Seismic Resistance of Frames Incorporating Precast Prestressed Concrete Beam Shells

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The use of precast concrete in building frames has a number of attractive features such as better quality control of the product and savings in formwork and construction time. The basic problem in the design of earthquake resistant building structures incorporating precast concrete elements is in finding an economical and practical method for connecting the precast elements together. The connection between the elements should ensure satisfactory strength and stiffness against seismic loads and enable the structure to achieve the necessary ductility during cyclic loading in the inelastic range.

Composite systems of concrete buildings, combining precast and cast-in-place (cast in situ) reinforced concrete, have a number of advantages in construction. The incorporation of precast concrete elements has the advantage of high quality control and speed of construction, and the cast-in-place reinforced concrete provides the structural continuity and the ductility necessary for adequate seismic performance.

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 cast-in-place 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-place concrete of the beam or column.

The typical structural organization of a building floor and frame system incorporating the precast pretensioned U-beam units is shown in Fig. 1. 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 (1.6 to 2.0 in.) and to place a 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 precast floor system is in place, the reinforcement may be placed, and the in-place concrete cast, inside the beam units, the topping slab and the columns of the next story. The section of the composite beam in the finished structure is shown in Fig. 2. Precast concrete columns have sometimes been used rather than cast-in-place concrete columns.

The precast prestressed concrete U-beam illustrated in Fig. 2 has webs tapered from the bottom to the top, to ensure ease of removing internal formwork when precast. 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-place 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. Note that the strands terminate in the end of the U-beam and hence are not anchored in the beam-column joint regions of the frames.

Initially in New Zealand, precast concrete U-beams were principally used in the construction of low rise buildings in which the horizontal seismic loads are resisted primarily by other elements such as totally cast-in-place reinforced concrete structural walls or frames. An early example of this type of construction is the Karioi Pulp Mill[1] (see Fig. 3).

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

This paper reviews seismic design considerations for frames with such composite beams. The results of tests

Synopsis

The performance of cast-in-place reinforced concrete moment resisting frames incorporating precast prestressed concrete U-beam shells, subject to seismic loading, is investigated. The precast beams act as permanent formwork and are not connected by steel to the cast-in-place concrete of the beam or column.

Three full scale beam-exterior column subassemblies were tested to determine their seismic performance characteristics when plastic hinge regions occur in the beams adjacent to the columns.

Provisions for the seismic design of such composite structures are discussed and additional design recommendations based on the test results are proposed where necessary. A numerical design example is included to illustrate the design approach.

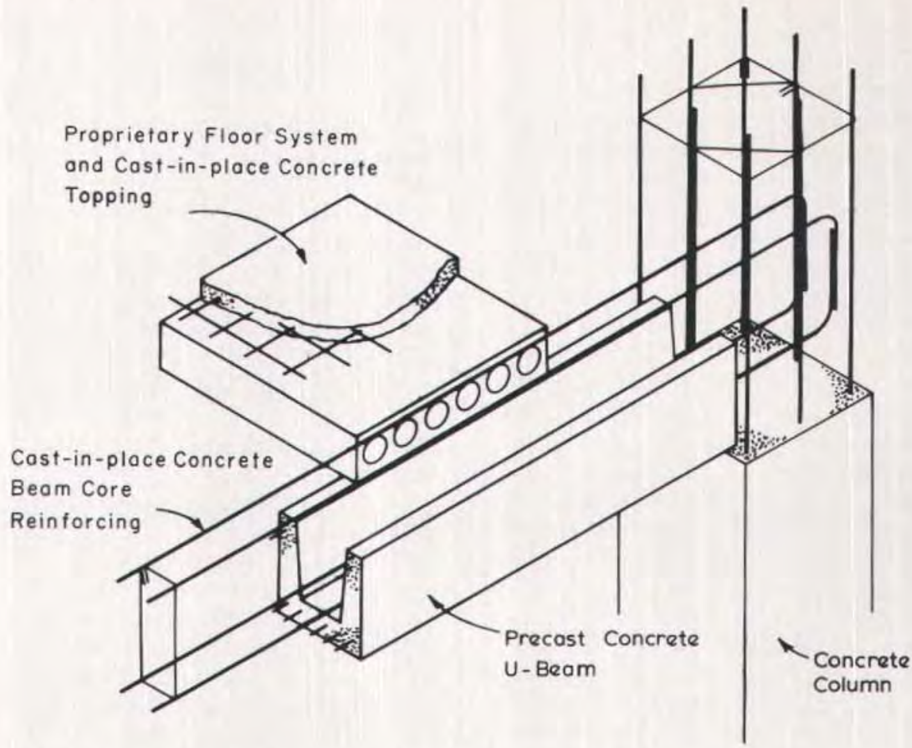


Fig. 1. Construction details of a composite structural system (not all reinforcement is shown).

conducted on three full scale composite beam-exterior column subassemblies, subjected to simulated seismic loading, are summarized. Design provisions based on the test results are proposed and a numerical design example is included to illustrate the design approach. The results of the tests may be seen reported in more detail in Ref. 2.

SEISMIC DESIGN CONSIDERATIONS

In the design of structures for earthquake resistance, a prime consideration is to ensure that the structure is capable of deforming in a ductile manner when subjected to several cycles of horizontal loading well into the inelastic range. This is because it is generally uneconomical to design a structure to respond in the elastic range to the large horizontal inertia loads induced by the greatest likely earthquake.

The recommended level of seismic design loads in codes is generally significantly lower than the elastic response inertia loads during severe earthquakes and the structure may be required to undergo horizontal displacements which are four to six times the horizontal displacement at the commencement of yielding of the frame. The ratio of the maximum displacement to the displacement at first yield is commonly referred to as the displacement ductility factor.

Ideally, the design concept for moment resisting frames should aim at dissipating seismic energy by ductile flexural yielding at chosen plastic hinge

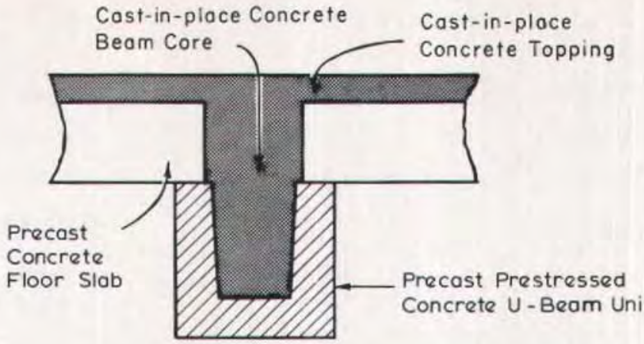


Fig. 2. Section of composite beam in finished structure (reinforcement is not shown).

Fig. 3. Precast concrete U-beams used as permanent formwork for cast-in-place reinforced concrete frames (Karioi Pulp Mill Building, New Zealand).

regions when the structure is subjected to the seismic design loads. The rest of the frame should be made sufficiently strong to ensure that it remains in the elastic range when flexural yielding occurs at the chosen plastic hinge locations. This means ensuring that shear failures and bond failures do not occur and that the preferred energy dissipating mechanism forms.

