BEHAVIOUR OF CAST IN SITU REINFORCED CONCRETE FRAMES INCORPORATING PRECAST PRESTRESSED CONCRETE BEAM SHELLS SUBJECTED TO SEISMIC LOADING

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SYNOPSIS

The performance of cast in situ reinforced concrete frames incorporating precast prestressed concrete U-beam shells, subjected to seismic loading, is investigated. The precast U-beams act as permanent formwork and are not connected by steel to the cast in situ concrete of the beam or column. A review of the design provisions of the New Zealand concrete design code NZS 3101 relevant to the design of such composite structures is made and supplementary design recommendations are proposed where necessary.

Three full scale reinforced concrete beam-exterior column subassemblies with precast prestressed concrete U-beam shells were constructed and tested to determine their seismic performance characteristics. Two of the subassemblies were designed for seismic loading with potential plastic hinge regions in the beams. One of these subassemblies had the bond between the precast and the cast in situ concrete in the beam deliberately broken in the potential plastic hinge region, while the other was bonded. The third subassembly was not designed for seismic loading. The test results for the two subassemblies designed for seismic loading demonstrated that the seismic provisions of the New Zealand concrete design code, in conjunction with the supplementary design recommendations, resulted in adequately ductile behaviour with satisfactory energy dissipating characteristics. It was observed that the U-beam was less damaged during seismic loading when the bond between the precast and the cast in situ concrete in the potential plastic hinge region was deliberately broken. The performance of the other composite beam-column subassembly, which was not designed for seismic loading, was unsatisfactory, since the energy dissipating characteristics were poor and excessive sliding shear displacements occurred in the plastic hinge region.

KEY WORDS:

Composite construction, ductility, earthquake resistance, frames, precast concrete beam shells, reinforced concrete.

LIST OF SYMBOLS

A_g	= gross area of column section	f'_c	= compressive cylinder strength of concrete
A_s	= area of top longitudinal reinforcement	f_{pu}	= ultimate tensile strength of prestressing strand
A'_s	= area of bottom longitudinal reinforcement	f_{su}	= ultimate tensile strength of reinforcing steel
A_v	= area of shear reinforcement at spacing s	f_y	= yield strength of reinforcing steel
b	= width of compression face of beam	M_i	= ideal flexural strength of section
b_v	= width of interface of section being investigated for horizontal shear	M_u	= design bending moment from factored loading
b_w	= width of beam web	P_i	= beam end load to cause positive moment ideal flexural strength of cast in situ concrete core alone to be reached
d	= distance from extreme compression fibre of concrete to centroid of tension reinforcement	$-P_i$	= beam end load to cause negative moment ideal flexural strength of cast in situ concrete core alone to be reached
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- $-P'_i$ = beam end load to cause negative moment ideal flexural strength of composite section to be reached
- s = centre to centre spacing of transverse reinforcement
- v_{dh} = total imposed horizontal shear stress
- v_i = total ideal shear stress resisted by concrete and shear reinforcement
- V_{max} = maximum applied shear force during the test
- V_u = design shear force from factored loading
- V_u^O = design shear force from overstrength beam moments and factored gravity loading
- Δ = beam end deflection
- Δ_y = beam end deflection at first yield
- $\rho = A_s/bd$
- $\rho' = A'_s/bd$
- ϕ = strength reduction factor
- $\mu = \Delta/\Delta_y$ = displacement ductility factor

1. INTRODUCTION

Composite systems of concrete buildings, combining precast and cast in situ reinforced concrete, have a number of advantages in construction. The incorporation of precast concrete elements has the advantages of high quality control and speed of construction, and the cast in situ reinforced concrete provides the structural continuity and the ductility necessary for adequate seismic performance. Further investigation and application of new methods of detailing composite structural concrete building systems has been advocated in New Zealand in the past, for example by Forest (1).

A building system which has become popular in New Zealand involves the use of precast concrete beam shells as permanent formwork for beams. The precast shells are typically pretensioned prestressed concrete U-beams and are left permanently in position after the in situ reinforced concrete core has been cast. The precast U-beams support the self weight and construction loads and act compositely with the reinforced concrete core when subjected to other loading in the finished structure. The precast U-beams are not connected by steel to the cast in situ concrete of the beam or column.

The typical structural organisation of a building floor and frame system incorporating the precast pretensioned U-beam units is shown in Fig. 1a and b. Current construction practice is to support the U-beam units on the cover concrete of the previously cast reinforced concrete column below, with a seating of 40 to 50 mm, and to place the proprietary precast concrete floor system between the U-beams of adjacent frames. Some propping

may be provided under the ends of the U-beam units as a back-up measure in case the U-beam seating on the column should prove inadequate to carry the construction load. Once the floor system is in place, the reinforcement may be placed, and the in situ concrete cast inside the U-beam units, the topping slab and the columns of the next storey. Precast concrete columns have sometimes been used rather than cast in situ concrete columns.

The precast concrete U-beam illustrated in Fig. 1 has webs tapered from the bottom to the top, to ensure ease of removing internal formwork. The inside surface is intentionally roughened, by the use of a chemical retarder and the removal of the surface cement paste, to facilitate the development of interface bond between the precast U-beam concrete and the cast in situ concrete core. The U-beams are pretensioned with seven wire strands and are designed to carry at least all of the self-weight and imposed loads during construction.

To date precast concrete U-beams have been principally used in the construction of low rise buildings in which the lateral seismic loads are resisted primarily by other elements such as totally cast in situ reinforced concrete structural walls or frames. An early example of this type of construction in New Zealand is the Karioi Pulp Mill (2).

In the case of one or two storey buildings an alternative design approach permitted by the New Zealand code for general structural design and loadings NZS 4203 (3) is to resist seismic loads by a column hinge sidesway mechanism. This approach would protect the composite beams from damage during seismic attack.

Recent trends have seen this form of composite beam construction used in multistorey moment resisting reinforced concrete framed structures. In this application, the composite beams will be required to act as the primary energy dissipating members during seismic loading. Doubts have been expressed by some designers and checking authorities concerning the ability of this form of composite construction to be able to fulfil that demand.

This paper summarises an investigation into the seismic performance characteristics of beam-column sub-assemblies in which the beams are formed from precast prestressed U-beam units with cast in situ reinforced concrete cores, and in which plastic hinge formation occurs in the beams during seismic loading. The investigation involved consideration of the influence of the precast U-beam on the behaviour of the composite beams, a review of appropriate seismic design provisions in the New Zealand concrete design code, and tests conducted on three full scale composite beam-exterior column sub-assemblies. The results of this

(a) Section of composite beam in finished structure (reinforcement not shown)

(b) Construction details of structural system (not all reinforcement shown)

Fig. 1 The Structural System

investigation may be seen in more detail elsewhere (4).

