{\text{eff}}/I_g \) should be interpolated.

ASCE/SEI 41-17:

\[
\frac{I_{\text{eff}}}{I_g} = 0.7 \text{ for } n \geq 0.5
\]  
(4.4g)

\[
\frac{I_{\text{eff}}}{I_g} = 0.3 \text{ for } n \leq 0.1
\]  
(4.4h)

For \( 0.1 \leq n \leq 0.5 \), the value of \( I_{\text{eff}}/I_g \) should be interpolated.

It can be observed in Table 4.5 that all the considered models i.e. ASCE/SEI 41-17, Paulay and Priestley (1992), ACI 318, Elwood and Eberhard (2009) and AS 3600, respectively, have a low coefficient of variation and as such, are recommended when assessing high strength RC columns with high axial load ratios. It is noted that the coefficient of variation in Table 4.5 is defined as the ratio of root mean squared error (RMSE) to the mean of the dependent variable and provides a comparison between experimental and estimated values by various models.

Figure 4.7 delineates the progression of stiffness degradation of the specimens with the increasing lateral drift levels. The column stiffness in each cycle has been normalized with respect to the effective yield stiffness and gross stiffness in Figures 4.7 (a) and 4.7 (b), respectively. It can be seen that the slope of stiffness degradation curve for all specimens is very steep in the beginning implying that stiffness degradation occurs at
a much rapid pace at smaller displacement excursions compared to larger displacements. It can also be observed that for a given drift level, stiffness degradation is greater in heavily loaded \((n = 0.45)\) columns as compared to lightly loaded columns \((n = 0.15 \text{ to } 0.3)\). Moreover, it is noted that column specimens with higher axial load ratios, higher concrete compressive strength and less lateral confinement collapse with higher normalized stiffness than other specimens. In other words, the stiffness of such columns degrades less before the collapse. For instance, the normalized stiffness \((K_i/K_g)\) of the specimen S1 \((n = 0.15)\) degraded to 0.05 before collapse, whereas specimen S3 \((n = 0.45)\) collapsed when its normalized stiffness \((K_{ix}/K_{gx})\) was much higher, i.e. 0.3. Similarly, stiffness degradation due to the repetition of the cycles was more pronounced in column specimens with high axial load ratios as compared to others.

Table 4.5. Comparison of the experimental effective moment of inertia with existing models

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ratio of effective to gross moment of inertia (I_{eff,x}/I_{gx})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Experimental</td>
</tr>
<tr>
<td>S1</td>
<td>0.36</td>
</tr>
<tr>
<td>S2</td>
<td>0.50</td>
</tr>
<tr>
<td>S3</td>
<td>0.63</td>
</tr>
<tr>
<td>S4</td>
<td>0.61</td>
</tr>
<tr>
<td>S5</td>
<td>0.53</td>
</tr>
<tr>
<td>S6</td>
<td>0.49</td>
</tr>
<tr>
<td>COV</td>
<td>0.07</td>
</tr>
</tbody>
</table>
Figure 4.7. Stiffness degradation of specimens with increasing lateral drifts a) ratio of stiffness in each cycle to the effective stiffness b) ratio of stiffness in each cycle to the gross stiffness

4.4 Summary and Conclusions

This chapter addressed the first research gap of the study by experimentally investigating the collapse behaviour of limited ductile HSRC columns, representative of construction practice in Australia and other regions of low to moderate seismicity, under unidirectional lateral actions and CAL. The variables assessed included the axial load ratio of the column, the amount of lateral confinement reinforcement and concrete compressive strength. The following conclusions can be drawn from the experimental study:

1. The axial load ratio of the column drastically affects the collapse drift capacity of HSRC columns with limited ductile detailing. Doubling the axial load ratio from $n=0.15$ to 0.3 reduced the collapse drift capacity by around 35%, whereas tripling the axial load ratio to 0.45 reduced the drift capacity by around 75%.

2. The heavily loaded column with $n=0.45$ exhibited a very low collapse drift capacity of around 1.0%, which would result in a high level of vulnerability to collapse in a rare (return period 500 years) or very rare (return period 2500 years) earthquake event. Hence, it is recommended that high-strength RC columns with detailing representative of Australian construction practice should be designed for $n \leq 0.4$ under uni-directional lateral actions to allow for a drift capacity of at least 1.5%.
3. An increase in axial load ratio accelerates the strength and stiffness degradation while simultaneously reducing the hysteretic energy dissipation. This results in axial load failure of the column occurring very quickly and suddenly after the maximum strength is exceeded and lateral load failure has occurred.

4. The stiffness degradation of the lightly loaded column specimens for a given drift level was observed to be higher than the corresponding heavily loaded columns. The comparison of the experimental effective moment of inertia with the predictions of different standards and models indicated that the predictions of all the considered models i.e. ASCE/SEI 41-17, Paulay and Priestley (1992), ACI 318, Elwood and Eberhard (2009) and AS 3600, respectively, were very close to the experimental results. Hence, all of these models can be deemed appropriate for estimating the effective moment of inertia of HSRC columns at the design stage.

5. Lateral confinement plays a vital role in improving the drift capacity, strength and stiffness degradation and energy dissipation of the columns at low to moderate axial load ratios; however, its effectiveness reduces at higher axial load ratios for high-strength concrete, as the collapse drift capacity just increased 20%, with 50% increase in the amount of transverse reinforcement. Similarly, high concrete strengths at high axial load ratios reduce the drift and energy dissipation, accelerate stiffness degradation and result in sudden axial load failure of the column very soon after the maximum strength is exceeded.
Chapter 5 Experimental Testing under Bidirectional Lateral Actions and Constant Axial Load

5.1 Introduction

The previous chapter investigated the collapse behaviour of limited ductile HSRC columns under unidirectional lateral actions and CAL. However, earthquake ground shaking is multidirectional in reality and results in the bidirectional displacement of columns in RC buildings. The bidirectional displacement of columns can be due to concurrent bidirectional ground motions from an earthquake, or from the irregularities in the building’s lateral load resisting system, which can induce the torsional response of the building structure. The majority of RC columns in the literature have been tested under unidirectional loading protocols, as opposed to more realistic bidirectional loading protocols. A review of existing literature in chapter 2 revealed that only 4 HSRC column specimens have been tested under bidirectional lateral loading so far. Furthermore, the tested columns were representative of moderately to fully ductile detailing practices in regions of higher seismicity.

The aim of this chapter is to experimentally investigate the bidirectional collapse behaviour of limited ductile HSRC columns commonly constructed in regions of low to moderate seismicity. The results of the experimental program are discussed in detail in terms of the general behaviour and failure modes, force-displacement behaviour, moment-curvature behaviour, energy dissipation and stiffness degradation. A detailed comparison with the corresponding unidirectional response of the identical columns tested in chapter 4 is also provided.
5.2 Test Specimens

The experimental program comprised of six HSRC columns shown in Table 5.1. The longitudinal reinforcement consisted of 6-N16 bars ($\rho_v=1.6\%$), whereas the specimens were provided with code-compliant AS 3600 [14] limited ductile transverse reinforcement detailing representative of typical Australian construction practice. More details about specimen design can be found in section 3.3 of the thesis.

Table 5.1. Design details of the column specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Width × Depth × Height (mm)</th>
<th>Concrete Cylinder Strength $f_{cm}$ (MPa)</th>
<th>Transverse Reinforcement Ratio $\rho_{hx}$ (%)</th>
<th>$\rho_{hy}$ (%)</th>
<th>Axial Load Ratio $n$ ($P$, kN)</th>
<th>Type of Loading Path</th>
</tr>
</thead>
<tbody>
<tr>
<td>S7</td>
<td>250×300 × 2550</td>
<td>86</td>
<td>0.42</td>
<td>0.35</td>
<td>0.15 (968)</td>
<td>Linearised Circular (1:1)</td>
</tr>
<tr>
<td>S8</td>
<td>250×300 × 2550</td>
<td>63</td>
<td>0.42</td>
<td>0.35</td>
<td>0.3 (1418)</td>
<td>Linearised Circular (1:1)</td>
</tr>
<tr>
<td>S9</td>
<td>250×300 × 2550</td>
<td>90</td>
<td>0.42</td>
<td>0.35</td>
<td>0.15 (1013)</td>
<td>Octo-Elliptical (1:1)</td>
</tr>
<tr>
<td>S10</td>
<td>250×300 × 2550</td>
<td>83</td>
<td>0.42</td>
<td>0.35</td>
<td>0.3 (1868)</td>
<td>Octo-Elliptical (1:1)</td>
</tr>
<tr>
<td>S11</td>
<td>250×300 × 2550</td>
<td>105</td>
<td>0.42</td>
<td>0.35</td>
<td>0.15 (1181)</td>
<td>Octo-Elliptical (0.6:1)</td>
</tr>
<tr>
<td>S12</td>
<td>250×300 × 2550</td>
<td>74</td>
<td>0.42</td>
<td>0.35</td>
<td>0.3 (1665)</td>
<td>Octo-Elliptical (0.6:1)</td>
</tr>
</tbody>
</table>

The orientation of the X, Y and Z axes of the specimen is shown in Figure 3.1 of chapter 3. The variables of the testing program were axial load ratio and the type of loading path. The specimens were tested under three different bidirectional lateral loading protocols, namely the linearised circular path (1:1) path, the octo-elliptical (1:1) path and the octo-elliptical (0.6:1) path as summarized in Figure 5.1. More details about the
development and description of the loading protocols can be found in section 3.7 of chapter 3.

Figure 5.1. Bidirectional loading protocols used in the experimental testing: a) linearised circular (1:1); b) octo-elliptical (1:1); c) octo-elliptical (0.6:1)

5.3 Experimental Test Results and Discussion

This section provides a detailed description and analysis of the experimental test results of the specimens tested under bidirectional lateral loading and CAL. The results are discussed in terms of general response and failure modes, force-displacement behaviour, moment-curvature behaviour, axial displacement-lateral drift behaviour, energy dissipation and stiffness degradation. The behaviour is also compared with the identical specimens tested in chapter 4 under unidirectional lateral loading and CAL.

5.3.1 General Response and Failure Mechanisms

The specimens exhibited a predominantly flexure failure mode just like the specimens tested under unidirectional lateral loading in the last chapter. ASCE/SEI 41-13 [5] defines the predominant failure mode of the column on the basis of the ratio of plastic shear demand to the shear capacity (i.e. $V_p/V_n$) of the column. According to this criteria, a column is flexure-critical if $V_p/V_n \leq 0.6$, flexure-shear critical if $0.6 \leq V_p/V_n \leq 1.0$ and shear-critical if $V_p/V_n > 1.0$. The expected failure modes of the tested specimens according to the classification criteria of ASCE/SEI 41-13 in the X and Y-directions of
the specimens are provided in Table 5.2, which indicates that all specimens were flexure-critical.

Table 5.2. Predicted failure mode according to the classification of ASCE/SEI 41-13

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$V_{px}/V_{nx}$</th>
<th>$V_{py}/V_{ny}$</th>
<th>Type of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>S7</td>
<td>0.42</td>
<td>0.40</td>
<td>Flexure</td>
</tr>
<tr>
<td>S8</td>
<td>0.45</td>
<td>0.44</td>
<td>Flexure</td>
</tr>
<tr>
<td>S9</td>
<td>0.43</td>
<td>0.44</td>
<td>Flexure</td>
</tr>
<tr>
<td>S10</td>
<td>0.40</td>
<td>0.45</td>
<td>Flexure</td>
</tr>
<tr>
<td>S11</td>
<td>0.36</td>
<td>0.50</td>
<td>Flexure</td>
</tr>
<tr>
<td>S12</td>
<td>0.30</td>
<td>0.44</td>
<td>Flexure</td>
</tr>
</tbody>
</table>

The first signs of distress appeared in the form of hairline flexural cracks in both the X and Y directions of the column at small displacement excursions, which were followed by vertical splitting cracks at relatively large displacements in the top and bottom plastic hinge regions, as would be expected for high strength concrete due to its inherently brittle nature. The vertical splitting cracks then opened up with increasing displacements and resulted in concrete spalling in the plastic hinge regions. Finally, longitudinal bar buckling and transverse bar fracture occurred, simultaneously leading to axial load failure of the specimen by the formation of a diagonal plane in the top plastic hinge region. It is noted that axial load failure of the specimens occurred simultaneously with longitudinal bar buckling and transverse bar fracture.

Overall, the general behaviour of the specimens under bidirectional lateral loading was quite similar to uniaxial test results described in chapter 4, except that each damage state under biaxial actions occurred at a considerably lower drift level compared to the latter. Table 5.3 summarises the drift limits associated with each damage level in the Y direction of the column and compares with the corresponding drift values of the identical columns (S1 and S2) under unidirectional lateral actions.
Table 5.3. Drifts at different damage states of the column

<table>
<thead>
<tr>
<th>No.</th>
<th>Visible Cracking</th>
<th>Concrete Spalling</th>
<th>Lateral Load Failure</th>
<th>Collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Uniaxial Loading</td>
<td>Biaxial Loading</td>
<td>Uniaxial Loading</td>
<td>Biaxial Loading</td>
</tr>
<tr>
<td>S1/S7</td>
<td>0.9</td>
<td>0.7</td>
<td>2.4</td>
<td>1.3</td>
</tr>
<tr>
<td>S2/S8</td>
<td>0.7</td>
<td>0.5</td>
<td>1.9</td>
<td>1.1</td>
</tr>
<tr>
<td>S1/S9</td>
<td>0.9</td>
<td>0.7</td>
<td>2.4</td>
<td>1.3</td>
</tr>
<tr>
<td>S2/S10</td>
<td>0.7</td>
<td>0.5</td>
<td>1.9</td>
<td>1.1</td>
</tr>
<tr>
<td>S1/S11</td>
<td>0.9</td>
<td>0.9</td>
<td>2.4</td>
<td>1.8</td>
</tr>
<tr>
<td>S2/S12</td>
<td>0.7</td>
<td>0.5</td>
<td>1.9</td>
<td>1.1</td>
</tr>
</tbody>
</table>

Note: Specimens S1 and S2 are identical specimens tested under uniaxial loading in chapter 4.

Figure 5.2 shows the damaged specimens at axial load collapse. It can be seen in Figure 5.2 that generally the damaged region (plastic hinge region) has a greater length for specimens with low axial load ratios as opposed to specimens with high axial load ratios. This is because the collapse of heavily loaded columns occurs suddenly without much damage progression.

5.3.2 Force-Displacement Behaviour

The force-displacement hysteretic response in the X and Y-direction directions of the specimens is presented in Figures 5.3 and 5.4, respectively. Lateral load failure denoting 20% degradation in the lateral strength and axial load failure point indicating a loss in the axial load carrying capacity of the column have been marked on each hysteresis. The effect of axial load ratio and the type of bidirectional loading path on the hysteretic response are discussed herein. The comparison of bidirectional test results with the corresponding unidirectional results is also provided in detail.
5.3.2.1 Effect of Bidirectional Loading Path

Two specimens were tested under each of the three bidirectional loading protocols considered in this study so that the effect of bidirectional loading path on the collapse behaviour of the columns could be understood properly. Specimens S7 and S8 were tested under linearised circular path (1:1), S9 and S10 under octo-elliptical (1:1) path and S11, S12 under octo-elliptical (0.6:1) path at axial load ratios of 0.15 and 0.3, respectively.
The specimens tested under linearised circular (1:1) and octo-elliptical (1:1) paths (i.e. S7, S8, S9 and S10) exhibited quite similar force and drift characteristics at a given axial load ratio. The peak lateral strength of the specimens was found to be around 10-15% lower than the theoretical capacity in the X and Y directions. Similarly, the lateral load failure drift and axial load failure (collapse) drift of the specimens subjected to these two protocols were also the same, as summarised in Table 5.4. All these specimens also had equal X and Y-direction collapse drifts.

On the other hand, S11 and S12, which were subjected to octo-elliptical (0.6:1) loading path, failed at 30% higher (Y-direction) and lower (X-direction) drift capacities in the Y and X-directions, respectively, in contrast with the corresponding specimens tested under the other two loading protocols. The peak lateral strength of the specimens S11 and S12 in the X-direction was 30-40% lower than the theoretical capacity, whereas it was 10-15% lower in the Y-direction. The column showed a considerably lower strength in the X-direction because the maximum imposed drift in the X-direction was 60% of the corresponding drift in the Y-direction. The column was, thus, unable to reach its ultimate capacity as damage in the Y-direction weakened the other direction. It can also be noticed in Figure 5.3 (c) that there is very sharp strength degradation in the X-direction of the specimen S11 as opposed to specimens S7 and S9, given the same axial load ratio, which may be attributed to the higher concrete strength (105 MPa) of S11 that resulted in the brittle strength degradation behaviour. Moreover, Figures 5.3 (c) and 5.3 (f) show that axial load failure occurred very soon after the commencement of strength degradation. This is in contrast with the behaviour of specimens S7-S10, where strength degradation in the X-direction was relatively gradual. On the other hand, Figure 5.4 indicates that strength degradation in the Y-direction of the column was similar to the other two loading protocols.
Figure 5.3. Force-drift response in the X-direction of the specimens S7 - S12.
Figure 5.5 shows the displacement paths of the column under different loading protocols. A comparison of specimens, S7, S9 and S11 at \( n = 0.15 \) shows that specimen S7 tested under linearised circular path was able to sustain a maximum displacement excursion of 2.4% in the first and the third quadrants and collapsed while entering the 2nd quadrant as shown in Figure 5.5 (a), whereas, specimen S9 subjected to octo-elliptical path (1:1) sustained four counterclockwise ellipses and two clockwise ellipses during the cycle with peak displacement excursion of 2.4% as indicated by Figure 5.5 (b). On the other hand, specimen S11 tested under octo-elliptical (0.6:1) path sustained three counterclockwise ellipses in the Y-direction during the cycle with peak displacement excursion of 3.1%. The corresponding peak drift sustained in the X-direction was 1.8% as shown in Figure 5.5 (c).