Mechanisms involving flexural yielding at plastic hinges are shown in Fig. 4. If yielding commences in the column before it occurs in the beams, a column sidesway mechanism can form as illustrated in Fig. 4b. Such a "soft story" mechanism can make very large curvature ductility demands on the plastic hinges of the critical story, particularly in the case of tall buildings.

On the other hand, if yielding occurs in the beams before it begins in the columns a beam sidesway mechanism, illustrated in Fig. 4c, can develop which makes more moderate demands on the curvature ductility required at the plastic hinges in the beams and at the column bases, even for tall frames. There-

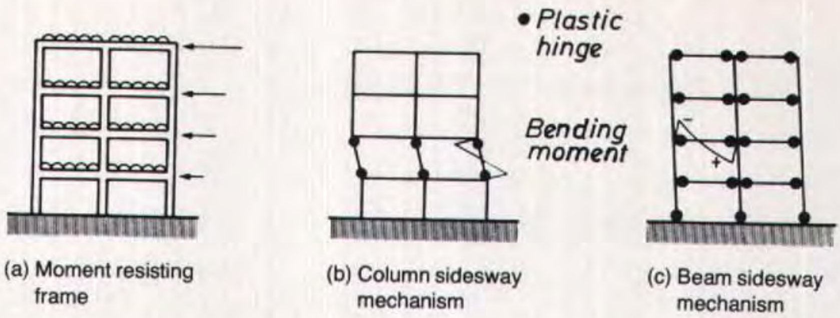


Fig. 4. Moment resisting frame with horizontal seismic loading and possible mechanisms.

fore, for tall frames, a beam sidesway mechanism is the preferred mode of inelastic deformation and a strong column-weak beam concept is advocated to ensure beam hinging.

For frames with less than about three stories, and for the top story of tall frames, the curvature ductility required at the plastic hinges if a column sidesway mechanism develops is not particularly high.[3] Hence, for one- and two-story frames, and in the top story of taller frames, a weak column-strong beam concept can be permitted.[4,5,6] This approach would protect the composite beams from damage during seismic motions.

However, for tall frames where a strong column-weak beam concept is necessary, the composite beams will need to be designed for adequate ductility. Seismic design considerations for moment resisting frames when plastic hinges form in the composite beams are discussed in the following sections.

Flexural Strength of Beams

The critical section for flexure in beams in moment resisting frames subjected to gravity and seismic loading is at or near the beam ends. In frames where gravity loading effects dominate, the critical sections for positive moment due to gravity plus seismic loading may occur in the beams away from the column faces. The critical negative moment sections will always occur at the beam ends. A distinctive feature of the behavior of the composite beam-column connection shown in Fig. 1 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.

The negative moment flexural strength at the end of the composite 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-place column concrete (see Fig. 5b). 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-place 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-place reinforced concrete core alone. The negative moment flexural strength away from the ends will be that due to the composite section.

The positive moment flexural strength at the end of the beam will be provided only by the longitudinal reinforcement and the cast-in-place concrete in the beam core and slab topping (see Fig. 5a). 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.

Hence, the dependable negative and positive flexural strengths of the composite beam at the beam ends should be taken as that provided only by the cast-in-place reinforced concrete beam core.

Plastic Hinge Behavior of Beams

The length of the plastic hinge region in the 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.[3] 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.

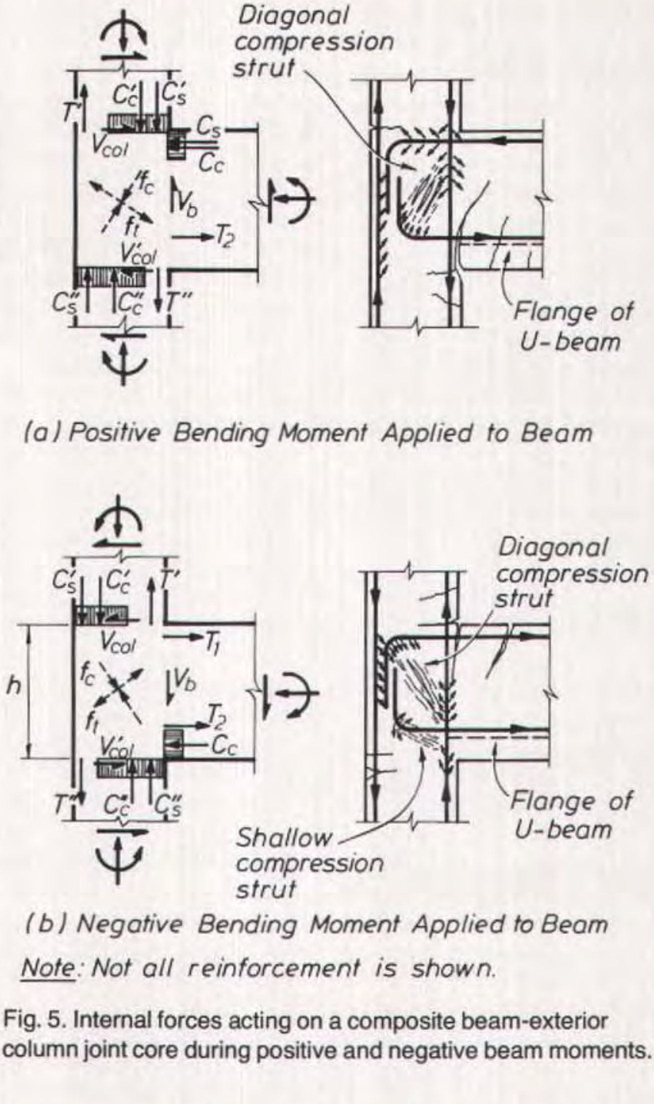
In the composite system considered here, in which there is no connection by steel between the end of the precast prestressed U-beam and the column, the length of the region of reinforcement yielding at the end of the composite beam when the bending moment is positive will be less than for a beam in a 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.

If the flexural cracking in the beam during positive bending moment does concentrate at the column face, the consequence would be higher beam curvatures in the plastic hinge region than for conventional reinforced concrete members. Hence, the concrete there would be subjected to high localized compressive strains and the longitudinal reinforcement in the beam there would suffer high localized plastic tensile strains 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 behavior during positive moment have resulted in reservations being expressed by some designers about the performance of this type of composite beam when required to act as primary energy dissipating members during seismic loading.

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

One possible approach, aimed at improving the plastic hinge behavior during positive moment, would be to construct a composite beam in such a manner that in the potential plastic hinge re-



(a) Positive Bending Moment Applied to Beam

(b) Negative Bending Moment Applied to Beam

Note: Not all reinforcement is shown.

Fig. 5. Internal forces acting on a composite beam-exterior column joint core during positive and negative beam moments.

gions at the ends of the beam the bond at the interface between the precast prestressed U-beam and the cast-in-place 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-place 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.

