Influence of Precast Prestressed Flooring on the Seismic Performance of Reinforced Concrete Perimeter Frame Buildings

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INFLUENCE OF PRECAST PRESTRESSED FLOORING ON THE SEISMIC PERFORMANCE OF REINFORCED CONCRETE PERIMETER FRAME BUILDINGS

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Abstract

Lateral load tests of reinforced concrete perimeter frames with diaphragms have shown that the addition of a floor slab (diaphragm) can have a major influence on structural performance. Three moment resisting frames were tested. Two of these frames were tested without a floor slab being attached to the beams, while the remaining frame was tested with the addition of a typical floor slab containing prestressed units. The tests showed that the addition of the floor slab increased the strength of the beams appreciably and as a result the lateral strength of the frame was increased by close to 80%. Clearly a strength increase of this order of magnitude is of major concern in seismic design in cases where it is essential to avoid the premature formation of a column sway mechanism. The test results presented together with an analytical study show the origins of this strength increase. Understanding these mechanisms is a first step in establishing a design method for assessing over-strength values in perimeter frames, which contain floors with prestressed units.
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Chapter 2

\[ A_g = \text{gross area of section.} \]
\[ b_w = \text{width of beam web.} \]
\[ d = \text{effective depth of beam, distance from extreme compression fibre of beam to centroid of tension reinforcement.} \]
\[ d_c = \text{distance from extreme compression fibre of beam to centroid of compression reinforcement.} \]
\[ \delta_{eu} = \text{elongation at mid-depth of beam containing uni-directional plastic hinges} \]
\[ e = \text{elongation of the reinforcement in the compression zone of the plastic hinges in the beam.} \]
\[ f'_{c} = \text{specified compressive strength of concrete.} \]
\[ F_c = \text{axial force required to just close the cracks in the compression zone due to contact stress effects.} \]
\[ l = \text{clear span length of beam, between column faces.} \]
\[ L = \text{span of beam.} \]
\[ L_d = \text{bar development length.} \]
\[ L_n = \text{clear span of hollowcore unit.} \]
\[ M_{pos} = \text{maximum positive flexural strength at end of beam.} \]
\[ M_{neg} = \text{maximum negative flexural strength at end of beam.} \]
\[ \theta = \text{rotation sustained by the plastic hinges.} \]
\[ \Sigma \theta = \text{sum of rotations of the plastic hinges in a beam bay.} \]
\[ t_s = \text{thickness of slab.} \]
\[ T_1 = \text{tension force in hollowcore topping reinforcement.} \]
\[ T_2 = \text{tension force in hollowcore topping reinforcement.} \]
\[ V_c = \text{nominal shear force resistance provided by concrete mechanisms.} \]
\[ V_o = \text{shear force due to truss-like action associated with shear resistance in reversing plastic hinges.} \]
\[ w_{\text{max}} = \text{maximum uniformly distributed load on that a beam can sustain for reversing hinges to form.} \]

Chapter 3

\[ \alpha = \text{angle of the diagonal members to the horizontal of a segment of beam instrumentation.} \]
\[ b_N = \text{displacement reading at the bottom of column ‘N’ (=A, B or C).} \]
\[ d_B = \text{distance from the centroid of segment to the bottom of column.} \]
**Notation**

\[ d_{ft} = \text{distance from the centroid of segment to the top of column.} \]

\[ \delta_{fl} = \text{lateral deflection at mid-span of the beam bay.} \]

\[ \delta_{x} = \text{change in length of member 'x'.} \]

\[ \delta_{b} = \text{lateral deflection due to shear.} \]

\[ \Delta_{x} = \text{as shown in Figures 3.23.} \]

\[ \Delta X = \text{change in the distance of segment X of a DEMEC arrangement.} \]

\[ E = \text{elongation in a segment of beam instrumentation.} \]

\[ e_{AB} = \text{average elongation of the beam between columns ‘A’ and ‘B’.} \]

\[ e_{BC} = \text{average elongation of the beam between columns ‘B’ and ‘C’.} \]

\[ e_{\text{max}} = \text{reinforcement strain at maximum stress.} \]

\[ f_{\text{max}} = \text{maximum stress of reinforcement.} \]

\[ f_{\text{c}} = \text{specified compressive strength of concrete.} \]

\[ f_{y} = \text{yield stress of reinforcement.} \]

\[ G = \text{change in depth of a segment of beam instrumentation.} \]

\[ h = \text{distance between top and bottom members of a segment of beam instrumentation.} \]

\[ h_{c} = \text{height of the column from the base to the beam centreline.} \]

\[ h_{i} = \text{initial vertical distance of DEMEC arrangement as shown in Figure 3.27.} \]

\[ L = \text{distance between the column centres.} \]

\[ l_{fl} = \text{flexural component of lateral deflection of column} \]

\[ l_{s} = \text{shear component of lateral deflection of column.} \]

\[ \pi = \text{pi (= 3.141593)} \]

\[ \theta = \text{rotation sustained by a segment of beam instrumentation.} \]

\[ \theta_{N} = \text{rotation of the segment N.} \]

\[ \theta_{x} = \text{rotations of the columns as shown on Figures 3.23.} \]

\[ \theta_{X} = \text{angle in triangular DEMEC arrangement as shown in Figure 3.27 (also } \theta_{y} \text{ and } \theta_{xz} \text{).} \]

\[ S = \text{shear deformation in a segment of beam instrumentation.} \]

\[ S_{n} = \text{shear deformation of segment } n. \]

\[ S_{N} = \text{shear deformation in segment N.} \]

\[ t = \text{the target displacement.} \]

\[ t_{AB} = \text{displacement reading of the displacement between the top of columns ‘A’ and ‘B’.} \]

\[ t_{BC} = \text{displacement reading of the displacement between the top of columns ‘B’ and ‘C’.} \]

\[ t_{N} = \text{displacement reading at the top of column ‘N’ (=A, B or C).} \]
Notation

\( u_t \) = horizontal displacement of DEMEC point.

\( u_v \) = vertical displacement of DEMEC point.

\( X \) = measured distance of a DEMEC arrangement as shown in Figure 3.27 (similarly segments \( Y \) and \( Z \)).

\( X_0 \) = initial measurement of a DEMEC arrangement.

\( x_n \) = distance from column centre to the centroid of each segment \( n \).

**Chapter 5**

\( C_e \) = crack width at central transverse beam.

\( E_s \) = elongation in floor slab.

\( \gamma \) = shear deformation of column.

**Chapter 7**

\( A_n \) = area of member ‘\( n \)’ of elongating hinge model.

\( A_v \) = area of transverse reinforcement in beam section.

\( \alpha \) = angle of diagonal compression strut.

\( B \) = width of beam section.

\( C_d \) = diagonal compression force.

\( C_{y_n} \) = compression force at yield of member ‘\( n \)’.

\( D \) = depth of beam section.

\( d_d \) = depth of the compression stress block of the flexible slab at the beam face.

\( \Delta_d \) = vertical deflection of diagonal slab.

\( E_n \) = modulus of elasticity of member ‘\( n \)’ of elongating hinge model.

\( \varepsilon_y \) = yield strain of member.

\( f_y \) = yield stress of transverse reinforcement in beam section.

\( I_{cr} \) = moment of inertia of cracked section.

\( I_e \) = effective moment of inertia of concrete section.

\( I_g \) = moment of inertia of gross concrete section.

\( L_{db} \) = basic development length of a straight bar.

\( L_{dh} \) = development length of hooked bars.

\( L_n \) = length of member ‘\( n \)’ of elongating hinge model.

\( L_t \) = length of tension tie under yield extension.

\( M_a \) = moment applied to a section.

\( M_{cr} \) = cracking moment of a section.

\( M_d \) = moment that can be resisted by the slab.

\( M_f \) = flexural strength based on assumed rectangular compression stress block.

\( \theta_d \) = rotation of diagonal slab.

\( s \) = spacing between transverse reinforcement.
Notation

$T_t$ = tension force in ties.
$T_y^n$ = tension force at yield of member 'n'.
$V_d$ = shear force along the interface between slab and beam.
$V_s$ = shear resistance provided by the transverse reinforcement.
$w_d$ = width of diagonal compression strut.

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$C$ = compression force in beam.
$\varepsilon_{face}$ = strain in reinforcement at column face.
$\varepsilon_s$ = strain measured across gauge length.
$\varepsilon_{yield}$ = strain in reinforcement at yield.
$T_{beam}$ = tension force passive longitudinal reinforcement in beam.
$T_{rein}$ = equivalent tension force from the slab reinforcement acting with the perimeter frame beams at the central column.
$T_{slab}$ = tension force in slab, equivalent to an additional force applied to the beam at mid-height of the floor slab.
1.1 Background

It was in the 1950s that the use of prestressed precast flooring began to expand concurrently with the building industry boom in the United States. Due to the high quality and cost-effectiveness demonstrated by the products, investment was placed into the promotion and commissioning in this form of construction. As a result, many precast prestressed flooring products were introduced, which became industry standards such as the double tee, single tee, flat slab, rib (or joist) and infill system followed by the hollow core system in the early 1960s [P1].

Since the 1960s there has been increase in the use of precast prestressed flooring at the expense of cast-in-place floors in the New Zealand construction industry. This can be attributed to the high cost of labour and formwork required for cast-in-place construction. General design provisions for the design of reinforced and prestressed concrete floor slabs are contained in the standards for design loadings and concrete structures, NZS 4203:1992 [S1] and NZS 3101:1995 [S2]. However, not all aspects of the design and construction of precast concrete components are covered. With such an extensive use of this form of construction, there is a need for more understanding of how this type of flooring interacts with other structural elements, particularly under seismic conditions.

It has only been in the last two decades that attention has been drawn to the interaction of floor slabs with moment resisting frame systems. Previous research on individual reinforced concrete beam-column joint assemblies with floor slabs identified the influence of slabs on beam-column connections, and the contribution they made to strength [S3, D1, P2, A1, C1 and F1]. However, moment resisting frames have a high level of indeterminacy, which is not present in tests on individual beam column sub-
assemblies. Indeterminacy allows forces to be redistributed and added to this, the phenomenon such as elongation of beams can induce significant additional actions. Comparisons of experimental results have shown that the response of individual beam-column joints sub-assemblies to that of multiple beam-column joint sub-assemblies (both with and without floor slabs) are significantly different, primarily due to the indeterminacy of a frame subassembly. Subsequently, researchers have undertaken tests of reinforced concrete frame subassemblies representing typical levels of a multistorey moment resisting building with and without floor slabs [Q1, Z1, Z2, F2]. While methods for design based on the results of tests of cast-in-place diaphragms can be validly extrapolated, the same may not be true of diaphragm with precast flooring elements.

1.2 General Concept of Seismic Design

New Zealand is situated in a seismically active region. Therefore, structural engineers must consider, and take precautionary measures to mitigate the effects of earthquakes. On the one hand, structures require a level of protection against damage to non-structural elements in order to minimise the disruption to normal operations in smaller magnitude earthquakes [A2]. This requires a minimum level of stiffness to be provided throughout the building to limit the amount of lateral movement in the building. On the other hand, it is generally uneconomical to design structures to resist the largest likely earthquake and remain undamaged. Therefore for the design of structures in seismic zones, most design standards have adopted the following recommendations as design performance criteria:

(a) Resists minor earthquakes with no damage.

(b) Resists moderate earthquakes without structural damage but with some non-structural damage.

(c) Resist major earthquakes without collapse, but sustaining structural and non-structural damage, even unrepairable, but most importantly, without the loss of life.
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The objectives above require structures to behave elastically in the event of moderate earthquakes that may be expected to occur in the life of the building. More importantly the building should be able to survive without collapse in a major earthquake. In order to avoid collapse, the structural members must behave in a ductile manner and be able to absorb and dissipate energy by inelastic deformation [P3]. The following specific structural properties have to be considered in order to satisfy the requirements for seismic design mentioned above:

(a) Stiffness - A realistic estimate of the stiffness needs to be made in order to reliably quantify, and thus control, the deformation of the structure. This property is estimated from section geometric properties and elastic moduli of construction materials. However it is not simple with reinforced concrete members, as cracking and the distribution of forces in the members influence the stiffness.

(b) Strength - For the past 70 years, since design for seismic resistance has been required by standards, strength and performance have often been considered to go hand in hand. However, there has been a realisation that increasing the strength alone does not necessarily enhance performance. The development of capacity design principles in the 1970s [P3], showed that the distribution of strength throughout a building was more important than the absolute design base shear value.

(c) Ductility - The ability of the structure and its members to deform in the inelastic domain is generally given the term ductility. In order to ensure survival in a major earthquake, the members have to be able to retain a high proportion of their initial strength while sustaining repeated inelastic deformations. This includes the ability of the members to absorb energy by hysteretic behaviour. The fundamental source of ductility is the material (or strain) ductility. This is exhibited by the stress-strain response of the material. Concrete is inherently a brittle material, and suited to carry compression stresses. On the other hand, steel is a ductile material, but when subjected to compression, is susceptible to buckling if not restrained. However in
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structural engineering, the curvature ductility of a section and the displacement ductility of a member is of more relevance. The moment-curvature relationship describes the response of the section. From a moment-curvature analysis, it is possible to determine displacement from a curvature distribution of a member. The displacement ductility of a member indicates the ability of the member to displace beyond its yield displacement. In this report where the term ‘ductility’ is used (given the symbol \( \mu \)), it is in reference to the displacement ductility of the member or structure.

Though not covered in detail in this thesis, another method of protecting structures from the damaging effects of earthquake ground motions is by base or seismic isolation, where the structure is uncoupled from the ground. To achieve this, additional flexibility is introduced at the base of the structure, which effectively increases the period of vibration of the structure. The combined structure-mounting system is designed such that the period of vibration is sufficiently long so that the structure is isolated from the greatest disturbing motions.

Energy dissipation devices are generally used in conjunction with base isolation to introduce extra damping into the system. This keeps the deflections of the structure relative to the foundation down to acceptable limits and absorbs the energy that would otherwise have to be adsorbed (with damage) by a structure without base isolation. Isolation may be provided for structures with longer fundamental periods, but the design is more complex as there may be more than one significant mode of vibration, and overturning effects may also be important [S5]. A common form of base isolation device is the lead-rubber bearing, which is similar to the laminated steel and rubber bearings used to allow movements due to thermal and other effects to occur on bridges, but with the addition of lead core as energy dissipator.

1.3 Background to Seismic Design of Moment-Resisting Reinforced Concrete Building

Moment resisting frames are one of the most widespread forms of structures used for modern multistorey commercial and residential buildings. Such frames can carry
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gravity loads while providing the necessary resistance for lateral forces generated from wind and earthquake ground motion. There are essentially two types of moment resisting frame structures. The first of these, given the term 'uniform frame building' (see Figure 1.1(a)), is where the frames are spaced at regular intervals and are designed to perform the dual act of supporting the gravity loads as well as providing lateral resistance.

![Diagram](image)

(a) Uniform frame building

![Diagram](image)

(b) Perimeter frame building

Figure 1.1: Typical plan configuration of moment resisting frame buildings.
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The second type of frame, as shown in Figure 1.1(b), is the ‘perimeter frame building’, where relatively stiff perimeter frames resist the majority of the lateral wind and seismic forces and laterally flexible internal framing supports the majority of the gravity loads. These structures have the advantage that they contain relatively few internal columns compared with the uniform frame building. A common feature of the perimeter frame structure is that the span of the floor slab is greater than the span of the beams in the perimeter frames.

Figure 1.2 shows three possible mechanisms of plastic deformation for a frame building as a result of lateral loading. The column side-sway mechanism, shown in Figure 1.2(a), has plastic hinges that form only in the columns of one storey. This mode of deformation requires large curvature ductility demand in the plastic hinges, which cannot be sufficiently supplied from the columns when the frame is several stories high [P3].

Figure 1.2: Possible modes of deformation in multi-storey building.

Alternatively more desirable mode of deformation for tall multistorey building can be obtained by designing for the partial and full beam side-sway mechanisms, as shown by Figures 1.2(b) and (c). In the full beam side-sway mechanism plastic hinges are confined to the beams and at the base of the columns. In the partial side-sway
mechanism most hinges are located in the beams, at the base of the columns and at selected locations in columns. For the same amount of displacement, the curvature demands in the members are much smaller for the beam side-sway mechanisms when compared to that of the column side-sway mechanism. P-delta actions should be considered as these can contribute to multiple column hinging.

In order to achieve the desired mode of failure, a design philosophy known as capacity design forms the core of seismic design of multistorey reinforced concrete buildings. This design procedure was initially proposed by Hollings [H1], and consequently has been developed and incorporated in New Zealand structural standards [P3, P4, P5, P6]. The approach involves the design of members in the frame in accordance with the 'weak beam and strong column' philosophy [P7].

This design process can be summarized in the following:

- The potential plastic hinge zones are selected by proportioning the members. To ensure that the chosen mechanism develops in preference to other failure modes, the potential plastic hinge zones are designed to be the weak links or fuses in the structure. In the case of the full beam side-sway mechanism the potential plastic hinge zones are located in the beams, with column hinging limited to just above the column bases (as shown by Figure 1.2(c)).

- The members within the potential plastic hinge zones are designed to have dependable flexural strengths and detailed to ensure they can sustain the required inelastic deformation.

- Shear and anchorage failures are inhibited within members that contain plastic hinges by ensuring that the strengths of these failure modes exceed that of the potential plastic hinges at flexural over-strength. The member over-strength is calculated based on the likely increased strength due to for example, higher than specified characteristic yield stress and strain hardening of reinforcement in reinforced concrete sections.
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- In regions outside potential plastic hinge zones, protection against inelastic deformation is provided by ensuring that the strength exceeds the maximum demands that can be imposed from over-strength actions sustained in the potential plastic hinges.

With this approach it is essential that the member over-strengths are realistically determined. Underestimates may result in the formation of unintended column failure modes, while overestimates increase the structural costs.

With the formation of plastic hinge zones, the tensile strains in the reinforcement are appreciably larger than the compressive strains in the concrete and as a consequence the member increases in length. Beams in ductile frames tend to elongate with the formation of plastic hinges. The restraint provided by floor slabs to this elongation increases the flexural strength of the plastic hinge zones. This strength increase is primarily a function of the beam elongation, the in-plane strength of the slab, the structural arrangement of the slab in relation to the frames and the strength of the connections between the slab and the frame. If the strength enhancement due to these factors is under-assessed, it is possible that in the event of a major earthquake, the intended ductile beam-sway mechanism may be replaced by the non-ductile column-sway mechanism, leading to premature structural collapse.

A model of a unidirectional plastic hinge was incorporated into a time history analysis of a six-storey three bay frame building [F3, F7]. The results were compared against a non-elongating analysis of the same structure. It was found that elongation caused the maximum interstorey deflection and the plastic hinge rotations of the column base in the first storey to be doubled in critical regions. In addition it was pointed out that elongation could result in precast floor units being pulled off their supports. This has important implications for the detailing of supports for precast flooring components and external cladding. It was observed that precast flooring systems performed poorly during the 1994 Northridge Earthquake [N1], with collapse occurring due to loss of seating.
1.4 Research Objectives

The seismic performance of ductile moment resisting frames has been extensively researched in New Zealand and elsewhere, and design rules have been developed from this work. As part of this effort the interaction of in-situ floor slabs with the beams of ductile moment resisting frames has been studied. In particular the interactions between beams and in-situ slabs have been considered and rules have been developed and included in the New Zealand Concrete Standard for the amount of slab reinforcement which should be included with the beam reinforcement, in assessing both the design and over-strength values. However, while the research work on which these rules are founded relates well to reinforced concrete slabs supported by beams at regular centres, such as illustrated in Figure 1.1(a), it does not model the case where perimeter frames are used. Two particular aspects are of concern:

- The use of precast prestressed units in floor diaphragm which will tend to restrain beam elongation, inducing forces that may increase the flexural strength of the perimeter frame.

- The effect of using precast prestressed units which span greater than the beams in the perimeter frame, which is likely to concentrate cracking at specific locations and induces failure which may not have been expected previously.

The research described in this thesis aims to bring more understanding to the interaction of plastic hinge zones and diaphragms and the potential problems that might arise with large deformations imposed on the structure. This is a natural continuation to research work on the behaviour of ductile reinforced concrete members in moment resisting frames carried out previously in the University of Auckland. In particular, the elongation of plastic hinge members and the effect it has on the overall behaviour of reinforced concrete frame elements was studied [M1, F2, F11].
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1.5 Outline of Thesis

The literature review on previous studies related to this work is presented in Chapter 2. Chapter 3 describes the experimental programme, including the details of the test units, the test arrangements, equipment and test procedures. Chapter 4 reports on the experimental results from the first test, Unit 1, while the experimental results of Unit 2 and 3 are contained in Chapters 5 and 6 respectively. Chapter 7 presents analytical models developed to model the effects of elongating hinge and interaction of floor slabs in frames. The experimental results and observations are discussed in Chapter 8. General conclusions are contained in Chapter 9.
Chapter 2

Literature Review

2.1 Introduction

In this chapter, the mechanisms of elongation in plastic hinges are described. The results of a number of tests in which elongation has been observed are reviewed. The influence of elongation of frames with insitu floor slabs and the stability of precast flooring elements are also examined. In-plane diaphragm actions in structures are also reviewed.

2.2 Plastic Hinges and Mechanisms of Elongation

When beams are subjected to inelastic cyclic displacements, two types of plastic hinge can form. The type that develops depends on the ratio of the gravity and seismic actions applied to the beam. The two forms of plastic hinges are:

1. Uni-directional or gravity dominated plastic hinges,
2. Reversing or seismic dominated plastic hinges.

With both plastic hinge types, the yielding in tension of the flexural reinforcement causes the beam to elongate. Structural actions associated with elongation have received little attention in the literature until recently, and its influence on the performance of seismically designed reinforced concrete structures has been largely neglected. Test results have indicated that member elongation is quite significant. Typically both uni-directional and reversing plastic hinges elongate by 2 to 5% of the beam depth before strength degradation occurs [F4, M1, R1, M2]. The inelastic load deformation characteristics of uni-directional and reversing plastic hinges are different, as discussed in the following subsections.
2.2.1 Uni-directional plastic hinge

In a frame, uni-directional plastic hinges may form in a beam in a severe earthquake if the level of gravity and vertical loading that is supported simultaneously with the earthquake induced lateral forces exceeds a critical value. Hence this form is known as a gravity dominated plastic hinge [F3]. The positive and negative moment plastic hinges develop at different locations along the beam. This is shown in Figure 2.1. This form of hinge may be expected in a severe earthquake in a proportion of the beams in uniform frame structures (see Figure 1.1(a) of Chapter I), which have been designed for the dual purpose of providing both seismic and gravity load resistance.