The displacement paths of the specimens, S8, S10 and S12 (tested at \( n = 0.3 \)), shown in Figures 5.5 (d), 5.5 (e) and 5.5 (f) indicate that specimen S8 failed when the peak displacement excursion of 1.4% was repeated for the second time and was able to sustain 7 quarter circles before collapsing. Specimen S10, however, sustained only first three counterclockwise ellipses in the cycle with peak displacement excursion of 1.4% and collapsed. Specimen S12, on the other hand, failed at a drift of 1.9% in the Y-direction and 0.9% in the X-direction.

This suggests that octo-elliptical path (1:1) is comparatively less severe (in terms of causing the collapse earlier) than the linearised circular path (1:1) at low axial load ratios and is more severe at high axial load ratios. However, it is worth noting that the peak positive and negative displacement excursions in the X and Y directions are repeated four times in the linearised circular protocol (1:1) (ref Figure 5.1 (a)), with the first two times during the first cycle of quarter circles in each quadrant and the remaining two times during the second cycle. In contrast, the peak positive and negative displacements in the X and Y directions are repeated only twice in the octo-elliptical path (1:1), since the column experiences less displacement than the peak value in the diagonal ellipses of the protocol (ref Figure 5.1 (b)). Therefore, it can be concluded that octo-elliptical path (1:1) is relatively more severe as it resulted in
Figure 5.4. Force-drift response in the Y-direction of the specimens S7 - S12.
collapse at the same drift as linearised circular path even though the column was subjected to fewer cycles of peak positive and negative displacement excursions.

The results under different loading paths essentially imply that the hysteretic response, especially the collapse drift capacity of the column is not overly dependent on the type of the bidirectional loading path but is rather more dependent on the overall enveloped magnitude of displacements in a bidirectional loading path. This is why, the force-displacement characteristics are almost identical (although the number of cycles of the collapse drift the columns were able to sustain differ) for linearised circular (1:1) and octo-elliptical loading paths (1:1), where the specimens were subjected to nearly same enveloped magnitude of displacements, whereas they are different for octo-elliptical (0.6:1) in which the magnitude of displacements was different in the X-direction of the column.

Figure 5.5. Bidirectional drift response history of the specimens S7-S12
5.3.2.2 Effect of Axial Load Ratio

The lateral load failure drift of the column specimens reduced by 35% on average, with the increase in axial load ratio from $n=0.15$ to 0.3, whereas axial load failure drift of the specimens reduced by 45% on average for the same increase in the axial load ratio. Table 5.4 shows the reduction in the drift capacity of specimens with the increase in the axial load ratio. It can also be seen in Figures 5.3 and 5.4 that at high axial load ratios, the strength degradation is relatively steeper compared to low axial load ratios. Moreover, for heavily loaded columns, axial load failure occurred soon after lateral load failure of the specimens, which is in contrast with the behaviour at low axial load ratios, where the collapse occurred more gradually and the difference between the lateral load failure and axial load failure drifts was relatively higher. The hysteretic plots also demonstrate that the lateral strength of the lightly loaded columns degraded to a much greater extent compared to the heavily loaded columns, which collapsed more suddenly after the occurrence of lateral load failure. The effect of axial load ratio on the failure drift in the Y-direction of the columns for different loading paths is illustrated in Figure 5.6, which shows that the reduction in drift capacity with the increase in axial load under bidirectional loading is slightly more than the unidirectional testing, as the lateral load failure and axial load failure drift capacity under unidirectional loading reduced by around 30% and 35%, respectively, with the increase in axial load ratio from $n=0.15$ to 0.3 as opposed to the reduction of 35% and 45%, respectively for bidirectional loading. This implies that the increase in axial load has a slightly more pronounced effect on the failure drift capacity of the column under bidirectional loading as opposed to unidirectional loading. It is noted that the results of specimens S7 and S8 tested under linearised circular (1:1) path are not shown in Figure 5.6 because they were identical to results under octo-elliptical (1:1) path.
Table 5.4. Drifts at lateral load failure and axial load failure

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Lateral Load Failure Drift (%)</th>
<th>Axial Load Failure/Collapse Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>S7 (n=0.15)</td>
<td>1.8</td>
<td>1.6</td>
</tr>
<tr>
<td>S8 (n=0.3)</td>
<td>1.4</td>
<td>1.3</td>
</tr>
<tr>
<td>S9 (n=0.15)</td>
<td>1.8</td>
<td>1.8</td>
</tr>
<tr>
<td>S10 (n=0.3)</td>
<td>1.1</td>
<td>1.1</td>
</tr>
<tr>
<td>S11 (n=0.15)</td>
<td>1.4</td>
<td>2.3</td>
</tr>
<tr>
<td>S12 (n=0.3)</td>
<td>0.9</td>
<td>1.4</td>
</tr>
</tbody>
</table>

Figure 5.6. Effect of axial load ratio on the drift capacity in the Y-direction: a) lateral load failure drift b) axial load failure drift

5.3.2.3 Comparison with Force-Displacement Behaviour under Unidirectional Lateral Actions

The comparison of the force-displacement hysteretic and backbone (positive Y-direction) response of the specimens tested under bidirectional lateral loading with that of the corresponding uniaxial response of identical specimens tested in chapter 4 is presented in Figures 5.7 and Figure 5.8, respectively. It is noted that the comparison is only provided for the Y-direction of the column as uniaxial tests were conducted in the Y-direction only. It can be observed in Figures 5.6, 5.7 and 5.8 that there is a
significant reduction in the drift capacity of the column under bidirectional loading. The Figure also shows that the uniaxial lateral load failure drift capacity of the column reduced by around 30% under octo-elliptical (0.6:1) loading path, whereas it reduced by around 45% under octo-elliptical (1:1) loading path. Similarly, the uniaxial collapse (axial load failure) drift capacity of the column reduced by around 35% under octo-elliptical (0.6:1) path and around 50% under octo-elliptical (1:1) loading path, which is quite significant. The reduction in the drift capacity under bidirectional loading compared to unidirectional loading can be attributed to bidirectional demands as well as increase in the number of cycles; though, bidirectional demands seem to be more dominant factor. This is because, under linearized circular path, the specimen was subjected to peak displacements in the X and Y directions four times, whereas, under octo-elliptical path, the specimen was subjected to peak displacements twice. Despite this, the collapse drift capacity under both loading protocols was the same. Nevertheless, it is noted that the available data is not conclusive as these two loading paths are different. Therefore, future studies need to consider these two parameters as variables to develop a definitive understanding of the issue. The peak lateral force under biaxial loading was higher than the corresponding uniaxial value for some specimens, which was due to the higher concrete compressive strength for specimens tested under bidirectional lateral loading. The plots in Figures 5.7 and 5.8 also demonstrate the acceleration in the rate of strength degradation under biaxial loading, which is even more enhanced at higher axial load ratios.
Figure 5.7. Force-drift hysteretic response under unidirectional vs bidirectional loading paths in the Y-direction of the specimens

Figure 5.8. Force-drift backbone response under unidirectional vs bidirectional loading paths in the (positive) Y-direction of the specimens

5.3.3 Moment-Curvature Behaviour

The moment-curvature hysteretic behaviour at the top of the column specimens is shown in Figures 5.9 and 5.10 for the Y and X-axis of the specimens, respectively. The point corresponding to the ultimate moment capacity of the specimens has been marked on the hysteresis. It can be observed in Figure 5.9 that the moment capacity in the Y-axis was smaller for the specimens (S11 and S12) tested under octo-elliptical (0.6:1) path compared to the specimens tested under the other two loading protocols. This is because, under the octo-elliptical (0.6:1) path, the column was subjected to larger bending in the X-axis, which also weakened the other axis, and as a result column
was not able to reach its theoretical moment capacity in the Y-axis. A comparison of curvature at the ultimate moment capacity in the X-direction of the specimens revealed that generally, the curvature at the ultimate moment was similar for linearised circular (1:1) path and octo-elliptical (1:1) paths. However, the curvature at the ultimate moment was generally smaller for specimens tested under octo-elliptical (0.6:1) loading path. Furthermore, the curvature at the ultimate moment in the X-direction reduced on average by 25% with the increase in axial load ratio from $n = 0.15$ to 0.3.

In the Y-direction of the column, the curvature at the ultimate moment was more or less the same under the three considered loading protocols for a given axial load ratio. On the other hand, there was a reduction of around 15% in the curvature at the ultimate moment in the Y-direction with the increase in the axial load ratio from $n = 0.15$ to 0.3.

![Diagram](image)

(a) S7  (b) S9  (c) S11
(d) S8  (e) S10  (f) S12

Figure 5.9. Moment-curvature response in the Y-axis of the specimens S7 - S12.
5.3.4 Axial Displacement-Lateral drift Behaviour

The relationship between axial displacement and lateral drifts in the X and Y-direction of the specimens is shown in Figures 5.11 and 5.12, respectively. The points corresponding to ultimate capacity and lateral load failure (20% degradation in lateral strength) have also been marked on the plots. It was observed that the column experienced axial elongation when the drift was zero in the X-direction for the specimens, S7, S9 and S11, that were tested at a low axial load ratio of $n = 0.15$ as indicated in, Figure 5.11. This is primarily because when the drift is zero in the X-direction, it is maximum in the Y-direction of the column as evident from Figure 5.1 and specimens experience axial elongation at maximum drifts in the Y-direction as shown in Figure 5.12. Among the specimens tested at the low axial load ratio of $n = 0.15$, the axial elongation behaviour at zero drift in the X-direction or at maximum drift in the Y-
direction was found to be more significant for specimen S11 that was subjected to octo-elliptical (0.6:1) loading path, whereas very negligible elongation was observed for specimen S7 tested under linearised circular (1:1) path.

Figure 5.11. Axial displacement-lateral drift behaviour of the specimens S7 – S12 – X-direction

At high axial load ratio (of \( n = 0.3 \)), on the other hand, the column specimens underwent extensive axial shortening in the beginning of the test when the axial load was being ramped up and didn’t undergo any elongation generally due to the presence of high axial loads.
A comparison between the axial displacement-lateral drift behaviour in the Y-direction of the specimens tested under bidirectional lateral loading and the identical specimens, S1 and S2, tested under unidirectional lateral loading in the previous chapter shows that under bidirectional loading axial shortening is a lot more significant than unidirectional loading and that is perhaps why the column collapses at a smaller drift under bidirectional loading as opposed to unidirectional loading. Furthermore, the extent of axial elongation at maximum drifts in the Y-direction is observed to be far less under bidirectional loading in contrast with unidirectional loading as delineated in Figures 4.5 (a), 4.5 (b) and 5.12.
5.3.5  Energy Dissipation

The energy dissipation was calculated by calculating the area enclosed under the force-displacement hysteresis at each increment of displacement for any given specimen. The accumulative energy dissipation at the collapse was then calculated by summing the hysteretic areas at all the displacement excursions till collapse in both the X and Y-directions of the column. Figure 5.13 shows the accumulative energy dissipation for the specimens tested under different loading protocols at axial load ratios of $n = 0.15$ and $n = 0.3$. It can be observed that specimens tested under bidirectional loading paths generally dissipated more energy compared to the specimens subjected to unidirectional loading, which is due to the coupling between the two loaded directions.

At both axial load ratios, the highest amount of energy was dissipated under octo-elliptical (1:1) loading protocol, followed by octo-elliptical (0.6:1) and linearised circular protocols, respectively, whereas the lowest amount of energy was dissipated under unidirectional loading protocol. However, the difference in the amount of energy dissipated reduced significantly as the axial load ratio was increased from $n = 0.15$ to 0.3.

For instance, at axial load ratio of 0.15, the specimen subjected to octo-elliptical (1:1) path dissipated around 14%, 34% and 55% more energy compared to the specimens tested under octo-elliptical (0.6:1), linearised circular (1:1) and unidirectional loading protocols, respectively, whereas at axial load ratio of 0.3, the specimen tested under octo-elliptical (1:1) path dissipated around 10%, 16% and 24% more energy than specimens subjected to octo-elliptical (0.6:1), linearised circular (1:1) and unidirectional loading protocols. It is worth noting here that the specimens subjected to octo-elliptical (1:1) and linearised circular (1:1) paths collapsed at the same drift capacity. However, since the displacement path in the octo-elliptical (1:1) loading protocol was as such that the column travelled more distance in this protocol (i.e. 8 elliptical loops), therefore the accumulative energy dissipated was higher for the specimens subjected to this protocol. This implies that the accumulative energy dissipation is not only dependent on the collapse drift of the column, but is also more
dependent on the axial load ratio and the distance travelled in a particular bidirectional loading path.

The hysteretic energy dissipation in the X and Y-direction of the specimens under different loading protocols is shown in Figures 5.14 and 5.15, respectively. Here the energy dissipation corresponding to the second cycle of the maximum displacement excursion in the X and Y-direction of the specimen has been plotted. It can be observed that the hysteretic energy dissipation increased with the increasing lateral drifts as a general trend. The energy dissipation at a particular drift limit can be seen to be similar for octo-elliptical (1:1), linearised circular (1:1) and octo-elliptical (0.6:1) paths in the Y-direction of the specimens. However, in the X-direction, the energy dissipation at a given drift was similar for octo-elliptical (1:1) and linearised circular paths, but was quite smaller for octo-elliptical (0.6:1) loading protocol. Moreover, the rate of increase in the hysteretic energy dissipation in the X-direction was smaller for octo-elliptical (0.6:1) loading path compared to the other two loading protocols. This is because under octo-elliptical (0.6:1) path more energy is dissipated in the Y-direction of the column as it is subjected to larger displacement excursions compared to the X-direction.

Figure 5.13. Accumulative hysteretic energy dissipation under unidirectional and bidirectional loading protocols: a) axial load ratio ($\eta = 0.15$); b) axial load ratio ($\eta = 0.3$)
Figure 5.14. Hysteretic energy dissipation in the X-direction of the column under different bidirectional loading protocols: a) axial load ratio \( n = 0.15 \); b) axial load ratio \( n = 0.3 \)

Figure 5.15. Hysteretic energy dissipation in the Y-direction of the column under different bidirectional loading protocols: a) axial load ratio \( n = 0.15 \); b) axial load ratio \( n = 0.3 \)

5.3.6 Effective Moment of Inertia and Stiffness Degradation

The effective moment of inertia of the specimens was determined from the experimental stiffness of the column. The effective moment of inertia \( (I_{eff}) \) for each
specimen was calculated based on the gradient of a line taken from the origin through the drift corresponding to the development of the theoretical yield capacity. The theoretical yield capacity was taken as the lateral force corresponding to the development of a theoretical yield moment, which was taken as the moment corresponding to the development of a compressive strain corresponding to the maximum unconfined compressive strength (calculated using the Karthik and Mander model [118]) being reached in the extreme compressive fibre of the concrete or tensile yielding of the reinforcement, whichever occurred first. The theoretical yield capacity was calculated using the actual material properties (i.e. for steel and concrete) for the respective specimens. The effective moment of inertia was normalized with respect to the gross moment of inertia \( I_g \) in the two directions of the columns to study the effect of the loading path on \( I_{\text{eff}} \). Table 5.5 presents a comparison of the effective moment of inertia of the specimens in the X and Y-axis for different axial load ratios (i.e. \( n = 0.15 \) and 0.3) under various loading protocols. It can be observed in Table 5.5 that the ratio of effective to the gross moment of inertia in the X-axis of the columns is slightly higher under uniaxial loading compared to biaxial loading. Data was not available to make a similar comparison for the Y-axis of the column.

No substantial difference could be observed between the effective moment of inertia of the specimens in X-axis under different bidirectional loading paths. However, in the Y-axis, the effective moment of inertia of the specimen tested under octo-elliptical (0.6:1) path at \( n = 0.3 \) was significantly lower than the corresponding moment of inertias under the other two loading protocols. This reduction in the moment of inertia may be attributed to the relatively higher drift in the Y-direction, which might have weakened the column significantly especially at high axial load ratio, and thus resulted in a lower effective moment of inertia.
Table 5.5. Effect of loading Path on the effective moment of inertia of the column

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Type of Loading</th>
<th>Ratio of Effective to Gross Moment of Inertia</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$I_{eff,y}/I_{g,y}$</td>
</tr>
<tr>
<td>S1 (n=0.15)</td>
<td>Unidirectional</td>
<td>-</td>
</tr>
<tr>
<td>S7 (n=0.15)</td>
<td>Linearised Circular (1:1)</td>
<td>0.29</td>
</tr>
<tr>
<td>S9 (n=0.15)</td>
<td>Octo-Elliptical (1:1)</td>
<td>0.32</td>
</tr>
<tr>
<td>S11 (n=0.15)</td>
<td>Octo-Elliptical (0.6:1)</td>
<td>0.29</td>
</tr>
<tr>
<td>S2 (n=0.3)</td>
<td>Unidirectional</td>
<td>-</td>
</tr>
<tr>
<td>S8 (n=0.3)</td>
<td>Linearised Circular (1:1)</td>
<td>0.47</td>
</tr>
<tr>
<td>S10 (n=0.3)</td>
<td>Octo-Elliptical (1:1)</td>
<td>0.48</td>
</tr>
<tr>
<td>S12 (n=0.3)</td>
<td>Octo-Elliptical (0.6:1)</td>
<td>0.29</td>
</tr>
</tbody>
</table>

In order to study the progression of stiffness degradation with the increasing lateral drifts, the stiffness ($K_i$) at the peak displacement excursion of each cycle is calculated and then plotted with the corresponding lateral drift. The average stiffness at each displacement excursion was determined from equation 4.3.

Figure 5.16 shows the progression of stiffness degradation in the Y-axis with the increasing lateral drifts for the three bidirectional loading protocols at two different axial load ratios. The stiffness has been normalized with respect to the gross stiffness in the Y-axis of the specimen. It can be observed in Figure 5.16 that, while the stiffness degradation for linearised circular (1:1) path and octo-elliptical (1:1) path followed nearly the same slope, the stiffness degradation for octo-elliptical (0.6:1) path was much more rapid and sharp. Also, Figures 5.16 (a) and 5.16 (b) delineate rapid stiffness degradation with the increase in axial load ratio.