In the plastic hinge regions of the beams the reinforced concrete cast-in-place core should have longitudinal and transverse reinforcing steel which is detailed according to the seismic design provisions for reinforced concrete ductile frames. This typically means a limitation on the maximum area of tension steel, the presence of compression steel with an area of at least one-half of the area of tension steel, and stirrup ties with a close spacing so as to confine the compressed concrete and to prevent buckling of longitudinal bars and shear failure.

In the New Zealand concrete design code[5] the spacing of stirrup ties in the potential plastic hinge regions of beams is required not to exceed the smaller of one-quarter of the effective depth of the beam or six longitudinal bar diameters or 150 mm (6 in.). The potential plastic hinge region is taken to extend over a length equal to twice the overall beam depth.

Shear Strength of Beams

In the plastic hinge regions at the ends of composite beams the cast-in-place 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-place concrete breaks down or if the bond is intentionally eliminated. Therefore, the beam core should be designed to have sufficient transverse reinforcement to resist the design shear force, using the seismic design provisions for reinforced concrete ductile frames. Away from the ends of the beam, the whole composite section may be considered to provide shear resistance.

In order to avoid a shear failure, and hence to ensure that ductile plastic hinging of the beams can occur during severe seismic loading, the design shear force for the beams should be that associated with the likely beam overstrength in flexure. To calculate 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 member, only the cast-in-place 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.

The stirrup ties provided in the potential plastic hinge regions of the beams should be capable of resisting the entire design shear force by truss action alone, since the shear carried by the concrete, V_c, diminishes during severe cyclic loading. That is, V_c tends to zero due to a breakdown in the shear transferred by dowel action of the longitudinal bars, by aggregate interlock, and across the compression zone.

Interface Shear Transfer Between Precast U-Beam and Cast-in-Place 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-place 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-place concrete core by the U-beam webs resisting dilation caused by relative shear movement 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 contact surfaces of the U-beam and the cast-in-place concrete core are the summation of stresses from a number of sources. The imposed horizontal shear stresses at the interface of contact surfaces between the U-beam and cast-in-place concrete core during positive bending moment arise from the transfer of the prestressing steel tension force from the U-beam to the core, and during negative bending moment arise from the transfer of the reinforcing steel force from the core to the U-beam flange.

The horizontal interface shear stress could be found from $V_u/b_v d$, where

V_u = vertical shear force at factored (ultimate) load

b_v = total width of interface (two sides plus bottom surface) and

d = effective depth of composite section

This is a simplistic approach to the more complex real behavior of the U-shaped interface.

The imposed vertical shear stresses at the interface during service loading arise from the superimposed live loads being supported by the floor system. The service live loads need to be transferred from the U-beam unit, on which the floor system is seated, to the cast-in-place concrete core by vertical shear stresses across the interface (see Fig. 2). The self weight of the U-beam unit, the precast concrete floor system, and the cast-in-place concrete core and floor topping during service loading are carried by the U-beam alone and therefore will not cause vertical shear stresses at the interface.

However, the transfer of vertical shear stresses across the interface will be more critical at the factored (ultimate) load if the end support of the U-beam in the column cover concrete is lost during seismic loading. In that case the vertical shear stresses will arise from the self weight of the U-beam, the precast floor system and the cast-in-place concrete core and floor topping, as well as from the live loads. This more critical case at ultimate should be used to determine the design vertical shear stress at the interface.

In the New Zealand concrete design code[5] an interface shear of 0.55 MPa (80 psi) is permitted at the factored (ultimate) load for interfaces that have no cross ties, but have the contact surfaces cleaned and intentionally roughened to a full amplitude of 5 mm (0.2 in.). A design approach to check interface shear transfer when the inside face of the precast U-beams have been so roughened would be to find the vector sum of the imposed design horizontal and vertical shear stresses at the interface at the factored (ultimate) load and to ensure that it is less than 0.55 MPa (80 psi).

Columns

Seismic design provisions for reinforced concrete ductile frames should be used to determine for the columns the longitudinal reinforcement required for flexure and axial load, and the transverse reinforcement required for shear, concrete confinement and restraint against buckling of longitudinal bars.

For tall frames a strong column-weak beam concept is adopted, in order to prevent as far as possible a column sidesway mechanism (soft story) from occurring during a major earthquake. Hence, the column bending moments found from elastic frame analysis for the code factored (ultimate) load combinations need to be amplified to give a higher column design moment, to take into account the likely beam overstrength in flexure, the higher mode effects of dynamic loading which can cause much higher column moments than calculated from code static loading, and the possible effect of seismic loading acting along both principal axes of the building simultaneously.[3,4,5,6]

Similarly, the column shear forces found from elastic frame analysis for the code factored (ultimate) load combinations need to be amplified to give a higher design shear force so as to avoid the possibility of brittle shear failure of the columns. Transverse reinforcement in the column ends is also necessary to provide flexural ductility there, since the amplified column design moments may not be sufficiently high to eliminate the possibility of some column hinging. In particular, a transverse bar spacing of not more than six times the longitudinal bar diameter to prevent premature buckling of compression steel is an important requirement.

Beam-Column Joints

The design shear forces for the beam-column joint cores can be based on the overstrength internal forces from beams.

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. 5b.

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-place concrete core alone and the joint core behavior will be that of a conventional reinforced concrete frame.

During positive bending moment, the cast-in-place reinforced concrete alone transfers the beam forces to the beam-column joint core. Hence, for positive moment in the beam the joint core behavior is that of a conventional reinforced concrete frame. A diagonal compression strut mechanism which transfers part of the joint core forces is shown in Fig. 5a.

It is apparent that the code approach for the design of cast-in-place reinforced concrete beam-column joints could be used ignoring forces from possible composite beam action. That is, the design horizontal joint core shear forces could be found for the cast-in-place concrete beam acting alone. This assumption is obvious for positive beam moments but is an approximation for negative moments. However, for negative beam moments the upper layers of bars introduce the horizontal joint core shear force over the greatest 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. 5b if those bars are in tension.

The joint core mechanism resisting the applied forces is made up partly by the diagonal compression strut mechanisms 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]

TEST PROGRAM

Three full-scale composite beam-exterior column units have been tested[2] to assess the seismic performance characteristics of the composite frame system described. The overall dimensions of the units are shown in Figs. 6, 9 and 10. For ease of construction of the units, the T-beam flanges typically resulting from the presence of the cast-in-place concrete floor topping were not modelled.