![Diagram](a) Frame sways to right
(b) Frame sways to left
(c) Bending moment diagram
(d) Deflected shape of beam

**Figure 2.1:** Uni-directional plastic hinges in beam.
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The formation of uni-directional plastic hinges in a beam of a seismic resistant frame is shown in Figure 2.1. As the frame sways to the right, as shown in Figure 2.1(a), a positive moment (sagging bending moment) plastic hinge forms to the left of the mid-span while a negative moment (hogging bending moment) plastic hinge forms next to the face of the right hand column. When the frame reverses in sway direction, as illustrated in Figure 2.1(b), the opposite situation develops as a positive moment plastic hinge forms in the span to the right of the mid-span as a negative moment plastic hinge forms adjacent to the face of the left hand column. The bending moment diagrams associated with these actions are shown in Figure 2.1(c).

Subsequent inelastic deformation of the structure in either direction causes the plastic hinge rotations to increase. The accumulation of rotation is accompanied by increasing deflection to produce the deflected shape illustrated in Figure 2.1(d). In order to survive an earthquake of high intensity, the plastic hinges must be able to sustain high rotations [M1], which are appreciably greater than the corresponding rotations sustained in reversing plastic hinges. Analyses indicate that with uni-directional plastic hinges imposed rotations are typically two to four times the corresponding values imposed on reversing plastic hinges [F5]. Additional reinforcement may be added to prevent formation of uni-directional plastic hinges. This consists of reinforcement lapped to the bottom longitudinal reinforcement along the beam length and stopped before the ends of the beam.

2.2.2 Reversing plastic hinge

Reversing plastic hinges are formed within the beams of a frame where seismically induced moments dictate the behaviour of the system. This type of plastic hinging occurs in a severe earthquake, provided that the maximum positive and negative bending moments occur at the ends of the beam. In beams with uniform longitudinal reinforcement, the maximum uniformly distributed load, \( w_{\text{max}} \), that a beam can sustain for reversing hinges to form is given by:

\[
 w_{\text{max}} = \frac{2 (M_{\text{pos}} + M_{\text{neg}})}{l^2}
\]

Equation 2.1
where \( M_{pos} \) and \( M_{neg} \) are the maximum positive and negative flexural strengths at each end of the beam,

\( l \) is the clear span of the beam between the column faces.

If the distributed force, \( w_{max} \), exceeds the value given by Equation 2.1, uni-directional plastic hinges form [F6].

---

**Figure 2.2:** Reversing plastic hinges in a beam.
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As the structure sways to the right, as shown in Figure 2.2(a), positive and negative moment plastic hinges form in the beam adjacent to the left and the right of the column faces respectively. The inelastic rotation of the plastic hinges reverses, shown in Figure 2.2(b), as the frame sways in the other direction. The bending moment diagram associated with this behaviour is shown in Figure 2.2(c). In this case, as shown in Figure 2.2(d), the deflected shape of the beam does not change with subsequent load cycles and the plastic hinge rotation is closely related to the interstorey drift. A significant amount of elongation in the beam length can result from repeated inelastic rotations of this type of plastic hinge.

2.2.3 Elongation in plastic hinge zones

Plastic hinge elongation was first observed by Paulay in 1969, in a series of tests carried out on spandrel beams of shear walls [P8]. The rotations of the plastic hinges in the beams develop mainly from the yielding of the reinforcement by flexural tension. Elongation occurs, as the tensile strains in the reinforcement are appreciably greater than the compressive strains.

Elongation in uni-directional plastic hinge zone

Elongation in uni-directional plastic hinges occurs primarily as a result of inelastic rotations. An idealised form of elongation sustained by a beam containing unidirectional hinges after recurring cyclic loading is shown in Figure 2.3. The extensive yielding of the reinforcement of a test beam is shown in Figure 2.4. It can be seen that the reinforcement in the compression zone sustains little strain and the rotation arises primarily from the tensile strains in the reinforcement. This demonstrated that the strains in the compression zone reinforcement are small and could be ignored. Based on this, the resultant elongation at mid-height of the beam, \( \delta_{eu} \), may be found by multiplying the plastic hinge rotation by half the distance between the top and bottom reinforcement, as given by Equation 2.2.

\[
\delta_{eu} = \frac{\Sigma \theta (d - d')}{2}
\]

Equation 2.2
where $\sum \theta$ is the sum of rotations of the plastic hinges in the beam bay,

$(d - d')$ is the distance between the centroids of the compressive and the tensile longitudinal reinforcement.

\[ \text{rotation in plastic hinge represented by a single crack} \]

**Figure 2.3:** Elongation in beam due to formation of uni-directional hinges [M2].

\[ \text{measurements in this zone} \]

\[ \text{At displacement ductility, } \mu : \]

<table>
<thead>
<tr>
<th>(\mu = 2)</th>
<th>(\mu = 6)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 - D28 (top)</td>
<td>2 - D28 (bottom)</td>
</tr>
</tbody>
</table>

\[ \text{at } \mu > 4 \text{ the } 1st \text{ and } 2nd \text{ cycle values are nearly identical} \]

**Figure 2.4:** Elongation of reinforcement in a uni-directional plastic hinge at column face of a test beam [F6].
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A uni-directional plastic hinge model has been developed in the University of Auckland, which has been implemented into the dynamic analysis program DRAIN-2DX and used to analyse a number of structures, which formed uni-directional plastic hinges [F7]. This model was later extended to include both the uni-directional and the reversing plastic hinges with the addition of shear deformation [D2].

Elongation in reversing plastic hinge zone

Figure 2.5 shows the strain patterns in of a reversing plastic hinge formed in a test beam [F6]. The strain patterns are markedly different from that of the uni-directional plastic hinge as shown in Figure 2.3.

![Diagram](Image)

**Figure 2.5:** Elongation of reinforcement in a reversing plastic hinge of a test beam [F6].

A reversing plastic hinge acts as a uni-directional plastic hinge on the application of initial inelastic displacement, where the reinforcement yields in the tension zone and a small compression strain is sustained by the compression zone reinforcement. On reversal of loading direction, the reinforcement in the compression zone, which had yielded in tension in the previous half cycle, does not fully yield back in compression.
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With additional inelastic loading cycles, the reinforcement continues to increase in length until buckling of the bars occurs. The extension of the reinforcement causes the cracks in the compression zones to remain open. Equation 2.3 was proposed to calculate the elongation, $\delta_{cr}$, in reversing plastic hinges [F6]:

$$\delta_{cr} = e + \frac{\Sigma \theta (d - d')}{2}$$  \hspace{2cm} \text{Equation 2.3}

where $e$ is the elongation of the reinforcement of the plastic hinge in the beam which had yielded in tension in the previous half cycle and has not fully yielded back in compression.

Two principle reasons were given for the explanation of why the longitudinal reinforcement in the compression zone of a reversing hinge does not fully yield back, thus preventing the cracks from closing [F6]. These are:

1. \textit{Contact stress effect} - arises when the concrete cracks in tension and the tensile longitudinal steel yields and dislodges aggregate particles into the cracks. When the loading direction reverses and the longitudinal steel goes into compression, these aggregate particles sustain local contact compression pressure, restricting the closure of the cracks [B1].

2. \textit{Mechanism of shear resistance in plastic hinge zones} - with the formation of intersecting diagonal cracks in the hinge zone, the shear resistance is provided by a truss like action, as shown in Figure 2.6. In this mechanism, the diagonal compression struts are sustained by the concrete while the tension ties are sustained by the transverse reinforcement. From the equilibrium requirements shown in Figure 2.6(b), it can be seen that the flexural tension force, $T$, is always larger than the flexural compression force, $C$, at the same section. As a consequence, the inelastic rotations in the hinge zone tend to occur more by the yielding of the tension reinforcement than the reinforcement in the compression zone. Under repeated cyclic loading, this results in longitudinal extension (or elongation) of the plastic hinge.
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(a) Diagonal compression

(b) Truss action

Figure 2.6: Mechanism of shear resistance in reversing plastic hinge [F6].

A model of plastic hinge zone substructure, as illustrated in Figure 2.7, was developed in the University of Auckland [D2, D3]. The steel and concrete truss elements, which are pinned at the end to the rigid arms, are placed at the centroids of the beam reinforcement. It can be seen from this model that once the steel truss element yields in tension, the plastic hinge increases in length. Under the tensile action, the concrete cracks and is assumed to have negligible stiffness, therefore the elongation has little impact on the cracked concrete element response while in tension. In the compression zone, the concrete truss element does not significantly reduce in length due to its large compression stiffness. The length of the plastic hinge is only updated once the loading direction reverses.

Significant shear deformations occur in plastic hinges of reinforced concrete beams subjected to inelastic cyclic loading. In the model shown by Figure 2.7, the shear resistance of the hinge is provided by the shear link. This element has no axial and flexural stiffness. The behaviour of this element incorporates a modified version of shear deformation theory developed by Fenwick and Thom [F8]. It applies to members containing equal areas of tension and compression reinforcement and assumes that axial loads are negligible. The following conclusions established from experimental observations formed the basis of the response of the shear link:

- Strain distribution is approximately linear along the length of the plastic hinge.
- The cracks in the compression zone remain open (zero axial load).
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- Extensive transverse reinforcement yielding occurs at the high strain end of plastic hinges.
- The shear resistance provided by the concrete (ie. the $V_c$ component) is negligible once intersecting diagonal cracks form in a reversing plastic hinge. The shear is resisted by tension forces in the stirrup and diagonal compression in the concrete.
- At load levels in excess of that causing yielding in the transverse reinforcement, repeated cycles of shear will cause the concrete in the web to spall in a manner which suggests a compression failure.

The hinge model was implemented into a two dimensional nonlinear computer package DRAIN-2D, which is capable of performing static and dynamic analyses. The cyclic response from analytical simulations of reinforced concrete members incorporating the hinge substructure were compared against experimental results from four cantilever beams, a portal frame and a two-storey three bay bent. The load versus displacement and moment versus rotation response closely followed those obtained experimentally.

![Diagram of plastic hinge model](image)

**Figure 2.7:** Model of elongating plastic hinge in beam [D2].

Analytical predictions of beam elongation, shear and flexural deformations were in reasonable agreement with values measured in experiments. However, the model was unable to represent accurately the effects of concrete contact stresses and reinforcement bar buckling and kinking. The effects of axial loading on the plastic hinge zone was not considered.
2.3 Previous Reporting on Elongation

The elongation that develops within the plastic hinges during testing of statically determinate units such as beams and beam-column subassemblies does not induce reactions within the members and is therefore easily overlooked. However, on formation of plastic hinge within statically indeterminate structures, a redistribution of internal forces occurs throughout the structure. The resulting internal actions are a function of the extent of plastic deformations, relative stiffness and overall configuration of the members within the structure. Hence elongation induced effects such as frame dilatancy (or expansion), loss of floor support and induced axial forces in floor slab can be important in indeterminate structures.

Elongation in reinforced concrete members has been observed by researchers, who have highlighted different aspects of plastic hinge elongation and its effects. These are briefly reviewed in the following sections.

2.3.1 University of Auckland, New Zealand

In this section, research carried out within the University of Auckland relating to elongation in reinforced concrete members is reviewed. Experiments where elongation was observed, and also tests where elongation was studied in detail, are presented. Also included are test programs of moment-resisting reinforced concrete frames, where their behaviour under cyclic loading was studied. In particular, the effect that elongation has on the overall performance was evaluated.

Elongation in reinforced concrete members

Elongation has been measured in many tests conducted at the University of Auckland. This action was first observed and reported by Fenwick and Fong in 1979 [F9], where five reinforced concrete cantilever beams were subjected to cyclic loading. These beams were 500mm deep by 200mm wide and had different span lengths. The increase in the length of beams was between 13 to 19mm in magnitude, which corresponded to elongation of 2.6 to 3.8% of beam depth. Fenwick and Nguyen tested a beam-column connection, where elongation of 4.4% of beam depth was measured [F10]. Another
series of tests reported by Fenwick et al in 1981 [F4], involved testing of eight cantilever beams. Elongation measurements of 2.5 to 4.1% of beam depth were recorded.

Two identical beams were made and tested to enable the performance of uni-directional and reversing plastic hinges to be compared [F6]. The first beam was subjected to cyclic loading, but the force in the upward direction was limited to 3/4 of the theoretical strength. The beam was loaded in the downward direction such that a uni-directional plastic hinge was formed. For this beam, measurements indicated that the strain sustained in the compression zone reinforcement was negligible provided that it had not been yielded in tension in previous load cycles. As shown in Figure 2.8, the elongation measured from the beam and from a test of a portal frame (see later) correlated well with the calculated values obtained from Equation 2.2 in which \( e \) was zero.

![Graph showing measured vs predicted elongation](image)

**Figure 2.8:** Predicted and measured elongation in members forming uni-directional hinges [F6].

For the second beam, cyclic loading was applied to impose equal displacement in both directions such that a reversing plastic hinge was formed. On the initial inelastic displacement, it acted as a uni-directional plastic hinge and a small compression strain was sustained by the compression zone reinforcement. On the reversal of the loading
direction, the reinforcement in the compression zone, which had yielded in tension during the previous half cycle, did not fully yield back in compression. With continued cyclic loading, the compression reinforcement continued to elongate.

Three more cantilever beams were tested under cyclic loading so that reversing plastic hinges were formed [F6]. Two of the beams were rectangular (500mm deep by 200mm wide), the first beam reinforced with five 20mm deformed longitudinal bars in both the top and bottom, and the second beam with five 20mm bars in the top and three 20mm bars as bottom reinforcement. A third beam had a tee-shaped cross-section with 500mm deep by 200mm wide web and 80mm thick flanges, 700mm wide both sides of the web. It was reinforced with five 20mm longitudinal bars in both the top and bottom and ten 10mm bars in the flanges. The test results showed that the beams with unequal areas of top and bottom reinforcement sustained slightly larger elongation when the larger area of steel is in compression, in comparison to the beam with equal areas of top and bottom reinforcement. Slightly smaller elongation was measured in the reverse direction than the corresponding beam with equal top and bottom reinforcement areas. It was found that the slab in the tee-section beam was ineffective in restraining the elongation.

Four beams, which were identical in section (500mm by 200mm), were subjected to cyclic loading under differing axial load levels for each beam [F7, T2]. Three of the beams were subjected to axial load levels of 0.039 $A_g f'_c$, 0.068 $A_g f'_c$ and 0.145 $A_g f'_c$ (where $A_g$ is the gross sectional area and $f'_c$ is the unconfined concrete compression strength). Three of the beams were tested dynamically at speeds comparable to that of a major earthquake. The fourth beam (subjected to axial force of 0.145 $A_g f'_c$) was tested slowly over a period of two days. It was found that elongation decreased with increasing axial load. This is shown in Figure 2.9.
Figure 2.9: Elongation of beams subjected to axial loads [F7].

To close the cracks in a reversing plastic hinge in a beam with equal top and bottom reinforcement an axial force must act. The axial force that acts must increase the magnitude of the force in the compression zone so that the compression force is equal to the tension force. The difference in the tension and compression forces is due to the mechanism of shear resistance in a reversing plastic hinge, which can be represented by a truss like action (see Section 2.2.3 and Figure 2.6).

It was proposed that and axial force of \( 0.05 A_g f_c' \) was required to close the cracks due to contact stress effects (wedging action of the dislocated aggregate particles in the crack). On this basis it was proposed that the axial force level, \( F_c \), required to just close the cracks in the compression zone is given by:

\[
F_c = V_o + 0.05 A_g f_c' \quad \text{Equation 2.4}
\]

where \( V_o \) is the shear force due to truss like action associated with shear resistance in reversing plastic hinges.
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Reinforced concrete portal frame

In an investigation into the performance of structures, which form uni-directional plastic hinges in a severe earthquake, a reinforced concrete portal frame was built and tested [M1]. Two constant point loads were applied to the beam to represent gravity loads, while a lateral force, which reversed in direction, acted just above the beam level to represent seismic actions, see Figure 2.10.

![Diagram of reinforced concrete portal frame](image)

**Figure 2.10:** Test arrangement of portal frame [M1].

During the test predominantly uni-directional plastic hinges formed. Positive moment plastic hinge rotations accumulated near the vertical load points and negative moment hinges accumulated in the beam near the column faces. The progressive increase in plastic hinge rotations was reflected in the increasing mid-span deflection, which reached 1.1% of the span during the second cycle to displacement ductility 6. Beam elongation was between 30 to 40mm depending on the direction of loading while the corresponding average lateral displacement was less than ±60mm. In a load cycle in which the imposed drift was ±3.4%, buckling occurred in the compression reinforcement, which led to the lateral strength being reduced to less than 80% of the theoretical strength.
Stiffness degradation in plastic hinges is largely a function of the shear deformation that occurs when the shear reverses in direction [F6]. For the portal frame, shear reversal did not occur and hence pinching of the load deflection curves did not develop to any appreciable extent, as shown in Figure 2.11. Consequently, there was very little stiffness degradation until failure was imminent and the longitudinal reinforcement started to buckle.

![Graph showing load versus displacement response at mid-point of portal frame.](image)

**Figure 2.11:** Load versus displacement response at mid-point of portal frame [M1].

**Three-bay reinforced concrete bent with and without slab**

Two 1/3 scaled test units were designed to model a level of an internal three-bay frame in a multistorey building [F2]. One of the units had a slab and the other was without. The unit with the composite slab had transverse beams, which cantilevered out from each column. The aim was to assess the influence of the slab on behaviour. The reinforcing details in the main beam and columns were identical for both units. The test arrangement for the composite frame slab unit is shown in Figure 2.12(a) and (b), and the beam and slab section is shown by Figure 2.12(c).
Figure 2.12: Two-bay reinforced concrete frame with floor slab [F2]. (continued)
Figure 2.12: Two-bay reinforced concrete frame with floor slab [F2]. (concluded)

One way hinges were fixed to the column bases and lateral forces were applied by reversing pin ended hydraulic actuators fixed to the top of each column, as shown in the figure. During the tests, the lateral forces applied to each of the external columns were kept at half the value applied to each the internal columns. With this arrangement, the columns provided no restraint to the elongation of the beams.

In both units, diagonal shear cracks in the beam hinges were well defined at the end of the ductility 2 cycles. Spalling of cover concrete started during the cycles to ductility 4. Some crushing of web concrete was evident in the ductility 6 cycles.

The presence of the slab significantly increased the strength and stiffness of the unit. The load versus deflection diagram for both tests is presented in Figure 2.13. The ductility 1 displacement for the unit without the slab was 7.4mm while the corresponding value for the unit with the slab was 5.2mm. These lateral displacements corresponded to interstorey drifts of 0.95% and 0.67% respectively. For the unit
without the slab the maximum lateral strength was 124kN, which was recorded during the ductility 4 cycle (equivalent interstorey drift of 3.7%). For the unit with the slab, at 2% interstorey drift, the reinforcement within the full width of the slab had yielded.

![Graph](image)

(a) With slab

![Graph](image)

(b) Without slab

Figure 2.13: Load versus displacement response of reference point of three-bay bent [F2]. (figures size adjusted from original to match scales)
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The maximum lateral strength of 196kN was reached during the ductility 4 cycles at an interstorey drift of 2.7%. The slab cantilevered out from the beam web by a clear 3.7 beam depths on each side. For comparison, the current New Zealand concrete design standard [S2] requires an equivalent width of 1.75 beam depths on each side of the web to be considered to contribute to the beam flexural strength in negative bending.

During the tests, it was evident that the plastic hinges adjacent to the exterior columns sustained more damage than the other plastic hinges. This was due to greater rotations that were forced onto these hinges as the elongation in the beams forced larger lateral displacement on the external columns relative to the internal columns. This is illustrated in Figure 2.14 and it was observed in both test units. However, this situation would not occur to the same extent in a multistorey building as the external columns are constrained by the beams in the higher levels.

For the unit without the slab, it was found that elongation amounted to a maximum of 35mm, which corresponded to an average elongation of 2.3% of beam depth per plastic hinge. At interstorey drift of 4%, the compression reinforcement in the plastic hinges started to buckle. This led to some reduction in the elongation of the beams. For the composite slab unit, elongation peaked at 28mm, or an average of 1.9% of beam depth per plastic hinge. The elongation versus interstorey drift is shown for both units in Figure 2.15. It can be seen that there is little difference between the units, indicating that the longitudinal slab had only a small influence on the magnitude of the elongation that developed.

![Diagram](image)

**Figure 2.14:** Differences in column lateral displacement [F2].
Figure 2.15: Comparison of elongation measurements from three-bay bents [F2].

2 ½-storey reinforced concrete frame

An approximately 1/3 scaled, 2 ½-storey three-bay reinforced concrete frame was built and tested by Fenwick et al [F11]. The unit was designed to a strength level that would correspond to a structure of about eight storeys. The reinforcement details were designed such that under cyclic loading, plastic hinges would be confined to the beams close to the column faces and to the columns close to the face of the foundation beam.

Lateral forces representing the storey seismic shear were introduced into the frame by pin-ended hydraulic actuators acting on each of the four columns, at a level equivalent to the mid-height of the third storey (as shown in Figure 2.16). The lateral forces were maintained throughout the test in a ratio of one to two for the external and internal columns respectively. With this arrangement, no artificial restraint to beam elongation was provided by the loading system.

The theoretical ultimate lateral strength of the frame was 192kN. The average displacement measured at 120kN was linearly extrapolated to obtain a ductility 1 displacement of 16.5mm, which corresponded to an interstorey drift of 0.63%.
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Figure 2.16: 2½ storey test frame by Fenwick et al [F11].

Figure 2.17: Load versus displacement response at reference point of 2 ½ storey frame [F11].

The lateral deflection of the reference point (as shown in Figure 2.16) is plotted against the total applied lateral force in Figure 2.17. It can be seen that stiffness degradation occurred in the ductility 2 cycles. This was due to shear deformation in the plastic hinge zones. With subsequent ductility 4 and 6 cycles, stiffness degradation continued
to increase with increasing shear deformation. Two complete cycles at ductility 6 were sustained with storey shears exceeding or reaching the calculated ultimate value. However, in the first half of the third ductility 6 cycle, the strength decreased to 71% of the theoretical ultimate value.

Plastic hinging in the beams was accompanied by significant elongation. This reached 53mm in the top line of beams and 45mm in the bottom beams (see Figure 2.19). These values correspond to an average elongation, which is in the range of 2.5 to 3.0% of beam depth for each plastic hinge. This elongation had a significant influence on the overall behaviour of the frame. As illustrated in Figure 2.18, the elongation of the lower beam forced the left hand side column outwards, increasing the shear sustained by it. In addition to this, axial tension was induced in the upper beams and axial compression in the lower beams. Axial tension in the upper beams led to an increase of shear deformation and concrete spalling in the associated plastic hinges, when compared to those in the lower level.