The effect of various loading protocols on the stiffness degradation in the X-axis of the column is shown in Figure 5.17. The progression of the stiffness degradation with the increasing lateral drifts is in the following order: unidirectional, octo-elliptical (0.6:1) and then both octo-elliptical (1:1) and linearised circular path (1:1).
Figure 5.16. Ratio of stiffness in each cycle to the gross stiffness – Y-axis of the column: a) axial load ratio, $n=0.15$; b) axial load ratio, $n=0.3$

Figure 5.17. Ratio of stiffness in each cycle to the gross stiffness – X-axis of the column: a) axial load ratio, $n=0.15$; b) axial load ratio, $n=0.3$

The slope of stiffness degradation under octo-elliptical (0.6:1) path is relatively less steep than the other two bidirectional loading protocols in the X-axis primarily because the column is subjected to less displacement in the other axis. Figure 5.17 also demonstrates that stiffness of the column degrades to a greater extent under unidirectional loading as opposed to bidirectional loading. This implies that at collapse, the column has relatively higher residual stiffness under bidirectional loading compared to unidirectional loading.
5.4 Summary and Conclusions

This chapter addressed the second gap of this research by conducting an experimental investigation into the collapse behaviour of limited ductile HSRC columns, representative of typical construction practice in Australia and other regions of lower seismicity, under bidirectional lateral actions and CAL. Axial load ratio and the type of loading path were the main variables of the testing program. The following conclusions can be drawn on the basis of the results presented in this chapter:

1. Each damage state such as cracking, concrete spalling, rebar buckling and fracture occurs at a considerably lower drift (30-50%) under bidirectional lateral loading compared to unidirectional lateral loading. This finding underscores the importance of considering the bidirectional drift capacity of columns in displacement-based design procedures, especially in columns that are expected to undergo strong bidirectional actions, such as corner columns in unsymmetrical/irregular buildings.

2. It was found that the force-displacement hysteretic response of the column is dependent on the X to Y-direction displacement ratio of the column. As such, the collapse drift capacity in the Y-direction of the column was 50% and 65% of the corresponding drift values under uniaxial loading when the ratio of displacements was 1.0 and 0.6, respectively. Similarly, the lateral force capacity of the column in both axes was 10-15% lower than the theoretical capacity when this ratio was 1.0 and was 30-40% lower in the X-direction when the ratio was 0.6.

3. The hysteretic response, especially the collapse drift capacity of the column, is not overly dependent on the type of the bidirectional loading path but is rather more dependent on the overall enveloped magnitude of displacements in a bidirectional loading path. Therefore, the drift capacity of specimens tested under linearized circular (1:1) and octo-elliptical (1:1) paths, which had the same overall envelope of displacements, were identical. This finding, if verified by further experimental testing, can have important implications regarding the selection of an appropriate loading protocol for estimating the bidirectional capacity of columns.
4. An increase in axial load ratio from 0.15 to 0.3 reduced the lateral load failure and axial load failure drift capacity of the column by around 35% and 45%, respectively, under bidirectional loading. It was also found that under bidirectional loading, an increase in axial load ratio has slightly more pronounced effect in reducing the drift capacity of the column as opposed to unidirectional loading.

5. The axial load failure/collapse drift of the AS 3600 code-compliant column specimens tested under axial load ratio of \( n = 0.3 \) and bidirectional loading with \( b/a = 1 \) was below 1.5%. Hence, it will be a good practice to keep the design axial load ratio below 0.3 for HSRC columns that are designed according to AS 3600 and are expected to experience strong bidirectional actions.

6. Bidirectional lateral loading accelerates the post-peak strength and stiffness degradation of the column and as such, the stiffness degradation was much steeper under biaxial testing as opposed to uniaxial testing. Like the reduced drift capacity, stiffness degradation was also found to be independent of the type of bidirectional loading path and was instead dependent on the overall enveloped magnitude of displacements in a loading path.

7. The effective moment of inertia was generally observed to be slightly higher under unidirectional lateral loading compared to bidirectional lateral loading. On the other hand, the effect of type of bidirectional loading path was found to be not very significant on the effective moment of inertia of RC columns.

8. The accumulative energy dissipation was found to be higher under bidirectional loading paths as opposed to unidirectional loading. However, the difference in the amount of energy dissipated reduced with the increase in axial load ratio. The highest amount of energy was dissipated under octo-elliptical (1:1) path. As such, at an axial load ratio of 0.15, the specimen subjected to octo-elliptical (1:1) path dissipated around 14%, 34% and 55% more energy compared to the specimens tested under octo-elliptical (0.6:1), linearised circular (1:1) and unidirectional loading protocols, respectively, whereas at an axial load ratio of 0.3, the specimen tested under octo-elliptical (1:1) path dissipated around 10%, 16% and 24% more energy than specimens.
subjected to octo-elliptical (0.6:1), linearised circular (1:1) and unidirectional loading protocols.

9. The accumulative energy dissipation of the column was observed to be highly dependent on the type of bidirectional loading path. It was observed that two specimens collapsing at the same drift under two different bidirectional loading paths can have significantly different accumulative energy dissipation depending upon the type of bidirectional loading path.
Chapter 6  Experimental and Numerical Study on Behaviour under Bidirectional Lateral Actions and Variable Axial Load

6.1  Introduction

In the last two chapters, the collapse behaviour of limited ductile HSRC columns was experimentally investigated under unidirectional and bidirectional lateral actions with CAL. However, the multidirectional nature of ground motion excitations results in the combined action of bidirectional lateral loading and variable axial (i.e. vertical) load on building columns. The variation in axial load on RC columns can occur due to the framing action in the building that resists the overturning moments from the lateral earthquake forces induced by the two concurrent and orthogonal horizontal components of the ground shaking. This type of variation is classified as synchronous axial load variation where any change in axial load is synchronous with and a function of the lateral loads, and therefore the extreme values (maximum or minimum) of both axial and lateral load occur at the same time. The second type of axial load variation is classified as nonsynchronous axial load variation where the variation of axial load and lateral loads are uncoupled and vary independently of each other. This kind of variation may arise due to the vertical component of the ground motion, which acts independently of its horizontal counterparts and usually has a relatively higher frequency content that results in different frequencies of axial load and lateral load variation. It is noted that the two types of axial load variation have also been referred to as proportional/in phase and non-proportional/out of phase axial load variation in Abrams [44], Saadeghvaziri [45] and ElMandooh and Ghobarah [46].

A common design approach in lower seismic regions for multi-storey RC buildings is to design the core walls in the building to take 100% of the lateral load and then assume the columns and beams/column strips are gravity frames that attract no lateral load
This means that RC columns are designed for their vertical gravity load and minimum moments only (as per the relevant code/standard), and any axial load variation due to potential framing action in these gravity frames is ignored. The literature review in chapter 2 showed that axial load ratio had a significant effect on the displacement capacity of the RC columns. As such, an inadvertent increase in axial load that has not been accounted for in the initial design could severely limit the displacement capacity of the gravity columns in the building, and thereby increase their vulnerability or probability of collapse during an earthquake event.

The first part of this chapter aims to numerically investigate the underlying mechanisms and range of axial load variation that would be seen in RC columns in regions of low to moderate seismicity. To this end, the case study RC building presented in chapter 3, with design and detailing representative of the conditions in Australia, is subjected to a suite of 15 unscaled earthquake ground motions that are representative of low to moderate seismic regions, and the resulting axial load variation in the columns is then investigated. This is followed by an experimental study in which limited ductile HSRC columns are tested under bidirectional lateral loading with two different axial load variation protocols, which were developed in chapter 3 of the thesis. The force-displacement behaviours are then compared with the identical specimens tested in the last two chapters under unidirectional and bidirectional lateral loading with CAL.

### 6.2 Axial Load Variation in Building Columns – General Concept

As mentioned earlier, the axial load variation in building columns arises due to either overturning moments and the subsequent push-pull forces from the horizontal components of the ground motion or the vertical component of the ground shaking or a combination of both. Variation in axial load in the corner perimeter columns of a building is primarily generated by the push-pull forces that are developed to resist the overturning moments on the building from the lateral forces imposed by the two orthogonal horizontal components of ground shaking, thereby resulting in an axial load variation that is typically synchronous to the lateral displacement of the building (i.e.
the peaks in axial load variation correspond and occur at the same instance in time as the peaks of the column’s lateral displacements).

Whereas, axial load variation in the internal non-perimeter columns is mainly controlled by the vertical component of the ground shaking since internal non-perimeter columns in a building frame typically generate negligible push-pull axial forces compared to corner perimeter columns. Since the vertical and horizontal dynamic properties of a building vary significantly (in terms of mode shapes and periods of vibration), the axial load variation in these columns is typically nonsynchronous to the lateral displacement of the building (i.e. the peaks in axial load variation do not occur at the same instance in time as the peaks of the column’s lateral displacements). Axial load variation in the internal perimeter columns though is often controlled by a combination of these behaviours.

Two typical scenarios of the push-pull forces generated in the columns of a building frame due to lateral loading, or lateral inter-storey drifts, are further illustrated in Figure 6.1, wherein two four-storey frames of varying structural proportions are presented. The frames have no gravity loading and are subjected to lateral loads with an inverted triangular load distribution varying from 100 kN at the first floor to 400 kN at the fourth floor. The lateral loads result in an overturning moment that is resisted by both bending moments at the base of the columns and push-pull axial forces in the columns. The distribution and relative magnitude of the axial forces induced are directly dependent on the structural proportions of the building frame, i.e. relative section size and modulus of beams and columns, inter-storey height and span length. In Frame A as shown in Figure 6.1 (a), the external columns develop large axial tension and compression forces, followed by a reversed axial load in the first adjacent internal column on each respective side. The central column has no variation in axial load due to the symmetrical nature of the frame. In contrast, Frame B in Figure 6.1 (b), which has different span lengths between adjacent bays, develops axial tension in the first and second column from the left and axial compression in the first and second column from the right. The difference between the two scenarios is due to the relative stiffness of the end span. The end span in Frame A has a higher stiffness relative to Frame B, which in
turns attracts larger push-pull forces in the two external columns. The central column in Frame B, similar to Frame A, attracts no variation in axial load due to the symmetrical nature of the frame. Regardless, it can be seen in both scenarios that the magnitude of the forces induced in the external columns is fairly large compared to the internal columns, particularly the central column.

![Figure 6.1. Axial load variation in building frame columns: a) frame A with shorter, stiffer end span; b) frame B with longer, more flexible end span.](image-url)
In view of the above, the axial load variation in the corner perimeter columns of a building can be considered to be dependent on the horizontal ground motion characteristics (particularly RSA), whereas axial load variation in internal non-perimeter columns can be considered to be correlated with the vertical RSA. The proposition that axial load variation is dependent on the RSA rather than any other ground motion characteristic, such as PGA or PGV, will be proven later in the section where results of the numerical analysis of a case study building are presented. The general expressions for axial load variation in corner perimeter columns and internal columns are proposed to be as follows:

\[ v_{n_{\text{corner}}}(\%) = \left( \alpha_1 \frac{S_a(T_x)}{\mu_{S_a}(T_x)} + \alpha_2 \frac{S_a(T_y)}{\mu_{S_a}(T_y)} \right) \times 100 \]  
\[ v_{n_{\text{internal}}} (\%) = \left( \alpha_3 \frac{S_a(T_z)}{\mu_{S_a}(T_z)} \right) \times 100 \]  

6.3 Numerical Investigation of Axial Load Variation in Columns Using a Case-Study RC Building

The same case study building utilized for the development of loading protocols in chapter 3 has been used here for the investigation of the range of axial load variation in columns of a typical RC frame wall building constructed in Australia and other regions of low to moderate seismicity and the controlling parameters affecting it. For the sake of brevity, the properties of the building and details about modelling and validation are not repeated here and the reader is referred to the subsections 3.7.1 and 3.7.2 for details. However, the building plan is reproduced in Figure 6.2 for the convenience of the reader.

The periods of the building in the three directions (X,Y,Z) were found out to be 1.14s, 1.48s and 0.17s, respectively. The building was subjected to a suite of 15 unscaled ground motions, representative of earthquake actions in low to moderate seismic regions, to investigate the axial load variation in the columns of the building. Readers are referred to Table 3.4 in chapter 3 of the thesis for further details about the ground motions.
6.3.1 Extent of Axial Load Variation in Corner and Internal Columns

The maximum percentage axial load variation under each ground motion occurred in the (symmetrical) corner perimeter columns A8/D1, which was likely due to coupling behaviour with the adjacent very stiff core wall. The minimum percentage variation in axial load was observed for internal non-perimeter columns, whereas for the internal perimeter columns, the range of axial load variation was in-between that of the corner perimeter columns and the internal non-perimeter columns. The variation in corner perimeter columns A8/D1 was found to be slightly higher than the other two corner perimeter columns (i.e. A1/D8), due to stronger framing action caused by the very stiff core wall and reduced beam length to the core. On the other hand, similar axial load variation was observed for all the internal perimeter and internal non-perimeter columns, respectively. It is also noted that the relative axial load variation of vertically adjacent columns along the height of the structure did not vary significantly (up to 5% only).
For the purpose of demonstration, a plot of axial load variation with time for the corner perimeter (A8), internal perimeter (B1) and internal non-perimeter column (B3) are shown in Figure 6.3 for the Christchurch (2011) earthquake event. The average (of maximum positive and negative) axial load variation for this earthquake event was found to be 33%, 8% and 6% in the corner perimeter, internal perimeter and internal non-perimeter columns, respectively. The average axial load variation in these columns for the rest of the ground motions considered in this study is presented in Table 6.1. It can be observed from these results that for each ground motion the axial load variation is significantly higher in the corner perimeter columns than the internal perimeter and internal non-perimeter columns, which is due to the greater push-pull forces generated in these columns. Table 6.1 shows that the mean axial load variation in the corner perimeter column, internal perimeter column and internal non-perimeter column is 43%, 12% and 10%, respectively, for the whole set of ground motions. This implies that even with more modest ground motions that are representative of low to moderate seismic regions, axial load variation in corner perimeter columns can be really significant, for this particular case study building.

Figure 6.3. Axial load variation for Christchurch (2011) earthquake: a) corner perimeter column (A8); b) internal perimeter column (B1); c) internal non-perimeter column (B3)

It should be noted that the variation in axial load for a given earthquake event will vary based on an individual building’s floor plan and structural proportions. The purpose of this study is not to establish a generalised level of variation that would apply to all
multi-storey buildings, but rather to assess the variation in axial load in a typical case study building representative of a low to moderate seismic region like Australia.

Table 6.1. Average axial load variation in ground floor columns of the case study building subjected to different ground motions.

<table>
<thead>
<tr>
<th>No.</th>
<th>Earthquake Event</th>
<th>Average Axial Load Variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Corner perimeter column-A8</td>
</tr>
<tr>
<td>1</td>
<td>Griva, Greece (1990)</td>
<td>26</td>
</tr>
<tr>
<td>2</td>
<td>Georgia, USSR (1991)</td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td>Little Skull Mountain, Nevada, USA (1992)</td>
<td>19</td>
</tr>
<tr>
<td>4</td>
<td>Joshua Tree, California, USA (1992)</td>
<td>74</td>
</tr>
<tr>
<td>5</td>
<td>Big Bear-01, California, USA (1992)</td>
<td>41</td>
</tr>
<tr>
<td>6</td>
<td>Northridge-04, California, USA (1994)</td>
<td>24</td>
</tr>
<tr>
<td>7</td>
<td>Double Springs, Alabama, USA (1994)</td>
<td>50</td>
</tr>
<tr>
<td>8</td>
<td>Dinar, Turkey (1995)</td>
<td>43</td>
</tr>
<tr>
<td>9</td>
<td>Kozani, Greece-01 (1995)</td>
<td>30</td>
</tr>
<tr>
<td>10</td>
<td>Umbria and Marche, Italy (1997)</td>
<td>71</td>
</tr>
<tr>
<td>11</td>
<td>Northwest China-04 (1997)</td>
<td>42</td>
</tr>
<tr>
<td>12</td>
<td>Chi Chi, Taiwan (1999)</td>
<td>134</td>
</tr>
<tr>
<td>13</td>
<td>Parkfield, California, USA (2004)</td>
<td>25</td>
</tr>
<tr>
<td>14</td>
<td>L'Aquila, Italy (2009)</td>
<td>16</td>
</tr>
<tr>
<td>15</td>
<td>Christchurch, New Zealand (2011)</td>
<td>33</td>
</tr>
<tr>
<td></td>
<td><strong>Mean</strong></td>
<td><strong>43</strong></td>
</tr>
</tbody>
</table>
6.3.2 Relationship between Ground Motion Intensity Measures and Axial Load Variation of RC Columns

The relationship between ground motion intensity measures such as peak ground acceleration, peak ground velocity and response spectral acceleration is investigated herein to identify the intensity measure that mainly controls the axial load variation in RC columns.

6.3.2.1 Peak Ground Acceleration

The highest axial load variation in the corner perimeter columns was observed for Chi Chi, Taiwan (1999) earthquake event, whereas the lowest variation was observed for L’Aquila, Italy (2009) earthquake. Apparently, this difference in variation of the axial load seems to be correlated with the PGA, which was about 2-3 times higher for Chi Chi, Taiwan (1999) compared to L’Aquila, Italy (2009). However, a closer investigation revealed that PGA is not related to axial load variation as such, as it was observed that for some of the other ground motions, axial load variation was more, even though PGA (in the three axes) was lower compared to the other ground motions. For instance, axial load variation for Double Springs, Alabama, USA ground motion was double of Griva, Greece (1990) as shown in Table 6.1, despite that PGA of the former was 25-30% lower than the latter. This warranted a more detailed investigation into the factors influencing the axial load variation of the building columns.

In order to further investigate the effect of PGA on axial load variation, average percentage variation in axial load for each ground motion is plotted with PGA (in the three axes) for all the four corner perimeter columns (A1, A8, D1 and D8) and four internal non-perimeter columns (B3, B6, C3, and C6). The resulting plots for corner perimeter columns and internal non-perimeter columns in Figure 6.4 show that there is no strong correlation between PGA of the ground motions and axial load variation, as the average correlation coefficient for the three PGA’s is found to be 0.75 and the data is also very scattered and trend line (in red) does not show any definite trend, especially for the corner perimeter columns. A better correlation, with an average correlation coefficient of 0.89 for the three PGA’s, can be seen for the internal non-perimeter
columns, however, data still exhibits scatter. It is noted that X and Y denote the two orthogonal horizontal components of ground motions and Z represents the vertical ground motion in Figure 6.4.