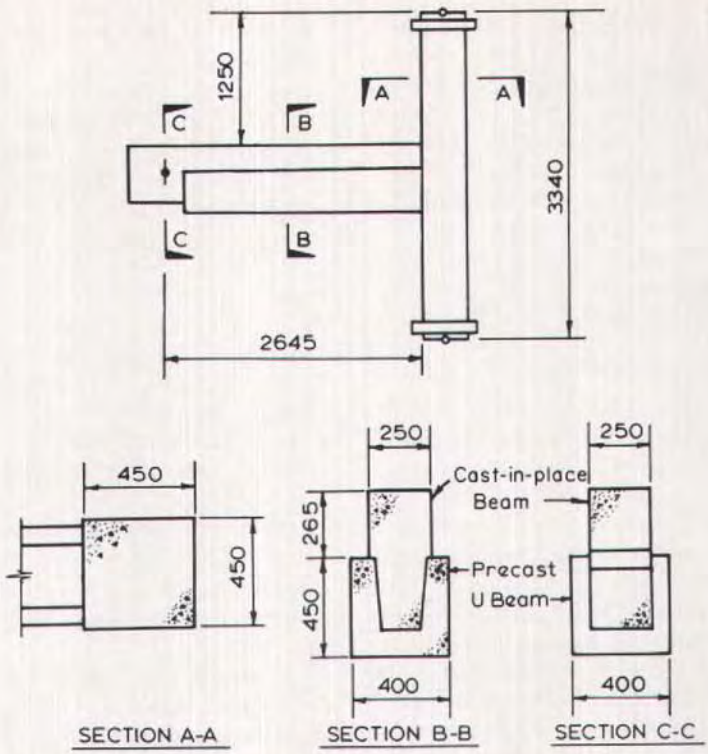


Fig. 6. Elevation and sections of test units (dimensions in mm).
Note: 1 mm = 0.0394 in.

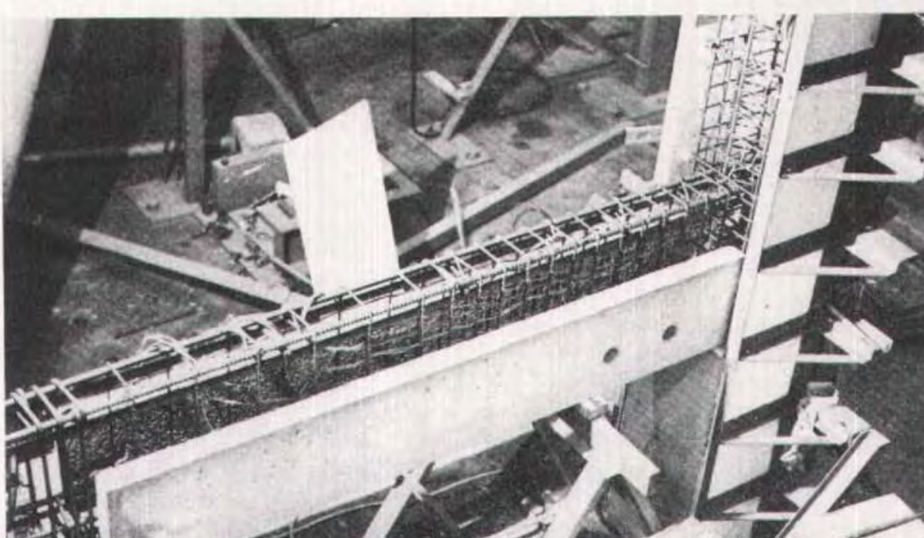


Fig. 7. View of beam and column reinforcement in place during construction of Unit 1.

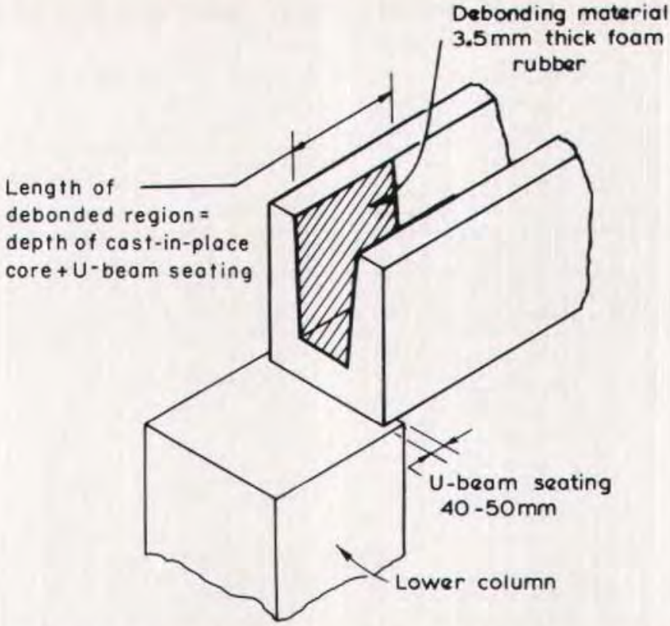


Fig. 8. Method of debonding potential plastic hinge region of Unit 3.

All units were designed using the New Zealand concrete design code[5] with the addition of the suggested supplementary seismic design recommendations where necessary as discussed in the previous section. The strength reduction factors were taken as $\phi = 1$ in all calculations and the overstrength factor for the longitudinal beam reinforcement, used for the calculation of the design shear forces, was taken as 1.25.

Details of Test Units

Unit 1 was detailed using code provisions for seismic loading, with a potential plastic hinge region in the beam adjacent to the column face. Unit 2 was not detailed for seismic loading, being designed using code provisions for gravity loading only. Unit 3 was detailed using code provisions for seismic loading and was identical to Unit 1 in all respects except that the interface between the precast U-beam and cast-in-place concrete core in the potential plastic hinge region was deliberately debonded in an attempt to improve the plastic hinge behavior. The details of the reinforcement in all units is shown in Figs. 9 and 10.

The interior surfaces of all precast U-beams had been roughened with an amplitude typically of 3 mm (0.12 in.). This surface roughness was achieved by chemical retarding of the interior surface after initial set and then removal of the surface cement paste from around the aggregate by washing with water and wire brushing.

The in-place concrete of the units was cast in the same orientation as for a prototype structure and according to anticipated site practice. There were two pours of in-place concrete for each unit. First, the lower column was cast up to the height where the precast U-beam would be seated on it. The precast U-beam was placed on the edge of the top surface of this column pour when concrete strength was gained (see Fig. 7). In