![Diagram](image)

**Figure 2.18:** Deformed shape of 2 ½ storey test frame [F11].
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The elongation in each level in the beams is shown in Figure 2.19. Beam elongation increased the rotation in the plastic hinges at the base of the external columns. This behaviour should to be considered when designing the shear and confinement reinforcement in the columns.

Figure 2.19: Elongation of beams in 2 ½ storey frame [F11].

2.3.2 University of Canterbury, New Zealand

Cheung recorded elongations between 2.5 to 4% of beam depth per plastic hinge in tests performed on beam-column with insitu slab units [C1]. Cheung et al [C2], postulated that beam elongation in the presence of a composite slab would cause the beam to go into compression, and act as a strut to resist the tension force sustained by the longitudinal (mesh) reinforcement in the slab. This strut and tie mechanism is shown in Figure 2.20. As indicated in the diagrams, the mechanism around the interior columns is dependant on the magnitude of the tension force sustained by the longitudinal reinforcement in the slab. The mechanism around the exterior columns relies on the anchorage of the slab reinforcement in the exterior transverse beams. In both cases, the strength of the beams was enhanced due to the introduction of the compression forces into the beams.
Restrepo tested a number of beam-column subassemblies and noted that elongation occurred when inelastic displacements were applied [R1]. He measured elongations between 2.1 to 2.8% of beam depth on various types of precast concrete beam-column units. From his and other tests, he concluded that the elongation in the test units containing two plastic hinges lay between \( \theta (d-d') \) and twice this value, where \( \theta \) is the rotation sustained by the plastic hinges. The expression \( \theta (d-d') \) was proposed by Megget and Fenwick for the prediction of elongation in uni-directional plastic hinges [M1].

### 2.3.3 United States of America

Beam elongation was reported by Zerbe and Durrani, who carried out tests on beam-column connections and two-bay beam-column subassemblies with and without slabs [Z1, Z2]. The lateral loads were applied to a stiff loading beam, which was connected to the tops of the columns. The columns were pinned at their bases to a rigid support, as illustrated in Figure 2.21. The authors compared the elongation in the two-bay frame with tests performed on one internal and two external statically determinate beam-column units. The elongation measured on the beam-column units were summed for comparison with the total elongation in the frame unit. It was found that the total
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elongation at 4.0% interstorey drift was 9.5mm for the frame unit, and 15.0mm for the beam-column units. These values correspond to 0.8 and 1.2% of beam depth per plastic hinge for the frame unit and the beam-column units respectively.

![Diagram of test arrangement of two-bay frame with floor slab](image)

Figure 2.21: Test arrangement of two-bay frame with floor slab [Z2].

The authors suggested that the difference in the elongation measured was due to the flexural stiffness of the columns, which resulted in axial compression in the beams, therefore restraining the elongation in the beams of the frame unit. Significant cracking of the outer faces of the external columns supported this suggestion. However, it appeared that it was the loading system that restrained the elongation since the tops of the columns were fixed in position relative to each other. The authors later commented that at large drift levels, the main beams could be restrained against axial elongation and that the testing arrangement more accurately represents the first storey in a multi-storey building. However, as shown by Figure 2.18, this may not be totally correct, where the test showed that the columns above the first level were forced outwards as the beams in the lower level elongated.

Qi and Pantazopoulou tested a 1/4 scaled two-bay beam-column subassembly with an insitu slab, as shown in Figure 2.22 [Q1]. The frame was built to represent the first one and a half storey of an internal frame. The researchers attempted to avoid introducing unrealistic restraint to elongation in the beams by controlling MTS actuators such that
the relative displacement at the ends of each actuator was set equal to the measured elongation within the respective span.

Figure 2.22: Test arrangement of two-bay frame with floor slab [Q1].

Figure 2.23: Effect of elongation in the beams in the level [Q1].

Throughout the test, they noted that the distribution of the base shear at the foundation was affected by the direction of loading. They attributed this alternating pattern of base shear distribution to a combination of overturning moments created by the application of lateral loads and residual beam deformations. The residual beam deformations, which resulted from inelastic strains in the reinforcement, cracking of the concrete and bond deterioration, accumulated to produce an overall expansion in the beams as shown in Figure 2.23. The elongation of the beams averaged 1.6% of the beam depth for each plastic hinge in the later loading cycles. This action introduced additional flexural moments and shear forces in the columns. The additional shear in the columns induced axial compression forces in the beams.
2.3.4 Japan

Hinge elongation was observed during the testing of a full scale seven-storey wall-frame building built on a large strong floor. This work was carried out as part of the U.S.-Japan Cooperative Earthquake Engineering Program [W1]. Substantial elongation was found in the wall due to the formation of a plastic hinge at its base. This resulted in axial tension forces being induced in the surrounding columns and axial compression being induced in the wall. This greatly increased the lateral strength above that predicted by standard methods of analysis. In practice it is unlikely that this level of strength enhancement would be obtained, as the additional axial compression force in the wall would, in all probability, have caused the foundations to fail.

Sakata and Wada tested 1/20 scaled concrete frame models to study the effects of deformation in multiple bay, medium height frame structures [S6]. Because of the small size of the models, they were able to devise a system of applying independent lateral loads to the columns of the test models without introducing additional restraint to the elongation of the beams. They showed that elongation could be expected to be larger in the higher levels when compared to the lower levels of the structure.

2.4 Stability of Precast Flooring Elements

Elongation of the beam plastic hinges may cause loss of seating for precast floor units. The situation is particularly severe for perimeter frame structures where the elongation from the plastic hinges in several beams can be applied to one span of precast units. The effect that this has on the connections of precast elements with cast in place topping and supporting beam has been investigated by Mejia-McMaster and Park, who tested three different connections between the ends of precast, pretensioned concrete hollowcore units [M3]. The first part of the test involved the downward loading on the floor unit, which had been constructed without bearing on the supporting beam. This was carried out to investigate the shear friction capacity provided by the topping slab. The second part involved application of horizontal load until seating was lost, followed by vertical loading to investigate the vertical reaction provided by the kinking action of tie bars.
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From this study, the authors recommended that special reinforcement should be used at the end supports of hollowcore floor units to prevent collapse in the event of inadequate seating lengths for imposed movements in an earthquake. Proper design of tie bars, by shear friction could be implemented in the event of loss of support without horizontal movement, or by kinking effect of bar if horizontal movement occurred. It was also concluded that straight lengths of tie bars with hooks should be placed within the units, as shown in Figure 2.24(a). The bars should be plain, rather than deformed, in order to allow yielding to propagate along the bar, therefore allowing for large plastic elongation. Figure 2.24(b) shows an alternative detail, which has a diagonal tie bar compared to the straight tie bar shown in Figure 2.24(a). However, this is a less desirable detail, as kinking of the tie bar may be accompanied by cracking of the floor unit along the diagonal tie bar.

Figure 2.24: Recommended connections tested by Mejia-McMaster & Park [M3].
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Three further tests on different connections at the ends of hollowcore units were carried out by Oliver et al [O1]. The first of these was the traditional detail of deformed starter bars going into cast-in-place topping slab with additional steel fibres. This unit behaved unsatisfactorily and it fell when the seating was lost. The second and third units were detailed with additional plain bar ‘paperclip’ ties (see Figure 2.25(a)), with the difference being steel fibres added to cast-in-place topping in unit three. Units 2 and 3 failed with fracture of the ‘paperclip’ legs at corresponding interstorey drift of 1.4% and 2.1% respectively.

![Diagram](attachment:image.png)

(a) ‘Paperclip’ connection

![Diagram](attachment:image2.png)

(b) ‘Staple’ connection

Figure 2.25: Hollowcore floor connection details tested by Oliver et al [O1].

It was concluded that this detail might be suited for buildings with structural walls or where precast floors span individual bays in the frame, but not in situations where large deformation demands are expected, when precast floors span multiple bays in the
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perimeter frame. It was also found that deformed bars traditionally placed in topping slabs to transfer seismic forces in the diaphragm may not have sufficient deformation capacity to resist the effects of beam elongation.

The authors recommended a ‘staple’ connection detail shown in Figure 2.25(b). This detail utilises flexural strength of plain 16mm bars to resist gravity loads when unseating occurs. The de-bonding of the plain bars within the cores should prevent significant axial strains from occurring, therefore allowing large horizontal displacements without the loss of support. A number of acceptable support arrangements of precast flooring are also shown by a publication titled ‘Guidelines for use of Structural Precast Concrete in Buildings’ [N2].

2.5 Effective Slab Width for Calculation of Beam Flexural Strength

The contribution of the strength of a slab in tension to the performance of reinforced concrete beams in frames has been examined in several projects. A number of researchers have proposed that the contribution of a slab can be assessed from the longitudinal reinforcement confined in a specified effective width for the calculation of beam flexural strength. It should be noted that the discussion below apply to cast insitu slabs, and not to necessarily to composite insitu slabs with precast elements (see later).

For beams at internal columns, Zerbe and Durrani [Z2] recommended including the width of the slab equal to two beam depths on each side of the beam for the calculation of the negative flexural capacity of the beams. For external columns they recommended that the width should be reduced to one beam depth on each side of beam if the torsional moment induced in the transverse beams by tension forces in the slab reinforcement (based on two beam depths on each side) exceeds the torsional strength of the transverse beam.

Pantazopoulou et al [P9], developed an analytical model to estimate the effective slab width and proposed an effective width of 1.5 beam depths from the beam face on each side of the beam should be considered for assessing the strength at first yield, and
increasing this to three beam depths on each side for large drift levels. The same values applied for both internal and external joints.

Qi and Pantazopoulou [Q1] found that for exterior connections, the computed flexural strength with an effective slab width of one beam depth on each side of the beam compared well with the estimated strength from experimental data. This was calculated by taking into account the torsional strength of the transverse beam at the exterior supports. For interior connections, the estimated effective beam depth at 2.0% interstorey drift was one beam depth on each side, and at 5.8% it was estimated to be 2.5 beam depths on each side.

The practical implications of amendments in the ACI 318-99 [A3] that include the estimation of nominal beam flexural capacity in seismic design of frame connections were reviewed by Pantazopoulou and French [P10]. ACI 318-99 calls for an effective width of slab reinforcement to be included for calculation of nominal flexural strength of beams under negative (hogging) bending for seismic response. The recommendation is that the effective width of slab on each side of the beam should be the lesser of:

\[ \frac{L}{4} \quad \text{or} \quad b_w + 16 t_s \quad \text{or} \quad \text{centre to centre spacing of beams.} \]

where \( L \) is the span of the beam

\( b_w \) is the width of beam web

\( t_s \) is the thickness of slab.

These values of effective slab participation were obtained from experimental results for tests and correspond to a lateral drift of 2% [P10]. As the effect of slab participation is greatly influenced by the structural drift, greater widths of slab might be effective at larger drifts.

In this paper the authors reviewed 15 years of previous related research from the United States, Canada, New Zealand and Japan. Pantazopoulou and French recommended that designers should consider the effects of slab participation on structural strength and stiffness, overall structure shear demand and capacity, beam shear demand, bar cut-offs and joint confinement.
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On the basis of test results, Cheung et al [C2] recommended that the effective slab width for the estimating of the negative flexural strength of beams should be taken as the lesser of:

- 1/4 of the span of the beam, extending each side from the centre of the beam section;
- 1/2 of the span of the slab, transverse to the beam under consideration, extending each side from the centre of the beam section;
- Where the beam frames into an exterior column, 1/4 of the span of the transverse edge beam, extending each side from the centre of the beam section;
- Where the beam frames into an exterior column but no transverse beam is present, 1/2 the column width extending each side from the centre of the beam section.

These recommendations were adopted in the 1995 New Zealand concrete structures standard [S2].

McBride et al [M2], tested a three-bay beam-column-floor unit (see Figure 2.12), and found that before yielding, the effective slab width was 1.4 beam depths on each side of the beam. At an interstorey drift beyond 2.0%, it was found that nearly the full slab width, which corresponded to 3.7 beam depths on each side of the beam, was effective for beam strength calculations. The authors suggested that the provisions in the 1995 New Zealand concrete code [S2], which gives a value of one quarter of the beam span between internal columns, should be revised to prevent significant underestimates of beam over-strengths.

2.6 Strength Enhancement from Precast-Prestressed Flooring

It is important to assess the likely strength enhancement of the beams due to the restraining forces from floor diaphragms. An underestimate of this strength
enhancement can lead to non-ductile failure modes, such as shear failure in beams or forcing plastic hinges into columns, leading to a possible column-sway failure mode.

Fenwick et al. in an analytical study consider a case where the elongation in the beams may be restrained by precast pretensioned units built into the floor [F12]. This is shown in Figure 2.26(a), where the precast units span more than one bay of the moment resisting frame. This situation is common in perimeter frame buildings. A number of outcomes are possible due to the interaction of the pretensioned units and the perimeter beams. Firstly, the restraint provided may induce high shear forces at the interface with the beam as illustrated in Figure 2.26(b). This might lead to a complete shear failure at this interface, or alternatively it could lead to a significant increase of the negative moment flexural strength in the beam.

![Diagram of perimeter frame and diaphragm](image)

(a) Plan of perimeter frame and diaphragm

![Diagram of shear forces](image)

(b) Shear forces at interface between beams and diaphragm

**Figure 2.26: Interaction of beams and diaphragm due to elongation in beams [F12]. (continued)**
Figure 2.26: Interaction of beams and diaphragm due to elongation in beams [F12]. (concluded)

The authors estimated that the potential flexural over-strength of plastic hinge C for the specific detail examined was 2.3 times the value that would normally be calculated if these actions were ignored [F12].

A further source of strength enhancement of beams due to diaphragm restraint to elongation of beams needs to be considered. Elongation of perimeter frame beams could generate wide cracks in the topping concrete on either side of the transverse beam. This situation is shown in Figure 2.26(c). Forces are transferred across these cracks by reinforcement in the insitu concrete. It can be seen that the floor is acting as a deep beam, with bending moment and shear forces being sustained. These actions together with the force transmitted across the cracks apply an axial force to the beams in the frame, which could increase in flexural strength in negative bending substantially. For the case examined, the estimated strength increase was 1.8 times the value that would be obtained ignoring the interaction with the floor slab [F12].

2.7 Preliminary Results from Testing of a Precast Hollowcore Floor Slab Subassembly

The structural interactions between reinforced concrete perimeter frames and commonly used precast, prestressed floor systems in modern buildings were investigated in a
collaborative effort between Universities of Auckland and Canterbury. At both universities, the main aims were to examine the effects that the elongation of beams had on structural integrity (e.g. seating lengths of floor units) and on strength enhancement of beams in negative bending.

A full scale subassembly was constructed and tested at the University of Canterbury [M4]. It consisted of a two-bay reinforced concrete frame with a single span on each end in the transverse direction. The floor system consisted of 300mm deep hollowcore units with 75mm cast insitu topping. The test setup is shown in Figure 2.27. A complex test rig was set up such that shear forces were applied at the top and bottom of the columns, while the drift angle between each column was kept the same. The displacements applied by the test rig were controlled such that elongation of the beams were neither promoted nor restrained.

Signs of distress to the seating detail were noticed at a relatively early stage of interstorey drift at 0.35%. At 1.9% drift, a significant tear developed in the floor along the joint between the first and second floor unit (see Figure 2.27). This was due to the elongation of the beams, which caused the central column to move outwards (orthogonal to direction of loading), taking the first hollowcore floor unit with it.

By the end of 2.0% drift in the reverse direction, the entire seating had been damaged with some of the units dropping by 10mm. Splitting of the web in the first hollowcore unit was also observed. At this stage the central column had moved outwards by 25mm. The structure was displaced up to 2.5% interstorey drift. The authors reasoned that in an actual building, the separation of the column and the floor unit closest to the perimeter frame from the rest of the floor could occur on several floors, and it is possible that columns would fail by buckling as the lack in effective restraint would increase their effective lengths, hence reducing axial load capacities.

The loading rigs were then moved to the outer frames and the structure was displaced in the transverse direction. The structure was subsequently displaced in steps, and eventually up to 3.5% interstorey drift in this direction. Splitting of the webs in the first hollowcore unit was extensive, and the floor had dropped by 60mm at this stage. The
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loading rigs were then transferred again to the main frame and the unit was displaced up to 2.0% interstorey drift. At this stage, the entire bottom section of the first hollowcore unit dropped. At reversal of displacements to 2.5% interstorey drift, the entire floor failed when design live loads were applied. Even though the floor failed, the perimeter frame beams, columns and beam-column joints were relatively undamaged.

Figure 2.27: Basic test setup of frame-floor slab subassembly [M4]. (continued)
One of the major observations from this work was the way in which the seating of the hollowcore units failed [M6]. Instead of sliding relative to the beam, there was enough bond or friction to cause the unit to fracture at the ends of the units (see Figure 2.28).

A technical advisory group [T1] was formed to discuss these results and recommended details were made and tested. These recommendations have been incorporated in a recent amendment to the New Zealand Concrete Structures Standard [S7]. The first recommended detail (see Figure 2.29(a)) requires a compressible backing material between the end of the floor unit and the beam, with a low friction bearing strip. This detail allows the end of the floor unit to slide and the beam to rotate relative to the floor unit without causing a fracture to the end of the floor unit. The second detail (see Figure 2.29(b)) require plain round bars placed at the bottoms of the filled cells of the
hollowcore units. However for both details, care is needed in placing reinforcing steel in the topping crossing the hollowcore unit and the supporting beam as very high strains could be induced at this section due to elongation and rotation. The tie forces resulting from this could induce flexural and axial tension failure or shear failure of the hollowcore unit (see later).

![Diagram of hollowcore with compressible backing](image)

(a) Hollowcore with compressible backing on low-friction bearing strip

![Diagram of hollowcore with 2-2 leg R6 hairpins](image)

(b) Hollowcore with 2-2 leg R6 hairpins on low-friction bearing strip

Figure 2.29: Hollowcore seating detail in amendment to New Zealand Concrete Structures Standard [S7].
The performance of the first hollowcore unit adjacent to the frame was also emphasised. In particular the relative vertical displacement between the frame beam and the floor slab caused the hollowcore unit to fail by the splitting of the webs in the hollowcore unit. It was proposed that the first hollowcore unit should be placed at some distance away from the perimeter beam with a cast insitu slab (or linking slab) in between, as shown by Figure 2.30, allowing a more flexible interface between the two elements. A detail similar to this was also incorporated in the recent amendment to the New Zealand Standard [S7].

![Diagram of starter bars crossing edge of hollowcore and lapping with topping reinforcement](image)

**Figure 2.30:** Hollowcore unit parallel to perimeter beam [S7].

The problem of the relative vertical displacement was also pointed out by Fenwick *et al* [F12]. In their experiment, the floor unit performed more favourably (no brittle failure of floor unit) due to a flexible insitu slab between the perimeter frame and the first precas: floor unit.

Due to the tear formed between the floor and the column, Matthews *et al* also proposed that tie reinforcement between the floor diaphragm and the column should be placed transverse to the perimeter frame (see Figure 2.31(b)). The New Zealand Concrete Structures Standard specified that the bars should be placed at $45^\circ$ to the beams (see Figure 2.31(a)), but Matthews commented that this could contribute to the perimeter beam over-strength actions. The recommended detail by the standard showed a transverse beam which can help in tying back the column. However, no clear provision was given for an intermediate column placed between floor spans.
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(a) Detail recommended by New Zealand Standard [S4].

(b) Detail recommended by Matthews [M6].

Figure 2.31: Details for tie-back of perimeter column.

Fenwick et al conducted an analytical study into the different actions that may arise in hollowcore floor diaphragms [F14]. The authors looked at the problems associated with the seating of hollowcore units, such as that listed below:

- unreinforced concrete core in hollowcore units due to placement of dam 75mm from the end of the unit (had been until recently standard practice),

- reinforced cores and insitu topping, similar to that shown in Figure 2.24 & Figure 2.25,

- details recently incorporated in the amendments to the New Zealand Structures Standard, shown by Figures 2.29.
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From their analysis, elongation and rotation of supporting beams relative to the floor unit can induce significant tension force in the topping reinforcement connecting the floor unit and the supporting beam. It was shown the force resulting from this, in combination with gravity loading and vertical seismic forces (high seismic zone for example Wellington), the floor units could fail in negative bending unless the topping is reinforced with passive reinforcement or prestressing strands near the top of the unit. An analysis of the shear stresses that could develop in the webs of the hollowcore units also indicated there is significant danger of diagonal tension failure in the units.

In addition to the above, the authors looked at the vertical differential displacement between the hollowcore units and the perimeter frame parallel to the units. They commented that three possible failure modes could result even with the recommended detail shown by Figure 2.30:

- Failure of the linking slab due to combined longitudinal shear, vertical shear and flexure.

- The longitudinal shear in the linking slab induces axial tension and negative moments to the hollowcore unit adjacent to it, and if the magnitude of the forces are large enough, the hollowcore unit could fail by breaking at the top.

- The vertical shear and bending moments transmitted by the linking slab induce torsion in the hollowcore unit and tension in the nearest web, which could result in splitting along the web.

The interaction of hollowcore units and beams transverse to the units (supporting beam) were also considered by Fenwick et al [F14]. Due to rotation of the plastic hinges in the beams (750mm deep beam), it was suggested that the vertical deflection over the width of a typical 300mm deep hollowcore unit (1200mm wide) could be 30mm. This would cause extensive damage to any hollowcore within this zone. Therefore, avoiding supporting hollowcore units on potential plastic hinge zones in beams can help to avoid the damage that would be induced.
When the transverse beam is subjected to bending, tensile strains are applied to the floor slab. As a result, any transverse reinforcement in the topping could sustain tension forces. Standard design practice requires designers to take the transverse reinforcement into account for calculating beam flexural over-strength. Any change in the tension force in the topping reinforcement ($T_1 - T_2$, see Figure 2.32) must be resisted by a shear force, such as friction at the seated edge of the floor unit if it is supported on a mortar bed. Therefore the hollowcore unit is subjected to Vierendeel truss type actions, which could result in a flexural shear failure of unreinforced webs. However, if the units are supported on low friction bearing strips, the tension force in the transverse reinforcement cannot change, and significant lateral displacements must develop between the beam and the hollowcore units. The effectiveness of transverse reinforcement in the topping concrete is also reduced as the tension force remains constant over the bay and makes no overall contribution to the lateral strength. Any force in this reinforcement results from the elongation of the beam only.