2. BEHAVIOUR OF COMPOSITE BEAMS IN FRAMES

2.1 Flexural Strength

The critical section for flexure in beams in moment resisting frames subjected to gravity and seismic loading is generally at the beam ends. In frames where gravity loading effects dominate, the critical section for positive moment due to gravity plus seismic loading may occur in the beams away from the column faces. A distinctive feature of the behaviour of the composite beam-column connection shown in Fig. 1b is that the prestressing strands of the precast concrete U-beam terminate at the end of the U-beam and hence are not anchored in the beam-column joint core. Therefore the positive moment flexural strength at the end of the beam will be provided only by the longitudinal reinforcement and the cast in situ concrete in the beam core and slab topping (see Fig. 2a). Away from the beam ends there will be some contribution from the precast prestressed U-beam to the positive moment flexural strength, but a full contribution from the prestressing strands (and hence full composite action of the section) can only occur at a distance greater than approximately 150 strand diameters from the beam end, which is the order of length required to develop the tensile strength of the strand (5).

The negative moment flexural strength at the end of the beam will be aided by the presence of the U-beam since the bottom flange of the U-beam will bear in compression against the cast in situ column concrete (see Fig. 2b). Hence the upper limit of the negative moment flexural strength at the ends of the beam will be that of the composite section. However should the beam end bearing on the column concrete and/or the interface bond between the cast in situ and precast beam concrete break down during seismic loading, the available negative moment flexural strength will reduce to less than the composite section value. The lower limit of negative moment flexural strength at the beam ends is that provided by the cast in situ reinforced concrete core alone. The negative moment flexural strength away from the ends will be that due to the composite section.

It is evident that the extent of the break down of end bearing and interface bond between the cast in situ and precast concrete, and the possible deterioration of the development of the force in the prestressing strands in the end regions of the precast beams, will mean that the dependable positive and negative moment flexural strength at the ends of the composite beams should be taken as that provided only by the cast in situ reinforced concrete beam core.

2.2 Plastic Hinge Region

The length of the plastic hinge region in beams is of interest in seismic design since the plastic hinge length has a significant effect on the level of displacement ductility factor which can be achieved by frames. Longer plastic hinge lengths lead to greater available displacement ductility factors for a given ultimate section curvature (6). In a conventional reinforced concrete frame the length of the beam region over which the tensile reinforcement yields is typically about equal to the beam depth and several flexural cracks will form in that region.

Opinions have been expressed in New Zealand that for the type of composite beam-column assembly considered in this study, in which there is no connection by steel between the end of the precast U-beam and the column, the length of the region of reinforcement yielding at the end of the composite beam when the bending moment is positive will be less than for a beam in a conventional reinforced concrete frame. This is because when positive moment is applied, the first crack to form will be at the contact surface between the end of the precast U-beam and the face of the column. It is possible that positive moment plastic rotations will concentrate at this one cracked section, since significant cracking may not occur in the flexurally stronger adjacent composite sections during subsequent loading. A consequence of a shortened plastic hinge length for positive moment would be higher beam curvatures in the plastic hinge region than for conventional reinforced concrete members. Also, if flexural cracking in the beam during positive moment does concentrate at the column face the longitudinal reinforcement in the beam there would suffer high localised plastic tensile straining which would perhaps lead to bar fracture when significant plastic hinge rotation occurs. Further, the extensive widening of that crack at large plastic hinge rotations may mean that the shear resistance mechanism due to aggregate interlock along the (vertical) crack will break down, leading to sliding shear displacements along that weakened vertical plane. These opinions concerning the plastic hinge behaviour during positive moment have resulted in reservations being expressed by designers about the performance of the type of composite beams considered in this study when required to act as primary energy dissipating members during seismic loading. Nevertheless, it should be noted that it has not been demonstrated by tests that the plastic hinging during positive moment is as concentrated as implied in the above comments.

The possible shortening of the length of the region of reinforcement yielding only applies when the beam moment is positive. When the beam moment is negative the behaviour should be more like

(a) Positive bending moment applied to beam

(b) Negative bending moment applied to beam

Fig. 2 Internal Forces Acting on a Composite Beam-Exterior Column Joint Core During Positive and Negative Beam Moments

a conventional reinforced concrete beam, since the top of the cast in situ concrete core does not have the precast U-beam surrounding it and the plastic hinge should be able to spread along the beam.

One possible approach, aimed at improving the plastic hinge behaviour during positive moment, would be to construct a composite beam in such a manner that in the potential plastic hinge region at the ends of the beam the bond at the interface between the precast U-beam and the cast in situ concrete core is intentionally eliminated. The effect of such a detail would be to allow the plastic hinge region to spread along the cast in situ concrete beam core without hindrance from the U-beam, and so avoid the possible concentration of the beam plastic hinge rotation in the region close to the end of the beam.

2.3 Shear Strength

In the plastic hinge regions at the ends of composite beams the cast in situ reinforced concrete core will need to resist all the applied shear force alone if the bond at the interface between the precast U-beam and the cast in situ concrete breaks down, or if the bond is intentionally eliminated. Away from the ends of the beam the whole composite section may be considered to provide shear resistance.

2.4 Interface Shear Transfer Between the Precast U-Beam and the Cast in Situ Concrete Core

Composite action of the beam can only occur if shear can be transferred across the interface between the adjoining precast and cast in situ concrete surfaces with practically no slip. Shear stress is transferred across the interface of concrete surfaces by concrete adhesion, interlock of mated roughened contact surfaces, and friction. Friction is reliant on a clamping force orthogonal to the contact plane. In the composite beam detail, reinforcement does not cross the contact surface and therefore does not provide a clamping force. Some small clamping force may be generated on the side faces of the cast in situ concrete core by the U-beam webs resisting, by flexural action, the dilatancy caused by relative shear moment along the roughened contact surfaces. Nevertheless it would seem appropriate to ignore friction and to rely only on shear transfer by adhesion and interlock of the mated roughened contact surfaces.

The imposed shear stresses at the interface of the surfaces are the summation of stresses from a number of sources. The imposed horizontal shear stresses at the interface during positive bending moment arise from the transfer of the prestressing steel tension force from the U-beam to the cast in situ concrete core, and during negative bending moment arise from the transfer of

the reinforcing steel force from the cast in situ concrete core to the U-beam flange. The imposed vertical shear stresses at the interface arise from the self weight of the U-beam unit, the floor system weight, and the superimposed dead and live loads being supported by the floor system. These vertical loads need to be transferred from the U-beam unit, on which the floor system is seated, to the cast in situ concrete core by vertical shear stresses across the interface (see Fig. 3). This transfer of vertical shear stresses across the interface may be particularly critical if the end support of the U-beam in the column concrete is lost during seismic loading.

As has already been discussed, the consequences of loss of bond at the interface of the precast U-beam and the cast in situ concrete core in plastic hinge regions need special consideration in design.