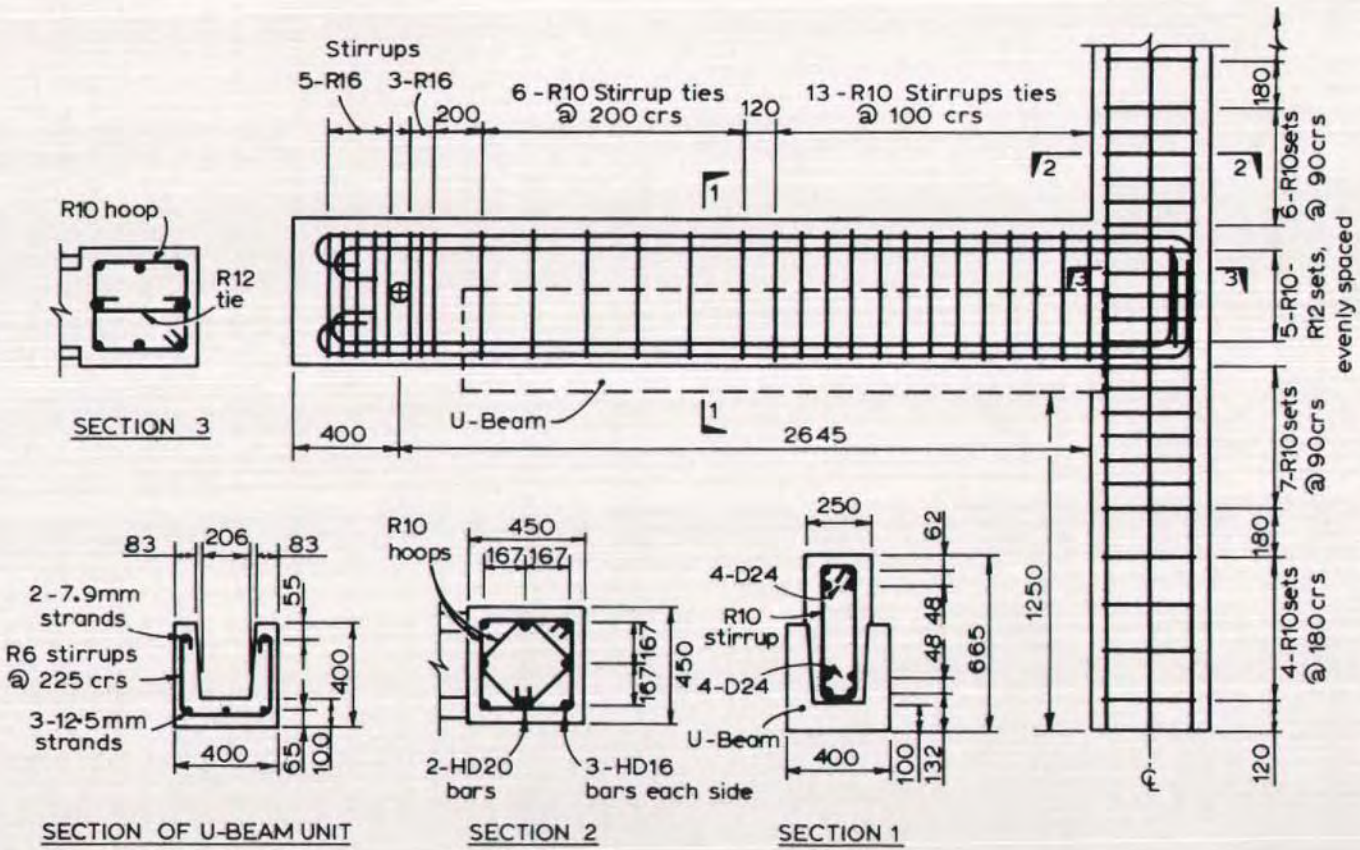


Fig. 9. Details of reinforcement of Units 1 and 3 (1 mm = 0.0394 in.).

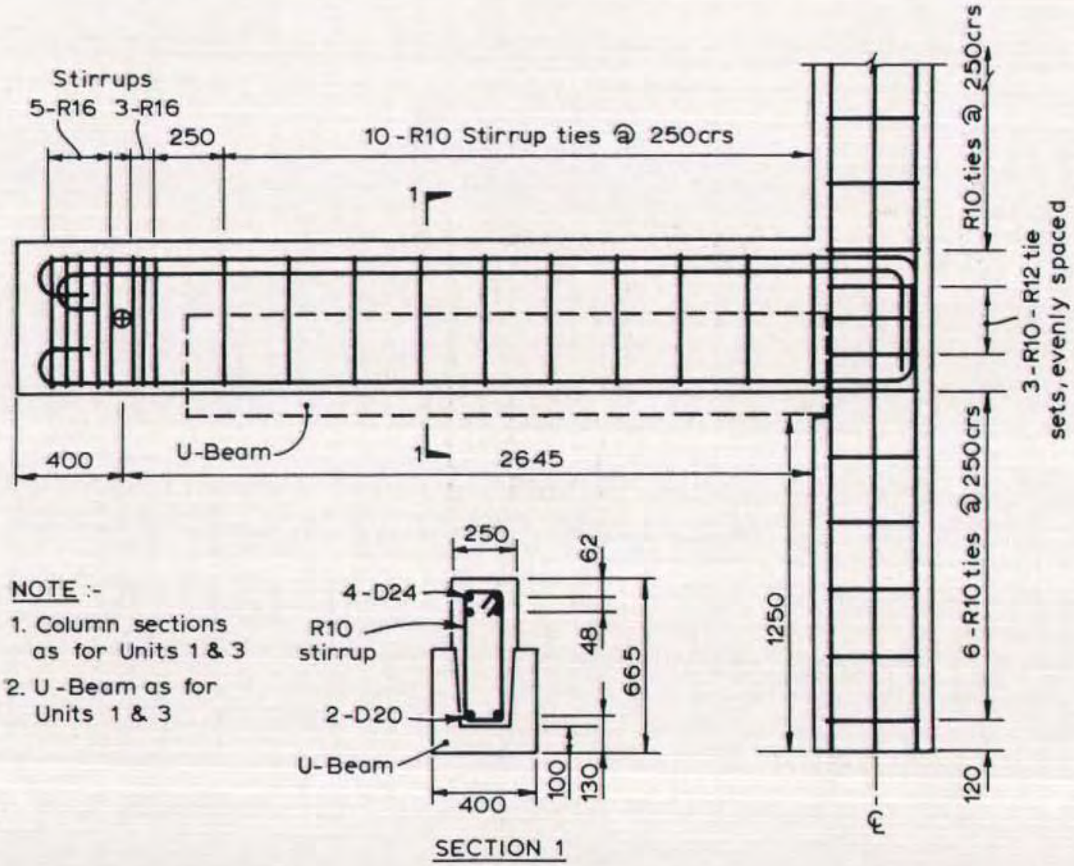


Fig. 10. Details of reinforcement of Unit 2 (1 mm = 0.0394 in.).

Table 1. Measured Properties of Steel and Concrete.

Reinforcing Steel

Bar designation	R10	R12	D20	D24	HD16	HD20
f_y MPa	336	311	285	308	402	444
f_{su} MPa	467	463	437	469	789	704

Note: R = plain round intermediate strength steel, D = deformed intermediate strength steel, HD = deformed high strength steel, No. = bar diameter in mm.

Seven-Wire Prestressing Strand

Diameter	7.9 mm	12.5 mm
0.2 percent proof stress MPa	1808	1678
f_{pu} MPa	1926	1793

Concrete Cylinders: f_c' at Day of Testing Unit, MPa

Unit	Lower column	Upper column and beam	U-beam
1	30.1	33.6	50.2
2	24.2	29.2	54.6
3	28.3	26.1	53.0

Note: 1 mm = 0.0394 in.; 1 MPa = 145 psi.

the next pour the beam core, beam-column joint and upper column were cast. Each unit was damp cured for not less than 7 days. The precast U-beams were supplied by a precasting firm.