![Graphs showing effect of PGA on axial load variation](image)

Figure 6.4. Effect of PGA on axial load variation in corner perimeter and internal non-perimeter columns: a) PGA-X; b) PGA-Y; c) PGA-Z

### 6.3.2.2 Peak Ground Velocity

The plot of PGV with average axial load variation shows a significantly better correlation than that of the PGA, especially for the corner perimeter columns, as indicated by Figures 6.5 (a), 6.5 (b) and 6.5 (c). As such, the axial load variation in corner perimeter columns was found to be strongly related to PGV in all the three directions, with an average correlation coefficient of 0.89. However, some slight scattering in the results can still be observed.

On the other hand, an average correlation coefficient of 0.82 was found between axial load variation and PGV for the internal non-perimeter columns. A lot of scatter can be observed in the plots for the internal non-perimeter columns, and the results were more or less similar to the behaviour observed for PGA.

The results in Figures 6.4 and 6.5 indicate that by and large, PGV can be considered to be a better intensity measure for axial load variation compared to PGA.
Figure 6.5. Effect of PGV on axial load variation in corner perimeter and internal non-perimeter columns: a) PGV-X; b) PGV-Y; c) PGV-Z

6.3.2.3 Response Spectral Acceleration

The effect of RSA on the axial load variation of columns was evaluated by calculating the RSA corresponding to the elastic period of the case study building in each direction. The dominant period of the building in X, Y and Z direction was 1.14s (Mode 2), 1.48s (Mode 1) and 0.17s (Mode 7), respectively. The plot of RSA with axial load variation of the corner perimeter and internal non-perimeter columns is shown in Figure 6.6. It can be seen in Figures 6.6 (a) and 6.6 (b) that RSA in the X and Y direction of the column is related very strongly to the axial load variation in the corner perimeter columns, with correlation coefficients of 0.93 and 0.94, respectively; however, RSA in the Z direction shows a relatively weak correlation (correlation coefficient of 0.79) with the axial load variation as shown in Figure 6.6 (c). This is because axial load variation in the corner perimeter columns is mainly controlled by the horizontal components of the ground motion.

On the other hand, the axial load variation in the internal non-perimeter columns is relatively weakly correlated (correlation coefficients of 0.86 and 0.73, respectively) with RSA in the X and Y direction and is very strongly correlated to RSA in the Z direction, with a correlation coefficient of 0.99. This is because smaller push-pull forces are generated in the internal non-perimeter columns due to the horizontal components
of the ground motion, and thus axial load variation in internal non-perimeter columns is primarily a function of RSA in the vertical direction of the building.

The results of the comparison of PGA, PGV and RSA with axial load variation imply that RSA can be considered to be the best intensity measure for the axial load variation of the columns, as, on the whole, it showed the best correlation with axial load variation compared to PGA and PGV.

![Figure 6.6](image)

Figure 6.6. Effect of RSA on axial load variation in corner perimeter and internal non-perimeter columns: a) RSA-X; b) RSA-Y; c) RSA-Z

### 6.4 Estimation of Axial Load Variation

The general expressions (6.1) and (6.2) proposed for axial load variation were a function of the RSA in the three axes of ground shaking and the coefficients, $\alpha_{1v}$, $\alpha_{2v}$ and $\alpha_{3v}$, which are dependent on the structural system and configuration. Equation (6.1) related the axial load variation in the corner perimeter columns with the RSA in both the X and Y directions (horizontal ground motions), whereas Equation (6.2) related the axial load variation in the internal non-perimeter columns with RSA in the Z direction (vertical ground motions). For the case study building considered in this thesis, the coefficients, $\alpha_{1v}$, $\alpha_{2v}$ and $\alpha_{3v}$, have been calibrated using the data in Figures 6.6 (a), 6.6 (b) and 6.6 (c), and are found to be 0.2, 0.18 and 0.1, respectively, for this particular case study building. Substituting these values into Equations (6.1) and (6.2) results in Equations (6.3) and (6.4) that can be used to estimate the axial load variation.
in the corner perimeter and internal columns of the case study building for any particular ground motion.

\[ v_{n_{\text{corner}}}(\%) = \left( 0.2 \frac{S_a(T_x)}{\mu_S a(T_x)} + 0.18 \frac{S_a(T_y)}{\mu_S a(T_y)} \right) \times 100 \]  

(6.3)

\[ v_{n_{\text{internal}}} (\%) = \left( 0.1 \frac{S_a(T_z)}{\mu_S a(T_z)} \right) \times 100 \]  

(6.4)

The estimates based on the proposed expressions for the case study building show a very good correlation with the actual axial load variation (in the numerical study), with coefficients of variation of 0.24 and 0.16 for equations 6.3 and 6.4, respectively. Figure 6.7 shows a comparison of the actual and estimated axial load variation.

It is recommended that future studies should investigate the impact of the type of structural system on the coefficients, \( \alpha_{1v} \), \( \alpha_{2v} \) and \( \alpha_{3v} \), by considering a variety of structural configurations.

**Experimental Testing**

The combined effect of axial load variation and bidirectional lateral actions on the overall behaviour of RC columns was evaluated by testing two limited ductile HSRC columns, representative of typical Australian construction, under bidirectional lateral
loading with axial load variation and comparing the results with identical specimens tested under unidirectional and bidirectional cyclic loading with CAL. An in-depth analysis of the effect of axial load variation on the collapse behaviour of the tested column specimens is provided in the following sections.

### 6.5.1 Test Specimens

The experimental program comprised of two limited ductile HSRC columns shown in table 6.2. The specimens were provided with the same longitudinal and transverse reinforcement as previous specimens tested under unidirectional and bidirectional lateral loading with CAL. As such, the longitudinal reinforcement consisted of 6-N16 bars (\( \rho_v = 1.6\% \)), whereas the specimens were provided with AS 3600 code-compliant limited ductile transverse reinforcement detailing, representative of typical Australian construction practice. More details about specimen design can be found in section 3.3 of the thesis.

The specimens were tested under two axial load variation loading protocols, namely synchronous and nonsynchronous axial load variation protocols, which were developed in subsection 3.7.8 based on the numerical analysis on a case study building. The synchronous and nonsynchronous axial load variation protocols are shown in Figures 6.8 and 6.9, respectively. A variation of 30% in the design axial load was considered. The bidirectional loading protocol considered was octo-elliptical (0.6:1) path shown in Figure 6.10 that was also used in the experimental testing of the columns in the last chapter. More details about the development and description of the octo-elliptical (0.6:1) loading protocol can be found in section 3.7 of chapter 3.
Table 6.2. Design and testing details of the column specimens

<table>
<thead>
<tr>
<th>No.</th>
<th>Width × Depth × Height (mm)</th>
<th>Concrete Cylinder Strength (MPa)</th>
<th>Transverse Reinforcement Ratio</th>
<th>Axial Load Ratio ( n ) ((P, kN))</th>
<th>Type of Loading Path</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>( f_{cm} ) ( \rho_{hx} )</td>
<td>( \rho_{hy} ) ( % )</td>
<td>( % )</td>
<td>( 0.15 \pm 0.045 ) ( (956 \pm 287) )</td>
</tr>
<tr>
<td>S13</td>
<td>250×300×2550</td>
<td>87</td>
<td>0.42</td>
<td>0.35</td>
<td>Octo-elliptical (0.6:1) with SS-VAL</td>
</tr>
<tr>
<td>S14</td>
<td>250×300×2550</td>
<td>85</td>
<td>0.42</td>
<td>0.35</td>
<td>Octo-elliptical (0.6:1) with NS-VAL</td>
</tr>
</tbody>
</table>

Figure 6.8. Synchronous axial load variation protocol used in the testing of specimen
S13: a) Y-direction; b) X-direction
6.5.2 Test Results and Discussion

The experimental results are discussed in detail in terms of general response and failure mechanisms, force-displacement behaviour, moment-curvature behaviour, axial displacement-lateral drift behaviour, energy dissipation and stiffness degradation. A detailed comparison with the corresponding behaviour of identical specimens tested under unidirectional and bidirectional lateral loading with CAL is also provided.
6.5.2.1 General Response and Failure Mechanisms

The specimens exhibited a predominantly flexure failure mode, as expected given the high shear-span ratio of the columns. Hairline flexural cracks were observed at small displacements, which were followed by vertical splitting cracks at larger displacements. The vertical splitting cracks then led to concrete spalling in the plastic hinge regions. Finally, longitudinal bar buckling and transverse bar fracture occurred simultaneously with the axial load failure of the specimen that occurred by the formation of a diagonal plane in the top plastic hinge region. Figure 6.11 shows the damaged specimens at axial load collapse. The figure indicates that the damaged region (plastic hinge region) has almost similar length under both axial load variation protocols.

Figure 6.11. Damage at axial load failure (collapse) – specimens S13-S14

Table 6.3 summarizes the drifts at the initiation of cracking, concrete spalling, lateral load failure and collapse for specimens S13 and S14 and compares with the corresponding drifts of the specimens tested in the previous chapters under unidirectional loading with CAL (S1) and bidirectional loading (octo-elliptical (0.6:1))
with CAL (S11), respectively. It can be observed in Table 6.3 that the type of loading does not have much effect on the initiation of cracking. With the exception of the specimen tested under NS-VAL (S14), the other specimens exhibited the same drift at the initiation of cracking. On the other hand, the effect of the type of loading can be clearly noticed on concrete spalling and rebar buckling and fracture drifts, which are around 35-50% lower for bidirectional loading compared to unidirectional loading. Table 6.3 further indicates that different damage states for specimens subjected to octo-elliptical (0.6:1) path with CAL (S11) and octo-elliptical (0.6:1) path with SS-VAL (S14) occur at the same drift. However, the drifts at different damage states of specimens tested under octo-elliptical (0.6:1) path with nonsynchronous axial load variation are lower than the other loading histories.

Table 6.3. Drifts at different damage states of the column

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of Loading</th>
<th>Y-direction drift (%) at the initiation of</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Visible Cracking</td>
</tr>
<tr>
<td>S13</td>
<td>Octo-elliptical (0.6:1) with SS-VAL (n=0.15±0.045)</td>
<td>0.9</td>
</tr>
<tr>
<td>S14</td>
<td>Bidirectional with NS-VAL (n=0.15±0.045)</td>
<td>0.7</td>
</tr>
<tr>
<td>S1</td>
<td>Unidirectional with CAL (n=0.15)</td>
<td>0.9</td>
</tr>
<tr>
<td>S11</td>
<td>Octo-elliptical (0.6:1) with CAL (n=0.15)</td>
<td>0.9</td>
</tr>
</tbody>
</table>
6.5.2.2 Force-Displacement Behaviour

The hysteretic curves for specimens, S13 and S14 in the X and Y-directions are shown in Figures 6.12 and 6.13, respectively. The points of lateral load failure (degradation of lateral strength by 20%) and axial load failure (loss of axial load carrying capacity) of the column have been marked on the hysteresis. It can be seen that the lateral load failure and axial load failure in the Y-direction occurred at 20-25% lower drift for specimen tested under nonsynchronous axial load variation (S14) compared to the specimen tested under synchronous axial load variation. The degradation in strength can also be observed to be steeper for specimen S14 as opposed to S13. The specimen, S13, showed different strengths (about 20%) at peak displacements in the positive and negative directions, especially in the Y-direction due to the different axial load ratios in the two directions (n=0.2 in positive direction and n=0.1 in the negative direction), whereas specimen S14 showed more symmetric strengths at peak displacements in the positive and negative cycles due to similar axial load ratio (n=0.2) in the two directions (refer Figure 6.9). The ultimate strength in the X-direction was 10-15% lower for nonsynchronous axial loading history compared to synchronous loading history, which is because, under nonsynchronous loading history, the axial load ratio was mostly minimum at peak displacements in the X-direction of the specimen.

Table 6.4 summarises the drifts at lateral load failure and axial load failure of all the 14 specimens tested in this study. The comparison of the results of specimens, S1, S11, S13 and S14 indicate that the drift capacity in the Y-direction under biaxial loading with CAL (S11) and VAL (S13 &S14) is significantly lower than the drift capacity under uniaxial loading (S1). Furthermore, the failure drifts were nearly similar for biaxial loading with CAL (S11) and biaxial loading with SS-VAL (S13). However, the failure drifts were 20-30% lower for specimens tested under biaxial loading with NS-VAL (S14).
Figure 6.12. Force-drift response in the X-direction of the specimens S13-S14.

Figure 6.13. Force-drift response in the Y-direction of the specimens S13-S14.
Table 6.4. Drifts at lateral load failure and axial load failure (Summary of all Tests)

<table>
<thead>
<tr>
<th>No.</th>
<th>$f_{cm}$ (MPa)</th>
<th>$\rho_h$ (%)</th>
<th>$n$</th>
<th>Type of Loading</th>
<th>Lateral Load Failure Drift (%)</th>
<th>Axial Load Failure Drift (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$X$</td>
<td>$Y$</td>
</tr>
<tr>
<td>S1</td>
<td>75.0</td>
<td>0.35</td>
<td>0.15</td>
<td>Unidirectional</td>
<td>3.0</td>
<td>4.8</td>
</tr>
<tr>
<td>S2</td>
<td>66.0</td>
<td>0.35</td>
<td>0.30</td>
<td>Unidirectional</td>
<td>2.2</td>
<td>3.2</td>
</tr>
<tr>
<td>S3</td>
<td>87.0</td>
<td>0.35</td>
<td>0.45</td>
<td>Unidirectional</td>
<td>1.0</td>
<td>1.1</td>
</tr>
<tr>
<td>S4</td>
<td>90.0</td>
<td>0.52</td>
<td>0.45</td>
<td>Unidirectional</td>
<td>1.1</td>
<td>1.4</td>
</tr>
<tr>
<td>S5</td>
<td>62.0</td>
<td>0.18</td>
<td>0.30</td>
<td>Unidirectional</td>
<td>1.8</td>
<td>1.9</td>
</tr>
<tr>
<td>S6</td>
<td>90.0</td>
<td>0.35</td>
<td>0.25</td>
<td>Unidirectional</td>
<td>1.9</td>
<td>2.6</td>
</tr>
<tr>
<td>S7</td>
<td>86.0</td>
<td>0.35</td>
<td>0.15</td>
<td>Linearised Circular (1:1)</td>
<td>1.8</td>
<td>1.6</td>
</tr>
<tr>
<td>S8</td>
<td>63.0</td>
<td>0.35</td>
<td>0.30</td>
<td>Linearised Circular (1:1)</td>
<td>1.4</td>
<td>1.3</td>
</tr>
<tr>
<td>S9</td>
<td>90.0</td>
<td>0.35</td>
<td>0.15</td>
<td>Octo-Elliptical (1:1)</td>
<td>1.8</td>
<td>1.8</td>
</tr>
<tr>
<td>S10</td>
<td>83.0</td>
<td>0.35</td>
<td>0.30</td>
<td>Octo-Elliptical (1:1)</td>
<td>1.1</td>
<td>1.1</td>
</tr>
</tbody>
</table>
### 6.5.2.3 Comparison of Force-Displacement Behaviour under Variable and Constant Axial Load Test

This subsection provides a comparison of the hysteretic behaviour of specimens tested under bidirectional loading with VAL (S13 and S14) and the identical specimen (S11) tested under bidirectional loading with CAL in chapter 5. It can be observed in Figures 6.14 (a), 6.15 (a) and Table 6.4 that the specimens tested under CAL and SS-VAL exhibited similar drift capacities, except in the positive X-direction where the specimen with CAL exhibited slightly lower drift capacity than the latter. The specimens also showed similar lateral strength in the positive direction, whereas in the negative direction, the lateral strength was lower for the specimen with synchronous axial load variation due to the lower level of axial load in the negative direction (refer Figure 6.8).

The hysteretic plots in Figures 6.14 (b) and 6.15 (b) indicate that specimen with nonsynchronous axial load variation exhibited lower drift capacity (about 25%) than the specimen with CAL. Moreover, the specimen with nonsynchronous axial load variation demonstrated considerably lower lateral strength capacity in the X-direction. This is because under nonsynchronous axial load variation protocol (refer Figure 6.9),

|    |   |   |   |   |   |   |   |   |
|----|---|---|---|---|---|---|---|
| S11| 105.0 | 0.35 | 0.15 | Octo-Elliptical (0.6:1) | 1.4 | 2.3 | 1.8 | 3.1 |
| S12| 74.0 | 0.35 | 0.30 | Octo-Elliptical (0.6:1) | 0.9 | 1.4 | 0.9 | 1.9 |
| S13| 87.0 | 0.35 | 0.15 ± 0.045 | Octo-Elliptical- (0.6:1) + SS-VAL | 1.4 | 2.3 | 1.9 | 3.1 |
| S14| 85.0 | 0.35 | 0.15 ± 0.045 | Octo-Elliptical- (0.6:1) + NS-VAL | 1.4 | 1.7 | 1.4 | 2.4 |

6.5.2.3 Comparison of Force-Displacement Behaviour under Variable and Constant Axial Load Test

This subsection provides a comparison of the hysteretic behaviour of specimens tested under bidirectional loading with VAL (S13 and S14) and the identical specimen (S11) tested under bidirectional loading with CAL in chapter 5. It can be observed in Figures 6.14 (a), 6.15 (a) and Table 6.4 that the specimens tested under CAL and SS-VAL exhibited similar drift capacities, except in the positive X-direction where the specimen with CAL exhibited slightly lower drift capacity than the latter. The specimens also showed similar lateral strength in the positive direction, whereas in the negative direction, the lateral strength was lower for the specimen with synchronous axial load variation due to the lower level of axial load in the negative direction (refer Figure 6.8).

The hysteretic plots in Figures 6.14 (b) and 6.15 (b) indicate that specimen with nonsynchronous axial load variation exhibited lower drift capacity (about 25%) than the specimen with CAL. Moreover, the specimen with nonsynchronous axial load variation demonstrated considerably lower lateral strength capacity in the X-direction. This is because under nonsynchronous axial load variation protocol (refer Figure 6.9),
the axial load ratio is mostly minimum, whenever the displacement is maximum in the positive or the negative X-direction.