![Figure 2.32: Vierendeel truss action in hollowcore (F14).](image)

From their study, Fenwick *et al* gave an alternative proposal for the seating of hollowcore units on supporting beams (F14). This is illustrated in Figure 2.33. The following comments expand on the corresponding numbered captions in the diagram:

1. Tie reinforcement used to transfer tension forces to enable the floor to maintain its function as a diaphragm. This is located at the ends of the units in filled concrete cells and is placed close to the pretensioned strands in the bottom such that the negative moments and associated shear stresses are reduced.
2. A hard board sheet placed against the end of the unit to create a break in the concrete. This is used to limit the magnitude of the forces transmitted into the hollowcore units.

3. Longitudinal reinforcement is added to the topping to ensure that the hollowcore unit has a greater flexural and axial strength than the critical section (end of unit at the support).

4. Additional pretensioned strands added to the top of the units to increase negative moment capacity. Also this increases the height of the zero stress fibre in the zone close to the supports which increases the shear strength of the section when subjected to negative bending.

5. Soft packing around the tie bars enable vertical movement to occur between the hollowcore unit and the beam without causing splitting cracks. This movement arises from the rotation of the beam and hollowcore unit about the low-friction strip.

![Diagram showing soft packing around tie bar, hardboard or equivalent, reinforcement in in situ concrete, pretensioned strands to top, bottom pretensioned strands, tie bar placed near bottom, low-friction bearing strip, and supporting beam (reinforcement not shown).]

**Figure 2.33:** Proposed detail to improve seismic performance of hollowcore units [F14].
Chapter 3

Experimental Programme

3.1 Introduction

In this chapter the experimental programme is described, starting from the design considerations, a description of the construction, instrumentation and the testing procedures adopted for the test units. Also included are the test results on the component materials.

3.2 Design Considerations

Three frame subassemblies were constructed for the experimental phase of the project. One of these frames was constructed integrally with a floor slab, which contained precast prestressed units. These were designed to represent one storey of a ductile, moment resisting perimeter frame of a multistorey building (see Figure 3.1), and as such it would be expected to form reversing plastic hinges in the beams in the event of a design level earthquake. Cyclic lateral forces were applied to the top and bottom of the columns to simulate seismic forces (more details in Section 3.6). These positions represent the mid-height of a storey in a frame building, where the points of inflexion in the columns are expected to form.

The experimental units were detailed in accordance with the New Zealand Concrete Structures Standard, NZS 3101:1995 [S2]. The choice of structural system for this project was based on the perimeter frame building design example contained in the ‘Examples of Concrete Structural Design to New Zealand Standard 3101’, or commonly known in New Zealand as the ‘Red Book’ [C3]. However, instead of incorporating the ‘hollowcore’ flooring system, the ‘interspan’ or ‘rib and infill’ flooring system was used for one of the units. The experimental units were scaled to approximately 1/3 of typical member sizes so that it could be accommodated in the space and equipment available to
the facilities in the University of Auckland Test Hall. A summary of the design calculations of the experimental units have been included in Appendix 1 of this report.

![Diagram of building](image)

**Figure 3.1:** Typical level of frame in a building modeled by test units.

### 3.3 Description of Experimental Units

The primary objective of this project was to investigate the influence of a precast-prestressed flooring system on the structural performance of a perimeter frame. To achieve this, at least two beam-column frame sub-assemblies were required. The first of these, Unit 1, involved a level of a two-bay bent with cantilever beam extensions on each end. The second test, Unit 2, contained the same frame as in Unit 1, but with the addition of precast floor ribs and floor concrete topping. The precast floor ribs were positioned parallel with the perimeter frame and were supported on three beams perpendicular to the perimeter frame. The depth of floor slab was approximately half the length of the frame in plan.

The first test was intended to serve as a benchmark for comparison against the second test, as well as reaffirming findings from previous research. It also allowed the method of loading to be trialled and refined before the more complicated second unit was tested. Subsequent to testing Unit 1, changes to the experimental arrangement and procedures

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were made for Unit 2 and a third unit (Unit 3) to be built and tested. This unit had a similar frame to Units 1 and 2, but was tested using the same procedure employed for Unit 2.

3.3.1 Test Frame Units: Units 1, 2 and 3

The dimensions and cross sections of the beams and columns of the experimental frames subassemblies are shown in Figure 3.2. The distance between the loading points at the top and bottom of the columns was 1230mm. The distance between the centres of each column was 2032mm. And the dimension from the centre of the outside columns to the end of the cantilevers was 1284mm.

The beams were 300mm deep and 130mm wide. They were reinforced with equal top and bottom longitudinal reinforcement, which consisted of three 12mm deformed bars along the entire length of the frame unit. Grade 300 bars, which had a design yield stress of 300MPa, were used. This grade was chosen as the use of high strength reinforcement reduces the stiffness of the beams and gives problems in the anchorage of the bars. Grade 300, 6mm diameter transverse reinforcement was placed in the beams at 65mm centres in the potential plastic hinge zones, in accordance with the anti-buckling and confinement requirements of the code [S2]. The stirrup spacing was increased to 100mm outside of the plastic hinge zones.

The columns were 300 deep and 200mm wide. These were reinforced longitudinally by twelve, Grade 430 (design yield stress of 430MPa), 12mm deformed bars running along the entire height of each column. The columns were deliberately over-designed for two reasons:

1. Obtain a ‘weak beam and strong column’ design, to ensure that the columns remained elastic throughout the test,

2. To allow for the anticipated strength enhancement effects of the beams in the second test, Unit 2, due to the addition of the slab.
Figure 3.2: Reinforcement details of the test frames. (continued)

(a) Elevation of frame unit

Notes: Half-hinge at cantilever ends only for Unit 2, see Figure 3.4 for details.

Column bars welded to 25mm steel plate reinforcement symmetrical about $C_L$
Chapter 3: Experimental Programme

(b) Section A-A: Column cross section

where:
H denotes $f_y = 430$ MPa
D denotes deformed round bar
R denotes plain round bar

(c) Section B-B: Beam cross-section
(flange not shown for Unit 2)

Figure 3.2: Reinforcement details of test frames. (concluded)

Grade 300, 6mm diameter bars were used for transverse reinforcement in the columns. The centre-to-centre spacing of stirrup sets was 50mm in the potential plastic hinge zones immediately above and below the beam face. This was increased to 65mm outside of these zones. Stirrup sets in the beam-column joints were spaced at 40mm centre-to centre in order to provide sufficient strength to these critical regions.

The sectional flexural strengths of the members (without adjacent flanges) are listed in Table 3.1 (see worked example calculation for Unit 1 in Section A1.5, Appendix I). These values were calculated using the average yield stress of the reinforcement determined from tension tests and the average compression strength found from concrete cylinder tests. The rectangular compression stress block defined in the New Zealand Concrete Structures Standard [S1] was used in these calculations. Generally for calculation of beam flexural over-strength in New Zealand, the design yield stress is
increased by an over-strength factor of 1.25. This factor is made up of two parts, the first of these being an allowance of 10% to account for the likely underestimate of the actual yield stress, and secondly 15% to allow for strain hardening characteristics of steel reinforcement. However, as the actual yield stress is known for these test specimens the flexural over-strength was taken as the product of the yield tension force in the reinforcement multiplied by 1.15, and the distance between the centroids of the top and bottom reinforcement.

It can be seen that the central column has a theoretical ultimate moment capacity equal to approximately 2.5 times the flexural over-strength of the beam. It was anticipated that this should give an adequate margin to protect the columns from failure for the strength increase in the beams in Unit 2 due to addition of floor slab.

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<tr>
<th>Table 3.1: Calculated theoretical flexural strength of sections</th>
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*flange not included for Unit 2, calculation including flanges see Chapter 5 pgs. 130-132.*

3.3.2 Test Frame-Slab Unit: Unit 2

The structural details of Unit 2 are shown in Figure 3.3. Where possible, the details were designed according to common best practice. The floor consisted of two spans of precast ‘rib and infill’ units with 40mm of in situ concrete topping. These units were Stahlton’s 900 Ti 200 rib and infill units (see Figure 3.3 b)). The in situ slab was cast on
timber, supported by the ribs. This timber was removed before the unit was tested. As shown in Figure 3.3(a), each precast unit spanned 3125mm from the outer transverse beam adjoined to the end of the cantilever in the main beam, to the central transverse beam extending from the central column.

(a) Plan view of Unit 2

(b) Section A-A: Frame beam and rib units

Figure 3.3: Diagrams of test Unit 2. (continued)
Figure 3.3: Diagrams of test Unit 2. (concluded)
The first precast unit was spaced at 225mm from the face of the frame beam, and five subsequent units were spaced at 450mm centre-to-centre. The sixth unit was joined to a solid end slab, which was 165mm deep and reinforced with six, high tensile 20mm reinforcing bars on each side (see Figure 3.3(e)). The end slab acted as a stiff boundary condition, to model the continuation of a floor diaphragm in a building. The floor was supported at two locations for each transverse beam by pedestals bolted down to the strong floor. PTFE bearings were used to allow the floor to slide over the pedestals during the test.

Figure 3.4: Corner half-hinge connection details.

The insitu concrete above the ribs was reinforced with mesh, which consisted of 3.125mm wires spaced at 75mm centre-to-centre in both directions. Joining the frame beam to the floor were 10mm deformed starter bars spaced at 225mm (see Figure 3.3(c)). The continuity reinforcing between each of the ribs and the central transverse
beam was provided by two 4.0mm wires (see Figure 3.3(d)). A seating width of 25mm was provided for the ribs along the transverse beams. The cantilever and transverse beams were connected with half-hinge joints as shown in Figure 3.4. A 12mm Reid bar (indicated by RB12H, $f_y = 500\text{MPa}$) with footplates screwed to the bar was used to provide the tension connection in the joint.

3.4 Testing of Materials

The materials used for the test units, namely steel reinforcement and concrete, were tested prior to the test of the units. The following subsections describe this process.

3.4.1 Steel reinforcement tension testing

The reinforcement was attained in two batches, the first was used for Units 1 and 2 and the second for Unit 3. Axial tension tests on samples of the reinforcement were carried out to determine the stress-strain relationships. The samples were obtained from each batch of reinforcement and were tested in direct tension without turning the bars to remove deformed patterns. The results of the tests on the reinforcement used in the beams and columns of the frames are summarised in Table 3.2, and details are given in Appendix 2. Additional steel reinforcement was required for the floor slab and transverse beams of Unit 2. The properties obtained from tests on these are shown in Table 3.3.

The 12mm bars were tested on the Avery Universal Testing Machine situated in the Structures Test Hall. The Grade 300, 12mm bar used in the beams of the frame for Units 1 and 2 had an average yield stress of 309MPa, while for Unit 3 it was 315MPa. These bars were ductile, as indicated by the high strain levels of 23 to 25% at maximum stress. The Grade 430, 12mm bars used in the columns yielded at an average of 461MPa for Units 1 and 2 and 466MPa for Unit 3. The strain at failure was between 25 to 26%. The stress-strain plot for tests on three samples of Grade 300 D12 bars are shown in Figure 3.5.

The plain round 6mm bars were tested on the Instron Testing Machine located in the Civil Materials Laboratory. These bars yielded at an average of 358MPa for Units 1
and 2 and 361MPa for Unit 3. An average ultimate stress of 468MPa for Units 1 and 2 was reached at strain levels ranging from 11 to 12%, and for Unit 3 it was 463MPa at strains between 9.5 to 10.5%.

![Figure 3.5: Stress-strain plot of 12mm diameter beam longitudinal reinforcement.](image)

The steel reinforcement properties used in the floor slab of Unit 2 are summarized in Table 3.3. The 3.125mm and the 4.0mm diameter wires were tested on the Instron Testing Machine in the Mechanics of Materials laboratory as shown in Figure 3.6. The 3.125mm and 4.0mm wires had an average yield stress of 408MPa and 431MPa respectively.

Deformed 10mm bars were used in the corner half-hinge joints, nominal reinforcing in the transverse beams and starter bars connecting the frame beam to the floor. These were found to have an average yield stress of 313MPa and an average ultimate stress of 437MPa. The strains at which ultimate stresses were sustained ranged from 18.7 to 19.6%.
Table 3.2: Steel reinforcement stress-strain properties.

<table>
<thead>
<tr>
<th>Description</th>
<th>yield stress $f_y$ (MPa)</th>
<th>maximum stress $f_{max}$ (MPa)</th>
<th>strain at maximum stress $\varepsilon_{max}$ (%)</th>
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<td>Beam longitudinal bars</td>
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<td>441</td>
<td>26.4</td>
</tr>
<tr>
<td></td>
<td>316</td>
<td>441</td>
<td>24.3</td>
</tr>
<tr>
<td><strong>HD12</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column longitudinal bars</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unit 1 and 2</td>
<td>457</td>
<td>606</td>
<td>14.9</td>
</tr>
<tr>
<td></td>
<td>469</td>
<td>609</td>
<td>15.6</td>
</tr>
<tr>
<td></td>
<td>456</td>
<td>609</td>
<td>17.8</td>
</tr>
<tr>
<td></td>
<td>462</td>
<td>614</td>
<td>15.8</td>
</tr>
<tr>
<td></td>
<td>462</td>
<td>609</td>
<td>16.3</td>
</tr>
<tr>
<td></td>
<td>475</td>
<td>612</td>
<td>14.6</td>
</tr>
<tr>
<td><strong>R6</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse reinforcement</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unit 1 and 2</td>
<td>369</td>
<td>470</td>
<td>11.2</td>
</tr>
<tr>
<td></td>
<td>350</td>
<td>470</td>
<td>11.2</td>
</tr>
<tr>
<td></td>
<td>355</td>
<td>463</td>
<td>12.3</td>
</tr>
<tr>
<td></td>
<td>354</td>
<td>464</td>
<td>9.7</td>
</tr>
<tr>
<td></td>
<td>363</td>
<td>462</td>
<td>10.5</td>
</tr>
<tr>
<td></td>
<td>365</td>
<td>464</td>
<td>9.9</td>
</tr>
</tbody>
</table>

Figure 3.6: Testing of wire reinforcement on Instron Test Machine.
Table 3.3: Steel reinforcement stress-strain properties - floor slab of Unit 2.

<table>
<thead>
<tr>
<th>Description</th>
<th>Yield stress $f_y$ (MPa)</th>
<th>Maximum stress $f_{\text{max}}$ (MPa)</th>
<th>Strain at maximum stress $\varepsilon_{\text{max}}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.125mm wire Floor mesh</td>
<td>398</td>
<td>479</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>423</td>
<td>483</td>
<td>11.40</td>
</tr>
<tr>
<td></td>
<td>403</td>
<td>480</td>
<td>15.76</td>
</tr>
<tr>
<td>4.0mm wire Rib end and central transverse beam continuity bars</td>
<td>439</td>
<td>502</td>
<td>11.32</td>
</tr>
<tr>
<td></td>
<td>420</td>
<td>483</td>
<td>11.59</td>
</tr>
<tr>
<td></td>
<td>433</td>
<td>497</td>
<td>12.39</td>
</tr>
<tr>
<td>D10 Starter bars, half-hinge joint and transverse beam bars</td>
<td>312</td>
<td>439</td>
<td>18.95</td>
</tr>
<tr>
<td></td>
<td>316</td>
<td>439</td>
<td>18.69</td>
</tr>
<tr>
<td></td>
<td>312</td>
<td>434</td>
<td>19.63</td>
</tr>
<tr>
<td>D12B Transverse beam longitudinal bars</td>
<td>312</td>
<td>423</td>
<td>22.50</td>
</tr>
<tr>
<td></td>
<td>315</td>
<td>455</td>
<td>22.40</td>
</tr>
<tr>
<td></td>
<td>316</td>
<td>452</td>
<td>22.52</td>
</tr>
<tr>
<td>HD20 End slab longitudinal bars</td>
<td>312</td>
<td>423</td>
<td>22.50</td>
</tr>
<tr>
<td></td>
<td>315</td>
<td>455</td>
<td>22.40</td>
</tr>
<tr>
<td></td>
<td>316</td>
<td>452</td>
<td>22.52</td>
</tr>
</tbody>
</table>

3.4.2 Concrete Compression Testing

Concrete cylinders were prepared with all concrete pours. These were tested using the Contest Concrete Testing Machine in the Civil Materials Laboratory. The results of the tests are summarized in Table 3.4.

Units 1 and 3 were constructed in a single pour (see Section 3.5). Concrete test cylinders were damp cured with the test unit for seven days. Three of these were tested twenty eight days after the pour and a further three immediately before the Unit 1 was tested. It was found that the average strength was 29.8MPa in both instances. Similarly, the concrete cylinder strength of Unit 3 was found to be 24.8MPa after twenty eight days, and was 26.1MPa just before testing commenced.

The frame-floor slab unit, Unit 2, was poured in four stages. This is described in more detail in Section 3.5. The concrete for the first three stages were obtained from ready
mixed concrete suppliers. The concrete for the final stage was prepared in the laboratory. The average strength of concrete from the ready mixed concrete suppliers was 31MPa, whereas the concrete mixed in the laboratory had an average strength of 39MPa.

Table 3.4: Concrete compression test results.

<table>
<thead>
<tr>
<th>Description</th>
<th>number of tests</th>
<th>age at test (days)</th>
<th>individual specimen strengths $f_c$ (MPa)</th>
<th>average strength $f'_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit 1: Frame Unit</td>
<td>3</td>
<td>28</td>
<td>29.6, 30.9, 29.2</td>
<td>29.9</td>
</tr>
<tr>
<td>Whole unit</td>
<td>3</td>
<td>130</td>
<td>30.4, 29.8, 32.2</td>
<td>30.8</td>
</tr>
<tr>
<td>Unit 2: Stage 1</td>
<td>3</td>
<td>28</td>
<td>25.6, 26.4, 24.0</td>
<td>25.4</td>
</tr>
<tr>
<td>Half of transverse beams</td>
<td>3</td>
<td>246</td>
<td>29.1, 29.2, 28.8</td>
<td>29.0</td>
</tr>
<tr>
<td>Unit 2: Stage 2</td>
<td>3</td>
<td>28</td>
<td>29.6, 29.9, 29.1</td>
<td>29.5</td>
</tr>
<tr>
<td>Bottom half of columns and frame beams</td>
<td>3</td>
<td>216</td>
<td>33.1, 31.7, 32.6</td>
<td>32.5</td>
</tr>
<tr>
<td>Unit 2: Stage 3</td>
<td>3</td>
<td>28</td>
<td>26.9, 32.7, 28.9</td>
<td>29.3</td>
</tr>
<tr>
<td>Floor and beams</td>
<td>3</td>
<td>167</td>
<td>28.4, 32.8, 32.2</td>
<td>31.9</td>
</tr>
<tr>
<td>Unit 2: Stage 4</td>
<td>2</td>
<td>28</td>
<td>37.4, 35.6</td>
<td>36.5</td>
</tr>
<tr>
<td>Top half of columns</td>
<td>4</td>
<td>155</td>
<td>43.1, 41.3, 36.3, 35.6</td>
<td>39.1</td>
</tr>
<tr>
<td>Unit 3: Frame Unit</td>
<td>3</td>
<td>28</td>
<td>24.2, 25.4, 24.8</td>
<td>24.8</td>
</tr>
<tr>
<td>Whole unit</td>
<td>3</td>
<td>130</td>
<td>26.0, 25.4, 26.9</td>
<td>26.1</td>
</tr>
</tbody>
</table>
3.5 Construction of Test Units

The test units were constructed in the Structures Test Hall. The majority of the steel reinforcement was obtained in straight lengths and cut to required lengths on site. The stirrups for the columns and the end slab (for Unit 2 only) were obtained pre-bent. However, the stirrups for the beams were bent on site from straight lengths of plain, round 6mm bars.

A base plate was used to attach the base of the columns to one-way pins, which were connected to the strong floor, and to the actuators that were used to adjust the position of the columns (see Figure 3.10). Details of the base plate are shown in Figure 3.7. The column longitudinal reinforcement were welded to slotted holes drilled into the 25mm thick base plate. Four threaded, 28mm diameter studs were welded to the plate to enable the pins to be bolted in place.

![Diagram of construction details](image)

**Figure 3.7:** Detail of connection at column base.

The formwork for the unit was constructed out of 20mm thick particle-board supported on timber lengths. The formwork for Units 1 and 3 was made so that the units could be cast on their side in one pour. The concrete was obtained from a ready-mixed concrete supplier. The specified maximum aggregate size was 10mm. After initial set, the
exposed surface was covered with wet sacks and polythene sheeting for a period of seven days for curing.

The reinforcing cage for the frame portion of Unit 2 was constructed in the same way as for the other units apart from the additional work needed to reinforce the corner half-hinge joints at the end of the cantilevers. The stages of construction of the unit are described in Table 3.5.

Table 3.5: Stages in construction of Unit 2.

<table>
<thead>
<tr>
<th>Stage</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>The reinforcing cages for the transverse beams were built and formwork made. Concrete was poured up to the seating level. These beams were placed into position on the pedestals.</td>
</tr>
<tr>
<td>II</td>
<td>The formwork for the main frame was assembled around the reinforcement, which had been placed and tied to the final position. Concrete was poured approximately to the mid-height of the frame beams.</td>
</tr>
<tr>
<td>III</td>
<td>The ribs were placed into position on the transverse beams and timber infills placed. This was done while the formwork was put in place. The reinforcing for the end slab was put into place. The floor mesh reinforcing was placed and concrete was poured up to the top surface of the beams and floor. Timber infills between each rib unit were removed on the following day.</td>
</tr>
<tr>
<td>IV</td>
<td>The top half of the columns were cast.</td>
</tr>
</tbody>
</table>

Concrete for stages I, II and III was obtained from a ready mixed concrete supplier. Concrete for stage IV was mixed in the laboratory as only a small amount was needed. In all cases, where a construction joint was required, a retarding agent was used to enable the treated surface to be brushed so that the aggregate was exposed at the
junction to the new concrete. At each stage, concrete was cured under damp conditions for seven days. Figure 3.8 shows the unit in stage III, immediately before concrete pour.

![Figure 3.8: Stage III of construction of Unit 2, immediately before concreting.](image)

### 3.6 Loading Equipment and Arrangement

The test equipment and loading arrangement were different for Units 1 and 2. The arrangement for Unit 2 was revised after the testing of Unit 1. Unit 3 was similar in arrangement to Unit 2. The subsections below describe these arrangements.