The debonded plastic hinge region in the beam of Unit 3 was achieved by fixing a 3.5 mm (0.14 in.) thick sheet of foam rubber to the inside face of the precast U-beam, over a length equal to the depth of the cast-in-place core, before casting the in-place concrete core. Fig. 8 shows the details of the debonded region.

Intermediate strength steel with a specified yield strength of 275 MPa (40,000 psi) was used for the beam longitudinal reinforcement and all transverse reinforcement. High strength steel with a specified yield strength of 380 MPa (55,000 psi) was used for the column longitudinal reinforcement. The measured concrete and steel strengths are shown in Table 1.

The steel ratio provided by the top longitudinal reinforcement in all the beams was 1.83 percent based on the b and d dimensions of the beam core. The steel ratio provided by the bottom longitudinal reinforcement in the beams was 1.42, 0.47 and 1.42 percent for Units 1, 2 and 3, respectively, based on the b and d dimensions of the beam core.

When the level of column axial load was $0.1 A_g f_c'$ where f_c' is the concrete compressive cylinder strength and A_g is the gross area of the column, 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, where the ideal flexural strength is that calculated using the actual strengths of the steel and concrete and assuming a strength reduction factor ϕ of 1.0. The Commentary of the New Zealand concrete design code[5] requires the design column bending moments to be at least 1.8 times the bending moments found from elastic frame analysis for the code factored (ultimate) load combinations.

The Test Rig

Figs. 11 and 12 show the loading arrangements and test rig. By alternating the directions of the load at the end of the beam, earthquake loading was simulated. The loading cycles were applied statically. The axial load on the column during the tests was held constant at $0.1f_c' A_g$. Superimposed dead loads were also applied to the beams to represent a 200 mm (7.9 in.) thick precast concrete voided slab floor with 65 mm (2.6 in.) thick concrete topping spanning between frames in the prototype building at 6.6 m (21.6 ft) centers. The positioning of the superimposed dead load was organized 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.

The New Zealand general design and loadings code[4] specifies that the performance of a ductile structural assembly is satisfactory for earthquake resistance if it retains at least 80 percent of its strength after being subjected to a minimum of four cycles of lateral loading to a displacement ductility factor of four in each direction. The displacement ductility factor μ is defined as the ratio of the maximum displacement Δ to the displacement at first yield Δ_y.

Note that in these tests, Δ_y was taken as 1.33 times the beam end deflection measured at three-quarters of the theoretical flexural strength of the unit. This definition of Δ_y, although arbitrary, gives a convenient reference first yield displacement which assumes elastic behavior up to ultimate, as is also assumed in many dynamic analyses of the earthquake response of structures.

In these tests a gradual increase in the imposed displacement ductility factor $\mu = \Delta/\Delta_y$ was chosen, to enable the behavior of the units to be examined at various ductility levels. The displacement controlled load cycles consisted of two cycles to $\mu = \pm 1$, four cycles to $\mu = \pm 2$, four cycles to $\mu = \pm 4$, and two cycles to $\mu = \pm 6$.

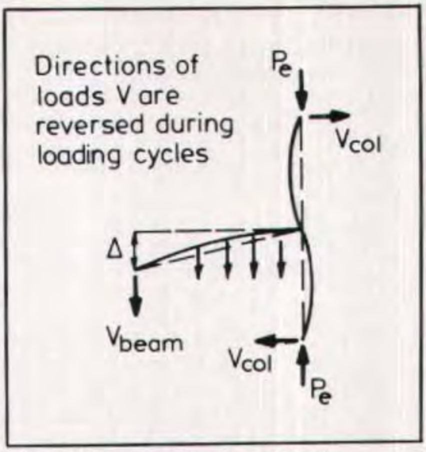


Fig. 11. External loads applied.

Test Results

The beam end load versus beam end deflection hysteresis loops measured for the units are shown in Fig. 13. In the figure the dashed lines marked $+P_i$ and $-P_i$ represent the loads at the theoretical ideal flexural strengths based on the beam core alone, and $-P_i'$ is the load at the theoretical ideal flexural strength based on the composite section, calculated using the measured material strengths and assuming a strength reduction factor ϕ of 1.0.

All theoretical ideal flexural strength calculations were conducted using the strain compatibility-equilibrium approach, incorporating the measured stress-strain curves for the prestressing steel and the reinforcing steel and assuming a rectangular concrete compressive stress block as in the ACI Building Code[7] and an extreme fiber concrete

Table 2. Ratio of Maximum Measured Beam Moments to the Theoretical Ideal Flexural Strengths of the Beams.

Unit	Positive moment	Negative moment	
	$\frac{M_{max}}{M_i}$	$\frac{M_{max}}{M_i}$	$\frac{M_{max}}{M_i'}$
1	1.12	1.33	0.90
2	1.09	1.43	1.03
3	1.03	1.04	0.75

M_i = ideal flexural strength of cast-in-place core alone

M_i' = ideal flexural strength of composite section (Both calculated using actual material strengths and $\phi = 1$)

compression strain of 0.003. Table 2 shows the maximum moments reached in the loading cycles as a percentage of the theoretical ideal flexural strengths. It is evident that in design the dependable flexural strength of the beams at the column face should be based on the cast-in-place concrete core alone.

Views of the units at the end of testing are also shown in Fig. 13. At the final stages of testing there was spalling of concrete at the column face in the region of the beam seating. This damage could have been prevented if a gap had been formed between the sides and bottom of the precast U-beam and the cast-in-place column concrete.

Units 1 and 3

Fig. 13a and c show that Units 1 and 3, which were designed for seismic loading, exhibited very satisfactory strength and ductility characteristics. 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-place reinforced concrete core within the precast concrete U-beam, even during positive bending moment. The strain readings recorded by electrical resistance strain gauges on the longitudinal beam reinforcement indicated that at the completion of the test the length of beam over which yielding of the tension steel occurred was about one-half of the depth of the cast-in-place concrete core in the case of positive moment and slightly greater than the depth of the core for negative moment. 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 measured plastic hinge length in the cast-in-place concrete core and the precast concrete U-beam was not damaged during the testing. Although both Units 1 and 3 displayed satisfactory ductile behavior during seismic loading, it may be considered that the debonded construction used in Unit 3 is to be preferred if it is considered important to reduce the damage occurring to the precast concrete U-beam shell during severe seismic loading.

No ill effects from beam shear were observed in Units 1 and 3. The maximum applied nominal shear stress in the plastic hinge region of the beams, assuming the beam core carried all the shear, was $V_{max}/b_w d = 0.28\sqrt{f_c'}$ MPa ($3.4\sqrt{f_c'}$ psi) for Unit 1 and $0.25\sqrt{f_c'}$ MPa ($3.0\sqrt{f_c'}$ psi) for Unit 3. The stirrup ties in the beam cores of these two units were capable of carrying a nominal shear stress by truss action of $A_v f_y/b_w s = 0.41\sqrt{f_c'}$ MPa ($4.9\sqrt{f_c'}$ psi) for Unit 1 and $0.47\sqrt{f_c'}$ MPa ($5.6\sqrt{f_c'}$ psi) for Unit 3.