Figure 6.14. Force-drift hysteretic response in the X-direction: a) constant (S11) vs synchronous axial load variation (S13); b) constant (S11) vs nonsynchronous axial load variation (S14).

Figure 6.15. Force-drift hysteretic response in the Y-direction: a) constant (S11) vs synchronous axial load variation (S13); b) constant (S11) vs nonsynchronous axial load variation (S14).

The comparison indicates that bidirectional lateral loading with synchronous axial load variation resulted in nearly similar displacement characteristics as bidirectional lateral
loading with CAL, whereas, on the contrary, bidirectional lateral loading with nonsynchronous axial load variation exhibited 20-25% lower collapse drift capacity than bidirectional lateral loading with CAL, thereby implying that such loading history is more damaging in the event of an earthquake. Figure 6.16 shows the effect of different loading paths on the axial load failure drift in the X and Y-directions of the specimens. It can be seen that the lowest drift capacity was exhibited under bidirectional lateral loading with nonsynchronous axial load variation, whereas the highest drift capacity was demonstrated under unidirectional lateral loading and CAL.

Figure 6.16. Effect of loading history on the axial load failure drift of the column: a) Y-direction; b) X-direction

6.5.2.4 Moment-Curvature Behaviour

The moment-curvature hysteretic curves at the top of the specimens are shown in Figures 6.17 and 6.18 for the Y and X-axis, respectively. The point corresponding to the ultimate moment capacity of the specimens has been marked on the hysteresis. It was observed that in the X-direction, the specimen tested under nonsynchronous axial load variation exhibited smaller curvature corresponding to the point of ultimate capacity compared to the specimen tested under synchronous axial load variation. This is because in nonsynchronous axial load variation protocol when the displacement is maximum in the X-direction, the axial load has the minimum value (refer Figure 6.9), whereas in synchronous axial load variation protocol when the displacement is
maximum in the X-direction, the axial load is closer to its mean value (refer Figure 6.8), instead of the minimum value.

In the Y-direction, the specimen tested under synchronous axial load variation showed quite different curvatures (corresponding to ultimate moment) in the positive and negative directions (due to the different axial load in the two directions in the synchronous axial load variation protocol) as opposed to the specimen tested under nonsynchronous axial load variation protocol, which showed more or less similar curvatures (due to similar axial load in the two directions in the nonsynchronous axial load variation protocol) corresponding to the ultimate moment capacity. Furthermore, the hysteretic response of the specimens tested under synchronous axial load variation is quite unsymmetric due to the unsymmetric axial load in the positive and the negative directions in the synchronous axial load variation protocol.

![Diagram](image)

Figure 6.17. Moment-curvature response in the Y-axis - specimens S13 - S14.
6.5.2.5 Axial Displacement-Lateral Drift Behaviour

The axial displacement-lateral drift behaviour in the X and Y-directions of the specimens S13 and S14 is shown in Figures 6.19 and 6.20, respectively. The comparison of behaviour in the X-direction shows that the specimen S13 tested under synchronous axial load variation exhibited a lot more axial elongation behaviour compared to the specimens S14 tested under nonsynchronous axial load variation. Specimen S13 underwent maximum axial elongation when the drift was zero in the X-axis, whereas S14 experienced most of the elongation when the drift was maximum in the X-axis. The pattern of axial displacement-lateral drift behaviour is actually a reflection of the imposed axial load and lateral displacement loading histories on the specimens. For the specimen tested under octo-elliptical (0.6:1) path and synchronous axial load variation protocol, when the displacement was zero in the X-axis, it was maximum in the Y-axis, which meant that the column would be in tension, thereby resulting in axial elongation. On the other hand, S14 experienced axial elongation when the drift was maximum in the X direction because under nonsynchronous axial load variation when the drift is maximum in the X-direction, the axial load is at its minimum value.

The axial displacement behaviour in the Y-direction of the columns as shown in Figure 6.20 indicates that the specimen under synchronous axial load variation protocol experienced a lot more axial elongation in the negative direction as opposed to the
positive direction. This is because the specimen was subjected to minimum axial load in the negative direction and was subjected to maximum axial load in the positive direction. Thus, axial displacement-lateral drift behaviour of the specimen, S13 was quite unsymmetric. On the other hand, specimen S14 experienced a concave down pattern of axial displacement-lateral drift in the Y-direction where the specimen underwent axial shortening at maximum displacements in the positive and the negative directions and experienced axial elongation at zero displacement. This is because, at peak displacements in the Y-direction, the axial load was maximum, whereas the axial load was minimum when the column was at the origin i.e. at zero displacement in the Y-axis.

Figure 6.19. Axial displacement-lateral drift behaviour of the column specimens S13 – S14 – X-direction
6.5.2.6 Energy Dissipation

The accumulative hysteretic energy dissipation of the specimens S13 and S14 subjected to octo-elliptical bidirectional load path and VAL is compared with the specimens tested previously under octo-elliptical path and CAL in Figure 6.21. The accumulative energy dissipation was determined by summing the energy dissipation in the X and Y directions of the specimens by calculating the area enclosed under the force-displacement hysteresis for each displacement increment. It was observed that among the specimens subjected to VAL, the specimen tested under synchronous axial load variation dissipated around 25% more energy than specimen tested under nonsynchronous axial load variation. This is because the specimen subjected to nonsynchronous axial load variation collapsed at 25% less drift than the specimen subjected to synchronous axial load variation.

On the other hand, the accumulative energy dissipation of the specimen tested under octo-elliptical (0.6:1) path and CAL was quite similar (difference of less than 6%) to the specimen subjected to octo-elliptical (0.6:1) path and synchronous axial load variation. However, the accumulative energy dissipation of the specimen subjected to octo-elliptical (1:1) path and CAL was around 10% and 30% higher than the specimens
tested under octo-elliptical (0.6:1) path with synchronous and nonsynchronous axial load variation, respectively. This is because the column travels more distance under octo-elliptical (1:1) path compared to octo-elliptical (0.6:1) displacement path. As such, the highest energy was dissipated by the column under octo-elliptical (1:1) path and CAL, whereas the lowest energy was dissipated by the column under octo-elliptical (0.6:1) path and nonsynchronous axial load variation due to smaller collapse drift capacity.

![Graph showing accumulative hysteretic energy dissipation under CAL and VAL.]

Figure 6.21. Comparison of accumulative hysteretic energy dissipation under CAL and VAL

### 6.5.2.7 Effective Moment of Inertia and Stiffness Degradation

The effective moment of inertia of the specimens was determined from the experimental stiffness of the column. The effective moment of inertia \(I_{eff}\) for each specimen was calculated based on the gradient of a line taken from the origin through the drift corresponding to the development of the theoretical yield capacity. The theoretical yield capacity was taken as the lateral force corresponding to the development of a theoretical yield moment, which was taken as the moment corresponding to the development of a compressive strain corresponding to the maximum unconfined compressive strength (calculated using the Karthik and Mander model [118]) being reached in the extreme compressive fibre of the concrete or tensile yielding of the reinforcement, whichever occurred first. The theoretical yield capacity was calculated using the actual material properties (i.e. for steel and concrete) for the respective specimens. The effective moment of inertia was then normalized with
respect to the gross moment of inertia ($I_g$) in the two directions of the columns to study the effect of the loading history on $I_{eff}$. Table 6.5 presents a comparison of the effective moment of inertia of the specimens in the X and Y-directions under various loading protocols.

It can be observed in Table 6.5 that the ratio of effective to the gross moment of inertia in the X-axis of the columns, S11 and S14, is generally 8-13% higher compared to specimens, S1 and S13. The ratio is higher for specimen S11 due to its higher concrete compressive strength ($f_{cm}=105$ MPa), whereas, the ratio is lower for specimen S1 tested under unidirectional loading due to its relatively lower concrete compressive strength ($f_{cm}=75$ MPa) than other specimens. Similarly, the ratio of effective to gross moment of inertia is higher for specimen S14 because it is subjected to maximum axial load ($n=0.15+0.045$) whenever the amplitude of displacement (whether positive or negative) is maximum in the Y-direction, whereas in the case of specimen S13, axial load was maximum ($n=0.15+0.045$) when the displacement amplitude was maximum in the positive direction and minimum ($n=0.15-0.045$) when the displacement amplitude was maximum in the negative direction and hence, the ratio of moment of inertia is relatively lower for specimen S13 due to the fluctuating axial load ratio at maximum amplitude of displacements.

In the Y-axis of the column, the ratio of effective to gross moment of inertia is relatively lower (13%) for specimen S14 than S13 because, under the nonsynchronous loading protocol, the axial load ratio stays mostly at its minimum value ($n=0.15-0.045$) in the X-direction of the column, which is in contrast with the synchronous loading protocol, where the axial load ratio is mostly within the intermediate range (between the maximum and minimum value) at the maximum amplitude of displacements.

In order to study the progression of stiffness degradation with the increasing lateral drifts, the stiffness ($K_i$) at the peak displacement excursion of each cycle was calculated and then plotted with the corresponding lateral drift. The average stiffness at each displacement excursion was determined using equation 4.3.
Figure 6.22 demonstrates that the progression of stiffness degradation with the increasing lateral drifts for various loading histories differs slightly. It can be observed in Figure 6.22 (a) that in the X-axis of the column, stiffness degradation is steepest for biaxial lateral loading with nonsynchronous axial loading history, which is followed by biaxial lateral loading with constant and synchronous axial loading histories, respectively. Unidirectional lateral loading with CAL, on the other hand, demonstrated a gradual stiffness degradation.

Similarly, in the Y-axis of the column, steepest stiffness degradation was exhibited by biaxial lateral loading with nonsynchronous axial loading history, followed by constant axial loading and synchronous axial loading histories, respectively. Data was not available to make a comparison with unidirectional loading history in the Y-axis of the column.

Table 6.5. Effect of loading path on the effective moment of inertia of the column specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Type of Loading</th>
<th>Ratio of Effective to Gross Moment of Inertia</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$I_{eff,y} / I_{g,y}$</td>
</tr>
<tr>
<td>S13 ($f_{cm}$ =87 MPa)</td>
<td>Octo-elliptical (0.6:1) with SS-VAL ($n=0.15\pm0.045$)</td>
<td>0.31</td>
</tr>
<tr>
<td>S14 ($f_{cm}$ =85 MPa)</td>
<td>Octo-elliptical (0.6:1) with NS-VAL ($n=0.15\pm0.045$)</td>
<td>0.27</td>
</tr>
<tr>
<td>S1 ($f_{cm}$ =75 MPa)</td>
<td>Unidirectional with CAL ($n=0.15$)</td>
<td>-</td>
</tr>
<tr>
<td>S11 ($f_{cm}$ =105 MPa)</td>
<td>Octo-elliptical (0.6:1) with CAL ($n=0.15$)</td>
<td>0.35</td>
</tr>
</tbody>
</table>
Summary and Conclusions

This chapter presented the results of a numerical and experimental study on axial load variation in RC columns under earthquake actions. In the numerical study, the typical patterns and range of axial load variation in columns were investigated by subjecting a case study building to a suite of 15 unscaled ground motions, representative of low to moderate seismic regions. This was then followed by an experimental study in which two limited ductile HSRC columns were respectively tested under the two generalised patterns of variable axial loading histories and results were compared with identical specimens tested in the previous chapters under uniaxial and biaxial lateral loading with CAL. The findings of the chapter are summarised as follows:

1. The numerical study revealed that the mean axial load variation in the corner perimeter, internal perimeter and internal non-perimeter columns of the case study building was 43%, 12% and 10%, respectively, for the ground motions considered in this study. The variation seems to be quite significant for the corner perimeter columns, considering that the selected ground motions were representative of low to moderate seismic regions, and were unscaled.

2. No definite correlation was observed between PGA of the ground motions and axial load variation of columns. The axial load variation in corner perimeters columns was
found to be a function of the RSA ($T_x=1.14s$ and $T_y=1.48s$) of the two orthogonal horizontal components of the ground motion, whereas the axial load variation in the internal non-perimeter columns was mainly dependent on the RSA ($T_z=0.17s$) of the vertical component of the ground motion.

3. Expressions were proposed that related the axial load variation in the columns with the RSA and the structural configuration of the building. The proposed expressions estimate the axial load variation in the corner perimeter and internal non-perimeter columns of the case-study building frame system with reasonable accuracy.

4. The lateral load failure and axial load failure drifts of the column specimen tested under biaxial lateral loading with CAL and SS-VAL histories were similar, whereas the drift capacity for specimen subjected to biaxial lateral loading with NS-VAL (in which variation of axial load is nonsynchronous to the lateral displacement of the building) was about 25% lower, thereby implying that nonsynchronous axial loading history is more damaging in the event of an earthquake. Furthermore, the accumulative energy dissipation was also found to be lowest for specimen subjected to nonsynchronous axial load variation.

5. The maximum strength of the specimen under nonsynchronous axial loading history was found to be lower (10-15%) in the X-direction of the column compared to synchronous loading history, whereas in the Y-direction of the column, the ultimate strengths were different in the positive and negative direction for synchronous loading history (due to different axial load ratios at peak displacements in the two directions) and similar in case of nonsynchronous loading history (due to similar axial load ratio at peak displacements in the two directions).

6. The type of variable axial loading history (synchronous or nonsynchronous) seem to influence the effective moment of inertia and stiffness degradation of the specimens only slightly, and as such the effective moment of inertia of the specimens under the two loading histories differed by just 10-13%.
Chapter 7 Model for Predicting Force-Drift Capacity of RC Columns

7.1 Introduction

Generally, structural design engineers are well versed with the strength behaviour of RC columns, whilst there is a lack of understanding of the corresponding drift behaviour, especially in the inelastic range. For the displacement-based design approach, it is very important to have a reliable prediction of the drift behaviour of RC columns in both the elastic and inelastic range. However, no unified drift models exist that could predict the drift capacity of both NSRC and HSRC columns in the inelastic range. Also, it was shown in the review of existing models in section 2.8 of chapter 2 that the models developed for NSRC columns are not suitable for HSRC columns. This chapter addresses this research gap by proposing a unified set of empirical drift models that have a unique ability to predict post-peak drift capacity of limited to moderately ductile NSRC as well as HSRC columns. This is followed by a detailed force-drift backbone model that can estimate the experimental response of an RC column in the elastic as well as inelastic range under unidirectional and bidirectional lateral loading. The last section of the chapter then discusses the applications of the proposed model, including the ability to check the seismic compliance of limited to moderately ductile RC columns in regions of low to moderate seismicity.

7.2 Development of Post-Peak Drift Models

The term ‘post-peak’ refers to lateral load failure and axial load failure limits in a force-displacement backbone envelope. Lateral load failure refers to 20% degradation in the lateral strength of the column, while axial load failure denotes loss of axial load carrying capacity of the column. These models have been developed by analysing the influence of the design parameters on the drift capacity of RC columns using general and detailed trend analysis. The incorporation of the effect of increasing concrete compressive
strength in reducing the drift capacity of RC columns is a distinct feature of these models, which makes them suitable for both NSRC and HSRC columns. This section presents the development of the drift models for lateral load and axial load failure limit states.

7.2.1 Experimental Database

The experimental database is the same as the one used in the review of existing drift models in chapter 2. The database comprised of 190 columns (79 HSRC and 111 NSRC columns) from 44 studies. Readers are referred to section 2.8.1 and Appendix A of the thesis for more details regarding the experimental database.

7.2.2 Lateral Load Failure Drift Model

The column drift capacity at lateral load failure was studied using an extensive database of 166 columns (96 NSRC and 70 HSRC). Due to the remarkable difference in the drift performance of columns failing in flexure and shear, separate drift models are proposed for flexure-critical and shear-critical RC columns. The influence of six design parameters, namely, aspect ratio \((L/h)\), longitudinal reinforcement ratio \((\rho_v)\), transverse reinforcement ratio \((\rho_h)\), transverse reinforcement yield strength \((f_{yh})\), concrete compressive strength \((f'_c)\) and axial load ratio \((n)\) on the lateral load failure drift capacity of the RC column is summarized in the next section.

7.2.2.1 Influence of Design Parameters on Lateral Load Failure Drift

The influence of the design parameters on the lateral load failure drift has been evaluated at two levels. First, using general trend analysis in which lateral load failure drift was plotted as a function of six design parameters as shown in Figure 7.1 and then using detailed trend analysis in which dependency from the other parameters was also taken into account. In detailed trend analysis, lateral load failure drift was plotted as a function of each of the six design parameters, while other parameters were kept constant as shown in Figure 7.2. Based on the general and detailed trend analysis, the influence of these six design parameters on lateral load failure drift can be summarized as follows:
**Aspect Ratio**: The results of the general trend analysis showed that the aspect ratio had more significant effect on lateral load failure drift of shear-critical columns as compared to flexure-critical columns. The detailed trends verified this observation that with the increase in aspect ratio, lateral load failure drift of shear-critical columns increases. Data was not available for detailed trend analysis of aspect ratio with lateral load failure drift of flexure-critical columns.

**Longitudinal Reinforcement Ratio**: The general trend analysis graph of longitudinal reinforcement ratio versus lateral load failure drift showed a lot of scatter. Similarly, detailed trends also showed mixed behaviour and hence no direct correlation was observed between lateral load failure drift and longitudinal reinforcement ratio for both flexure and shear-critical NSRC and HSRC columns.

**Transverse Reinforcement Ratio**: Both general and detailed trend analyses showed an increase in lateral load failure drift with the increase in transverse reinforcement ratio for both flexure and shear-critical RC columns.

**Transverse Reinforcement Yield Strength**: Not much data was available to study the influence of the variation of transverse reinforcement yield strength on the lateral load failure drift of RC columns, as most of the tests in the literature did not consider it as a variable. Moreover, the results of detailed trend analysis showed an insignificant increase in the lateral load failure drift with the increase of transverse reinforcement yield strength.

**Concrete Compressive Strength**: The results of general trend analysis showed that concrete compressive strength had a more significant effect on lateral load failure drift of flexure-critical columns as compared to shear-critical columns. This observation was confirmed by detailed trend analysis results, which also showed a decrease in lateral load failure drift with the increase in concrete compressive strength for flexure-critical columns.

**Axial load ratio**: It was observed that amongst all the design parameters, the axial load ratio had the most significant effect on lateral load failure drift of RC columns. As such, detailed trend analysis graphs showed a substantial reduction in lateral load failure
drift with the increase in axial load ratio for both flexure and shear-critical NSRC and HSRC columns.