2.5 Strength of Beam-Column Joint Core

During positive bending moment, the cast in situ reinforced concrete alone transfers the beam forces to the beam-column joint core. Hence for positive moment in the beam the joint core behaviour is that of a conventionally reinforced concrete frame. A diagonal compression strut mechanism which transfers part of the joint core forces is shown in Fig. 2a.

During negative bending moment, the greatest beam flexural strength arises from composite action when the precast U-beam flange transfers most of the compressive force in the beam to the joint core by direct bearing against the column. Then both the upper and lower layers of longitudinal reinforcement in the beam may be in tension. A joint core diagonal compression mechanism involving two struts which transfer part of the joint core forces is shown in Fig. 2b. One strut forms between the bends in the upper tension steel and the lower concrete compression zone. The other strut forms at a shallow angle to the horizontal between the bends in the lower tension steel and the lower compression zone. Should the flange of the precast U-beam cease to transfer compression to the column during seismic loading, the negative beam moment will be due to the cast in situ concrete core alone and the joint core behaviour will be that of a conventional reinforced concrete frame.

The joint core mechanism resisting the applied forces is made up partly by the diagonal compression strut mechanism described above and partly by a truss mechanism involving transverse hoop reinforcement and intermediate column bars. During cyclic loading in the inelastic range the joint core shear transferred by the diagonal compression strut mechanism decreases, mainly due to the presence of full depth cracking in the beam at the column face, and the shear

transferred by the truss mechanism increases (5,6).

3. REVIEW OF CODE REQUIREMENTS FOR THE DESIGN OF CAST IN SITU REINFORCED CONCRETE FRAMES INCORPORATING PRECAST CONCRETE BEAM SHELLS

3.1 General

Design provisions for cast in situ reinforced concrete frames incorporating precast beam shells are not fully covered by the New Zealand concrete design code NZS 3101 (5). The code provisions where applicable are reviewed below and proposals made for supplementary design recommendations where necessary.

3.2 Flexural Strength

The flexural strength may be calculated using the code assumptions of an equivalent rectangular concrete compressive stress distribution, an extreme compression fibre strain of 0.003, a linear strain distribution down the section depth, a bilinear stress-strain relationship for the longitudinal reinforcement (nonprestressed steel), and the code equation for the stress in the prestressing steel at the flexural strength. If the stress-strain curve for the prestressing steel is known the flexural strength may be calculated most accurately using the requirements of strain compatibility and force equilibrium of the beam section (7). This procedure is more accurate since it avoids the use of the approximate code equation for the stress in the prestressing steel at the flexural strength and enables more precise consideration to be given to the contribution of the prestressing tendons down the section depth.

The flexural strength of the composite beam section is calculated assuming that the cast in situ reinforced concrete core and the precast prestressed concrete U-beam act monolithically. The flexural strength of the beam section ignoring composite action is that of the cast in situ reinforced concrete beam core alone.

In general design, the dependable flexural strength of members must be at least equal to that required by the factored applied loads

$$\phi M_1 \geq M_u \quad (1)$$

where the ideal strength M_1 should be based on the cast in situ concrete beam core only at or near the beam ends, or the composite section away from the beam ends where the interface shear between the precast and cast in situ concrete can be transferred satisfactorily and development length requirements of the prestressing strand are satisfied. The strength reduction factor ϕ for flexure is 0.9.

In seismic design, the likely overstrength in flexure is required, as the

beam overstrength actions influence the beam shear design and the column flexure and shear design. For finding the likely upper limit of flexural overstrength of the beam, composite action should be assumed in plastic hinge regions where negative moment is applied since the flange of the U-beam can act as the compression zone of the composite member, as previously discussed. However for positive bending moment in plastic hinge regions at the ends of the members, only the cast in situ reinforced concrete beam core need be considered. If positive moment plastic hinges form away from the beam ends, the composite section flexural strength should be used if the interface shear and strand development length requirements are satisfied.

In complying with the code requirements for the longitudinal steel ratios, and other design parameters, the beam section dimensions are required. The width of the compression face b , the effective depth d , and the overall depth h , all depend on the moment direction and the location of the section in the beam, and are defined in Fig. 4.

In the design of beams that may form plastic hinges during seismic loading, the special detailing requirements for ductility specified by the code must extend over a potential plastic hinge region of length equal to twice the beam depth. This length conservatively represents the likely region of yielding of the longitudinal reinforcement. It is suggested conservatively that the beam depth here be defined as the full depth of the composite beam section. One of the requirements for the spacing of stirrup ties in potential plastic hinge regions is that the spacing shall not exceed $d/4$. It is suggested that the d used for this requirement is the effective depth of the cast in situ concrete core and not of the composite section.

3.3 Shear Strength

Design of cross sections subjected to shear is based on the requirement that in general design the dependable shear strength shall be at least equal to that required by the factored applied loads, namely

$$\phi v_i b_w d \geq V_u \quad (2)$$

The strength reduction factor ϕ for shear is 0.85. In seismic design the requirement is that ideal shear strength shall be at least equal to that associated with the overstrength beam moments and the factored gravity load, namely

$$v_i b_w d \geq V_u^0 \quad (3)$$

In both equations v_i is the total ideal shear stress resisted by the concrete mechanisms and the truss mechanism of the shear reinforcement.

Fig. 3 Transfer of Vertical Shear Across Interface of the Cast in Situ Concrete Beam Core and the Precast Concrete U-Beam

Fig. 4 Beam Section Parameters

In the plastic hinge region at the ends of the beams only the cast in situ reinforced concrete beam core should be relied upon to provide shear resistance, and all design parameters are related to that core section. The dimension b_w used in the shear equations could be taken as the mid-depth width of the in situ core and the dimension d is as defined in Fig. 4a and b. In the plastic hinge region the shear reinforcement in the cast in situ concrete beam core shall be designed to resist all the shear force.

Away from the potential plastic hinge regions shear will be resisted by the composite section and the shear force can be allocated to both the concrete and to the shear reinforcement. The composite section values for d defined in Fig. 4c and d should be used, and b_w taken as the width of the upper half of the cast in situ beam core.

3.4 Interface Shear Stresses

In the section of the code dealing with composite concrete flexural members there are recommendations for checking the permitted horizontal shear forces between elements cast at different times when the elements are expected to behave compositely. The "horizontal" shear forces may be interpreted to mean those shear forces acting on any plane other than the plane that is orthogonal to the longitudinal axis of the member.

As discussed in Section 2.4, the interface of the precast U-beam and the cast in situ concrete core of the composite beam investigated in this study involves two near vertical side surfaces and a bottom horizontal surface, and no clamping force across the surfaces should be assumed to exist. It is proposed that to check the interface shear transfer at the factored loads an allowable "horizontal" shear stress of 0.55 MPa be permitted, which according to the code applies to interfaces that have no cross ties, but have the contact surfaces cleaned and intentionally roughened to a full amplitude of 5 mm. All three surfaces (the two side surfaces and the bottom surface) could be assumed to make up the total "width" of the cross section being investigated and therefore the horizontal shear stress v_{dh} could be taken as the average value found from section analysis.