The transverse steel in the potential plastic hinge regions of the beams was governed by spacing required by the code ductility provisions. The peak stress reached by the beam-column joint core shear reinforcement, measured by electrical resistance strain gauges, was 83 percent of the yield strength for Unit 1 and one hoop reached yield in Unit 3.

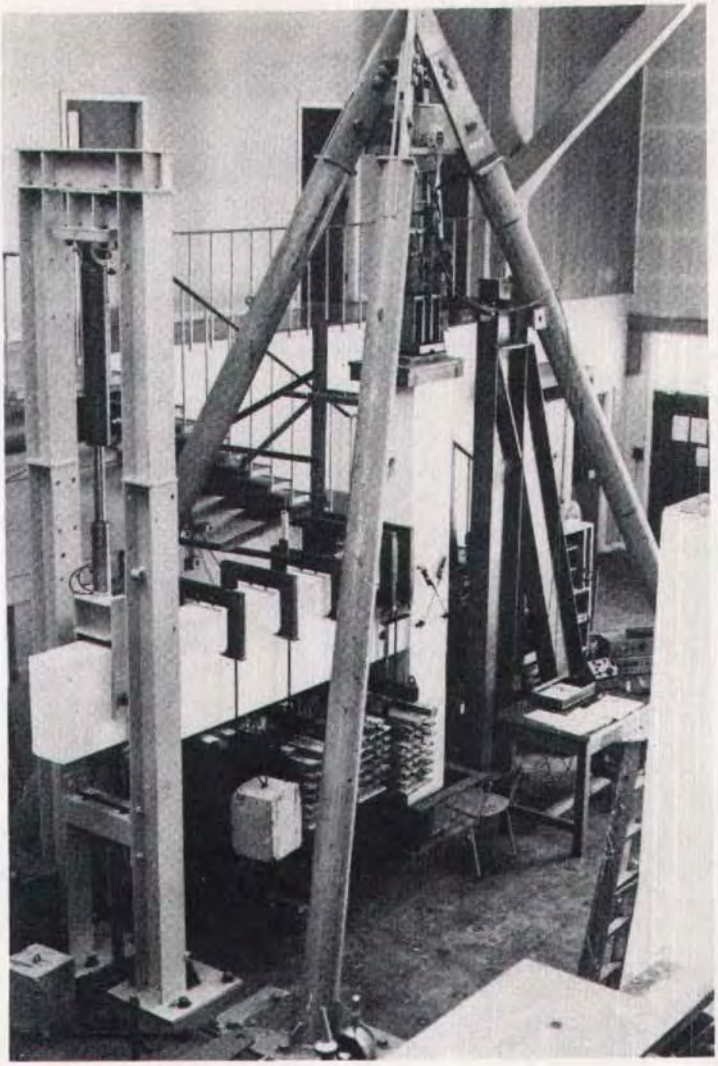
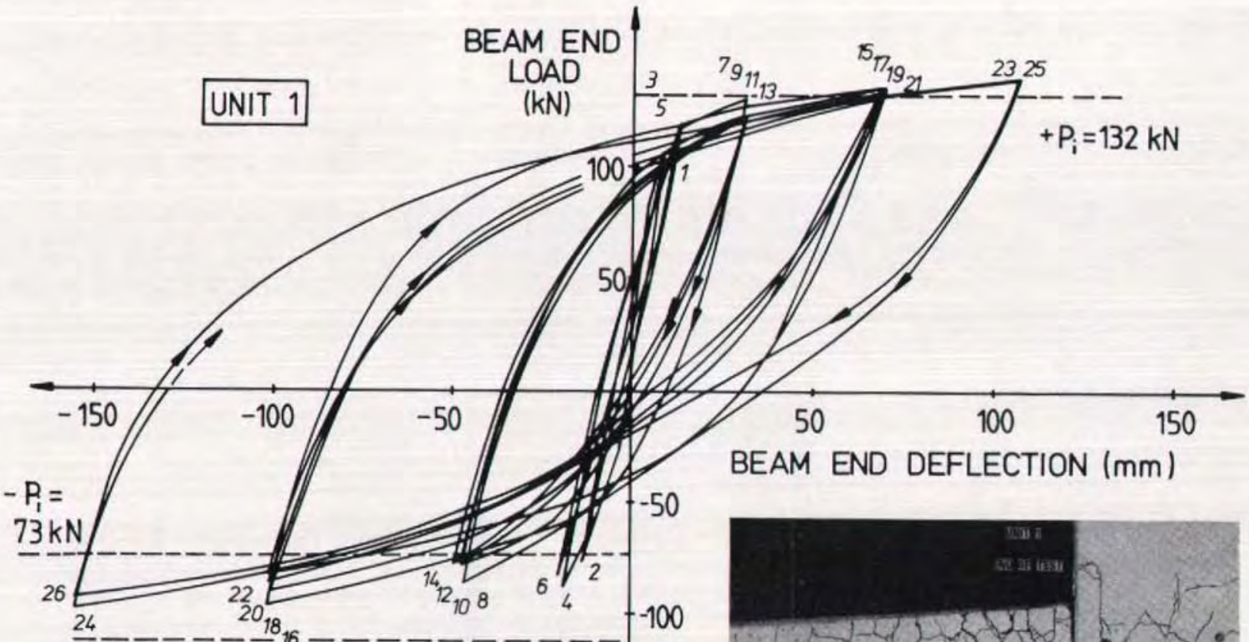


Fig. 12. View of test rig.

Diagonal tension cracks were observed in the joint cores (see Fig. 13a and c). The columns of both units remained in the elastic range with limited cracking during the tests.

Unit 2

Unit 2 was not designed for seismic loading. That is, the potential plastic hinge region was not detailed with closely spaced stirrup ties for ductility. Extensive sliding shear displacements occurred along the vertical cracks in the beam at the face of the column as the test progressed, which was the main reason for the pinched hysteresis loops with low included area (see Fig. 13b). In Unit 2 the spacing of stirrup ties in the potential plastic hinge region was 250 mm (9.8 in.) compared with the 100 mm (3.9 in.) spacing used in Units 1 and 3.



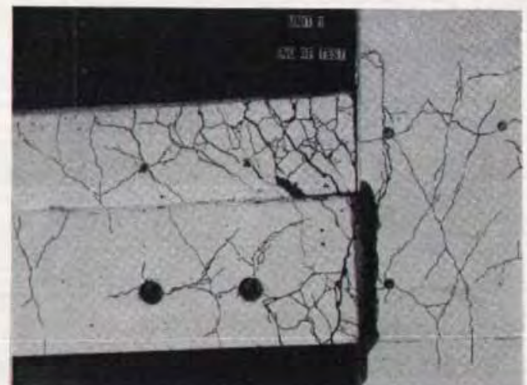


Fig. 13a. Beam end load versus beam end deflection and view of units at end of testing (1 kN = 224 lb, 1 mm = 0.0394 in.).