#### 3.6.1 Test arrangement of Unit 1

A photograph of the test arrangement of Unit 1 is shown in Figure 3.9 and the overall schematic in Figure 3.10. Reversing hydraulic actuators with a 200kN push and 150kN pull capacity were placed in the bays between columns ‘A’ and ‘B’ and ‘B’ and ‘C’. These were operated by reversing hydraulic hand pumps. The pins to which the hydraulic actuators were attached to, at the each of the top of the columns, were held by bolts and packed to the side of the columns using sand and cement mortar. The main loading hydraulic jack with a capacity of 380kN push, 280kN pull was attached to the top of column ‘A’. It was held by a buttress bolted on top of two thick walled steel universal sections (see Figure 3.10). These in turn were held together by bolts and
stressed down to the strong floor by high tensile McAlloy bars. Plates of steel were welded between the buttress and the steel section to provide adequate shear transfer.

The bottom of each column was pinned and supported vertically by 40mm thick steel ‘sway’ plates attached to pins, which were bolted to the strong floor as shown in Figure 3.10. This arrangement enabled the bottom the columns to displace laterally, along the direction of the frame. The bottom displacements of columns ‘A’ and ‘C’ were controlled by hydraulic actuators (with capacity of 150kN push, 100kN pull) pinned to the sides of the base plate of the respective columns. The hydraulic actuators were held on the other side by buttresses bolted to the strong floor. The bottom of column ‘B’ was restrained from moving to any large degree. By this arrangement, the loading components did not provide additional restraint to the elongation of the members, by allowing the outside columns to extend outwards from the middle column. Struts, which were pin ended, were attached to the strong wall and the top and bottom of the columns (see Figure 3.10). These struts allowed displacements to occur in the plane of the frame but restrained out of plane movement.

Figure 3.9: Test Arrangement of Unit 1 - motorised hydraulics control (foreground), datalogger (right), loading hydraulic actuators (top of columns), hand-pumps (on floor).
Figure 3.10: Schematic of test arrangement of Unit 1.
3.6.2 Test arrangement of Unit 2

The loading arrangement with Unit 1 was difficult to operate and control, and consequently a different arrangement was used for Unit 2. The main difference was in the way the top columns were loaded. This can be seen by comparing Figure 3.10 of Unit 1 and Figure 3.12 of Unit 2. The revised arrangement allowed the actuators at the top of the columns to be operated independently each other.

The central hydraulic actuator had a push capacity of 500kN and 280kN in the other direction. This had a shorter cylinder in comparison to the actuators used for the outside columns, which were had capacities of 380kN and 280kN in the push and pull directions respectively. This shorter length made it possible to fit this arrangement in the space available without obstructing the movement of the columns. The reaction force for the actuator to column ‘B’ was provided by a cantilever 250x250x9mm RHS, which was welded at the base to a 40mm thick square plate. This was stressed to strong wall with stressing bolts. Two additional braces were welded to the RHS and the plate. A vertical steel frame provided the reaction frame to the actuator at column ‘C’. This was stressed to the strong wall and also braced diagonally against the floor. Apart from these changes, the other arrangements were similar to that employed for Unit 1.

Figure 3.11: Loading arrangement on Unit 2.
Figure 3.12: Schematic of test arrangement of Unit 2.
3.6.3 Additions to Unit 2

Concern was expressed over the possibility of splitting of the concrete above the starter bars that connect the main beam to the floor slab. These bars were placed on top of the mesh, and the cover concrete over these bars was approximately 15mm. It was decided that some form of retrofit work was needed to prevent possible premature failure by this mode. Strips of steel measuring 20mm wide, 100mm long and 2mm thick were epoxied onto the surface of the insitu concrete directly above the positions of the starter bars (see Figure 3.13(a)). Figure 3.13(b) shows a photograph taken of two of these during the test. The line on the floor, to the left in this photograph is a crack in the floor, which shows that the concrete was splitting above the bar.

![Diagram of Column A and B with precast ribs and steel strip](image)

(a) Typical placement on floor

![Photograph of two steel strips on floor during test](image)

(b) Two steel strips on floor during test

Figure 3.13: Steel strips epoxied to concrete floor.

Additional weight was placed on the floor before testing commenced. The prestressing force in the bottom half of the ribs results in the negative bending of the composite section (floor rib and topping concrete). Stress redistribution due to creep and
shrinkage reduces the average flexural cracking moment of the section. Combined with tension in the floor slab (due to diaphragm forces in the plane of the floor generated from the interaction of the frame with the floor) this could result in failure in negative bending unless the floor is sufficiently reinforced with passive reinforcement. The addition of extra weight can contribute to balance out some of this action by increasing the negative moment capacity. An equivalent surface load of approximately 3.0kPa was applied to the first 1000 mm of the floor width.

Lead bars were used as additional weight nearer to the frame to allow strain measurements to be made with DEMEC gauges. When necessary the lead bars were moved temporarily when DEMEC gauge readings were taken. Sand bags, each weighing approximately 250N, were placed further away from the frame. A total of approximately 18kN of extra weight was placed onto the floor. Figure 3.14 shows the placement of the lead bars and the sand bags. During the test, a few sand bags were moved at a time to enable DEMEC gauge readings to be made.

Figure 3.14: Placement of extra mass on the floor.
3.6.4 Test arrangement of Unit 3

The loading arrangement for this test frame is shown by Figure 3.15. This arrangement was similar to Unit 2, which was described in Section 3.6.2. However the three 40mm thick sway plates at the bottom of the columns were replaced with load cells. These were used to monitor the axial forces acting in the columns, which enabled the shears in the beams to be determined.

![Figure 3.15: Test arrangement of experimental Unit 3.](image)

3.7 Measurement and Instrumentation

A substantial amount of instrumentation to record forces, displacements, and deformations was placed on the experimental units. Displacement measurements were made using portal displacement transducers and force readings were obtained from load cells. Other manual checks were made using tapes and rulers, DEMEC readings, theodolite and level sightings. The number of individual portal gauge and load cell readings that could be used for the Unit 1 test was restricted to 110 due to the channel capacity of the data acquisition system. A total of 104 displacement transducers were used for this test. For Unit 2, a second data acquisition system was acquired and used in conjunction with the first to allow for up to 206 channels to be monitored. A total of 170 displacement transducers and 153 DEMEC gauge reading points were to be
Chapter 3: Experimental Programme

recorded. For Unit 3, a total of 131 channels were monitored during the experiment. The measurements and instrumentation for the experiments are further described in the following subsections.

3.7.1 Measurement of forces

In all the tests, lateral loads applied to the top of all the columns and the bottoms of the outside columns were measured by load cells coupled with the reversing hydraulic actuators. A load cell was used to monitor the lateral reaction force at the bottom of column ‘B’. In Unit 3, additional load cells were placed at the bottom of the columns to measure the axial forces acting on the columns. Prior to the test, the load cells were calibrated on an Avery Universal Machine.

3.7.2 Measurement of displacements

The displacements of the top and bottom of the columns were measured using portal displacement transducers. Calibration of the transducers was conducted on the MTS load frame before the test. At the top of each column on Unit 1, a portal transducer was attached to the hydraulic jack and screwed on to the pinned joint at the side of the column (see Figure 3.16). With this arrangement, the absolute displacement at the top of column ‘A’ was monitored together with the displacement of column ‘B’ relative to column ‘A’, and column ‘C’ relative to column ‘B’. This arrangement was modified for Units 2 and 3. Instead of using the standard sized portal transducers, larger portal transducers with an effective measurement range of ±60mm were manufactured. These transducers were attached to the column pin on one end and to an independent stand on the other end to enable the absolute displacement of each column to be measured.

During the testing of Unit 1, steel rulers and tapes were used to check the displacements against the electronic measurements obtained at displacement peaks. For Unit 2, additional portal transducers were mounted to measure the changes in the distance between the columns as a check against the absolute displacements of the columns. For Units 2 and 3 checks were made using theodolite sightings to steel rulers fixed to the columns.
Figure 3.16: Typical placement of portal transducer at top of columns.

Portal transducers were used to measure the displacements of the bottom of the columns (see Figure 3.17). Steel rulers and tapes were used to check against the electronic readings at zero load positions and cycle peaks during testing. In Unit 2, additional portal transducers were mounted to measure the distance between the bottoms of the columns.

Figure 3.17: Typical placement of portal transducer at bottom of columns.
3.7.3 Measurement of beam elongation

The elongation between the column centrelines for the two internal beams was measured directly using portal transducers as shown in Figure 3.18. Two portal transducers were used to check one against the other during the tests, as well as obtaining an average value for controlling the test (see test procedure in section 3.8). The centres of the beam-column joints contained 12mm reinforcement bars, which had previously been positioned and cast in with the concrete. The 12mm bars were tapped so that aluminium discs could be attached to the bars by screws. Steel rods connected these to the portal transducers. Hooks glued along the beam were used to prevent sagging of the rods (also see Figure 3.19).

In Unit 2, due to the extension of the central transverse beam from column ‘B’, the aluminium disc could not be placed as in Units 1 and 3. Instead, a steel bar was cast into the transverse beam parallel to the longitudinal beam at mid depth so that the steel rods could be screwed into the bar. This was placed at the level of the mid-depth of the beams underneath the floor, as shown in Figure 3.19. The same arrangement was used to measure the elongation from the centre of each outside column to the respective end of the cantilever beam.

Figure 3.18: Instrumentation to measure direct elongation of beam between beam-column joints.
Figure 3.19: Measurement of elongation in beam, Unit 2.

3.7.4 Measurement of beam deformation

Steel studs were welded to the top and bottom line of the beam longitudinal reinforcement at specified positions after the fabrication of the reinforcement cages. These studs were used for mounting portal transducers. The studs were welded to the reinforcement such that the average strains could be measured as well as preventing studs from being dislodged as the concrete cracks. As illustrated in Figure 3.20, an aluminium disc was screwed to each steel stud and shown in Figure 3.21(b), 4mm rods were attached to the disc and the portal transducers. Prior to casting a grease impregnated tape was wrapped around the studs. These were removed after the concrete was cast to give a clear gap between the stud and the concrete (see Figure 3.20).

Figure 3.20: Measurement points on beams and columns.
The overall instrumentation scheme on the frame of the test units is shown in Figure 3.21(a). There was change of level at the beam-column face from the difference in the width of the columns to the beams. As shown in Figure 3.21(b), the steel rod was bent in order to allow a transducer to be placed. However, any slippage of beam reinforcement in the beam-column joints could not be effectively quantified and can only be visually observed.

Figure 3.21: Positions of portal transducers on test unit frames.

The portal transducers were arranged in a pattern so that the portal readings could be reduced to find the following components of beam deformation:
1. Flexural deformation (or rotation of each segment).

2. Shear deformation (in segments with diagonal gauges).

3. Elongation of the beam.

4. Expansion in depth of beam (in segments with gauges normal to the span of the member).

These deformation components were found from the readings as shown graphically in Figure 3.22. The summation of the components from each segment allows the resultant deformation of the beam to be found. As shown in Figure 3.22(a), pure rotation in this segment affects the length of top and bottom members (i.e. members 'a' and 'b'). The effects of shear and depth expansion components on members 'a' and 'b' are typically so small as to be considered negligible, whereas elongation causes equal extension in both gauges. Therefore the rotation, $\theta$, in a segment is given by:

$$\theta = \frac{\delta a - \delta b}{h}$$ \hspace{1cm} \text{Equation 3.1}

where $\delta a$, $\delta b$ are the changes in length of members 'a' and 'b', and $h$ is the distance between the top and bottom members.

From Figure 3.22(b), it can be seen that the shear component causes a change in length of the diagonal members. The elongation and depth expansion components cause the members to extend the same amount. Assuming that there is uniform curvature over the length of the gauges, an approximation of shear deformation, $S$, in one segment can be calculated from:

$$S = \frac{\delta e - \delta f}{2 \cos \alpha}$$ \hspace{1cm} \text{Equation 3.2}

where $\delta e$, $\delta f$ are the changes in length of members 'e' and 'f', and $\alpha$ is the angle of member 'f' to the horizontal (equal angle to 'e').
Figure 3.22: Deformation components in a single grid segment.

Elongation causes the top and bottom members to extend the same amount (Figure 3.22(c)) while rotation causes one of the members to extend while the other contracts. Shear and depth expansion have negligible effect to these members. Therefore the elongation in the segment, $E$, is given by the average change in length of the top and bottom members, as given by Equation 3.3:

$$E = \frac{\delta a + \delta b}{2}$$  \hspace{1cm} Equation 3.3

As shown in Figure 3.22(d), the depth expansion, $G$, is given by the change in length of the vertical member:

$$G = \delta d$$  \hspace{1cm} Equation 3.4

where $\delta d$ is the change in length in the member 'd'.

• 85 •
Figure 3.23: Elongation and flexural deformation component of mid-span deflection in each bay.

The lateral deflection of each bay due to flexural deformation and elongation can be calculated. Assuming that the columns are rigid, the lateral deflection of the mid-span of the bay is given by Equation 3.5 and illustrated by Figures 3.23.
\[ \delta_R = \frac{\left( \theta_1 + \theta_2 \right) h_c}{2} \]  \hspace{1cm} \text{Equation 3.5}

where \( \delta_R \) is the lateral deflection at mid-span of the bay,
\( \theta_1 \) and \( \theta_2 \) are the rotations of the columns, and
\( h_c \) is the height of the column from the base to the beam centreline.

Referring to the geometry in Figure 3.23(a), the column rotations are given by:

(a) \[ \theta_1 = \frac{\Delta_2}{L} \]  \hspace{1cm} \text{and}  \hspace{1cm} (b) \[ \theta_2 = \frac{\Delta_1}{L} \]  \hspace{1cm} \text{Equations 3.6}

where \( \Delta_1 \) and \( \Delta_2 \) are as shown in Figure 3.23(a), and
\( L \) is the distance between the column centres.

The rotation of the discrete segments (see Equation 3.1) can be used to calculate \( \Delta_1 \) and \( \Delta_2 \), as shown in Figure 3.23(b), and are given by Equation 3.7.

(a) \[ \Delta_2 = \theta_a x_a + \theta_b x_b + \theta_c x_c + \theta_d x_d \]  \hspace{1cm} \text{Equations 3.7}

(b) \[ \Delta_1 = \theta_a (L - x_a) + \theta_b (L - x_b) + \theta_c (L - x_c) + \theta_d (L - x_d) \]

where \( \theta_n \) is the rotation of segment \( n \), and
\( x_n \) is the distance from column centre to the centroid of each segment ‘\( n \)’.

The lateral deflection of the mid-span of each bay due to shear deformation of the beam can be calculated from summing the beam shear deformations of each segment. From consideration of Figure 3.24, the lateral deflection due to shear, \( \delta_s \), is given by Equation 3.8.

\[ \delta_s = \frac{(S_a + S_b + S_c + S_d) h_c}{L} \]  \hspace{1cm} \text{Equation 3.8}

where \( S_n \) is the shear deformation of segment \( n \) given by Equation 3.2.
Figure 3.24: Shear component of mid-span lateral deflection of each bay.

Portal transducers were added to the columns immediately above and below the beam-column joint zone for Units 2 and 3. These gave additional information on the flexural and shear deformations at these regions. The equations below give the lateral displacement of a column between the top and the bottom of the column. The displacements at the top of the columns measured from the test can be compared against the interpolated flexural and shear components of lateral deflection in the bays (given by Equations 3.5 - 3.8) together with the average lateral displacement of the columns.

Figure 3.25 shows the flexural component of the lateral deflection of a column. The flexural component of lateral displacement at the top relative to the bottom of the column, \( l_f \), is given by:

\[
l_f = \theta_1 d_T + 2(\theta_c h_c) + \theta_2 d_B
\]

\textit{Equation 3.9}

where \( \theta_N \) is the rotation of the segment \( N \) as shown in Figure 3.25,

\( d_T \) is from the centroid of segment to the top of column,

\( d_B \) is from the centroid of segment to the bottom of column, and

\( h_c \) is the height to the top/bottom of column from the centre of column.
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Figure 3.25: Flexural component of lateral deflection of column.

The shear component of lateral displacement, $\ell_s$, is given by the sum of the shear deformation measured directly in each segment.

$$\ell_s = S_I + S_C + S_B \quad \text{Equation 3.10}$$

where $S_N$ is the shear deformation in segment $N$ given by Equation 3.2

3.7.5 Measurement of floor deformation in Unit 2

Portal transducers were attached to rods, which were epoxied into the slab at the ends of the first two rows of precast ribs. This was done to obtain a history of the cracking that was likely to occur at these locations. DEMEC points were placed along the entire length of the floor in a triangular pattern. This enabled local shear movement between the floor and the frame to be measured. Figure 3.26 shows the scheme of measurement points placed on the floor.

The DEMEC gauges were spaced at 200mm apart. The wider spaced gauges further from the frame were measured using a Vernier scale. Three separate DEMEC reference gauges were measured each time measurements were taken in order to allow corrections to be made for thermal strains. The localised shear movement in each triangular DEMEC arrangement (Figure 3.27) was determined in the following way:
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(a) \( X = X_0 + \Delta X \)

where \( X \) is the measured distance as shown in Figure 3.27,
\( X_0 \) is the initial measurement, and
\( \Delta X \) is the change in the distance.

(b) \( Y - Y_0 + \Delta Y \)

(c) \( Z = Z_0 + \Delta Z \)

Equations 3.11

And:

\[ \theta_x = \cos^{-1} \left( \frac{Z^2 + Y^2 - X^2}{2ZY} \right) \]

Equation 3.12

\[ \theta_{x2} = \pi - \theta_x \]

Equation 3.13
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Figure 3.26: Scheme of measurement and instrumentation of floor of Unit 2.

Once the geometry of the deformed shape has been determined from the data and Equations 3.10-13 the unknown displacements, \( u_1 \) and \( u_2 \) can be found using Equations 3.14 and 3.15.

\[
\begin{align*}
  u_1 &= Y \cos \theta_{x2} \\
  u_2 &= Y \sin \theta_{x2} - h_d
\end{align*}
\]

Equation 3.14
Equation 3.15

where \( h_d \) is the distance shown in Figure 3.27.

![Figure 3.27: Geometry of triangular DEMEC arrangement.](image)

3.8 Testing Procedure

Loads were applied at the top and bottom of columns to induce moments into the beams. It was important that the displacement of the three columns would remain parallel to one another during the test, as the columns in levels above the first storey of a building would remain relatively parallel as the building displaces during an earthquake. The procedure was also designed to minimise any restraining forces into the beams as these could influence the strength and elongation.
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After completing the test for Unit 1, as mentioned in Section 3.6, the loading arrangement was re-designed. Therefore the procedure for testing was modified to suit. The following subsections describe the plan for the testing of both units.

3.8.1 Procedure for testing Unit 1

The unit was loaded up to approximately 3/4 of the design strength of the frame (based on design strength of beam section shown in Table 3.1) for two cycles. By linear extrapolation of the column displacement at this point, the ductility 1 displacement was determined. Two complete load cycles were then applied to give displacement ductilities of two, four and six. The planned lateral displacement sequence for the test is shown in Figure 3.28. As shown on the figure, for example the expression ‘+2Di’ represents the peak displacement to first cycle in the positive direction to ductility 2, and ‘-4Dii’ represents second cycle in the negative direction to ductility 4 (ie. +/- sign indicates direction, the number and ‘D’ indicates ductility displacement and the letter indicates the i\textsuperscript{th} cycle). Columns ‘A’, ‘B’ and ‘C’ were loaded and readjusted at a 1:2:1 ratio respectively, while adjustments were made to keep the columns parallel to one another. As the actuators were connected in a single line at the top of the columns, the load reading at column ‘A’ represented the total load on the unit. The difference of the load readings at column ‘B’ and ‘C’ represented the actual load on column ‘B’.

![Figure 3.28: Planned displacement history for Unit 1](image_url)
At least 20 load increments were taken to reach each peak displacement. For each load increment, the top of the columns were loaded to the target ratios. Then a scan of the readings was taken. Following this, the bottom actuators (external columns) were adjusted from the displacement at the top, such that the columns were parallel. The following equations were used:

\[ b_A = t_{AB} + b_B \quad \text{Equation 3.16} \]

where \( b_A \) is the displacement reading at the bottom of column ‘A’, \( b_B \) is the displacement reading at the bottom of column ‘B’, and \( t_{AB} \) is the displacement reading of the displacement between the top of columns ‘A’ and ‘B’.

Similarly:

\[ b_C = t_{BC} + b_B \quad \text{Equation 3.17} \]

where \( b_C \) is the displacement reading at the bottom of column ‘C’, \( b_B \) is the displacement reading at the bottom of column ‘B’, and \( t_{BC} \) is the displacement reading of the displacement between the top of columns ‘B’ and ‘C’.

After making adjustments to the bottom actuators, the load ratio at the top changed. This process was repeated until a satisfactorily small difference was achieved. At this stage the readings from the portal gauges and load cells were scanned and saved.

3.8.2 Procedure for testing Units 2 and 3

Unit 2 could not be loaded in the same way as Unit 1. This was due to a number of factors. Firstly, the loading arrangement and the instrumentation of the critical displacements had been modified. Secondly there was no way of determining the actions in the unit, therefore the columns could not be loaded at a pre-determined ratio. Thirdly, the point of first ductility could not be accurately calculated, since the yield strength of the unit could not be found due to uncertainty about the interaction of the
frame and diaphragm. Therefore, it was decided that this unit was to be displaced to predetermined interstorey drift levels. Unit 3 was also tested in a similar way as Unit 2.

For the initial elastic cycles, the unit was displaced to 0.1% interstorey drift for at least two cycles, and then up to 0.2% for another two cycles. Then the unit was displaced up to 0.5% interstorey drift. Two displacement cycles for each target drift level were taken and in increments of 0.5% drift until there was substantial falling off in performance.

The columns were loaded to a target displacement at each loading step. For each step within a cycle, not more than one-twentieth of the target displacement was imposed. Each column was displaced to:

(a) \[ t = t_A + b_A \]
(b) \[ t = t_B + b_B \]
(c) \[ t = t_C + b_C \]  \hspace{1cm} \text{Equations 3.18}

where \( t \) is the target displacement,

\( t_N \) is the displacement reading at the top of column \( N \) (\( =A, B \) or \( C \)), and

\( b_N \) is the displacement reading at the bottom of column \( N \).

Then the bottom actuators were adjusted, such that the distance between the bottoms of the columns were equal to the average of the elongation of the beam in the respective bays. The following equations were used:

\[ b_A = e_{AB} + b_B \]  \hspace{1cm} \text{Equation 3.19}

where \( b_A \) is the displacement reading at the bottom of column ‘A’,

\( e_{AB} \) is the average elongation of the beam between columns ‘A’ and ‘B’,

\( b_B \) is the displacement reading at the bottom of column ‘B’.