The general design requirement is that the dependable shear strength shall be at least equal to that required by the factored applied loads, namely

$$0.55 \text{ MPa} \geq v_{dh} = \frac{V_u}{\phi b_v d} \quad (4)$$

where d is the effective depth of the composite section, b_v is the total width of the interface, and the strength reduction factor ϕ is 0.85. Where the bending moment diagram is linear, for example as a good approximation in

negative moment regions, the value of $V_u/b_v d$ can be obtained by calculating the maximum total longitudinal tensile force in the reinforcement in the cast in situ concrete core and dividing this force by the contact area of the interface between the maximum moment section and the section where the moment reduces to zero. Note that this is a simplistic approach to the more complex real behaviour at the U-shaped interface.

In seismic design Eq. 4 can be used with V_u replaced by the vertical shear associated with overstrength beam moments and factored gravity loads, V_u^o , and ϕ may be taken as unity. Therefore

$$0.55 \text{ MPa} \geq \frac{V_u^o}{b_v d} \quad (5)$$

The effect of imposed vertical shear stress at the interface, originating from the U-beam and floor system dead weight and the imposed live loading supported by the floor system, may also need to be included. If the seating of the U-beam ends at the column face is lost, through spalling of the seating concrete, these gravity loads are transferred from the U-beam entirely via the interface to the in situ concrete beam core (and then out to the columns supporting the beam core). Typically the vertical shear stress due to gravity loading is small enough to be ignored, but allowance for it may be made if desired. One approach would be to find the vector sum, of the vertical shear stress due to the gravity load and the average horizontal design shear stress described previously, and to check this vector sum against the allowable value of 0.55 MPa.

Note that the design approach should assume that dependable composite action can only occur away from the potential plastic hinge zones. One possible effect of the loss of bond between the precast U-beam and the cast in situ concrete is shown in Fig. 5. The vertical loads carried by the precast concrete U-beam may cause failure of the seating of the precast beam at the column face and could result in flexural cracks in the top, or even flexural failure, of the precast beam.

3.5 Strength of Beam-Column Joint Core

It is recommended that the code approach for the design of beam-column joints should be used ignoring forces from possible composite beam action. That is, the design horizontal joint core shear forces should be found for the cast in situ concrete beam acting alone. This assumption is obvious for positive beam moments but is an approximation for negative moments as is discussed in Section 2.5. However for negative beam moments the upper layers of bars introduce the horizontal joint core shear force over the greater part of the core depth. The horizontal shear force introduced

by the lower layers of bars may be assumed to be equilibrated by the very shallow diagonal compression strut shown in Fig. 2b if those bars are in tension.

4. TEST PROGRAM

4.1 General

Three full scale composite beam-exterior column units were designed, constructed and tested. The overall dimensions of the units are shown in Fig. 6.

For ease of construction of the units the T beam flanges typically resulting from the presence of the cast in situ concrete floor topping (see Fig. 1a) were not modelled.

All units were designed using the New Zealand concrete design code NZS 3101 (5), with the addition of the suggested supplementary design recommendations where necessary, as discussed in Section 3. The strength reduction factor was taken as $\phi = 1$ in all design calculations, and the overstrength factor for the Grade 275 longitudinal reinforcement in the beams was taken as 1.25.

Unit 1 was designed for seismic loading, with a potential plastic hinge region in the beam. Unit 2 was not designed for seismic loading. Unit 3 was designed for seismic loading and was identical to Unit 1 in all respects except that the interface between the precast concrete and the cast in situ concrete in the potential plastic hinge region of the beam was deliberately debonded in an attempt to improve the plastic hinge behaviour.

4.2 Unit 1 Details

Unit 1 was designed for seismic loading. Capacity design principles were used to ensure that beam plastic hinging was the energy dissipating mechanism when seismic loading caused the unit to enter the inelastic range of behaviour. The beam flexural overstrengths were calculated assuming composite behaviour for negative moment and no composite behaviour (that is, the cast in situ concrete core acting alone) for positive moment. The critical beam section for flexure was at the face of the column.

The details of the reinforcement for Unit 1 are shown in Fig. 7. The beam flexural reinforcement consisted of four D24 Grade 275 steel bars in the top and bottom of the cast in situ concrete core ($\rho = 1.83\%$ and $\rho' = 1.42\%$, based on the beam core b and d). The column flexural reinforcement consisted of three D16 Grade 380 bars at each face of the column, and two D20 Grade 380 intermediate bars.

At the column load level of $0.1A_g f'_c$ used in the tests, 68% of the horizontal joint core shear forces, calculated for the actions at overstrength,

was required to be resisted by joint hoops and ties according to the code. Five sets of two leg R10 Grade 275 hoops plus single leg R12 Grade 275 cross ties were provided to meet this steel requirement. The level of vertical joint core shear force necessitated the provision of the two intermediate column bars referred to above. At the level of column load applied, $0.1A_g f'_c$, the sum of the ideal flexural strengths of the column sections above and below the beam was 1.59 times the beam section ideal flexural strength, based on actual material strengths.

The code requirements for transverse steel in the beam and column typically resulted in a shear strength in excess of the design shear force, since spacing limitations usually governed the transverse steel arrangements. The beam shear strength was calculated for the cast in situ reinforced concrete core acting alone.

4.3 Unit 2 Details

Unit 2 was not designed for seismic loading. The design followed the code recommendations for reinforced concrete frames which are not intended as primary seismic load resisting systems. This design approach could be used, for example, if the seismic loads acting on the structure are resisted primarily by structural walls. However, the members of such a frame in a building would be required to undergo the deformations associated with maintaining compatibility with the rest of the structure under seismic displacements. The purpose of testing this unit under simulated seismic loading was to determine the potential the unit had for ductility and energy dissipation.

The details of the reinforcement for Unit 2 are shown in Fig. 8. Four D24 Grade 275 bars were used in the top of the cast in situ concrete core and two D20 Grade 275 bars were used in the bottom ($\rho = 1.83\%$ and $\rho' = 0.47\%$, based on the beam core b and d). The column flexural reinforcement was identical to that of Unit 1. Simplicity of construction, rather than strength requirements, was the rationale for the choice of column flexural steel. The same axial load level was maintained in this test, namely $0.1A_g f'_c$. At this column load level, 38% of the horizontal joint core shear force calculated for actions at overstrength was to be carried by joint hoops and ties according to the code. Three sets of two leg R10 Grade 275 hoops plus single leg R12 Grade 275 cross ties were provided. The transverse steel provisions for the beam and columns were governed by minimum shear steel requirements of the code.