Figure 7.1. General trends of the relationship between design parameters and lateral load failure drift of columns
Figure 7.2. Detailed trends of the relationship between design parameters and lateral load failure drift of columns
7.2.2.2 Proposed Models

The results of the general and detailed trend analysis indicated that lateral load failure drift of flexure-critical columns is much dependent on axial load ratio, transverse reinforcement ratio, transverse reinforcement yield strength and concrete compressive strength, whereas lateral load failure drift of shear-critical columns is dependent on axial load ratio, transverse reinforcement ratio and aspect ratio. Hence, expressions were developed by curve fitting of experimental data to predict the lateral load failure drift of flexure-critical and shear-critical RC columns. It was observed that if very high strength transverse reinforcement was used then it did not contribute much to increase the drift at lateral load failure. Thus a maximum limit of $f_{yh} = 500$ MPa was imposed on the expressions developed for predicting lateral load failure drift. These expressions can be used for NSRC as well as HSRC columns. Separate expressions were developed to be used with area ratio and volumetric ratio of transverse reinforcement, respectively.

\[
(\delta_{lf})_{flexure} = 3(1 - 2n) + \rho_h \sqrt{\frac{f_{yh}}{f'_c}} \quad (7.1a)
\]

or

\[
(\delta_{lf})_{flexure} = 3(1 - 2n) + 0.5 \rho_s \sqrt{\frac{f_{yh}}{f'_c}} \quad (7.1b)
\]

\[
(\delta_{lf})_{shear} = 1.75(2.3\rho_h - n) + 0.8 \left(\frac{L}{h} - 1\right) \quad (7.2a)
\]

or

\[
(\delta_{lf})_{shear} = 1.75(1.15\rho_s - n) + 0.8 \left(\frac{L}{h} - 1\right) \quad (7.2b)
\]

The proposed empirical expressions (7.1a and 7.1b) are applicable within the following range of parameters for flexural critical columns: $12.1 \leq f'_c \leq 104.3$ MPa, $0.07\% \leq \rho_h \leq 1.0\%$, $0.15\% \leq \rho_s \leq 2.47\%$, $f_{yh} \leq 500$ MPa and $0.027 \leq n \leq 0.5$, whereas the proposed empirical expressions (7.2a and 7.2b) are applicable within the following range of
parameters for shear critical columns: $13.5 \leq f'_c \leq 100$ MPa, $0.07\% \leq \rho_h \leq 0.4$, $0.15\% \leq \rho_s \leq 0.9\%$, $0.072 \leq n \leq 0.44$ and $1.0 \leq L/h \leq 2.0$

The models were calibrated using a comprehensive database of 166 columns (70 HSRC and 96 NSRC) from the literature and did not include the experimental tests conducted as part of this research. The proposed expressions offer very good accuracy and fit the experimental data quite well as shown in Figures 7.3 and 7.4. The coefficient of variation of the proposed model for flexure-critical columns (46 HSRC and 61 NSRC) is 0.3, whereas the coefficient of variation of the proposed model for shear-critical columns (24 HSRC and 35 NSRC) is 0.33. It can be seen from the graphs that the lateral load failure drift model for flexure-critical columns is giving very conservative results for a few NSRC columns. These columns have large transverse reinforcement ratio in the range of 0.5-1.0\%, which is very rare in practice for limited ductile columns. Hence, the authors chose to fit the curve more closely to the columns that are representative of the standard detailing practice for limited ductile columns.

![Figure 7.3. Comparison of experimental drifts with the proposed model drift predictions for flexure-critical columns at lateral load failure: (a) area reinforcement (using Eq 7.1a); (b) volumetric reinforcement (using Eq 7.1b)]
7.2.3 Axial Load Failure Drift Model

Axial load failure drift of column was studied using a database of 82 columns (47 NSRC and 35 HSRC) from the literature. The influence of six design parameters, namely, aspect ratio ($L/h$), longitudinal reinforcement ratio ($\rho_v$), transverse reinforcement ratio ($\rho_h$), transverse reinforcement yield strength ($f_{yk}$), concrete compressive strength ($f'c$) and axial load ratio ($n$) on the drift capacity of the column at axial load failure is summarized in the next section.

7.2.3.1 Influence of Design Parameters on Axial Load Failure Drift

Based on the general and detailed trend analysis shown in Figures 7.5 and 7.6, the influence of the six design parameters on the axial load failure drift of columns can be described as follows:

Aspect Ratio: No definite relationship was observed between aspect ratio and axial load failure drift of both NSRC and HSRC columns from general trend analysis. Similarly, detailed trend analysis graphs also showed mixed trends, thereby confirming that axial load failure drift has no direct correlation with the aspect ratio.
*Longitudinal Reinforcement Ratio:* The detailed trend analysis showed mixed results, with axial load failure drift showing both increasing and decreasing trends with the increase of longitudinal reinforcement ratio. Hence, no definite relationship was observed between the longitudinal reinforcement ratio and axial load failure drift.

*Transverse Reinforcement Ratio:* Both general and detailed trend graphs indicate that axial load failure drift shows a strongly increasing relationship with the increasing transverse reinforcement ratio. The scatter in the general trend graph; however, indicates some dependency on the other parameters.

*Transverse Reinforcement Yield Strength:* Nothing conclusive could be established about the influence of transverse reinforcement yield strength on axial load failure drift of the column from the general trend analysis. However, detailed trend analysis, in which all other parameters were kept constant, showed an increase in axial load failure drift with the increase of transverse reinforcement yield strength.

*Concrete Compressive Strength:* General trend analysis showed that concrete compressive strength did not have much effect on axial load failure drift of NSRC columns. However, for HSRC columns increase in concrete compressive strength significantly reduced the axial load failure drift. Data was not available for delineating the influence of concrete compressive strength on the axial load failure drift while keeping other parameters constant.

*Axial Load Ratio:* Axial load ratio has the most significant impact on axial load failure drift of both NSRC and HSRC columns. Both the general and detailed trend analysis showed a strongly decreasing relationship with the increasing axial load ratio for both NSRC and HSRC columns.
Figure 7.5. General trends of the relationship between design parameters and axial load failure drift of columns
Figure 7.6. Detailed trends of the relationship between design parameters and axial load failure drift of columns
7.2.3.2 Proposed Models:

It can be concluded from the results of general and detailed trend analysis that axial load failure drift of normal and HSRC columns is dependent on axial load ratio, transverse reinforcement ratio, concrete compressive strength and transverse reinforcement yield strength. Using these parameters, empirical expressions are developed for predicting the axial load failure drift of NSRC and HSRC columns by curve fitting the results of 82 columns (47 NSRC and 35 HSRC) from the literature. It is noted that the expressions were not calibrated with the experimental results of this study. These expressions can be used with transverse reinforcement ratio by area as well as transverse reinforcement ratio by volume and are applicable to both NSRC and HSRC columns. The proposed expressions fit the experimental data very well as shown in Figure 7.7. The expressions were calibrated with columns having both flexure and shear modes of failure, so they are equally applicable to both type of columns. Moreover, the proposed models have a very good coefficient of variation of 0.25. The proposed expressions are as follows:

\[ \delta_{af} = 5(1-2n) + \rho_h \sqrt{\frac{f_{yh}}{f'_c}} \]  

(7.3a)

or

\[ \delta_{af} = 5(1-2n) + 0.5 \rho_s \sqrt{\frac{f_{yh}}{f'_c}} \]  

(7.3b)

The proposed empirical expressions are applicable within the following wide range of parameters: 13.5 \( f'_c \) \leq 104.3 MPa, 0.07% \leq \rho_h \leq 0.92%, 0.15% \leq \rho_s \leq 2.65%, 240 \leq f_{yh} \leq 1360\,\text{MPa} and 0.05 \leq n \leq 0.5
This section presents a piecewise backbone model that can estimate the force-displacement capacity of the NSRC and HSRC columns in both the elastic and inelastic range. The model was originally proposed by Wilson et al. [79] for NSRC columns subjected to unidirectional lateral actions and has been herein refined and further developed to estimate the capacity of both NSRC and HSRC columns subjected to unidirectional as well as bidirectional lateral actions. The unidirectional model is first presented and later some factors are proposed to account for bidirectional actions based on the results of the experimental testing program conducted by the author and previous research in the literature. It is shown that the proposed model reproduces the experimental backbone envelope of the specimens tested in this study under unidirectional and bidirectional loading very well.

### 7.3.1 Force-Drift Model for Unidirectional Lateral Loading

The piecewise Force-Drift backbone model is defined by five points, namely, cracking point, yield point, ultimate point, lateral load failure (20% degradation in ultimate force capacity) and axial load failure (50% degradation in ultimate force capacity) as shown in Figure 7.8. The model calculates the backbone envelope of a cantilevered column.
bent in single curvature. For columns bent in double curvature, this model can be used by taking the shear span length (i.e. \( L \)) as half of the column height and then multiplying the predicted displacements by a factor of two.

![Figure 7.8. Piecewise force-drift model](image)

**Point A (Cracking):** Cracking point refers to the point when a RC column starts developing cracks. Force and drift corresponding to cracking point can be determined using the principles of basic mechanics (i.e. Euler’s beam theory).

\[
(F_{cr})_{\text{uniaxial}} = \frac{(f_t + \frac{P}{A_g}) I_g}{L \bar{x}} \quad (7.4a)
\]

\[
(\delta_{cr})_{\text{uniaxial}} = \frac{M_{cr} L}{3E_c I_g} \quad (7.4b)
\]

**Point B (Yield):** Elastic behaviour is maintained by the column until the yield point on the force-drift backbone model. The yield force and curvature can be calculated corresponding to the point of first yield, based on the development of a compressive strain corresponding to the maximum unconfined compressive strength (calculated using the Karthik and Mander [118] model) being reached in the extreme compressive fibre of the concrete or tensile yielding of the reinforcement, whichever occurred first. Subsequently, yield drift can be determined from equations (7.5b) or (7.5c) given below.
\[(F_y)_{\text{uniaxial}} = \frac{M_y}{L}\]  
(7.5a)

\[(\delta_y)_{\text{uniaxial}} = \frac{1}{3} \bar{\theta}_y L\]  
(7.5b)

\[(\delta_y)_{\text{uniaxial}} = \frac{M_y L}{3E_c I_{\text{eff}}}\]  
(7.5c)

Effective second moment of area \(I_{\text{eff}}\) can be determined using ASCE 41-17 (2017) equations 4.4 (g) and 4.4 (h) given in chapter 4.

**Point C (Ultimate):** The ultimate strength point refers to the point of maximum force capacity. Ultimate strength design method can be used for the calculation of force capacity. The sum of yield drift and plastic drift gives the ultimate drift of the column. The plastic drift can be determined using plastic hinge length and ultimate curvature of the section.

\[(F_u)_{\text{uniaxial}} = \frac{M_u}{L}\]  
(7.6a)

\[(\delta_u)_{\text{uniaxial}} = (\delta_y)_{\text{uniaxial}} + \delta_{\text{pl}}\]  
(7.6b)

\[\delta_{\text{pl}} = (\bar{\theta}_u - \bar{\theta}_y)L_p\]  
(7.6c)

**Point D (Lateral Load Failure):** This point corresponds to 20% reduction in the ultimate force capacity of the column. It is considered as conventional failure point in high seismic regions. The following empirical equation developed in the last section can be used to determine lateral load failure drift of flexure-critical HSRC and NSRC columns:

\[\left(\frac{F_{lf}}{F_u}\right)_{\text{uniaxial}} = 0.8 \times F_u\]  
(7.7a)

\[\left(\delta_{lf,\text{flexure}}\right)_{\text{uniaxial}} = 3(1 - 2n) + \left(\rho_h \frac{f_{yh}}{f'_c}\right)\]  
(7.7b)

The proposed equation for drift is valid within the following range:

12.1 \leq f'_c \leq 104.3 \text{ MPa}, 0.07\% \leq \rho_h \leq 1.0\%, f_{yh} \leq 500 \text{ MPa and } 0.027 \leq n \leq 0.5

**Point E (Axial Load Failure):** Axial load failure refers to the loss of axial load carrying capacity of the column. The lateral force at axial load failure can be conservatively
assumed as 50% of the ultimate force capacity, whereas the equation proposed in the last section can be used for calculating the axial load failure drift of HSRC and NSRC columns.

\[
(F_{af})_{uniaxial} = 0.5 \times F_u \tag{7.8a}
\]

\[
(\delta_{af})_{uniaxial} = 5(1 - 2n) + \left( \rho_h \frac{f_{yh}}{f'_c} \right) \tag{7.8b}
\]

The proposed equation is valid within the following range:

\[13.5 \leq f'_c \leq 104.3 \text{ MPa}, 0.07\% \leq \rho_h \leq 0.92\%, 240 \leq f_{yh} \leq 1360 \text{ MPa} \text{ and } 0.05 \leq n \leq 0.5\]

Figure 7.9 presents a comparison of the experimental force-drift hysteretic results of the specimens S1-S6, tested in chapter 4 of the thesis with the backbone envelope predicted by the model. It can be observed that the predictions of force and drift are in a very good agreement with the experimental results for all the six specimens. In particular, the post-peak drift capacity predicted by expressions (7.7b) and (7.8b) is in a very close agreement with the experimental results. The coefficient of variation between the experimental results and the empirical predictions was found to be 0.17 and 0.18 for the lateral load failure and axial load failure drift models, respectively. It is worth noting that the experimental results of these specimens were not used to develop the models for lateral load failure and axial load failure drifts. Therefore, the good correlation with the experimental test results of this study further emphasises the validity of the model.
A comparison of the drift prediction at axial load failure by the proposed model (Eq 7.8b) and other existing models in the literature is provided in Table 7.1. The coefficient of variation of the predicted drifts to experimental drifts for various models is summarised in the table. The coefficient of variation shows that Zhu et al. (2007), Elwood and Moehle (2003) and the proposed model give the closest approximations to the experimental results. On the other hand, the models by Tran and Li (2013) and ASCE 41-17 (2017) largely underestimate, whereas models of Eurocode 1998-3 (2005) and Wibowo et al. (2014) largely overestimate the axial load failure drifts of the specimens.

It can also be seen in Table 7.1 that the proposed model has the lowest coefficient of variation. The other models from the literature were primarily intended (developed) for normal-strength RC columns and were not calibrated for high-strength RC columns, and that perhaps explains some of the discrepancies between the estimated drifts and the experimental values.
Table 7.1. Comparison of Experimental Drifts with Existing Predictive Models

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**Design Axial Load Ratio Limit under Unidirectional Earthquake Actions:**

Eq 7.8b can be readjusted to give Eq 7.9 that can then be used to determine the upper limit of design axial load ratio \(n_d\) that will result in any required drift capacity for a given column. It is noted that a factor of safety \(\varphi_f = 0.85\) has been added in the expression to account for uncertainties in the prediction. The factor of safety is suggested based on the observed coefficient of variation, which was 0.18 (i.e. 18% variability from actual values) for the prediction given by Eq 7.8b for the experimental tests conducted in this study.

\[
n_d = \frac{1}{2} \left( 1 - 0.25 \left( \frac{\delta_{af}}{\varphi_f} - \rho_h \frac{f_{yh}}{f'_c} \right) \right) \tag{7.9}
\]

### 7.3.2 Force-Drift Model for Bidirectional Lateral Loading

The force-drift backbone model presented in the previous section is further developed and modified in this section to account for the bidirectional actions using a set of modification factors. The modification factors are developed on the basis of the observed biaxial response in the experimental study presented in chapter 5 of the thesis and also from the observations in previous studies in the literature.
It was reported in the chapter 5 that, when equal displacements were applied in the two axes of the column, the peak lateral force capacity in both axes of the tested column specimens was 10-15% lower than the theoretical force capacity, whereas, the force capacity in the X-direction was found to be 30-40% lower than the theoretical capacity when displacements in the X-direction were 60% of the corresponding value in the Y-direction. To account for this behaviour, a force reduction factor $\alpha_{1,2}$ is proposed.

A significant reduction in the drift capacity was also reported under bidirectional loading as opposed to unidirectional loading. A displacement reduction factor $\gamma_{1,2}$ is proposed to account for the ratio of imposed displacements in the two axes of the column. This factor applies to all five points of the backbone envelope (i.e. points A to E). In addition, another displacement reduction factor called the ‘biaxial interaction factor’ (i.e. $\beta$) is used to further reduce the post-peak displacement capacity (i.e. points D and E), based on the observations from previous studies in literature and the current study.

Table 7.2 presents a summary of the uniaxial post-peak drift capacity reductions found in the literature (and also from the current study) where RC columns were tested under bidirectional loading. The results of the previous studies on the bidirectional testing of RC columns in the literature show a substantial decrease in the uniaxial drift capacity of the column under bidirectional loading. However, there is a lot of variability in results as far as the percentage reduction in drift capacity is concerned. The results are summarized in terms of the ratio of displacements imposed in the two axes of the columns (i.e. $x/y$). It should be noted that the drift reduction percentages given in the table are the maximum reductions that were observed under a given protocol. In view of the results presented in Table 7.2, it can be concluded that the uniaxial drift capacity of the column can generally be expected to reduce between 15-55% under bidirectional loading, depending upon the type of the loading path and the $x/y$ ratio. Hence, the proposed biaxial interaction factor, $\beta$ is dependent on the $x/y$, that is the ratio of the enveloped non-concurrent maximum displacement in the smaller loading direction and the larger loading direction.
Table 7.2. Summary of literature on drift reduction under different bidirectional loading protocols

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</tbody>
</table>
It was reported in Chapter 5 of the thesis that the uniaxial drift capacity of the column reduced by 50% when $x/y$ was 1. On the other hand, the summary results of previous studies in Table 7.2 show that depending upon the type of bidirectional loading path, the reduction in drift capacity can be in the range of 20-55% under bidirectional loading when $x/y = 1$. Thus, for the sake of conservatism, the expression for $\beta$ is proposed as such that $\beta=0.5$ (50% reduction), when $x/y = 1$. Similarly, it was observed in this study that the drift capacity reduced by around 35% when $b/a$ was 0.6, whereas studies in literature which used $x/y = 0.6$ and $x/y = 0.5$ reported a maximum upper bound reduction of 38% and 32%, respectively, in the uniaxial drift capacity of the column as shown in Table 7.2. These observations were taken into account and accordingly, the proposed expression conservatively gives $\beta=0.63$ (37% reduction) and $\beta=0.67$ (33% reduction), when $x/y = 0.6$ and $x/y = 0.5$, respectively.