And:
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\[ b_c = e_{bc} + b_b \]

*Equation 3.20*

where \( b_c \) is the displacement reading at the bottom of column ‘C’,

\( e_{bc} \) is the average elongation of the beam between columns ‘B’ and ‘C’,

\( b_b \) is the displacement reading at the bottom of column ‘B’.

This process was repeated until the errors were marginal. At each step the data was saved on file. This procedure was much easier in comparison to the procedure used for Unit I, as the actuators were decoupled and as a result, changes made in any one of the actuators did not affect the others to a large extent. The new data acquisition programme also enabled the values given by *Equations 3.18-20* to be calculated simultaneously.
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Chapter 4

Test Results of Unit 1

4.1 Introduction

This chapter presents the results from the testing of the plane frame unit, Unit 1. It contains general description of the crack patterns, load versus deflection characteristics, flexural and shear deformation and the elongation which developed in the beam members.

4.2 Displacement History

It had been intended to follow the loading sequence set out in Section 3.8.1. However, the displacements were recorded from the top of column ‘A’ instead of column ‘B’, which was the originally planned location. As a result of beam elongation, the exterior columns (columns ‘A’ and ‘C’) moved outwards, and this led to greater displacements being imposed on these columns than had been intended. The actual displacement history measured for column ‘B’ is shown in Figure 4.1 (the figure shows the intended displacement steps eg. +2Di, +4Di etc. and the actual drift displacement). As indicated in the figure, it was displaced up to a maximum of 5.9% interstorey drift in the positive direction, compared against 2.9% drift the other direction.

The controlling displacement should have been the difference in the lateral displacements of the top and the bottom of column ‘B’. However, this would have been difficult to achieve with the test arrangement that was used, as the hydraulic actuator at column ‘B’ was coupled to column ‘A’. In light of the difficulties encountered in this test, modifications were made to the test arrangement and procedure used for Units 2 and 3. As indicated by Figure 4.2, the columns were adequately parallel to one another throughout the test. The lateral displacement at any load stage was taken as the difference in displacement of each column measured between the lateral support and load points for the individual column (ie. between the top and bottom of column where
Chapter 4: Test Results of Unit 1

the actuator loads were applied as shown on Figure 3.10 of Chapter 3). The largest out of parallel between the columns throughout the test was 1.6mm during the cycle to +5.9% interstorey drift.

Figure 4.1: Actual displacement sequence applied to Unit 1.

Figure 4.2: Lateral displacement history of columns.
4.3 General Behaviour and Observations During Test

The duration of the test was seven days. Early in the test a fault was found with the hydraulic hose fitting connected to the hydraulic actuator attached to column ‘A’. The movement in the hydraulic cylinder was very small with large build up in pressure at the pumps. There was a sudden release of this pressure to the actuator, and as a result the frame was laterally displaced up to 9.5mm. This movement exceeded the yield displacement of the unit, described in Section 4.4. As a result, the test unit would have sustained a limited amount of damage in that it is expected that the initial stiffness was reduced. The unit was brought back to its zero position, actuator pressures released and the test was restarted.

Figure 4.3: Central beam-column joint at the end of test.

At the end of the elastic cycles (up to 0.45% drift), fine flexural cracks were identified at the column faces and potential plastic hinge zones. There were six fine cracks less than 0.1mm wide on the tension side of the central column and two flexural cracks on each of the outside columns. As shown by Figure 4.3, a diagonal shear crack at about 45°, which was less than 0.1mm wide formed within the beam-column region of the central column during the second elastic positive half cycle. As shown in Figure 4.3, further diagonal cracking developed during the displacement: to +1.8% and -0.8% drift
(+2Di and -2Dii on Figure 4.3, where ‘i’ represents the first cycle at the target displacement, and ‘ii’ represents the second cycle). At no time did these diagonal cracks exceed 0.4mm in width. The three columns appeared to remain elastic throughout the test with only fine cracks measuring at a maximum of 0.5mm wide. At the peak displacement to +5.9% drift half cycle (intended +6Dii), six flexural cracks had formed on the tension sides of the central column.

Cracking in the beams was mainly confined in the plastic hinge zones, with only minor cracking occurring in the mid-span regions. Inclined diagonal shear cracking occurred near the column faces, which appear to initiate at the first and second stirrups in the beam. In the plastic hinge zone just to the right of the central column, these cracks were almost vertical and diverted to about 45° at the top and bottom of the beam. For the other plastic hinges, these cracks were approximately 70° to the horizontal through the middle third of the depth beam but were inclined at about 45° above and below this zone. The development of these cracks is shown in Figure 4.4(a) to (b) for the plastic hinge zone located to the left of the central column. The black lines indicate cracks that opened or grew in length as the unit was loaded in the positive direction (southwards). The red lines indicate cracks formed in the negative direction. Also shown in these figures are the greater progressions of the black crack lines compared against the red crack lines as larger rotations were sustained by the plastic hinges in the positive direction of loading.

As Figure 4.5(a) shows, a 2mm wide crack formed near the column face at +1.5% drift (intended +2Di). This figure also shows cracking of the beam at the column face. By the end of the displacement to +3.9% drift cycles (intended +4Dii), this crack developed further to show signs of a possible pull out of top reinforcement bars from the joint. The strength of test unit decreased to 80% of its maximum value during displacement to +5.9% drift (intended +6Dii) when the beam longitudinal reinforcement slipped through the central beam-column joint, as shown in Figure 4.5(b).
Figure 4.4: Progression of cracking in plastic hinge zone to the left of the central column (column ‘B’).
Figure 4.5: Cracking near column face of the central column.
4.4 Force versus Displacement Response

This section describes the force versus displacement response of the unit, as well as the response of each of the columns.

4.4.1 Overall unit response

Lateral forces of up to a maximum of 67.4kN were applied to the unit in the elastic cycles, which corresponded to 64% of the calculated theoretical lateral strength of 105.4kN (see section A1.5, Appendix 1). This was calculated based on the beam flexural strengths shown in Table 3.1 in Chapter 3. Figure 4.6 shows the force versus displacement response for the third elastic cycle in terms of the sum of the lateral load applied (load on main loading actuator at column ‘A’) against the lateral displacement at the top of column ‘A’.

The extrapolated ductility 1 displacement was extrapolated as 8.1mm in the positive direction and 7.7mm in the negative direction, which corresponds to interstorey drifts of 0.68% and 0.64% respectively. The actual yield displacement would have been less, as the stiffness of the unit was reduced after the unplanned displacement that was applied at the beginning of the test (see Section 4.2). There seemed to be approximately 1mm of movement at low lateral force level or on changeover of loading direction. This movement could be attributed to the slack in the pins at the base of the columns.

The lateral force versus displacement response for the test unit is shown in Figure 4.7 (data included in Appendix 3). Part (a) of this figure shows the total lateral force plotted against the absolute displacement at the top of column ‘A’ (ie. measured displacement between the top of column ‘A’ and a stationary reference point), while Figure 4.7(b) is the total lateral force versus the average of the relative lateral displacement between the top and bottom of the three columns. The maximum lateral force in both directions of loading for the ductility 2 cycles was 115kN, which corresponds to 1.09 times the theoretical strength.

The maximum lateral force was 133kN (1.26 times theoretical strength) at +5.2% drift in the positive direction and 132kN (1.24 times theoretical strength) at -1.7% drift in the...
negative direction of loading. Some pinching of the force versus displacement curve at the -1.6% drift half cycle (-4Dii) was noted. At this stage it was noticed that diagonal cracks had increased in width coupled with the spalling of cover concrete, particularly in the plastic hinge just to the right of the central column.

![Graph showing lateral load versus lateral displacement](image)

**Figure 4.6**: Elastic range total lateral force on frame against displacement at top of column ‘A’.

The unit failed to reach 80% of the maximum strength in the final displacement cycle (+5.9% drift). The unit failed by loss of bond of the longitudinal beam bars in the central beam-column joint. At this stage, most of the cover concrete in the plastic hinges had spalled, revealing the main reinforcement. The reinforcement did not buckle greatly even though large rotations had been induced.

The differences in the sum of the lateral force applied at the top of the columns and the sum of the lateral forces resisted at the column bases at different stages of the experiment were small. This is shown by Table 4.1. The difference between the sum of the top and bottom lateral forces were typically 1 to 5 percent. At lower loads (between 10 to 30kN), the difference between the forces were typically 5 to 15 percent.
Chapter 4: Test Results of Unit 1

Table 4.1: Difference in sum of force applied to top and bottom of columns at peaks of displacement cycles.

<table>
<thead>
<tr>
<th>displacement ductility, D</th>
<th>load at top (kN)</th>
<th>load at bottom (kN)</th>
<th>difference (kN)</th>
<th>difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>at 0.64 of theoretical strength</td>
<td>67.1</td>
<td>65.0</td>
<td>2.1</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>-65.1</td>
<td>-60.6</td>
<td>-4.5</td>
<td>7</td>
</tr>
<tr>
<td>2D</td>
<td>95.2</td>
<td>94.5</td>
<td>0.7</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>-108.3</td>
<td>-102.6</td>
<td>-5.7</td>
<td>5</td>
</tr>
<tr>
<td>4D</td>
<td>128.9</td>
<td>129.9</td>
<td>1.0</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>-124.4</td>
<td>-121.6</td>
<td>-2.8</td>
<td>2</td>
</tr>
<tr>
<td>6D</td>
<td>127.4</td>
<td>132.7</td>
<td>5.3</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td>-120.5</td>
<td>-121.6</td>
<td>-1.1</td>
<td>1</td>
</tr>
</tbody>
</table>

Note: Values are for 1st cycle of displacement step.

Figures 4.7: Total lateral force versus displacement at top of Column ‘A’. (continued)
Figures 4.7: Total lateral force versus displacement at top of Column 'A'.
(concluded)

4.4.2 Individual column response

The force versus displacement response for each column is presented in Figures 4.8. The forces plotted are those applied at the top of each corresponding column and the displacements are the relative lateral displacements measured between the top and bottom of each column. The irregularity of the plots is due to the nature of the test procedure, which involved very frequent adjustments to the loading equipment and also due to the relaxation of forces in the hydraulic actuators during the period when the displacement measurements were recorded.

It can be seen from Figure 4.8(a), (b) and (c) that in the cycle to +1.5% drift (+2Di), the beams, particularly around joints 'A' and 'C', responded elastically to reach the maximum load, followed by degradation in strength prior to reaching the peak displacement of that cycle.
Figure 4.8: Force versus displacement response of individual columns. (continued)
Figure 4.8: Force versus displacement response of individual columns. (concluded)

Figure 4.9: Peak lateral force versus displacement comparison of the columns.

The displacements for each column were similar to each other as adjustments were made at each step to keep them parallel. This is illustrated in Figure 4.9 (data included
in Appendix 3). Also noticeable in this chart is the smaller load in the forward direction sustained by column ‘C’ in comparison with column ‘A’. The reverse is true for the reverse direction of loading, but to a lesser extent. The arrangement of the actuators (joined at the top of the three columns, see Section 3.6, Chapter 3) may have contributed to axial forces inadvertently applied through the beams as adjustments were made to the hydraulic actuators during the experiment.

4.5 Components of Deformation

The method and equations set out in Section 3.7.4 were used to calculate the average lateral displacement at the top of the columns from the flexural and shear deformations measured on the beams and columns. Figure 4.10 compares the average lateral displacement measured directly at the top and bottom of the columns with the calculated displacements derived from the displacement measurements made on the test unit.

![Graph showing flexural and shear components of deformation in Unit 1.](image)

**Figure 4.10:** Flexural and shear components of deformation in Unit 1.

The results shown Figure 4.10 are the average values from the two beam bays and the three columns. The closure error shown could be attribute to a number of factors such
as slipping of bars in joints (especially at greater drifts), bending and twisting of rods between transducers (also see Figure 4.16), twisting of transducers and in accurate readings in transducers. Despite the closure error, it can be seen from Figure 4.10 that the calculated displacement closely matched the displacement from direct measurements of the displacements in the positive direction. In the negative direction, the match was good until the ductility 6 cycles. As expected, the contribution from shear deformation increased, as the magnitude of the peak displacements increased.

The plot of lateral displacement due to shear deformation is shown in Figure 4.11. This shows that the displacements due to shear in the positive direction was similar to that in the negative direction. Shear deformations became more prominent from the ductility 4 cycles and beyond, as shown in Figure 4.11.

The lateral displacement due to flexural deformation is shown in Figure 4.12. It can be seen from this figure that significant reduction in flexural stiffness of the frame occurs during the ductility 6 cycles.

![Figure 4.11: Lateral displacements from shear component of beams deformation against sum of lateral load.](image-url)
Figure 4.12: Lateral displacements from flexural component of beams deformation against sum of lateral load.

4.6 Elongation of Beams

Elongation along the centreline of the beams was measured by two means, firstly in a direct way by a displacement transducer spanning between the centres of the beam-column joints, and secondly by using the top and bottom line of instruments set up to measure beam deformations.

The total elongation being the sum of elongation for the two beams measured at peak positions is plotted against the displacement ductility in Figure 4.13 (data included in Appendix 3). As shown in the plot, the measurements by both methods were in close agreement until the second positive cycle of ductility 4 (+4Dii). The error between the two measurements could be due to the local buckling of the beam bars combined with the bending of the steel rod between the measurement points at the step between the beam and the column face. This is illustrated by Figure 4.14.
Figure 4.13: Total elongation of beams at peaks of ductility displacement.

Figure 4.14: Error in instrumentation due to local buckling of beam bar and bending of steel rod.

The maximum recorded elongation of the frame in the elastic cycle was 0.3mm, but this likely to be on the low side, as the test was restarted after two earlier cycles had already been conducted (see Section 4.3). There is reason to believe from previous research that greater elongation of the beam occurs in the elastic cycles [M2]. Significant elongation of the beams started in the first ductility 2 cycle, with a total elongation of 5mm. This
Chapter 4: Test Results of Unit 1

corresponded to an average elongation of 0.4% of the beam depth for each of the four plastic hinges. The largest increase in elongation was during the first ductility 4 half cycle (+4Di), where the total elongation increased to a value of 18mm. The maximum elongation measured was 32.9mm, recorded at the peak of the second positive ductility 6 half cycle (+6Dii), which corresponds to an average elongation of 2.7% of beam depth for each plastic hinge. The reduction of elongation in the final half cycle was due to the loss of bond of the top row of beam reinforcement in the central beam-column joint, which resulted in the loss of stiffness in that region.

Figure 4.15 traces the history of total elongation with the application of lateral loads to the unit. It can be seen that the majority of the total elongation in the beams was during the positive direction of loading. This was due to the greater displacements imposed on the unit in the forward direction.

Figure 4.15: History of total elongation in terms of total applied lateral load.

Figure 4.16 shows the total elongation of the unit plotted against the ductility displacement cycles and subdivided into the two components, namely the rotational component and the component due to the extension of the compression reinforcement, e (see Section 2.2.3 and Equation 2.3). The rotations in the elastic cycle resulted in the
small increase in length of the tension reinforcement. This increase was then negated by the reduction in length of the compression reinforcement at load reversal. In the first inelastic cycle, the tension reinforcement yielded, such that a permanent extension, $e$, was induced. In the first half cycle of ductility 4, it seemed that the extension had been compressed back to its original state. However, on load reversal, a large permanent extension had been induced, which accounted for 70% of the total elongation at that stage. This increased to 80% for the ductility 4 cycle, but was then reduced in the following ductility 6 cycle. The reduction in the extension of the bars was caused by the buckling of the compression reinforcement (top and bottom reinforcement, depending on direction of loading) during the ductility 6 cycles. By the end of the test, the permanent extension in the compression reinforcement accounted for more than 50% of the total elongation measured.

![Graph showing total elongation and rotational component](image)

**Figure 4.16:** Compression reinforcement extension and rotational components of elongation.
Chapter 5

Test Results of Unit 2

5.1 Introduction

This chapter presents the results from the experiment on the frame with floor slab unit, Unit 2. The results include general observations made during testing and information collected and processed to determine the deformations that were sustained.

5.2 Displacement History

As previously described in Section 3.8.2 of this report, the theoretical yield strength of Unit 2 could not be determined. Therefore, the unit could not be loaded to a predetermined load to determine the ductility 1 displacement. Instead, it was subjected to the displacement history shown in Figure 5.1 (drift displacement measured between pins at top and bottom of columns).

Initially, the unit underwent displacements up to ±0.1% interstorey drift for three complete cycles, followed by two cycles of displacements up to ±0.2% interstorey drift. At these stages, the equipment and testing procedure were fine tuned. After some reduction of data from the initial elastic phases, the unit was subjected to displacement cycles of ±0.5% interstorey drift for two complete cycles. As shown in Figure 5.1, further displacement cycles were set at steps of 0.5% interstorey drift, for two complete cycles at each step. One exception was for the displacement to ±3.5% interstorey drift, where the unit underwent three cycles at this level. This was done to provide more assurance that the unit was not going to fail before another displacement step was taken. The unit was then loaded for two further cycles up to ±4.0% then up to ±4.5% interstorey drift each, before testing ended.

Figure 5.2(a) shows the displacement history for each of the columns from 0.2 to 2.0% interstorey drift. As indicated by the figures, the columns were kept parallel to one
another. Throughout these displacement cycles, the largest out of parallel difference between the columns was 0.6mm at the peak of displacement cycle to ±1.5% interstorey drift.

The displacement history for displacement cycles from ±2.5% to ±4.5% interstorey drift is shown in Figure 5.2(b). From 2.5 to 3.5% interstorey drift levels, at no time in the loading phases did the out of parallel difference between the columns exceed 0.2mm. In the worst case at ±4.0% interstorey drift cycles was 0.4mm. The test unit was more difficult to control for the ±4.5% interstorey drift cycles. At some stages the out of parallel difference was up to 0.7mm.

There were some differences during the unloading phase of each cycle (from peak to zero displacement) as the pressure release in the pumps was very difficult to control. The pattern was much the same for the other displacement cycles.

Figure 5.1: History of displacement levels applied to Unit 2.
Chapter 5: Test Results of Unit 2

(a) From 0.2 to 2.0% interstorey drift

(b) From 2.5 to 4.5% interstorey drift

Figure 5.2: Lateral displacement history of columns.
5.3 General Behaviour and Observations During Test

Shrinkage cracks in the beams and floor were located and marked before the test. In the beams, cracks ran from the beam soffit to just above mid-depth. The shrinkage cracks formed in the floor topping around the mid-span region of both floor spans and were typically about 400mm in length. These ran across the slab, in directions perpendicular to the perimeter frame. A long crack formed along the support between the central transverse beam and the floor slab on the south side (which spans from column ‘B’ to the end support near column ‘C’). This crack ran along the length of the transverse beam to the end-slab. The shrinkage cracks were difficult to see with the naked eye, and were less than 0.1mm in width.

Displacement cycles to 0.2% interstorey drift

The test lasted for a period of thirty two days. The initial elastic cycles were performed over four days. The unit was displaced up to ± 1.3mm at the top relative to the bottom of the column for three cycles. This corresponded to an interstorey drift of 0.1%. At these stages, the sum of lateral force peaked at ± 53kN. The unit was then loaded up to a maximum of 83kN in either direction for two cycles. The displacement reached at this level was consistently around the ± 2.2mm mark (± 0.2% interstorey drift). Over the duration of elastic loading, some problems were encountered with the instrumentation and the loading equipment. This involved replacing of four portal transducers, three data cables and a load cell. The load arrangement was also altered at the bottom of column ‘C’, where the extension bar connected to the actuator was buckling. A steel bracket was added to shorten the length and eliminate the need for an extension bar. The final arrangement is shown in Figure 3.11 of Chapter 3. At this stage as only small displacements imposed on the unit and consequently it was not damaged. Any deformation was limited to minor cracking which closed on unloading. The data collected from the test was checked to ensure that it was consistent. In particular, physical measurements were made to check against the column displacements and the elongation readings.
Displacement cycles to 0.5% interstorey drift

The next step in testing was to displace the unit up to ±0.5% interstorey drift. Two displacement cycles to this level were applied. Minor flexural cracking was apparent in the potential plastic hinge zones and there was some diagonal cracking in the beam-column joint zones. In addition a limited amount of cracking occurred at the interface of the beam and floor around the columns and the beam potential plastic hinge zones. At this stage, cracks formed in the floor slab between the central transverse beam and the end supports of both floor spans (along the central transverse beam). At the peak displacement, this crack extended from the frame to the second rib unit of the south slab, while the crack formed up to the first unit in the north slab. In both cases the cracks were less than 0.2mm in width.

Displacement cycles to 1.0% interstorey drift

Crack patterns developed on the floor, which clearly showed that reinforcement in the floor slab was acting with the beam reinforcement. Diagonal cracks formed along the interface of the beam and floor were clear, showing shear transfer in this zone. This is shown in Figure 5.3 and in Figure 5.26(a), which shows a sketch of the cracks in the floor up to 1.0% interstorey drift. The red lines represent cracks formed at displacement in the positive direction (southward), while the black lines indicate cracks formed at negative displacement direction. It can be seen here that next to column ‘A’, diagonal cracks formed on the cantilever extension at the beam-floor interface to the left of the column. On reversal of loading direction, cracks were formed in the beam and diagonal cracking occurred at the beam-floor slab interface. Diagonal cracks were also found on the floor, starting from the end of the north slab (or left in figure) and cracking diagonally towards column ‘A’. These appear to indicate compression struts, or compression forces from the precast rib flowing into the beam-column region. The crack pattern at the other south end of the unit (column ‘C’) was similar to that described above.

Cracks also formed parallel to the precast ribs, as shown in Figure 5.26(a), particularly around the beam-column joint zones next to the first row of precast ribs. This indicates bending of the insitu slab connecting the perimeter frame to the first precast rib. This is illustrated by Figures 5.8 and described in greater detail by the associated text.
Chapter 5: Test Results of Unit 2

There was further extension in the length of the cracks formed along the central transverse beam (see Figure 5.26(a)). Cracks were less than 0.3mm in width at this stage. Diagonal cracking was found on the beams in positive bending (sagging moment) in the plastic hinge zones. In negative bending (hogging moment), cracks formed at a smaller angle to the vertical than those formed in the previous half cycle.