4.4 Unit 3 Details

Unit 3 was designed for seismic loading. The reinforcement details are shown in Fig. 7. Unit 3 was identical to Unit 1 in all respects except that the

Fig. 5 Possible Failure of Precast Concrete U-Beam in Debonded Plastic Hinge Region of Composite Beam After Loss of Seating at Column

(a) Elevation of test units (dimensions in mm)

(b) Column and beam sections (dimensions in mm)

Fig. 6 Dimensions of the Test Units

Fig. 7 Details of Reinforcement of Units 1 and 3

Fig. 8 Details of Reinforcement of Unit 2

interface of the precast and cast in situ concrete was deliberately debonded in the potential plastic hinge region of the beams. This was achieved by fixing a thin sheet of foam rubber to the precast U-beam over a length equal to the depth of the cast in situ core concrete, measured from the face of the column.

4.5 Construction of the Units

The interior surfaces of the precast concrete U-beam had been roughened by the precaster so that composite action between the cast in situ beam core and the U-beam could be achieved. The roughening was attained by chemical retarding of the interior surface of the precast unit after initial set, and then removal of the surface cement paste from around the aggregate by washing with water and wire brush. The amplitude of the roughening, defined in NZS 3101 (5), was typically on average 3 mm. Fig. 9 indicates the surface conditions of the U-beam units used. The NZS 3109 (8) classification of this roughening would be a Type B construction joint.

The in situ concrete of the units was cast in the same orientation as for the beams and columns in a prototype structure and according to anticipated site practice. There were two concrete pours for each unit. The lower column was poured first, up to the height where the precast U-beam would be seated on the lower column. The top surface of this column pour was roughened to the code requirements for construction joints and a ledge prepared for seating the precast U-beam. In the second pour, the beam core, beam-column joint region, and the upper column were cast. The formwork was of plywood. Compaction of concrete was achieved by internal vibrators. Each unit was damp cured for not less than seven days. Fig. 10 shows the beam reinforcement in the precast concrete U-beam and the column reinforcement in place for Unit 1.

The debonded plastic hinge region of Unit 2 was achieved by fixing a 3.5 mm thick sheet of foam rubber to a length of the inside surface of the precast concrete U-beam before casting the in situ concrete core. The length of the debonded region was equal to the depth of the cast in situ concrete core, measured from the face of the column, plus the U-beam extension into the column. Fig. 11 shows the details of the debonded region. The inner surface of this U-beam region was plastered smooth before fixing the foam rubber sheet. This smoothing and foam rubber treatment of the inner surface of the U-beam effectively prohibited bond and friction developing between the cast in situ beam core and the interior surface of the U-beam in that region.

4.6 Steel Properties

The measured yield and ultimate strengths of the Grade 275 and 380

reinforcing steel are shown in Table 1, and representative stress-strain curves in Fig. 12, as obtained from laboratory tests.

The properties of the prestressing strand used in the U-beams were provided by the strand manufacturer and are shown in Table 2 with the stress-strain curves shown in Fig. 13. Information on the actual properties of the Grade 275 steel stirrups in the precast U-beam was not available.

4.7 Concrete Properties

The cast in situ concrete was supplied by a ready-mix plant. The maximum aggregate size was 12 mm. The 100 mm diameter by 200 mm test cylinders were cured in the fog room. The concrete slump and compressive cylinder strengths are shown in Table 3.

The precast concrete U-beams were provided by a precasting firm. The compressive cylinder strength of the U-beam concrete is also shown in Table 3.

4.8 The Test Rig

The test rig and loading arrangements are shown in Figs. 14 and 15. The rig consisted of a tubular steel tetrapod to carry the axial compressive load applied to the column, and two independent reaction frames to carry the lateral loads. The column ends were held in a vertical line and the beam ends were displaced vertically. By alternating the direction of the beam end loads earthquake loading was simulated. The beam end load cycles were applied statically over a number of days.

In the rig the ends of the columns were grouted into steel caps and the axial column load of $0.1f_c A_g$ was applied by a hydraulic jack which acted through a load cell. Steel pins at the column ends allowed free rotation in the plane of the specimen.

The beam loading consisted of its self weight, a superimposed "dead load" and the alternating vertical end load applied by a hydraulic jack. The jack, acting through a load cell, allowed control of load or displacement in the vertical direction. This jack load was transmitted via steel yokes and pins to the ends of the beam. As the units were of full size dimensions, their self weight contributed significantly to the loading. The superimposed dead load represented a 200 mm thick Dycore slab floor with 65 mm thick topping, spanning between frames in the prototype building at 6.625 metre centres. The positioning of the superimposed dead load was organised so that this load was applied only on the top horizontal surface of the precast U-beam webs, which is how the slab system in a real structure would be supported (see Fig. 1). For the tests the superimposed dead load consisted of four assemblies suspended from the U-beams

Fig. 9 View of Typical (Unit 1) Roughened Interior Surface of the Precast Concrete U-Beams

Fig. 10 Views of Beam and Column Reinforcement in Place During Construction of Unit 1

(a) Construction Details

(b) Foam Rubber Sheet in Place

Fig. 11 Method of Debonding Potential Plastic Hinge Region of Unit 3

Fig. 12 Stress-Strain Curves for Reinforcing Steel

Fig. 13 Stress-Strain Curves for Prestressing Strand

Table 1 : Measured Properties of Reinforcing Steel

Grade of Bar	Grade 275				Grade 380	
Bar Designation	R10	R12	D20	D24	D16	D20
f_y (MPa)	336	311	285	308	492	444
f_{su} (MPa)	467	463	437	469	789	704

Table 2 : Measured Properties of Prestressing Steel

7-wire Strand	7.9 mm Diameter	12.5 mm Diameter
0.2% Proof Stress (MPa)	1808	1678
f_{pu} (MPa)	1926	1793

Table 3 : Measured Properties of Concrete

Unit	Slump (mm)		f'_c at Day of Testing Unit (MPa)		
	Lower Column	Upper Column and Beam	Lower Column	Upper Column and Beam	U-Beam
1	185	140	30.1	33.6	50.2
2	190	180	24.2	29.2	54.6
3	190	190	28.3	26.1	53.0

at appropriate spacings to represent the floor slab weights (see Fig. 14). The weights for the assemblies were obtained using lead ingots, concrete blocks, and scrap steel.

4.9 Instrumentation

Displacements of the units measured during the tests were the end and midspan deflections of the beam, column end deflection, joint core distortions, and beam rotations and slip between the cast in situ beam core and the precast U-beam interior surface within the plastic hinge region. Linear potentiometers were used to measure displacements.

The beam rotations in the potential plastic hinge regions were obtained using linear potentiometers connected to two steel frames fixed to two pairs of horizontal steel rods cast in the in situ concrete. To facilitate the placing of the lower pair of rods, 64 mm diameter holes were diamond cut through the webs of the U-beam. The cast in situ beam core concrete was not allowed to enter these holes, thus avoiding any horizontal shear transfer there between the cast in situ and precast concrete. These holes allowed easy viewing of any differential movement between the cast in situ concrete core and the U-beam walls.