The modifications factors for direction 1 and direction 2 loading are presented in Table 7.3. Direction 1 loading corresponds to the direction of the column (i.e. Y or X-direction), which has the dominant (i.e. larger) lateral displacement applied to it.

Table 7.3. Modification factors for biaxial behaviour

<table>
<thead>
<tr>
<th>Force reduction factor (applies to Points A to E)</th>
<th>Direction 1 loading</th>
<th>Direction 2 loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_1 = 1 - 1/9 \times (x/y)$</td>
<td>$\alpha_2 = 0.625 \times (x/y) + 0.275$</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Displacement reduction factor (applies to Points A to E)</th>
<th>Direction 1 loading</th>
<th>Direction 2 loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_1 = 1.0$</td>
<td>$\gamma_2 = x/y$</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Biaxial interaction factor (applies to Points D and E only)</th>
<th>Direction 1 loading</th>
<th>Direction 2 loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta = \left( \frac{1}{1 + x/y} \right)$</td>
<td>$\beta = \left( \frac{1}{1 + x/y} \right)$</td>
<td></td>
</tr>
</tbody>
</table>

Note: These modification factors are proposed for $0.1 \leq x/y \leq 1.0$. If $x/y < 0.1$, the behaviour is considered predominantly unidirectional, i.e. $\alpha = \gamma = \beta = 1.0$.

The expressions for each point of the force-drift backbone model with the incorporation of modification factors that take into account the bidirectional behaviour are presented below:
**Point A (Cracking):** The following expressions can be used to determine the force and corresponding drift at the cracking point under bidirectional lateral loading.

\[
(F_{cr})_{\text{biaxial}} = \alpha_{1,2} \frac{(f_t + \frac{P}{A_g}) l_g}{\bar{x} L} \quad (7.10a)
\]

\[
(\delta_{cr})_{\text{biaxial}} = \gamma_{1,2} \times \frac{M_{cr} L}{3E_c l_g} \quad (7.10b)
\]

**Point B (Yield):** The force and drift at yield point of an RC column subjected to bidirectional lateral actions can be calculated as follows:

\[
(F_y)_{\text{biaxial}} = \alpha_{1,2} \times \frac{M_y}{L} \quad (7.11a)
\]

\[
(\delta_y)_{\text{biaxial}} = \gamma_{1,2} \times \frac{1}{3} \delta_y L \quad (7.11b)
\]

\[
(\delta_y)_{\text{biaxial}} = \gamma_{1,2} \times \frac{M_y L}{3E_c l_{eff}} \quad (7.11c)
\]

**Point C (Ultimate):** The following expressions give the force and the corresponding drift at the ultimate point on the force-drift backbone envelope under bidirectional actions:

\[
(F_u)_{\text{biaxial}} = \alpha_{1,2} \times \frac{M_u}{L} \quad (7.12a)
\]

\[
(\delta_u)_{\text{biaxial}} = \gamma_{1,2} \times (\delta_y + \delta_{pl}) \quad (7.12b)
\]

\[
\delta_{pl} = (\delta_u - \delta_y) L_p \quad (7.12c)
\]

**Point D (Lateral Load Failure):** The force and drift at lateral load failure of an RC column subjected to bidirectional earthquake actions can be calculated as follows:

\[
(F_{lf})_{\text{biaxial}} = 0.8 \times F_u \quad (7.13a)
\]

\[
(\delta_{f_{\text{flexure}}})_{\text{biaxial}} = \beta \times \gamma_{1,2} \times \left(3(1 - 2n) + \left(\rho_h \frac{f_{yh}}{f_c}\right)\right) \quad (7.13b)
\]
**Point E (Axial Load Failure):** Axial load failure point under bidirectional lateral actions can be determined as follows:

\[
(F_{af})_{\text{biaxial}} = 0.5 \times F_u
\]  \hspace{1cm} (7.14a)

\[
(\delta_{af})_{\text{biaxial}} = \beta \times \gamma_{1.2} \times \left( 5(1 - 2\eta) + \left( \rho_h \frac{f_{yh}}{f'_c} \right) \right)
\]  \hspace{1cm} (7.14b)

The effect of \(x/y\) ratio on the force-drift behaviour of the column is illustrated in Figure 7.10 using the proposed model, where the uniaxial response in the two directions of a case study column is plotted with the corresponding biaxial response for different ratios of displacements \((x/y)\) in the two directions. The case study column has a concrete strength of 75 MPa and axial load ratio of 0.15, whereas the cross-section and reinforcement properties are same as columns tested in this study (refer Table 3.1). It is noted that the biaxial force-drift response has been calculated using expressions 7.10 to 7.14, whereas for the corresponding uniaxial response all the proposed modification factors are set equal to 1.

It can be observed in Figure 7.10 that as the \(x/y\) ratio is decreasing from 1 to 0.3, the force-drift behaviour in the direction 1 is approaching the uniaxial capacity, whereas at the same time, the force-drift capacity in the direction 2 is moving further away from the uniaxial capacity of this particular direction. This implies that as the \(x/y\) approaches 0.3, we will have the minimum capacity in the direction 2 of the column, as the damage in the direction 1 would have weakened the direction 2 to an extent that column would collapse without dissipating much energy in the direction 2.
The comparison of the bidirectional force-drift model predictions with the experimental results in the X and Y-direction of the specimens S7-S12 tested in chapter 5 is provided in Figures 7.11 and 7.12, respectively. It can be observed that the model predictions correlate very well with the experimental results, indicating that the proposed model can predict the biaxial force-drift behaviour of the RC columns with reasonable accuracy. The coefficients of variation for the drift predictions at lateral load failure and axial load failure in the X-direction are 0.13 and 0.1, respectively. Similarly, the coefficients of variation for lateral load failure and axial load drifts in the Y-direction are 0.17 and 0.1, respectively. The lower values of the coefficient of variation serve as a validation of the proposed model.

The proposed models do not take into account the effect of variation in axial load on the drift behaviour of the RC column. This is because there was not enough experimental data to develop and calibrate the models that incorporate the effect of axial load variation on the drift capacity. However, it is noted that the specimen S13 tested in this study under bidirectional lateral actions and synchronous axial load variation of 30% exhibited the same post-peak drift capacity as specimen S11 tested under bidirectional lateral actions and constant axial load. On the other hand, specimen S14 tested under bidirectional lateral actions and nonsynchronous axial load variation resulted in 25%
less drift capacity than the specimen tested under bidirectional lateral actions and constant axial load. This implies that nonsynchronous axial load variation would result in the least post-peak drift capacity of the column. This aspect should be studied in detail in the future experimental testing, so that appropriate drift models that incorporate the effect of axial load variation can be developed.

**Design Axial Load Ratio Limit Under Bidirectional Earthquake Actions:**

Eq 7.14b can be readjusted to give Eq 7.15 that can then be used to calculate the upper limit of design axial load ratio that will result in any required drift capacity. No factor of safety is suggested for the bidirectional model as it was already developed to give conservative estimates and had a very low coefficient of variation (i.e. 0.1), which was also because it mostly under predicted the actual values.

\[
n_d = \frac{1}{2} \left( 1 - 0.25 \left( \delta_{af} (1 + x/y) - \rho \frac{f_{yh}}{f'c} \right) \right)
\]

(7.15)
Figure 7.11. Comparison of bidirectional experimental results with the backbone model predictions in the X-direction of specimens S7-S12

Figure 7.12. Comparison of bidirectional experimental results with the backbone model predictions in the Y-direction of specimens S7-S12
7.4 Applications of the Force-Drift Prediction Model

This section discusses potential applications of the proposed lateral force-drift model for limited to moderately ductile RC columns.

7.4.1 Two-Tier Seismic Assessment of Soft-Storey Buildings

The piecewise lateral force-drift model can be used to evaluate the seismic performance of soft-storey buildings. A soft storey in a building is that storey which has less stiffness and is weaker than the adjacent storeys. Soft-storeys can often exist in the ground floor of residential buildings, due to architectural requirements for the provision of open space so that parking garages, swimming pools, etc. can be accommodated. Typical soft-storey buildings shown in Figure 7.13 indicate that large open space is created by providing columns in the ground storey, thereby reducing the whole structure to a rigid block of heavy mass supported on columns at the bottom. Thus, columns in the soft-storey not only serve the purpose of providing vertical support to the building but also act as lateral load resisting mechanism under seismic actions.

During an earthquake, energy and displacement demands tend to concentrate in the bottom columns of the soft-storey structures. Moreover, all the levels of a soft-storey structure displace by a similar amount, hence it is possible to idealize it as a single degree of freedom system. Seismic performance of such soft-storey structures can be reasonably assessed by evaluating the performance of its bottom storey columns. The next sub-section presents a two-tier assessment procedure to evaluate the seismic performance of soft-storey buildings using the proposed lateral force-drift model.

Figure 7.13. Typical soft-storey buildings (photos taken by Hing-Ho Tsang)
**7.4.1.1 Tier 1 Assessment**

The lateral force-displacement model proposed in section 7.3 can be used with ‘two-tier’ assessment approach proposed by Wilson and Lam [123] to assess the seismic performance of soft-storey structures in regions of low to moderate seismicity.

A ‘first-tier’ simple check can be to compare peak displacement demand (PDD) for a rare or very rare earthquake event with the axial load failure drift ($\delta_{af}$) capacity of the soft-storey columns. A soft-storey structure can be considered resilient against collapse if the drift corresponding to axial load failure of the soft-storey columns is greater than the peak displacement demand during an earthquake. Peak displacement demand for any region can be obtained directly from the respective seismic design code of that region. As an illustration, peak displacement demands for a rare earthquake event (return period of 500 years) determined in accordance with the Australian earthquake standard, AS 1170.4 are summarised in Table 7.4 for different soil sites for a seismic hazard factor of 0.08g, which is the minimum threshold value for Australia.

<table>
<thead>
<tr>
<th>Return Period</th>
<th>Rock (mm)</th>
<th>Shallow Soil (mm)</th>
<th>Soft Soil (mm)</th>
<th>Very Soft Soil (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>500</td>
<td>26</td>
<td>36</td>
<td>58</td>
<td>90</td>
</tr>
</tbody>
</table>

The application of ‘first-tier’ assessment procedure is demonstrated herein using a case study building that has a soft-storey at the ground floor. The case study resists lateral earthquake loads using two columns with a 500x500 cross-section and an inter-storey height of 3.2 metres, which are bending in double curvature. The design properties of the columns are summarised in Table 7.5. The columns are designed with limited ductile detailing in accordance with the specifications of AS 3600 [14].
### Table 7.5. Design properties of case study column

<table>
<thead>
<tr>
<th>Width×Depth ×Height (mm)</th>
<th>Aspect Ratio</th>
<th>Concrete Grade Strength $f'_c$(MPa)</th>
<th>Longitudinal Rebars $\rho_v$(%)</th>
<th>Ties (mm) $\rho_h$(% (By Area))</th>
<th>Transverse Reinforcement Yield Strength $f_{yh}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>500×500×2000</td>
<td>4</td>
<td>65</td>
<td>12N28 (2.96)</td>
<td>4-legged N10@300 (0.21)</td>
<td>500</td>
</tr>
</tbody>
</table>

The soft-storey structure is assumed to be in a low seismic region (hazard factor, $Z=0.08g$) with the bottom storey columns of the structure supporting an axial load ratio, $n=0.3$. The uniaxial axial load failure drift capacity of the soft-storey columns calculated according to equation (7.8b) is summarised in Table 7.6. As explained earlier, a soft-storey structure is essentially a rigid block of heavy mass supported on columns at the bottom, so its performance can be assessed by simply assessing the performance of its ground storey columns as seismic displacement demands in such a structure accumulates at the bottom. This case study column has an axial load failure drift of 83 mm, which is greater than the peak displacement demand for the rock, shallow soil and soft soil sites summarised in Table 7.4. This means the building can achieve a collapse prevention performance objective for a 500-year return period event on these sites. However, since the maximum displacement is less than the peak displacement demand of 90 mm for a very soft soil site, the building is potentially vulnerable on sites of this nature.
Table 7.6. Drift predictions for case study soft storey building

<table>
<thead>
<tr>
<th>Point</th>
<th>Drift Capacity (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking Point</td>
<td>6.0</td>
</tr>
<tr>
<td>Yield Point</td>
<td>19.0</td>
</tr>
<tr>
<td>Ultimate Point</td>
<td>22.0</td>
</tr>
<tr>
<td>Lateral Load Failure</td>
<td>58.0</td>
</tr>
<tr>
<td>Axial Load Failure (Collapse)</td>
<td>83.0</td>
</tr>
</tbody>
</table>

7.4.1.2 Tier 2 Assessment

If a soft-storey column fails the ‘first-tier’ check, an advanced ‘second-tier’ assessment comprising of the capacity spectrum method can be employed. In the capacity spectrum method, the structural capacity curve and seismic demand curves are superimposed on each other to get the performance point of the structure. The performance point represents the maximum acceleration and displacement demands on the structure during the earthquake. If the capacity and demand curves intersect, then a structures performance can be deemed satisfactory. The lateral force-drift backbone model presented in section 7.3 can give the capacity curve of an RC column under the unidirectional lateral actions, whereas seismic demand curve is site-specific and region dependent and can be obtained from the design response spectrum of the relevant seismic design codes.

The application of the second-tier check is demonstrated on the soft-storey column mentioned previously, by plotting the capacity curve of the column obtained from the proposed force-displacement model against the seismic demand curves of rock, shallow soil, soft soil and very soft soil sites for a rare earthquake event, as shown in Figure 7.14. Because the soft storey has two columns, the lateral forces calculated for one column are multiplied by two. The seismic demand curve in Figure 7.14 is in accordance with the requirements of AS 1170.4 and has been converted into the force-displacement format by multiplying acceleration with the mass of the structure, which
is assumed to be 1000 tons for the purpose of this illustration. The damping ratio for response spectrum is 5%.

![Graph showing lateral force-displacement relationship for different soil types.](image)

**Figure 7.14. Second-tier assessment of the case study soft-storey structure**

The second-tier assessment shows that the soft-storey building is able to meet the strength and displacement requirements on rock, shallow soil and soft soil sites, hence it can be deemed practically safe on these sites. However, both the strength and displacement demands are exceeding the structure’s capacity on a very soft soil site, thereby making it vulnerable to collapse in a rare earthquake event.

The outcomes from the second-tier assessment are in agreement with those from the simplified first-tier check. When analysing a large building stock, the first-tier approach can allow designers to quickly assess and eliminate the less vulnerable buildings. Thus allowing the more complex analysis to be performed on the truly vulnerable buildings.

### 7.4.1.3 Seismic Performance Level of Soft-Storey Buildings

The piecewise lateral force-displacement model can also be used to assess the performance level of soft-storey buildings during earthquakes in low to moderate seismic regions. Table 7.7 relates each point on the piecewise lateral force-displacement model with a corresponding performance level specified in Vision2000 document drafted by Structural Engineers Association in California (SEAOC) [124].
Table 7.7. Correlation between performance levels and the lateral force-displacement model

<table>
<thead>
<tr>
<th>Performance Level</th>
<th>Corresponding Limiting Point of the Lateral Force-Displacement Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fully Operational</td>
<td>Cracking Point</td>
</tr>
<tr>
<td>Operational/ Immediate Occupancy</td>
<td>Yield Point</td>
</tr>
<tr>
<td>Life Safety</td>
<td>Ultimate Point</td>
</tr>
<tr>
<td>Near Collapse</td>
<td>Lateral Load Failure Point</td>
</tr>
<tr>
<td>Collapse</td>
<td>Axial Load Failure Point</td>
</tr>
</tbody>
</table>

Table 7.7 can serve as a convenient tool for the design engineers to assess the performance level of a soft storey building in a particular earthquake scenario, as the performance of a soft-storey building is largely dependent on the performance of its bottom storey columns, which can be evaluated by using the proposed lateral force-displacement model. For instance, seismic performance levels of the soft-storey building previously evaluated using the two assessment methods for a rare earthquake scenario are summarised in Table 7.8.

Table 7.8. Predicted seismic performance levels of the case study soft-storey building

<table>
<thead>
<tr>
<th>Soil Site</th>
<th>Performance Level-Tier 1</th>
<th>Performance Level-Tier 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock</td>
<td>Life Safety</td>
<td>Operational / Immediate Occupancy</td>
</tr>
<tr>
<td>Shallow Soil</td>
<td>Near Collapse</td>
<td>Life Safety</td>
</tr>
<tr>
<td>Soft Soil</td>
<td>Near Collapse</td>
<td>Near Collapse</td>
</tr>
<tr>
<td>Very Soft Soil</td>
<td>Collapse</td>
<td>Collapse</td>
</tr>
</tbody>
</table>

The performance levels stated in Table 7.8 are in accordance with the results of Tier 1 and Tier 2 assessment of the soft storey building analysed in the previous section. For example, under Tier 1 assessment, rock soil has a peak displacement demand of 26 mm, which is close to the ‘ultimate’ displacement capacity of the soft storey column i.e. 22 mm (refer Table 7.6), hence the performance level is considered to be ‘life safety’. However, under Tier 2 assessment, capacity and demand curves of the soft-storey
structure intersect close to the yield point for a rock site, hence the performance level is specified as operational/immediate occupancy. The performance levels for other soil sites are also determined accordingly.

It can be observed in Table 7.8 that the performance levels of the case study building determined in accordance with Tier 1 assessment are generally more conservative as compared to Tier 2 assessment. This implies that Tier 1 assessment is simple, yet conservative, and hence, can be employed as a quick safety check to assess the seismic performance of the soft storey structures at the design stage.