![Image of cracks](image)

**Figure 5.3:** Diagonal cracking on floor in vicinity of column ‘A’, at 1.0% drift.

**Displacement cycles to 1.5% interstorey drift**

At displacements up to ±1.5% interstorey drift, more floor cracks formed parallel to the perimeter frame (see Figure 5.26(b)). These were found in both spans, between the first and second row of rib units. Figure 5.4 shows a typical crack in the north span. This crack formed from the central transverse beam and propagated along next to the second rib unit (between the first and second rib units). The following are possible reasons for the formation of cracks parallel to the perimeter frame:

- The cracks formed parallel to the perimeter frame at the interface between the perimeter frame and the floor, and at the first precast rib are most likely due to vertical differential movement between the frame and the floor. This movement caused bending in the insitu slab linking the frame to the first rib (also see Figure 5.8).

- Elongation of the beams could cause the outer columns to move outwards, away from the floor slab (or perpendicular to the plane of the frame). This
Chapter 5: Test Results of Unit 2

movement is restrained by tension in the slab reinforcement, therefore causing cracks to form parallel to the frame (see Figure 5.5(a)). The test unit was restrained (perpendicular to the frame) at the top and bottom of the columns, and the movement at the centre of the column was not measured. However, in an actual building, this movement could be expected as the columns can only move outwards away from the floor to accommodate beam elongation and cracks in the floor.

- Beam elongation could cause the floor to act as a deep beam, as shown in Figure 5.5(b). Compression force due to this bending action is resisted partly by the floor slab and the outer transverse beam. The corresponding tension force is resisted partly by the floor slab and the central transverse beam. Due to this, shear is also resisted along the interface between the central transverse beam and the floor slab. These forces resulted in formation of cracks that branched from the central transverse beam and continued parallel to the perimeter frame (see Figure 5.4, denoted 'crack parallel to frame').

![Diagram showing crack patterns](image)

**Figure 5.4:** Longitudinal crack on floor alongside precast rlb unit, 1.5% drift.
Figure 5.5: Formation of cracks parallel to perimeter frame.

The crack along the central transverse beam and the floor opened to 2.1mm wide for the south span (see Figure 5.4), and 1.8mm for the north span at the end of the second cycle to 1.5% drift. There were also separation cracks at both ends of the floor (cantilevered ends). This is shown in Figures 5.6 and 5.26(b). These extended to mid-way between the first and second row of rib units. There was some limited spalling of cover concrete on the columns in compression at the beam face, above the floor. Cracks in the beam at column faces were about 3mm wide.
Displacement cycles to 2.0% interstorey drift

On the first cycle to 2.0% interstorey drift, damage was inflicted to the corner half-hinge joint at the north end. By the end of the second cycle, the Reid bar footplate in the half-hinge connection (see Figure 3.4 of Chapter 3) was exposed, as shown by Figure 5.6. This was due to the lifting up of the end of the transverse beam from the vertical movement at the end of the cantilever. Also visible in this figure is the separation crack between the floor slab and an external transverse beam.

There were further extensions in the lengths of the cracks parallel to the ribs. New cracks were found between the second and third ribs on both floor spans. The crack along the central transverse beam had extended to the end slab. There was some spalling of concrete at the beam and slab interface adjacent to column ‘C’ (see Figure 5.26(c)).

![Image of separation crack and footplate](image)

Figure 5.6: Damage to corner half-hinge joint at north end, at 2.0% drift.

Displacement cycles to 2.5% interstorey drift

On the first negative half cycle to 2.5% interstorey drift, spalling of concrete occurred at the beam and slab interface adjacent to column ‘A’. This was similar to the event adjacent to column ‘C’ in previous cycle to 2.0% drift, and the positive half cycle to 2.5%. A diagonal crack in the floor from the south end towards column ‘C’ widened to 3.5mm. On reversal of loading direction, a diagonal crack in the north end slab, which extended towards column ‘A’ widened to a similar extent (see Figure 5.26(d)).
Longitudinal cracks (parallel to frame) were formed between the third and fourth ribs on both spans.

At this stage the vertical differential movement between the frame and the floor slab was obvious when a large crack formed at the beam to slab interface adjacent to column ‘C’. This is shown in Figure 5.7(a). On the second negative half cycle to 2.5% drift, the same situation arose in the region close to column ‘A’, as shown in Figure 5.7(b). Figure 5.8(a) illustrates the vertical differential movement of the floor and the beam. As the frame is displaced laterally, the beams rotate. However, the slabs remained near straight because of the relatively stiff prestressed ribs. This relative displacement causes the floor to separate from the beam under positive rotation (sagging), giving rise to the crack at the floor and slab interface (see Figure 5.8(b)). In practise this movement could still be expected in an actual building as the crack patterns and widths indicate that the stiff end slab had little influence over this mode of local deformation, as the bending of the slab was limited to the slab between the frame and the first precast rib. However, the movement of the building in the direction perpendicular to the direction modelled in the test could influence the magnitude of this movement. Further studies would be required to investigate this effect.

![Figure 5.7: Vertical movement of floor from beam at 2.5% interstorey drift.](image)

(a) Next to column ‘C’

Figures 5.7: Vertical movement of floor from beam at 2.5% interstorey drift.
(continued)
Figure 5.7: Vertical movement of floor from beam at 2.5% interstorey drift.
(concluded)

Figure 5.8: Illustration of relative vertical movement between floor and beam.
(continued)
Chapter 5: Test Results of Unit 2

(b) Section view of floor slab and beam

Figures 5.8: Illustration of relative vertical movement between floor and beam. (concluded)

Displacement cycles to 3.0% interstorey drift

In the cycles up to ±3.0% interstorey drift, more damage occurred at the large crack due to vertical differential movement of the floor and the beam adjacent to columns ‘A’ and ‘C’. The first and second starter bars were exposed at these locations. In beam bay ‘B’-‘C’, the crack had extended past the mid-span, as shown in Figure 5.9. There was some diagonal cracking in the first row of precast ribs at the ends of both spans. However, these cracks were small due to the restraint provided by the prestressed ribs. A sketch of the cracks in the floor at this stage is shown in Figure 5.26(d).

Figure 5.9: Elevation view of perimeter frame of extension of crack past mid-span of bay ‘B’ - ‘C’, at 3.0% interstorey drift.
Displacement cycles to 3.5% interstorey drift

In the first positive cycle to 3.5%, the crack associated with the vertical differential movement of the slab and beam extended past the mid-span of beam bay ‘A’-‘B’. The vertical movements between the slab and beam were more evident on the south cantilever extension. On reversal of loading, damage inflicted on the north cantilever extension was similar to the south extension in the previous half cycle. This is shown in Figure 5.10.

As a result of the concrete spalling at the beam and floor slab interfaces, the longitudinal beam reinforcement closest to the floor slab was exposed, particularly in plastic hinges next to columns ‘A’ and ‘C’. Chunks of concrete from the floor around columns ‘A’ and ‘C’ had fallen off after the second cycle. The unit was subjected to a third cycle to 3.5% interstorey drift. However, no significant change was observed in this cycle.

![Image](image_url)

Figure 5.10: Shearing and uplift of floor at north cantilever, at 3.5% drift.

Displacement cycles to 4.5% interstorey drift

More damage at the interface of the beams and floor was sustained around columns ‘A’ and ‘C’ after displacements to 4.0% interstorey drift. Damage to the plastic hinges next
to columns 'A' and 'C' was more severe, with buckling of longitudinal reinforcement and wide flexural cracks forming near to the column faces. Figures 5.11(a) and (b) show where concrete had fallen off around column 'C' and the starter bars exposed. The test was ended after two further cycles to 4.5% drift.

(a) Cantilever extension adjacent to column 'C'

(b) Region around column 'C'

Figure 5.11: Damage around column 'C' at the end of test.
5.4 Force versus Displacement Response

The force versus displacement response in the initial stages is shown in Figure 5.12. The displacement in this figure is the average relative displacement between the top and bottom for the three columns. This is referred to as 'lateral displacement' in the remainder of this document. It can be seen that at this stage, there was very little degradation in stiffness. The two cycles to ±0.2% drift were virtually identical. The maximum lateral force applied at this stage was 83kN at a lateral displacement of 2.2mm, and -81.5kN in the reverse direction at a lateral displacement of -2.2mm.

![Graph showing force versus displacement response](image)

**Figure 5.12: Force versus displacement response in initial loading stages, up to 0.2% interstorey drift.**

The lateral force versus displacement response for the whole test is shown in Figure 5.13. Solid lines indicate the first cycle for each interstorey drift level and subsequent cycles are marked with dashed lines. Generally the unit was softer in the second cycle than in the first, which can be attributed to the flexural and shear cracking that occurred in the first cycle. There was more pinching in the load displacement relationship in the second cycle for each displacement step. The maximum lateral forces were reached at
the peak of displacement to 2.5% interstorey drift. The value in the positive direction was 313kN at 30.8mm and was -284kN at -30.8mm in the negative direction. The strength of the unit started to decline gradually in the subsequent cycles until it failed to reach 80% of the maximum value at the second cycle to 3.5% drift in the negative direction. It failed to reach 80% of the maximum value at the third cycle to 3.5% interstorey drift in the positive direction. However, the unit was taken to further displacement levels, as the lateral strength was still high compared to the test of the frame without the floor slab (Chapter 4). Testing was stopped after the second cycle to 4.5% interstorey drift, where the lateral strength reached at these levels was 166kN and -150kN in the positive and negative directions respectively.

![Diagram showing lateral force versus displacement response](image)

**Figure 5.13: Lateral force versus average displacement response of Unit 2** (see values in Appendix 3).

The theoretical strength (lateral force) shown in Figure 5.13 was calculated based on the recommendations in the New Zealand Concrete Structures Standard [S2]. The strength was calculated by including effectively anchored longitudinal reinforcement within an effective slab width for the beams in negative bending. For this unit, the effective width
Chapter 5: Test Results of Unit 2

of floor slab included in beam flexural strength calculation was 443mm (equivalent to one quarter of the span of the beam, from the beam centreline). A precast prestressed floor rib was within this width (see Figure 5.14). Two theoretical strengths (lateral force) were considered; the first by ignoring the contribution of the prestressed strands in the floor rib, and the second by including the strands.

The values from material tests (see section 3.4 of Chapter 3) were used in calculation the theoretical strength (see worked example section A1.6, Appendix 1). Within the effective width, there were six 3.125mm wires, and two 10mm bars. However, adjacent to the central column, column ‘B’, the two lengths of 10mm reinforcement above the prestressed rib cannot be considered to contribute to beam strength as these were not effectively anchored. These bars were terminated at the ends of the prestressed ribs (i.e. not connected at the transverse beam). The tension force at yield from the reinforcement within the effective slab width was 68kN. At joint ‘B’ the value was 18.8kN. The resultant flexural strength of the beam sections are shown in Table 5.1. By interpolating the bending moments to the column centrelines, the theoretical lateral strength of the unit was 188kN, neglecting the contribution of the prestressed rib within the effective slab width.

The tension force acting at mid-height of the slab is eccentric to the prestressed section. In calculating the strength of the prestressed rib, allowance must also be made for the bending moments due to gravity loads. Assuming that the compression strength of the rib concrete is 50MPa, and the prestressing strands (56mm² each) were initially stressed to 980MPa, an ultimate strength analysis indicates that each rib can resist an eccentric force of 69kN at the beam section adjacent to joint ‘A’ and an eccentric force 72kN adjacent to joint ‘C’. The eccentric force acting in the rib adjacent to joint ‘C’ is greater due to slightly larger gravity load bending moment than the corresponding moment acting at the rib adjacent to joint ‘A’. Adding to this the 68kN sustained by the passive reinforcement within the effective slab width gives a tension force capacity of 137kN at joint ‘A’ and 140kN at joint ‘C’. The flexural strength of the beam sections are shown in Table 5.1. From these, the theoretical lateral strength of 217kN was calculated for the unit.
Table 5.1: Theoretical flexural strength of beams excluding prestressed strands.

<table>
<thead>
<tr>
<th></th>
<th>Joint 'A'</th>
<th>Joint 'B'</th>
<th>Joint 'C'</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-ve bending</td>
<td>+ve bending</td>
<td>-ve bending</td>
</tr>
<tr>
<td>excluding prestressed strands</td>
<td>-43.5 27.0</td>
<td>-31.7 27.0</td>
<td>-43.5 27.0</td>
</tr>
<tr>
<td>including prestressed strands</td>
<td>-59.4 27.0</td>
<td>-31.7 27.0</td>
<td>-58.8 27.0</td>
</tr>
</tbody>
</table>

Figure 5.14: Effective width of slab contributing to beam flexural strength in negative bending.

The theoretical strengths calculated significantly underestimate the strength of the unit. In the negative direction of lateral displacement, the theoretical strength (including prestressed rib) was 76.4% of the measured peak force of 284kN. In the positive direction of displacement, the theoretical strength was 69.3% of the measured peak force of 313kN. These values clearly indicate that the theoretical strength calculation based on recommendations by the New Zealand Concrete Structures Standard may not be appropriate for this type of structure.

Allowing for two times the code recommended effective slab width and including two ribs for strength calculations, the lateral strength of the structure was 273kN. This value is closer in comparison with the measured peak forces. However extensive cracking should have occurred in the floor slab, across the top surface two prestressed ribs for the
tension force to develop. Such cracking was not observed (see Figures 5.26 (c) and (d)). Clearly the increase in strength of the unit had to come from other sources. Another source of strength enhancement is the deep beam action of the floor, resulting from the opening of crack along the central transverse beam due to beam elongation, as shown in Figure 5.5 (b). This is discussed in greater detail in Chapter 8.

Table 5.2: Difference in sum of force applied to top and bottom of columns at peaks of displacement cycles.

<table>
<thead>
<tr>
<th>Interstorey Drift (%)</th>
<th>Load at Top (kN)</th>
<th>Load at Bottom (kN)</th>
<th>Difference (kN)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1%</td>
<td>49.5</td>
<td>41.0</td>
<td>8.5</td>
<td>17</td>
</tr>
<tr>
<td></td>
<td>-52.8</td>
<td>-45.4</td>
<td>-7.5</td>
<td>14</td>
</tr>
<tr>
<td>0.2%</td>
<td>82.2</td>
<td>70.4</td>
<td>11.8</td>
<td>14</td>
</tr>
<tr>
<td></td>
<td>-81.2</td>
<td>-73.9</td>
<td>-7.3</td>
<td>9</td>
</tr>
<tr>
<td>0.5%</td>
<td>176.3</td>
<td>160.5</td>
<td>15.8</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td>-169.1</td>
<td>-161.8</td>
<td>-7.3</td>
<td>4</td>
</tr>
<tr>
<td>1.0%</td>
<td>262.9</td>
<td>247.3</td>
<td>15.5</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>-242.3</td>
<td>-232.8</td>
<td>-9.5</td>
<td>4</td>
</tr>
<tr>
<td>1.5%</td>
<td>285.1</td>
<td>277.2</td>
<td>7.9</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>-261.4</td>
<td>-248.1</td>
<td>-13.3</td>
<td>5</td>
</tr>
<tr>
<td>2.0%</td>
<td>304.4</td>
<td>297.2</td>
<td>7.2</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>-272.4</td>
<td>-259.9</td>
<td>-12.5</td>
<td>5</td>
</tr>
<tr>
<td>2.5%</td>
<td>313.2</td>
<td>307.4</td>
<td>5.8</td>
<td>2</td>
</tr>
<tr>
<td></td>
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<td>-268.4</td>
<td>-15.6</td>
<td>5</td>
</tr>
<tr>
<td>3.0%</td>
<td>303.0</td>
<td>298.7</td>
<td>4.2</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>-277.8</td>
<td>-262.5</td>
<td>-15.3</td>
<td>6</td>
</tr>
<tr>
<td>3.5%</td>
<td>276.4</td>
<td>273.5</td>
<td>2.9</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>-246.5</td>
<td>-230.1</td>
<td>-16.4</td>
<td>7</td>
</tr>
<tr>
<td>4.0%</td>
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<td>243.8</td>
<td>-21.5</td>
<td>-10</td>
</tr>
<tr>
<td></td>
<td>-213.3</td>
<td>-179.6</td>
<td>-33.8</td>
<td>16</td>
</tr>
<tr>
<td>4.5%</td>
<td>211.5</td>
<td>240.5</td>
<td>-29.0</td>
<td>-14</td>
</tr>
<tr>
<td></td>
<td>-182.6</td>
<td>-150.5</td>
<td>-32.1</td>
<td>18</td>
</tr>
</tbody>
</table>

Note:
Interstorey drift %: displacement imposed on test.
Values are for 1st cycle of displacement step.
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The sum of the lateral forces measured at the top of the columns did not equal the sum of forces measured at the bottom of the columns. Table 5.2 gives a summary of the forces at the peaks of the first lateral displacement cycle for each interstorey drift level. Up to 3.5% interstorey drift, the out of balance was between 2.9kN to 15.8kN for displacements in the positive direction, and was 7.3kN to 16.4kN for displacements in the negative direction. These differences were between 1 to 17% of the sum of the forces at the top of the columns in the positive direction up to 3.5% drift. In the negative direction, the corresponding values were 1 to 14%. This difference may be attributed to the friction force, which arose from the sliding of the floor unit on the supporting pedestals (as shown in Figure 3.3(a) in Chapter 3). The difference was high for the displacement cycles to 4.0 and 4.5% interstorey drift levels.

5.5 Moment Input to Beam-Column Joints

The moment applied to each beam-column joint was calculated from the sum of the lateral forces applied at the top and bottom of each column multiplied by the corresponding distance to the centre of the joint. Figure 5.15 shows a plot of the moment input for each of the beam-column joints at peaks of each cycle. Clearly for all stages, the moment applied to the central joint, joint ‘B’, was higher than the moment applied at both joints ‘A’ and ‘C’. In the positive displacement cycles (top axis), the moment applied in joint ‘C’ was larger than that applied to joint ‘A’. The reverse is true in the negative direction, though the margin was less. There were differences in the mechanism of slab contribution to the performance of the frame depending on the direction of loading. This was because two hinges in negative bending would form to one side of the frame centreline compared to one hinge in negative bending to the other side. This reverses on the change in loading direction. The strength increased when the slab was subjected to tension. Also noticeable for the plot was the drop off in moment input for the second cycle of each displacement step.

The theoretical bending strengths of joints at the column centrelines are shown on Figure 5.15. These were interpolated from the values shown in Table 5.1 (including prestressed rib). At the central joint, joint ‘B’, the value was 70kNm, while for the outside joints the greater value was 99kNm (98kNm for the lesser). The maximum
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recorded moment input to joint ‘B’ was 150kNm, more than twice the theoretical value. For the outer joints, the maximum value was 136kNm, which is 1.37 times the theoretical value. It is clear from the comparisons that the code recommended method used for the calculation of beam strength is inappropriate. While the theoretical strength for the outer joints is comparable, the difference between the theoretical strength and the actual strength is significantly large. Clearly there is another source of strength enhancement to joint ‘B’.

![Graph showing moment input to each beam-column joint at cycle peaks (values in Appendix 3).](image)

**Figure 5.15: Moment input to each beam-column joint at cycle peaks (values in Appendix 3).**

5.6 Components of Deformation

The method and equations set out in Section 3.7.4 were used to calculate the average lateral displacement at the top of the columns from the flexural and shear deformations measured on the beams, columns and beam-column joint zone. Figure 5.16 compares the average lateral displacement measured directly at the top and bottom of the columns with the calculated displacements derived from the measurements. The results shown in the figure are the average values from the three columns.
It can be seen from Figure 5.16 that the calculated displacement closely matched the displacement from direct measurements until the displacements exceeded 3.0% interstorey drift. As expected, the contribution from shear deformation increased as the displacements got larger. The components of deformation were not calculated for the displacements to 4.5% interstorey drift, as some of the portal transducers were no longer operating due to excessive out of plane deformation.

The load versus displacement plot of the lateral displacement due to flexural deformation, against the sum of lateral load is shown in Figure 5.17. The plot of lateral displacement due to shear deformation is included as Figure 5.18. From Figure 5.17, it can be seen that the lateral displacement from flexural deformation was greater in the positive direction than the negative from displacements to 2.5% drift and onwards. This was compensated by greater lateral displacement due to shear deformation in the negative direction than the positive, shown by Figure 5.18. The decrease in the strength of the unit at displacements of 3.0% drifts and greater was accompanied by greater reduction in flexural and shear stiffness with increasing cycles and lateral drift. Comparison of the two plots shows that the reduction in shear stiffness was greater than the reduction in flexural stiffness.

Figure 5.16: Flexural and shear components of deformation in Unit 2.
Figure 5.17: Lateral displacements from flexural deformations against the sum of lateral load.

Figure 5.18: Lateral displacements from shear deformations against the sum of lateral load.
Figure 5.19: Shear deformation in beam-column joints.

Figure 5.19 shows the shear deformation within the beam-column joints at peak drift displacements. As expected, the shear deformation in joint ‘B’ was greater than the outer joints. Shear deformation in joint ‘A’ was about 20 to 30% greater than joint ‘C’ at displacements of 1.0% drift and greater in the positive direction. In the negative direction deformation in joint ‘A’ was a maximum of 15% greater than joint ‘C’. This graph also shows that shear deformation in the beam-column joints increased at a greater rate up to 1.0% drift than at greater than 1.0% drift. This coincides with the pattern of strength increase shown in the lateral force against average lateral displacement plot shown by Figure 5.14.

5.7 Elongation of Beams

The recorded elongation over the whole length of the unit is shown in Figure 5.20 (see data included in Appendix 3). The values plotted were elongation readings taken at the peak of each lateral drift cycle. The solid line in Figure 5.20 is the elongation
measurement taken directly between the column centres, as described in Section 3.7.3 (in Chapter 3) together with the measurements taken from each of the outside columns to the end of the cantilever extensions. The dashed line represents the calculated elongation from the measurements taken from the portal transducers mounted on the beams, which were used to calculate the flexural deformation. In this case the elongation was calculated from the sum of the elongation measured in each segment, as indicated in Equation 3.3 (in Chapter 3). The closeness of the two lines in both plots is important, as it shows that the method used to measure the elongation directly was appropriate. This was important as the elongation from the direct measurement was used for controlling the test, where the columns were displaced parallel to one another in accordance with the elongation measured.

The maximum elongation measured at displacements up to 0.5% interstorey drift was 0.42mm. The initial elongation up to this stage was small and was mainly due to flexural cracking of the beams in the regions adjacent to the column faces. In both plots, it can be seen that the increase in the elongation of the beams was reasonably linear in two stages. Total elongation steadily increased from 0.5mm at displacement of 0.5% of interstorey drift up to a total of 16.5mm for the last cycle of 3.0% interstorey drift at an approximate rate of 1.4mm per cycle. Elongation increased at a much greater rate of approximately 6.2mm per cycle for the remaining displacement steps. This is coincidental with the decrease in the strength of the unit, and greater damage incurred at the beam-floor slab interface around columns 'A' and 'C' during the test. At these stages, the initial restraint to elongation of the beams from the prestressed flooring system was reduced, as the concrete around these areas fell off and the starter bars started to bend.