Some electrical resistance strain gauges were also placed on the beam longitudinal reinforcement in the potential plastic hinge region and on the column

transverse reinforcement in the potential plastic hinge region and on the column transverse reinforcement within and adjacent to the joint core.

Potential sliding shear along vertical cracks within the plastic hinge region of the beams was anticipated. A datum was set up on the column, with a grid pattern marked on the beam, to measure vertical shear slip between the beam and column.

4.10 Testing Procedure

The New Zealand loadings code, NZS 4203 (3), specifies that the performance of a ductile structural assembly is satisfactory if it retains at least 80% of its strength after being subjected to a minimum of four cycles of loading to a displacement ductility factor μ of four in each direction.

In these tests a gradual increase in the displacement ductility factor was chosen, to enable the behaviour of the units to be examined at various ductility levels.

In the first cycle of loading the beam was taken to three-quarters of the first yield load, where the first yield load was calculated as that load when the first yield strain was reached in the outermost flexural bars. The beam end deflection at first yield, Δ_y , was taken as 1.33 times the beam end deflection measured at three-quarters of the first yield load. This definition of the

Fig. 14 View of Test Rig

Fig. 15 Simulated Seismic Loading of Units

"deflection at first yield", although arbitrary, gives a convenient reference first yield deflection. The displacement ductility factor is then defined as

$$\mu = \Delta / \Delta_y \quad (6)$$

where Δ is the imposed beam end deflection.

The subsequent loading cycles were displacement controlled. The load pattern used in the tests is shown in Fig. 15, and consisted of two cycles to $\mu = \pm 1$, four cycles to $\mu = \pm 2$, four cycles to $\mu = \pm 4$, and two cycles to $\mu = \pm 6$,

5. TEST RESULTS

5.1 Unit 1 Results

The measured beam end load-beam end deflection hysteresis loops are shown in Fig. 16a. It is evident that the performance of the unit was acceptable according to the performance criterion of the loadings code NZS 4203 (3) for ductile frames. The energy dissipation characteristics of the plastic hinge region appeared very satisfactory as demonstrated by the shape of the loops. The loss of stiffness during cyclic loading was due to the presence of open cracks in the compression zone caused by yielding of the longitudinal tension steel in the previous half cycle (what was a tension zone in the previous half cycle was now a compression zone), and due to the Bauschinger effect which caused a reduction in the tangent modulus of the longitudinal steel at stresses less than the original yield stress during reversed loading.

Some sliding shear occurred along the vertical crack in the beam at the column face, particularly in the final stages of testing, but did not result in pinching of the load-deflection hysteresis loops. Loss of the U-beam seating occurred, due to spalling of concrete at the column face, but the gravity loads applied to the U-beam were transferred across the interface to the cast in situ beam core by bond without adverse effects.

The theoretical beam end load capacities, based on the ideal flexural strengths calculated using the measured material properties, are also shown in Fig. 16a. For positive moment displacement ductility factors the peak measured beam moments reached were 89 to 112% of the ideal capacity, based on the cast in situ concrete core section alone. For negative moment displacement ductility factors, the peak beam moments reached at least 133% of the ideal capacity based on the cast in situ concrete core section alone, and ranged between 75 and 90% of the ideal capacity based on the composite section.

The visible cracking and other damage observed during testing is illustrated in Fig. 16c to e. The cracks

were traced with felt pen on the white painted surface of the concrete. Bond break down at the interface of the cast in situ concrete core and the precast beam concrete, and flexural cracking of the precast beam concrete, occurred to various degrees in the plastic hinge region during positive bending moment. This allowed the plastic hinge region during positive bending moment to propagate along the cast in situ concrete core and thus reduced the concentration of plastic hinging in the beam at the column face. The cracking in the bottom of the precast U-beam during positive moment increased as bond was gradually lost along the pretensioned strands at the end of the beam. During negative moment there was flexural cracking in the top of the cast in situ concrete core along the beam as expected.

Strain readings recorded on the longitudinal beam reinforcement (see Fig. 16b) indicated that at the completion of the test the length of the beam over which the yielding of flexural steel occurred was about one-half of the depth of the cast in situ concrete core for positive moment, and was slightly greater than the depth of the cast in situ concrete core for negative moment. The shorter plastic hinge region for positive moment was anticipated, but nevertheless the length of this region was sufficient to permit plastic hinge rotation without undue concentration of curvature.

The maximum applied nominal shear stress $V_{max}/b_w d$ in the plastic hinge region of the beam during the tests was $0.28/f'_c$ MPa, assuming that the cast in situ concrete core resisted all the shear force. The stirrup ties provided in the cast in situ concrete core were capable of resisting a nominal shear stress of $0.41/f'_c$ MPa, based on the code equation $A_f V_y / b_w$ given by the conventional truss mechanism.

The column remained in the elastic range with limited cracking during the tests. Diagonal tension cracking occurred in the beam-column joint core but was not extensive. The peak stress reached by the joint core hoop reinforcement was 83% of the yield strength. Yielding of the longitudinal beam reinforcement eventually penetrated through the joint core to reach the bend in those bars.

5.2 Unit 2 Results

The beam end load-beam end deflection hysteresis loops shown in Fig. 17a indicate that in terms of strength during cyclic loading the unit satisfied the performance criterion of the loadings code NZS 4203 (3) for ductile frames. However the energy dissipation characteristics of the plastic hinge region were not particularly satisfactory as is evident from the pinched shape of the loops. Part of this pinching in the early loading runs was due to the

(a) Beam end load versus beam end deflection, Unit 1

(b) Beam longitudinal reinforcement strains for first cycle of each displacement ductility factor group, Unit 1

(c) View of Unit 1 at
peak of fourth
load run to $\mu = -2$

(d) View of Unit 1 at
peak of fourth
load run to $\mu = +4$

(e) View of Unit 1 at
completion of test

Fig. 16 Unit 1 Behaviour (Continued from previous page)

relatively early yielding of compression steel during negative moment loading, since for that direction of moment the area of compression steel is smaller than the area of tension steel. However most of the pinching was due to sliding shear displacements.

Sliding shear along the vertical cracks in the beam at the face of the column dominated the load-deflection behaviour of the beam. The observed sliding shear displacements were much more extensive than in the case of Units 1 and 3. This behaviour can be related to the fact that the amount of transverse reinforcement in the cast in situ concrete core of Unit 2 was only 40% of that in Units 1 and 3. The 250 mm centre to centre spacing of the transverse reinforcement used in Unit 2 was 0.44 of the overall depth of the cast in situ concrete core. The maximum applied nominal shear stress in the plastic hinge region of the beam of Unit 2 during the tests was $0.29\sqrt{f'_c}$ MPa, whereas the stirrup ties provided were capable of resisting a nominal shear stress of $0.18\sqrt{f'_c}$ MPa by the conventional truss mechanism. Hence the remaining nominal shear stress of $0.11\sqrt{f'_c}$ MPa needed to be carried by concrete shear resisting mechanisms. These nominal shear stresses are based on the properties of the cast in situ concrete core alone. The behaviour of Unit 2 provided evidence of the need to provide the total shear resistance by transverse reinforcement in the plastic hinge regions in order to achieve ductile behaviour and to control sliding shear displacements. Note that the 100 mm spacing of stirrup ties used in Unit 1 satisfactorily controlled sliding shear in that unit.