7.4.2 Non-linear Analysis of RC Structures

The proposed lateral force-displacement model can also be used for non-linear analysis of RC structures. Most of the earthquake design standards require non-linear analysis for the seismic design of structures with high importance levels. Realistic models of stress-strain, moment-curvature and force-displacement behaviour are required to capture the non-linear response of structures reliably. The proposed lateral force-displacement model can be employed by structural design engineers for non-linear analysis of RC structures due to its simplicity, a wide range of applicability (both NSRC and HSRC columns) and accuracy, especially in capturing post-peak displacement behaviour of RC columns prevalent in regions of low to moderate seismicity. This can be achieved by using the proposed model to define the force-displacement behaviour of RC columns in the commercial design software packages such as ETABS, SAP2000 etc. while conducting a non-linear analysis of the RC structures.

7.4.3 Assessment of Gravity Columns in RC Wall Buildings

The majority of low-, mid- and high-rise buildings in Australia are RC wall buildings, which utilise RC rectangular walls and building cores as the primary lateral load resisting system of the building. This is a common practice observed in many other regions of low to moderate seismicity. Buildings of this nature are generally designed for lateral loads using 3D linear elastic analysis programs (e.g. ETABS) with all the columns as ‘pin-ended’ elements. Using this approach, all the lateral load is distributed
to the walls and buildings cores and the columns ‘go the ride’ when the floor diagram
displaces and moves as the walls and building cores resist the lateral earthquake loads.

The seismic compliance of the columns can quickly be assessed by determining the
maximum displacement of the columns from the analysis model and then ensuring that
it is less than the axial load failure displacement of the respective column (i.e. Equation
7.8b under unidirectional actions and 7.14b under bidirectional actions). However, it
should be noted that when force-based seismic analysis procedures are adopted, the
maximum displacement of the columns is not the maximum displacement in the 3D
linear elastic analysis model, but rather the displacement from the model multiplied by
the force reduction factor (i.e. $R_f = \frac{\mu}{S_p} = 2.6$ for limited ductile RC structures, according
to AS 1170.4) adopted when calculating the seismic actions. The force reduction factor
is used to account for inelastic behaviour in the structure.

The application of the proposed model in the assessment of gravity columns in RC
buildings is illustrated by evaluating the seismic compliance of the top storey corner
column A1 of the case study RC frame-wall building presented and analysed in chapter
3 under DBE scaled Christchurch (2011) ground motion. Table 7.9 presents the
maximum average drift on the column from the analysis model in the two orthogonal
horizontal directions and compares it with the axial load failure drift capacity
calculated from the equation 7.14b. It is noted that the maximum drift from the analysis
model has been multiplied with the force reduction factor of 2.6 specified by AS 1170.4
for limited ductile buildings. The column has a cross-sectional size of 500×500 mm with
a transverse reinforcement ratio of $\rho_h=0.21\%$ and is supporting an axial load ratio of
$n = 0.035$. The displacement demand ratio $\left(\frac{x}{f}\right)$ in the two axes of the column is 0.5.
Table 7.9. Comparison of drift demand from the analysis model with drift capacity from the proposed model for the top storey corner column A1 under DBE Christchurch (2011) ground motion of the case study building (Figure 3.5)

<table>
<thead>
<tr>
<th>Drift Demand from Analysis Model×$R_f$ (%)</th>
<th>Axial Load Failure Drift Capacity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>Y</td>
</tr>
<tr>
<td>1.6</td>
<td>3.2</td>
</tr>
</tbody>
</table>

It can be seen in Table 7.9 that the drift capacity of the top storey corner column A1 is greater than the maximum drift demand. Thus, the selected column can be deemed compliant under the considered ground motion. In a similar manner, the seismic compliance of any other column from the analysis model can also be checked.

### 7.5 Summary and Conclusions

This chapter presented models for predicting the force-displacement backbone envelope of both NSRC and HSRC columns subjected to unidirectional and bidirectional lateral actions. Of particular note, are the post-peak drift models that were developed and calibrated with an extensive database of RC columns in literature and relate the drift capacity of the column with design parameters such as axial load ratio, transverse reinforcement ratio, transverse reinforcement yield strength, concrete compressive strength and aspect ratio. The force-displacement backbone envelopes predicted by the model showed a very good correlation with the experimental results of the specimens tested under unidirectional and bidirectional lateral actions in this research. The applications of the proposed model include seismic performance assessment of soft-storey structures and assessment of gravity columns in RC wall buildings. The proposed model can also serve as a tool for conducting a reliable non-linear analysis of RC buildings through its use in commercial design software packages for defining the force-displacement behaviour of an RC column. Furthermore, the model can also aid in decision-making regarding the need for retrofitting of RC columns in existing buildings.
Chapter 8  Conclusions and Recommendations

8.1  Summary

This research focused on developing and improving the fundamental understanding of the collapse behaviour of limited ductile HSRC columns under multidirectional earthquake actions. Limited ductile HSRC columns are commonly constructed in mid to high-rise RC buildings in Australia and other regions of low to moderate seismicity and are characterised by wide spacing of transverse reinforcement, thereby resulting in inadequate confinement of the concrete core, and thus may be prone to collapse in a rare or very rare earthquake event.

In the first stage of the study, a set of realistic loading protocols were developed for bidirectional lateral loading and axial load variation based on the results of a numerical study, in which a case study RC frame-wall building was subjected to a suite of 20 ground motions, representative of low to moderate seismic regions. The typical patterns of the bidirectional lateral displacement and axial load variation in the columns of the case study building were identified by rigorous statistical analysis of the results and were then generalized to develop the loading protocols.

The collapse behaviour of the limited ductile HSRC columns was then experimentally studied by testing 14 full-scale columns, designed according to Australian concrete standard, under unidirectional and bidirectional lateral loading with CAL or VAL, using the developed loading protocols. The first six specimens (S1-S6) were tested under unidirectional lateral loading and CAL, with axial load ratio, transverse reinforcement ratio and concrete compressive strength being the variables of the testing program. The next six specimens (S7-S12) were tested under bidirectional lateral loading and CAL, with axial load ratio and bidirectional loading path as the variables, whereas the last two specimens (S13-S14) were tested under bidirectional lateral loading and VAL, with axial load variation protocol as the variable of the testing. The performance of the tested specimens was thoroughly investigated in terms of force-displacement, moment-
curvature, axial displacement-lateral drift, energy dissipation and stiffness degradation behaviour. Detailed comparisons were also made regarding the behaviour of the specimens under different loading protocols.

Apart from testing the specimens under axial load variation protocols, a numerical study was also conducted on a case study building to study the range of axial load variation and the governing parameters that control the axial load variation in the corner and internal columns of the RC buildings. Accordingly, a generalised expression was proposed to estimate the axial load variation in the columns of the RC buildings.

Finally, a model was proposed for predicting the force-displacement backbone envelope of limited to moderately ductile RC columns subjected to unidirectional and bidirectional lateral actions. The expressions for the post-peak displacement capacity were calibrated with a comprehensive database of experimental tests from the literature. The model predicted the experimental force-displacement backbone envelope of the specimens tested in this study with reasonable accuracy. The applications of the proposed model, particularly relating to the seismic performance assessment of limited to moderately ductile RC columns in regions of low to moderate seismicity, were also discussed in detail with the aid of a case study example.

The outcomes of this research will have important implications for the earthquake safety of our building stocks in Australia and globally. In particular, the study will lead to a better understanding of the collapse behaviour of RC columns under multidirectional earthquake excitations and will result in the improvement of the existing seismic design procedure of HSRC columns, particularly in Australia and other regions of low to moderate seismicity.

8.2 Conclusions

Loading Protocols

- The rigorous statistical processing of the results of the numerical study showed that the actual displacement path of the RC columns under earthquake excitations is in the form of elliptical loops of different aspect ratios and orientations. Accordingly,
a bidirectional lateral loading protocol namely, the octo-elliptical path, was proposed that generalised the displacement path of the columns under multidirectional earthquake excitations. Also, two axial load variation protocols, namely synchronous and nonsynchronous axial load variation, were developed in which the axial load was synchronous and nonsynchronous to the lateral displacement of the building, respectively.

Effect of Axial Load Ratio

- The experimental study demonstrated that the axial load ratio drastically affects the collapse drift capacity of limited ductile HSRC columns. As such, doubling the axial load ratio from $n=0.15$ to 0.3 under unidirectional lateral loading reduced the collapse drift capacity by around 35%, whereas tripling the axial load ratio to 0.45 reduced the drift capacity by around 75%. Similarly, under bidirectional lateral loading, doubling the axial load ratio from $n=0.15$ to 0.3 reduced the collapse drift capacity of the column by around 45%. This indicates that under bidirectional lateral loading, an increase in axial load ratio has a more pronounced effect in reducing the drift capacity of the column as opposed to unidirectional lateral loading.

Effect of Bidirectional Lateral Loading

- The uniaxial collapse drift capacity of the column reduced by 35-50% under bidirectional loading, which is quite significant. The reductions of 35% and 50% were observed when the ratio of X to Y-direction displacements were 0.6 and 1.0, respectively, in the bidirectional loading path. Similarly, the lateral force capacity of the column in both axes was 10-15% lower than the theoretical capacity when this ratio was 1.0 and was 30-40% lower in the X-direction when the ratio was 0.6. As such, the hysteretic response of the column was found to be highly dependent on the ratio of X to Y-direction displacements ($x/y$) in a bidirectional loading path.
- The accumulative energy dissipation was found to be higher under bidirectional loading paths as opposed to unidirectional loading. However, the difference in the amount of energy dissipated reduced with the increase in axial load ratio.
Furthermore, the accumulative energy dissipation of the column was observed to be highly dependent on the type of the bidirectional loading path. As such, the highest amount of energy was dissipated under octo-elliptical (1:1) path with CAL and lowest energy was dissipated under linearised circular (1:1) path with CAL and octo-elliptical (0.6:1) path with NS-VAL.

**Axial Load Variation**

- The numerical study revealed that the mean axial load variation in the corner perimeter, internal perimeter and internal non-perimeter columns of the case study building were 43%, 12% and 10%, respectively, for the ground motions considered in this study. The variation seems to be quite significant for the corner perimeter columns, considering that the selected ground motions were representative of low to moderate seismic regions, and were unscaled.

- The axial load variation in the corner perimeter columns was found to be a function of the RSA of the two orthogonal horizontal components of the ground motion, whereas the axial load variation in the internal non-perimeter columns was mainly dependent on the RSA of the vertical component of the ground motion.

- The experimental study showed that the collapse drift of the column specimens tested under bidirectional lateral loading with CAL and SS-VAL histories were similar, whereas the drift capacity for specimen subjected to bidirectional lateral loading with NS-VAL (in which variation of axial load is nonsynchronous to the lateral displacement of the building) was about 25% lower, thereby implying that nonsynchronous axial loading history is more damaging in the event of an earthquake.

**Vulnerability to Collapse**

- The collapse drift capacity of AS 3600 code-compliant HSRC column with axial load ratio of 0.45 under unidirectional lateral loading and axial load ratio of 0.3 under bidirectional lateral loading ($x/y = 1$) was found to be less than 1.5%, thereby implying that such columns can be more vulnerable to collapse in a rare or very rare earthquake event in regions of low to moderate seismicity.
**Proposed Force-Drift Model**

- The model proposed for predicting the force-displacement backbone envelope of NSRC and HSRC columns subjected to unidirectional and bidirectional lateral actions showed a very good correlation with the experimental results of the specimens tested in this research.

### 8.3 Design Recommendations

1. A good design practice would be to limit the design axial load ratio to $n_d \leq 0.40$ for limited ductile HSRC Columns under unidirectional lateral loading to allow for a drift capacity of at least 1.5%. This limit has been calculated based on Eq 7.9 for the design axial load.

2. Similarly, a good design practice would be to limit the design axial load ratio to $n_d \leq 0.25$ for limited ductile HSRC Columns that are expected to experience strong bidirectional actions (i.e. $\frac{x}{y} = 1$) to allow for a drift capacity of at least 1.5%. This limit has been calculated based on Eq 7.15 for the design axial load.

Figure 8.1 illustrates the variation of axial load ratio with the concrete compressive strength for a drift capacity of 1.5% using equations 7.9 and 7.14. The column has the same cross-sectional properties and reinforcement detailing as specimen S1 tested in this study (refer Table 3.1). Concrete compressive strength is the only variable, which is increasing from 25 MPa to 100 MPa. It can be seen that under unidirectional lateral actions, the column with high-strength concrete ($f'_c \geq 50$ MPa) exhibited a drift capacity of 1.5% at an axial load ratio of approximately 0.4 or less. On the other hand, for normal-strength concrete ($f'_c < 50$ MPa), the axial load ratio can even be higher than 0.4 to have a drift capacity of 1.5%.

Similarly, under strong bidirectional actions (i.e. $\frac{x}{y} = 1$), the column exhibited a drift capacity of 1.5% at an axial load ratio of less than or equal to 0.25 (approximately) for high-strength reinforced concrete columns, whereas for normal-strength reinforced concrete columns, the axial load ratio could even be greater than 0.25. However, when bidirectional lateral actions were not that strong (i.e. $\frac{x}{y} = 0.5$), the column exhibited a
drift capacity of 1.5% at an axial load ratio of less than or equal to 0.35 (approximately) for high-strength reinforced concrete columns, which increased to more than 0.4 for normal-strength reinforced concrete columns.

The results in Figure 8.1 essentially support the recommendation that the maximum allowable design axial load ratio for attaining a drift capacity of 1.5% for high-strength reinforced concrete columns should be less than or equal to 0.4 and 0.25, respectively, under unidirectional and strong bidirectional lateral actions.

Figure 8.1 Variation of axial load ratio with concrete compressive strength at the axial load failure drift of 1.5%

3. The proposed force-displacement backbone envelope model can be used in conjunction with the two-tier assessment procedure mentioned in chapter 7 for seismic performance assessment of soft-story structures at the design stage. The proposed model can also serve as a tool for conducting a reliable non-linear analysis of RC buildings through its use in commercial design software packages for defining the force-displacement behaviour of an RC column.

8.4 Recommendations for Further Research

This research investigated the collapse behaviour of limited ductile HSRC columns under multidirectional earthquake actions. Following recommendations are being made for future research in this regard:
1. Future studies should focus on incorporating the findings of the experimental, numerical and analytical work presented in this thesis to further develop the displacement-based design procedure so that it can be incorporated in the design standards. The study should particularly be extended to determine the equivalent viscous damping under multidirectional earthquake actions, which is one of the most important parameter that represents the ductility and energy absorption of the substitute structure in the displacement-based design procedure. It is noted that the current design procedure followed by the Australian Earthquake standard is force-based design in which substitute structure is represented by elastic damping instead of equivalent viscous damping (that is the sum of elastic and hysteretic damping), and is thus considered deficient.

2. This thesis provides a comprehensive experimental data on the nonlinear force-displacement behaviour of high-strength RC columns under unidirectional and bidirectional lateral loading with constant and variable axial load up to the point of collapse of the specimens. The experimental results can be used to develop and validate appropriate numerical models for modelling the nonlinear behaviour of RC columns under these loading conditions. Such models can consequently be used to accurately estimate the nonlinear response of RC buildings under earthquake actions.

3. The protocols proposed for bidirectional lateral loading and axial load variation were developed using ground motions representative of low to moderate seismic regions. The applicability of these protocols in high seismic regions should be evaluated by studying the typical patterns of bidirectional lateral loading and axial load variation in columns of RC buildings under strong ground motions representative of higher seismic regions.

4. In future studies, HSRC columns with different cross-sectional sizes and longitudinal and transverse reinforcement ratios should be tested, as the experimental data on the collapse behaviour of HSRC columns subjected to multidirectional earthquake loading is still very limited. It is noted that this was only the second experimental study (first study on limited ductile HSRC columns) that
evaluated the behaviour of HSRC columns under bidirectional lateral loading with CAL or VAL.

5. This study employed bidirectional loading protocols with X to Y-direction displacement ratios \( x/y \) of 0.6 and 1, respectively. Future experimental programs need to study the collapse capacity of RC columns at other \( x/y \) displacement ratios to better understand the behaviour under different loading conditions.

6. Future research should study the individual contribution of bidirectional demands and number of cycles in reducing the drift capacity of the column. It should be particularly investigated that which factor among these two has a more dominant role in reducing the drift capacity of the column.

7. The column specimens were subjected to axial load variation of just 30\% in the present study. It would be interesting to see the effects of higher percentages of axial load variation on the collapse behaviour of the RC columns. Therefore, future studies need to test the specimens under bidirectional lateral loading with different percentages of variation in axial load.

8. The proposed generalized expressions (i.e. Eq 6.1 and 6.2) for estimating the axial load variation in the corner and internal columns of RC buildings need to be validated by studying the axial load variation in buildings of other configurations. Particularly, the coefficients \( \alpha_{1v}, \alpha_{2v} \) and \( \alpha_{3v} \) should be determined for different configurations of RC buildings, as the coefficients proposed in this research are only specific to the case study building considered in this study.

9. Finally, the proposed force-displacement backbone envelope model should be validated against the experimental results of other studies that tested the column specimens under bidirectional lateral loading with CAL. The model could also be modified and refined to predict the force-displacement behaviour for columns subjected to lateral loading coupled with axial load variation.
References


[17] ACI Committee 318 (2014), “Building code requirements for structural concrete (ACI 318M-14) and commentary (ACI 318RM-14),” American Concrete Institute, Farmington Hills, MI.


[66] ACI 374.2R-13 (2013), ”Guide for testing reinforced concrete structural elements under slowly applied simulated seismic loads, American Concrete Institute,” Farmington Hills, MI, USA.


[81] Umehara, H., Jirsa, J. O. (1982), “Shear strength and deterioration of short reinforced concrete columns under cyclic deformations,” University of Texas at Austin, Austin, TX, USA.


Appendix A. Experimental Database Used for the Drift Capacity Models

Table A.1: NSRC Column Database (F= Flexure, S= Shear, FS =Flexure-Shear, n.a. = Not available)

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<th>Specimen</th>
<th>$f'_c$ (MPa)</th>
<th>$L/h$</th>
<th>$\rho_v$ (%)</th>
<th>$\rho_h$ (%)</th>
<th>$f_{yh}$ (MPa)</th>
<th>$n$</th>
<th>$\delta_f$ (%)</th>
<th>$\delta_{af}$ (%)</th>
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Appendix B. Details of Instrumentation

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