Figure 5.20 also plots the elongation in the beams in terms of the average elongation by percentage of the beam depth per plastic hinge. The average elongation per hinge for the entire frame was 0.92% of beam depth at 3.0% interstorey drift, and the maximum was 2.64% at the end of test (4.0% drift). However, these values do not represent actual level of elongation in the hinges. Figure 5.21 shows the average elongation for each plastic hinge within the two beam bays (four plastic hinges) and the average elongation per plastic hinge within the cantilever extensions (two plastic hinges).
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Figure 5.20: Total elongation of beams at peaks of lateral displacement steps (see values in Appendix 3).

Figure 5.21: Elongation in hinges within beam bays compared to elongation in hinges in cantilever extensions (see values in Appendix 3).

Figure 5.21 shows that there is a large difference in the elongation recorded between the hinges within the beam bays and those in the cantilever extensions. At the peak of 3.0% interstorey drift, the average elongation per plastic hinge was 1.25% per plastic hinge,
while in the cantilevers the corresponding value was 0.25%. At the end of the test, elongation was 3.28% of beam depth per plastic hinge within the beam bays, while in the cantilevers it was 1.35%. The difference in elongation of the inner hinges and those in the cantilever extensions were due to the rotation sustained by the hinges. Figure 5.22 shows the average change in rotation of the plastic hinges, between the positive and negative peaks of the drift cycles. The rotations were measured over a distance of approximately one beam depth. As shown by the figure, the changes in rotation sustained by the inner hinges were greater than the hinges in the cantilever extensions.

Figure 5.22: Change in rotation of plastic hinges between cycle peaks.

5.8 Slab Measurements

DEMEC points were placed on the slab as described in Section 3.7.5 (Chapter 3). Figures 5.23 shows the elongation calculated from the DEMEC measurements obtained from the first three rows of measurement points compared against the direct elongation measurement of the beams. Measurements were only taken up to 3.0% drift as a number of the DEMEC points near the beam and floor interface had fallen off. The first
row of points (row a) were 110mm away from the inside beam face where the direct elongation was measured. The second row of points (row b) was 450mm, while the third (row c) was 900mm away from the beam face. The locations of the rows are shown in Figure 3.26 of Chapter 3.

There is good agreement between the direct elongation measurements and those taken from DEMEC points. In both floor spans (though more prevalent in the north span), as shown in Figure 5.23(a), elongation decreases as the frame reversed in drift direction. As the frame was displaced in the positive direction, in the north span, two plastic hinges were subjected to negative (hogging) bending and one plastic hinge was in positive bending. Therefore the reinforcement at the top of the two hinges elongated in tension and shortened in compression at the other hinge. On reversal of direction, at the top of the beams, two hinges shortened and one elongated. The elongation measured on the floor slab shown in Figure 5.23(a) exhibited this behaviour as the floor was located in the same plane as the top of the beams. As expected, as shown in Figures 5.23, the measured elongation in the floor was less than the elongation in the beams as the distance away from the beams increased.

The distribution of elongation on the floor slabs is shown in Figures 5.24. By comparing Figure 5.24(a) and (b), it can be seen that as the frame was displaced in the positive direction, the elongation in the north span increased while elongation in the south span decreased. The reverse is true as the frame was displaced in the negative direction. The crack width at peak displacements between the floor spans and the central transverse beam is shown in Figures 5.25. From comparison of Figures 5.24 and 5.25, it can be seen that a large proportion of the total elongation in the floor was due to the crack at the central transverse beam. The crack widths at the central transverse beam and the elongation of the floor slabs (measured at 100mm from the face of the beam) are compared in Table 5.3. It can be seen in both slabs that the crack width at the central transverse beam make up 55 to 79% of the total slab elongation in the north slab when the unit was displaced in the positive direction. In the south slab, it was 64 to 72% when displaced in the negative direction. On reversal of direction, the cracks at the central transverse beam close up, making up 8 to 20% of the total elongation in the north slab, and 0 to 16% in the south slab.
Chapter 5: Test Results of Unit 2

Table 5.3: Crack width between central transverse beam and north slab compared to elongation in floor slab at 110mm away from beam face.

<table>
<thead>
<tr>
<th>drift displacement</th>
<th>elongation in floor slab, $E_s$ (mm)</th>
<th>crack width at central transverse beam, $C_c$ (mm)</th>
<th>$C_c / E_s$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>+1.0%</td>
<td>1.1</td>
<td>0.6</td>
<td>55%</td>
</tr>
<tr>
<td>-1.0%</td>
<td>1.2</td>
<td>0.1</td>
<td>8%</td>
</tr>
<tr>
<td>+2.0%</td>
<td>4.4</td>
<td>3.2</td>
<td>73%</td>
</tr>
<tr>
<td>-2.0%</td>
<td>3.1</td>
<td>0.6</td>
<td>19%</td>
</tr>
<tr>
<td>+3.0%</td>
<td>8.1</td>
<td>6.4</td>
<td>79%</td>
</tr>
<tr>
<td>-3.0%</td>
<td>5.6</td>
<td>1.1</td>
<td>20%</td>
</tr>
</tbody>
</table>

South Slab

<table>
<thead>
<tr>
<th>drift displacement</th>
<th>elongation in floor slab, $E_s$ (mm)</th>
<th>crack width at central transverse beam, $C_c$ (mm)</th>
<th>$C_c / E_s$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>+1.0%</td>
<td>0.8</td>
<td>0.1</td>
<td>13%</td>
</tr>
<tr>
<td>-1.0%</td>
<td>1.4</td>
<td>0.9</td>
<td>64%</td>
</tr>
<tr>
<td>+2.0%</td>
<td>3.3</td>
<td>0</td>
<td>0%</td>
</tr>
<tr>
<td>-2.0%</td>
<td>4.7</td>
<td>3.4</td>
<td>72%</td>
</tr>
<tr>
<td>+3.0%</td>
<td>5.6</td>
<td>0.9</td>
<td>16%</td>
</tr>
<tr>
<td>-3.0%</td>
<td>8.8</td>
<td>6.1</td>
<td>69%</td>
</tr>
</tbody>
</table>

By comparing Figures 5.24(a) and 5.25(a) for displacements in the positive direction, it can be seen for the south slab that the majority of the total elongation near the face of the beam (first row of DEMEC points) was in the floor, as the crack width at the transverse beam was small. This pattern is similar for the negative direction in the north floor slab. This is explained by the opening of the diagonal cracks near the outer columns (see Figures 5.26) as the cracks at the central transverse beam reduced in width.

Figures 5.25 also show that the crack width between the floor slab and the transverse beam was larger in the south slab than in the north slab, particularly from the third rib and beyond. The plots show that the crack in the north slab closed more in comparison to the south slab as direction of displacement was reversed. The crack in the south slab also propagated further into the slab than the crack in the north slab throughout the entire test. This is shown by the crack patterns on the floor slab in Figures 5.26.
Figure 5.23: Total elongation measured on floor spans compared against elongation from direct measurements at peak displacements.
Figure 5.24: Distribution of floor slab elongation at peak displacements.
Figure 5.25: Crack width between floor and central transverse beam measured at peak displacements.
(a) Displacements up to 1.0% interstorey drift

(b) Displacements up to 1.5% interstorey drift

Figure 5.26: Crack pattern on floor at different displacement levels. (continued)
Chapter 5: Test Results of Unit 2

(c) Displacements up to 2.0% interstorey drift

(d) Displacements up to 3.0% interstorey drift

Figure 5.26: Crack pattern on floor at different displacement levels.  (concluded)
Chapter 6: Test Results of Unit 3

Chapter 6

Test Results of Unit 3

6.1 Introduction

This chapter presents the results from the experiment on the second of the plane frame units, Unit 3, together with general observations made during the experiment.

6.2 Displacement History

This unit was subjected to a similar displacement history that was applied to the frame-floor slab unit, Unit 2. This displacement history is shown by Figure 6.1. The percentage interstorey drift is the displacement applied, measured between the pin at the top and the bottom of the columns. Three displacement cycles up to ± 0.1% interstorey drift were applied to the frame, followed by two cycles of displacements up to ± 0.2% interstorey drift. The data collected from these test cycles were checked to ensure correct operation of the instrumentation. Then it underwent two cycles to 65% of the theoretical lateral strength calculated (flexural strength of beams calculated assuming rectangular stress block in beam compression) for the test unit based on material tests. As shown in Figure 6.1, further displacement cycles were set at steps of 0.5% interstorey drift, with two complete cycles at each step. The unit was displaced for two cycles at ± 3.5% interstorey drift before testing was concluded.

Figure 6.2 shows the displacement history for each of the columns from displacement at 65% of the theoretical lateral strength to 3.5% interstorey drift. As can be seen in the plot, the columns were kept parallel to one another up to 2.5% interstorey drift. Over these displacement cycles, the largest out of parallel difference between the columns was 0.7mm during a displacement cycle to ± 2.5% interstorey drift. On the following cycles, it became increasingly difficult to keep the columns parallel. This was due to some twisting at the base of column ‘A’ (discussed later). During the cycles to ± 3.0%
interstorey drift the largest out of parallel displacement was 2.4mm while during the cycles to ±3.5% interstorey drift, the largest difference was 7.1mm.

Figure 6.1: History of displacement levels applied to Unit 3.

Figure 6.2: Lateral displacement history of columns from 65% of $F_y$ to 3.5% interstorey drift.

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6.3 General Behaviour and Observations During Test

Shrinkage cracks in the frame were located and marked before the test. The shrinkage cracks were difficult to see with the unaided eye as they were less than 0.1mm in width. It took two weeks to complete the test. The initial elastic cycles were performed in four days.

Displacement cycles from 0.1% interstorey drift to displacement at 65% of frame theoretical strength

The unit was displaced to ±0.1% interstorey drift for three cycles. This corresponded to a displacement of ±1.2mm at the top of the columns relative to the bottom (hence referred to as relative displacement in this document). Over these cycles, the maximum lateral load was 24.6kN. The unit was then displaced up to ±0.2% interstorey drift. In these load cycles fine flexural cracks had formed in the beams. However, cracks were not present in the beam-column joint zones.

![Joint cracks in joint 'B' at 65% of frame theoretical lateral strength.](image)

Figure 6.3: Joint cracks in joint ‘B’ at 65% of frame theoretical lateral strength.
Chapter 6: Test Results of Unit 3

The unit was then loaded up to around of 67kN in both directions for two cycles. This corresponded to 65% of the theoretical lateral strength of the test unit. The average displacement was 0.35% interstorey drift. During these load cycles diagonal cracks had formed in beam-column joint ‘B’ (central joint, also refer to Figure 3.11 in Chapter 3) as shown in Figure 6.3. The red lines represent cracks formed at displacement in the positive direction (southward), while the black lines indicate cracks formed at negative displacement direction.

Data collected from the experiment was downloaded for checks on readings from the instrumentation. Tape measurements and theodolite readings taken between a reference point and the top of the columns were used to check against the electronic readings of the column displacements. Tape measurements were also taken between the centres of the joints to check against elongation readings from the displacement transducers.

Displacement cycles up to 1.5% interstorey drift

The next step in testing was to displace the unit to ±0.5% interstorey drift. Two displacement cycles to this level was applied. More flexural cracking was apparent in the beams and columns, but these cracks did not exceed 0.15mm in width.

Figure 6.4: Beam-column joint ‘A’, at 1.5% interstorey drift.
The unit was then subjected to displacements up to ±1.0% interstorey drift for two cycles. Cracking was mainly confined to the plastic hinge zones and beam-column joints. Most of the cracking was concentrated in the beam at the column face. In the beam plastic hinge zones next to joints ‘A’ and ‘C’ diagonal cracks formed at approximately 60° to the horizontal. Diagonal cracks also formed in the joint zones of all the columns. As expected, at displacements up to ±1.5% interstorey drift, the cracks formed in the previous cycles had extended and widened. A picture of the beam-column joint of joint ‘A’ is shown by Figure 6.4.

Displacement cycles up to 3.5% interstorey drift

The unit was then displaced to ±2.0% interstorey drift for two cycles. Diagonal cracks formed in the beam plastic hinge zones next to joint ‘B’. During this stage, some abnormal movement was detected at the base of column ‘B’. This was thought to be caused by the slack in the pin connections at the base. The test unit was then displaced up to ±2.5% interstorey drift. On closer inspection during the first cycle of this displacement step, the loading frame at the base of column ‘B’ was actually sliding by around 8mm both ways. This was thought to be caused by the shrinkage of the mortar bed at the base of the loading frame as the frame was put in place some months before testing began. The test was stopped and loads released after the first cycle to 2.5% interstorey drift. The mortar was replaced and the frame was restressed down to the strong floor. Testing recommenced on the following day.

On displacement up to ±3.0% interstorey drift, diagonal cracks in the beams had increased in width and damage had also spread along the plastic hinge zone by about one beam depth. Significant spalling of cover concrete had occurred resulting in the exposing of beam reinforcement in the beams adjacent to the outside columns. At the central columns, spalling of concrete on the column faces next to beams indicate some pulling out of the beam reinforcement in the joint zone. The unit was then displaced to ±3.5% interstorey drift for two cycles before testing was concluded. The plastic hinges in the beams at the end of the test is shown by Figure 6.5.
Figure 6.5: Test unit at end of test.
In these latter stages, it became increasingly difficult to control the test by adjusting the hydraulic pumps such that the columns would remain parallel. It was discovered during displacement step to ±3.5% interstorey drift that column 'A' had rotated out of the plane of the frame. This rotation was caused by the rotation of the swivelling pin which connected the load cell to the base of the column. This meant that the elongation measurement along the beam of bay 'A'-‘B’ had been less than the actual elongation (see Figure 6.6). This measurement was critical for accurate control of the test as shown by Equations 3.18 – 3.20 in Chapter 3.

Figure 6.6: Rotation of column ‘A’.

This problem was highlighted during the negative displacement cycles (in northwards direction). Due to the under-measured elongation of the beam between column ‘A’ and column ‘B’, the base of column ‘A’ was displaced by a lesser amount than what should have been had the column remained plane (see Equation 3.19), and in order to maintain parallel columns (see Equation 3.1), the top of column ‘A’ had been subjected to lesser displacements that what would have been likely. Therefore very little load was recorded at the top of column ‘A’ while higher loads were measured at the base of the column. Due to the distribution of forces in the frames, a higher force was applied at the top of column ‘B’ and lower force at base of column ‘B’. This meant that the beam in bay ‘A’-‘B’ sustained an appreciable level of axial force. This problem with the rotating pin was not detected in columns ‘B’ and ‘C’.
6.4 Force versus Displacement Response

The force versus displacement response in the elastic stages is shown in Figure 6.7. The maximum lateral force applied at this stage was 67.8kN at a lateral displacement of 4.7mm, and 68.8kN in the reverse direction at a lateral displacement of 4.3mm. By interpolation, the average yield displacement is 6.7mm, which corresponds to an interstorey drift of 0.55%.

![Graph showing force versus displacement](image)

Figure 6.7: Force versus displacement response in the elastic range, up to 65% of nominal lateral strength of frame.

The lateral force versus displacement response for the whole test is shown in Figure 6.8. Solid lines indicate the first cycle for each interstorey drift level and subsequent cycles are marked with dashed lines. The maximum lateral force in the positive direction was reached at the peak of displacement to 1.0% interstorey drift. The maximum value in the positive direction was 114.2kN. In the other direction, the maximum lateral strength of 125.6kN was reached at 2.5% interstorey drift. The figure also shows the theoretical lateral strength of 103.6kN, calculated based on material tests and assuming a rectangular compression stress block as defined in the New Zealand Concrete Standard
Chapter 6: Test Results of Unit 3

[S1]. The maximum value in the positive direction was 10.2% greater than the theoretical value, while it was 21.2% more in the negative direction.

The test unit failed to reach 80% of the maximum lateral strength in the negative direction following the second cycle to 3.0% interstorey drift. As expected, due to strength degradation and opening of cracks, the lateral strength recorded in the second displacement cycles at each displacement step was less than the value recorded in the first cycles. Pinching of the curves was more evident following the displacement cycles to ±2.0% interstorey drift as shear deformations increased as displacements got larger.

Figure 6.8: Lateral force versus displacement response of test unit.

The differences in the sum of the lateral force applied at the top of the columns and the sum of the lateral forces resisted at the column bases at different stages of the experiment were small. This is shown by Table 6.1. The difference between the sum of the top and bottom lateral forces were typically no more than 5 percent at the peak of the displacement cycles.
6.5 Moment Input to Beam-Column Joints

The moment applied to each beam-column joint was calculated from the sum of the lateral forces applied at the top and bottom of each column multiplied by the corresponding distance to the centre of the joint. Figure 6.9 shows a plot of the moment input for each of the beam-column joint at peaks of each cycle. Clearly for all stages, the moment applied to the central joint, joint ‘B’, was higher than the moment applied at either joints ‘A’ and ‘C’. The moment input to joints ‘A’ and ‘C’ were in reasonable agreement. Further from the displacement steps to 2.5% interstorey drift, it can be seen that most of the loss or decrease in the strength of the unit was associated with the decrease in the strength of joint ‘B’.
Figure 6.9: Moment input to each beam-column joint at cycle peaks.

The theoretical joint strength calculated from the flexural strength of the beam sections are shown on Figure 6.9. For the outer joints, the theoretical strength was 32kNm, and was 64kNm for Joint ‘B’. The experimental peak value for Joint ‘B’ in the positive direction was 2% greater, and was 15% greater in the negative direction. For the outer joints, the experimental peak values were 33% greater than the theoretical value in both directions.

The comparisons also show that while the outer hinge strengths were maintained in the larger drift cycles, but the central joint strength decreased markedly in these cycles (from 2.5% drift onwards).

6.6 Components of Deformation

The method and equations set out in Section 3.7.4 were used to calculate the average lateral displacement at the top of the columns from the flexural and shear deformations measured on the beams and columns and joint zones. Figure 6.10 compares the average
lateral displacement measured directly at the top and bottom of the columns with the calculated displacements derived from the measurements taken. The results shown in the figure are the average values from the two beams bays and the three columns. The figure shows that the calculated displacements are in reasonable agreement with the displacement derived from direct measurements, up to displacement cycles to ±2.0% interstorey drift. However further from 2.0% interstorey drift, the difference became increasingly large. This may be due to errors in the transducers and also combined with the twisting of column ‘A’ (see Figure 6.6) where the deformation recorded at the column face, on the east face of the frame (location of the transducers) would have been greater than the centre-line displacements.

The displacements from direct measurements and the displacements derived from beam and column deformations are also compared by Figure 6.11 with the exception that only columns ‘B’ and ‘C’, and the beam between columns ‘B’ and ‘C’ were considered. Similar to Figure 6.10, the derived measurements tended to overestimate the lateral displacements of the frame obtained from direct measurements. However, in this case the curves are in reasonable agreement up to ±2.5% interstorey drift, and the errors in both directions are more uniformly spread compared to Figure 6.10.

As expected, the contribution from shear deformation increased as the lateral displacements got larger. Figure 6.12 shows the lateral displacement due to flexural deformation versus the sum of lateral load and the graph of lateral displacement due to shear deformation shown in Figure 6.13. From Figure 6.12, it can be seen that the lateral displacement from flexural deformation was greater in the positive direction than the negative from displacements to ±2.5% drift and onwards. Conversely, the shear deformations were greater in the negative direction than the positive direction. Lateral displacements in the second cycles due to flexural deformation were smaller than that in the first cycle of the same displacement step for the later cycles. This observation can be linked to the increased lateral displacement due to shear deformation in the second cycle of each displacement step. This also explained the pinching in the lateral force versus displacement plot for the second cycles, shown earlier in Figure 6.8.
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Figure 6.10: Flexural and shear components of deformation in Unit 3.

Figure 6.11: Flexural and shear components of deformation of only bay ‘B’-’C’ of Unit 3.
Figure 6.12: Lateral displacements from flexural deformations against the sum of lateral load.

Figure 6.13: Lateral displacements from shear deformations against the sum of lateral load.
Figure 6.14 shows the shear deformation within the central beam-column joints at peak drift levels in the positive direction of lateral displacement. As expected, the shear deformation in joint ‘B’ was greater than the outer joints. This figure shows that shear deformation in joint ‘B’ increased at a greater rate up to 1.0% drift then slows from 1.0 to 2.0% drift. It then held an approximately constant level from 2.0% onwards.

Figure 6.14: Shear deformation in beam-column joints.

6.7 Elongation of Beams

The total elongation of the unit is shown in Figure 6.15 in terms of elongation at the peak of interstorey drift. The elongation of the beam bay ‘B’-‘C’ is shown in Figure 6.16. Two lines were plotted for each of the figures. The solid line was the measurement taken directly between the column centres for both cases, as described in Section 3.7.3 The dashed line represents the calculated elongation from the measurements taken from the portal transducers mounted on the beams, which was used to measure the flexural deformation. The elongation was calculated from the sum of the elongation measured in each segment, as indicated by Equation 3.3.
Figure 6.15: Elongation of beams at peak of displacement steps.

Figure 6.16: Elongation of beam bay ‘B’ – ‘C’ at peak of displacement steps.
Figure 6.17: Axial load levels in beams.

From Figures 6.15 and 6.16, it can be seen that the elongation of the beams started to increase in a linear fashion from the onset of displacement to ± 1.0% interstorey drift. At 3.0% interstorey drift, which corresponds to about a displacement ductility of 5.5, the average elongation per plastic hinge was 2.4% of beam depth, while for the plastic...
hinges between columns ‘B’ and ‘C’ the corresponding value was 3.6% of beam depth. The maximum total elongation measured for the unit was 31.7mm or 2.6% of beam depth per plastic hinge (see Figure 6.15). This was recorded at 3.5% interstorey drift. In comparison, the maximum elongation recorded in beam bay ‘B’-‘C’ was 24.7mm or 4.1% of beam depth (see Figure 6.16). The elongation of the beam between columns ‘A’ and ‘B’ was restricted by the axial compression force applied due to the errors in testing caused by the rotation of column ‘A’. This is shown by Figure 6.17 which plots levels of axial loads sustained by each beam. These values were calculated from the out of balance forces applied at the top and bottom of each column.

The beam between columns ‘B’ and ‘C’ was subjected to axial tension forces, which would explain the higher elongation measurements. However, the tension forces were low compared to the axial compression forces sustained by the beam between columns ‘A’ and ‘B’, particularly further from ±2.0% interstorey drift.