For positive moment displacement ductility factors, the peak measured beam moments ranged between 90 and 109% of the ideal capacity based on the flexural strength of the cast in situ concrete core alone, calculated using measured material strengths. For negative moment displacement ductility factors the peak beam moments exceeded the ideal capacity based on the cast in situ concrete core alone and ranged between 83 and 103% of the ideal capacity based on the composite section, calculated using measured material strengths.

The visible cracking and other damage observed during testing is illustrated in Fig. 17c to e. Unlike Unit 1, bond break down at the interface of the cast in situ concrete core and precast beam concrete was not observed during positive bending moment. The precast U-beam suffered little damage, with only flexural cracks propagating down from the cast in situ concrete above during negative bending moment.

During negative moment there was spread of longitudinal beam steel yielding along the beam much as in Unit 1 (see Fig. 17b). However during positive

moment no flexural cracks developed in the precast U-beam and hence only one major crack formed at the end of the U-beam at the column face. The strain gauge readings for bottom bars in tension in Fig. 17b did not indicate a very high peak strain at the beam end, presumably because the critical crack at the end of the precast beam was 65 mm away from the nearest strain gauge and hence the peak strain was not recorded. The curvature ductility demand in this very localised region (a single crack) during positive bending moment would have been much higher than in Unit 1. Evidently the small area of bottom reinforcement in the cast in situ beam core meant that the positive moment at the flexural strength of the beam core was not high enough to crack the precast concrete U-beam in the composite section and hence the positive moment plastic hinge was unable to spread along the beam. This points to the importance of having sufficient bottom reinforcement in the cast in situ concrete core to cause the precast part of the composite beam to crack during positive bending moment.

The precast U-beam lost its seating during the tests, due to the spalling of the concrete at the column face, but the gravity load applied to the U-beam was transferred across the interface to the cast in situ concrete by bond without adverse effects.

The column remained in the elastic range with limited cracking during the tests. Diagonal cracking of the beam-column joint core formed only during negative moment loading since the positive moment shears were too small to cause cracking. The joint core hoops remained in the elastic range. Yield penetration of the longitudinal beam steel into the joint core was recorded.

5.3 Unit 3 Results

The measured beam end load-end deflection hysteresis loops shown in Fig. 18a satisfied the performance criterion of the loadings code NZS 4203 (3) for ductile frames. For positive and negative moment ductility factors the peak measured beam moments ranged up to 103% and 104%, respectively, of the ideal capacities based on the cast in situ concrete core alone, calculated using measured material strengths. It is to be noted that the maximum negative beam moment reached ranged between 69 and 75% of the ideal capacity of the composite section.

The visible cracking and damage is illustrated in Fig. 18c to e. The main flexural cracking in the cast in situ concrete core during both positive and negative moments gradually extended during the test along the whole length of the region where the bond between the cast in situ concrete core and the precast U-beam had been deliberately broken. The region of the beam beyond the debonded length appeared to remain fully composite,

(a) Beam end load versus beam end deflection, Unit 2

(b) Beam longitudinal reinforcing strains for first cycle of each displacement ductility factor group, Unit 2

(c) View of Unit 2 at
peak of fourth
load run to $\mu = -2$

(d) View of Unit 2 at
peak of fourth
load run to $\mu = +4$

(e) View of Unit 2 at
completion of test

Fig. 17 Unit 2 Behaviour (Continued from previous page)

(a) Beam end load versus beam end deflection, Unit 3

(b) Beam longitudinal reinforcement strains for first cycle of each displacement ductility factor group, Unit 3

(c) View of Unit 3
at peak of fourth
load run to
 $\mu = -2$

(d) View of Unit 3
at peak of fourth
load run to
 $\mu = +4$

(e) View of Unit 3
at completion
of test

Fig.18 Unit 3 Behaviour (continued from previous page)

as no slip along the interface of the cast in situ concrete and the precast U-beam was apparent there. The only cracking which occurred in the precast U-beam was very minor cracking at the top, and hence the U-beam at the end of the test gave the appearance of being undamaged. The major vertical crack that formed at the ends of the precast beam at the face of the column had a smaller width in Unit 3 than in Units 1 and 2. The distribution of cracking along the plastic hinge region was more extensive in Unit 3 than in Units 1 and 2, and hence the concentration of beam plastic rotation at the major crack was not so great in Unit 3.

The strain readings on the longitudinal beam reinforcement (see Fig. 18b) indicated that the length of steel yielding was about equal to the depth of the cast in situ concrete beam core.

No slip at the interface of the cast in situ beam core and the precast U-beam appeared to occur, even after the loss of the seating support of the debonded U-beam when the concrete at the column face spalled. The debonded length of U-beam acted as a cantilever, carrying the gravity load applied to it. This gravity load had to be transferred to the cast in situ concrete core at the end of the debonded region, and this was satisfactorily achieved.

The maximum applied nominal shear stress in the plastic hinge region of the beam was $0.25\sqrt{f'_c}$ MPa during the test, whereas the stirrup ties provided were capable of resisting a nominal shear stress of $0.47\sqrt{f'_c}$ MPa by the conventional truss mechanism.^c This nominal shear stress was based on the properties of the cast in situ concrete core alone.

The column remained in the elastic range with limited cracking during the tests. Diagonal tension cracks in both directions were observed in the beam-column joint core and all joint core hoops except one remained in the elastic range. Yield penetration of the longitudinal beam bar steel into the joint core occurred as in Unit 1.

6. DISCUSSION OF THE TEST RESULTS AND DESIGN ASSUMPTIONS

As discussed in Section 3, the seismic design provisions of the New Zealand concrete design code NZS 3101 (5) do not cover all aspects of the design of composite beam-column subassemblies. The findings from the tests concerning the design recommendations made in addition to those in NZS 3101 are discussed below.

The length of the potential plastic hinge region in the cast in situ concrete beam core was taken as twice the full depth of the composite member. However the test results indicated that twice the depth of the in situ concrete core would be a more appropriate length for use in design.

The development length of the prestressing strand in the precast portion of the composite beam must be considered so that a shortfall in the positive moment flexural strength does not arise if the positive moment plastic hinge region occurs within the span. It is suggested that the first 200 mm of strand be disregarded when determining development lengths since the test results from Unit 1 indicated some degradation of bond along the strands during testing of up to this distance from the end of the U-beam unit.

The minimum dependable flexural strength of the composite beams for both positive and negative moments at the column face were found to be given by the cast in situ concrete core alone. The maximum flexural strengths required for the beam overstrength considerations in the design of the column, beam-column joint core, and the beam shear resistance were assumed to come from the composite beam for negative moment and from the cast in situ concrete beam core alone for positive moment. These assumptions were found to be valid.

The test results from Unit 1 showed that the cast in situ concrete beam core tended to act independently from the precast U-beam in the plastic hinge regions at the ends of the beams once deterioration of the interface bond and cracking of the bottom of the precast U-beam had occurred. This bond deterioration and cracking allowed the yielding of the longitudinal beam reinforcement in tension during positive bending moment to spread along a reasonable length of the cast in situ concrete beam core, and hence a concentration of plastic rotation at the crack at the column face did not occur. However in Unit 2 the area of longitudinal reinforcement in the bottom of the cast in situ concrete core was only 0.47% of the core bd and hence the positive moment flexural strength of the core was insufficient to cause cracking of the precast U-beam. As a result, in Unit 2 the plastic rotation in the beam during positive bending moment was undesirably localised at a single crack at the column face. Therefore, if the area of the bottom steel in the core is not high (it was 1.4% of the core bd in Unit 1) a debonded plastic hinge zone such as in Unit 3 should be used to ensure a spread of yielding during positive bending moment. The spread of yielding during negative bending moment was satisfactory in all units. Note that the design possibility of anchoring exposed ends of strand, from the bottom flange of the precast U-beam, in the cast in situ concrete beam-column joint core would not be particularly effective, since a long development length of 150 strand diameters would be required. Also, debonding of a short length of the bottom reinforcement bars from the column face into the beam core would spread tension yielding during positive moment, but this debonding would have the disadvantage that such bars during negative moment may not act effectively as compression reinforce-

ment due to a tendency to buckle. Note also that placing ties across the U-beam to improve interface shear transfer in the plastic hinge region would not be helpful to the spread of plastic hinging in the region which requires some bond deterioration at the interface.

The bond deterioration observed at the interface of the cast in situ concrete core and the precast U-beam in the plastic hinge region indicated that the cast in situ reinforced concrete core eventually provided the full shear resistance in the plastic hinge region. Hence in design only the shear resistance associated with the cast in situ concrete core should be used in the potential plastic hinge region. Outside the plastic hinge region composite action was observed, and hence the shear resistance associated with the composite section could be used in that part of the member.

All design section parameters (b , b_w , d and h) for the beam were redefined in terms of either the cast in situ concrete core alone or the composite section as appropriate when applying the code provisions. The redefinitions appear to have led to satisfactory design.

The design requirements for shear stresses at the interface of the cast in situ concrete and the U-beam are not specifically covered by the code. However, application of a conservative limit (0.55 MPa) to the calculated nominal horizontal shear stresses outside the plastic hinge regions appeared to result in a satisfactory solution. In Units 1 and 3, assuming composite action, this calculated nominal shear stress was slightly in excess of 0.55 MPa. Bond break down at the interface during plastic hinging should be considered to have occurred over an end length of member equal to the depth of the cast in situ concrete core.

In the final stages of the tests the U-beam seating at the column face was lost due to the spalling of the column concrete in that zone. While this did not apparently affect the strength or ductility of Units 1 or 3, it is evident that some form of strengthening of the column at the beam seating zone would prevent unnecessary damage. It is suggested that a rectangular steel collar formed of welded structural steel angle, approximately 40 mm x 40 mm, could be placed in the cover concrete of the column around all the column reinforcement. The vertical flange of the angle could be in the column face. Alternatively, if the beam ends are propped during construction, a gap could be formed between the bottom of the precast beam and the cast in-situ column concrete.

Column flexure and shear, and beam shear design, based on the beam over-strength considerations recommended above

appear appropriate, as the observed behaviour in the tests matched the intentions of the design. Beam-column joint core design utilising the code approach resulted in satisfactory behaviour.

7. GENERAL CONCLUSIONS FROM TEST RESULTS

Tests were conducted on three full scale reinforced concrete composite beam-exterior column subassemblies subjected to simulated seismic loading. The composite beams were formed from a precast prestressed concrete U-beam shell with a cast in situ reinforced concrete core. The pretensioned strand in the U-beam terminated at the beam end and hence was not anchored in the column. The columns were of cast in situ reinforced concrete.

The conclusions reached as a result of the study are as follows:

1. The provisions of the New Zealand concrete design code NZS 3101 do not cover all aspects of the design for seismic loading for this type of construction. Proposals are made for additional design recommendations where necessary to take into account the presence and directional influence of the precast concrete U-beam during severe seismic loading.
2. Units 1 and 3 were composite beam-exterior column subassemblies designed for seismic loading with the potential plastic hinge regions in the beams. Unit 3 had the bond at the interface of the precast concrete U-beam and the cast in situ concrete core deliberately broken in the potential plastic hinge region, in an attempt to increase the length of the plastic hinge region. In Unit 1 the precast and cast in situ concrete were bonded along the whole length of beam. When subjected to simulated seismic loading both Units 1 and 3 exhibited satisfactory strength and ductility characteristics in terms of the performance criterion of the New Zealand loadings code NZS 4203 for ductile frames. The strength of the units was maintained at acceptable levels when displacement ductility factors of up to ± 6 were imposed on the units. In addition the hysteresis loops were not pinched and indicated satisfactory energy dissipation characteristics. In Unit 1 there was a tendency for the plastic hinging to spread along the cast in situ reinforced concrete core within the precast concrete U-beam, even under positive bending moment, and hence the plastic hinge rotation did not concentrate in the beam at the column face and no undesirable concentration of curvature resulted. In Unit 1 the precast concrete U-beam became extensively cracked during the tests. In Unit 3 the deliberate debonding of the interface concrete resulted in a longer plastic hinge length in the

- cast in situ concrete core and the precast concrete U-beam was not damaged during the testing. Although both Units 1 and 3 displayed satisfactory ductile behaviour during seismic loading, it may be considered that the debonded construction used in Unit 3 is to be preferred, in order to reduce the damage to the precast concrete U-beam shell during seismic loading.
3. Unit 2 was a composite beam-exterior column subassembly which was not designed for seismic loading. That is, the potential plastic hinge region was not detailed with closely spaced stirrup ties for ductility. When the unit was subjected to simulated seismic loading it was judged that the energy dissipation characteristics for seismic loading were not acceptable. Extensive sliding shear displacements occurred in the plastic hinge region of the beam and resulted in pinched hysteresis loops with low included area. However Unit 2 would be suitable for non-seismic resisting frames, where seismic loads are carried by walls or other structural systems.
 4. The performance of the column and the beam-column joint core of all three units during simulated seismic loading was satisfactory.
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