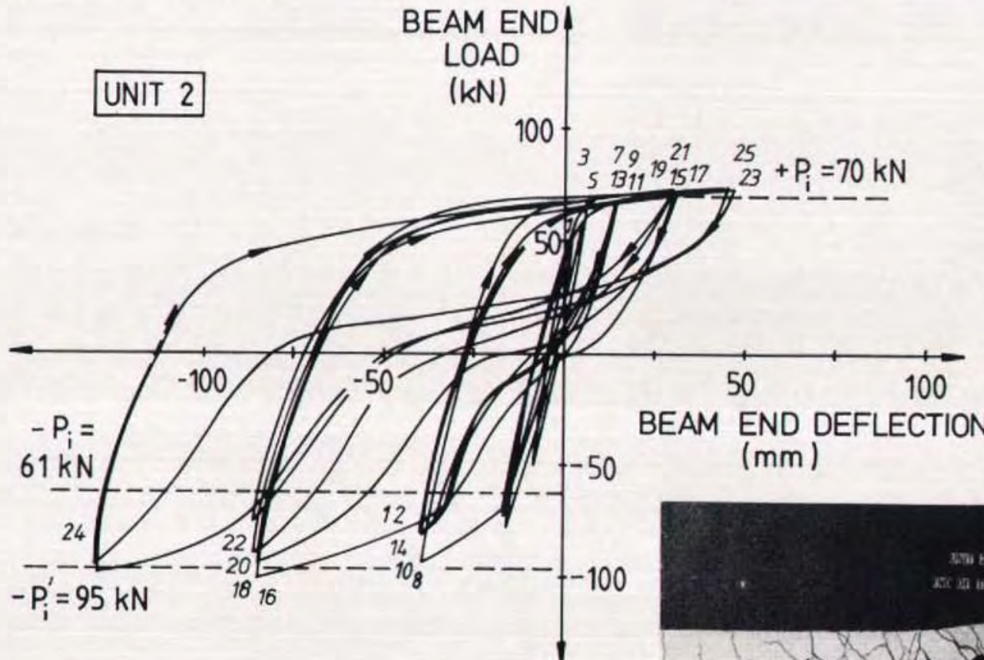


Fig. 13b. Beam end load versus beam end deflection and view of units at end of testing (1 kN = 224 lb, 1 mm = 0.0394 in.).

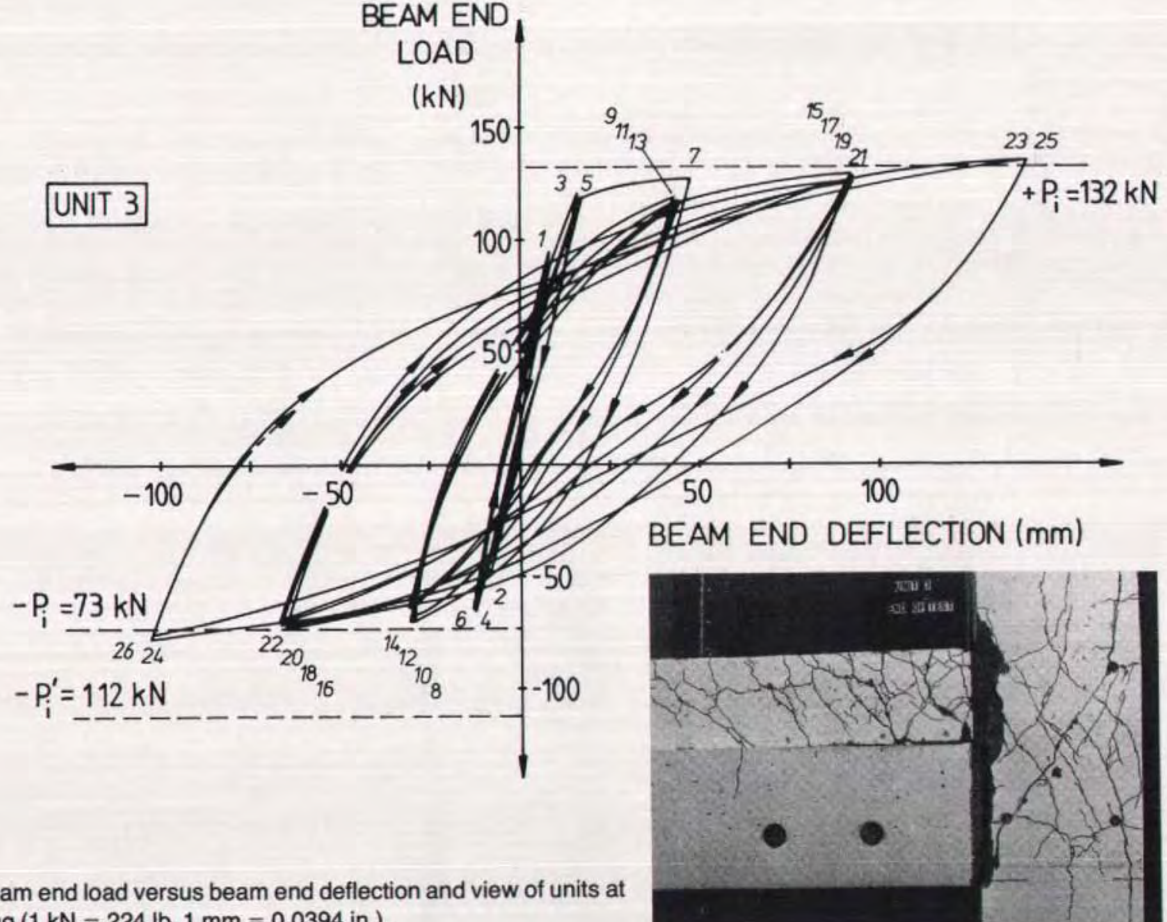


Fig. 13c. Beam end load versus beam end deflection and view of units at end of testing (1 kN = 224 lb, 1 mm = 0.0394 in.).

Evidently, that spacing in Unit 2 was too large to assist dowel action of the longitudinal beam bars to prevent sliding shear from occurring at the column face.

The maximum applied nominal shear stress in the plastic hinge region of the beam, assuming that the beam core carried all the shear, was $0.29\sqrt{f_c'}$ MPa ($3.5\sqrt{f_c'}$ psi). The stirrup ties provided were capable of resisting a nominal shear stress of $0.18\sqrt{f_c'}$ MPa ($2.2\sqrt{f_c'}$ psi) by the truss mechanism. Hence, the remaining nominal shear stress in the plastic hinge region needed to be carried by the concrete shear resisting mechanisms.

Also, in Unit 2 the area of longitudinal reinforcement in the bottom of the cast-in-place concrete core was only 0.47 percent of the core bd, where b is the beam core width and d is the beam core effective depth. The positive moment flexural strength of the beam core at the column face was insufficient to cause cracking of the precast U-beam when acting compositely with the core in the region adjacent to the column face. As a result, in Unit 2 the plastic rotation in the beam during positive bending moment was undesirably localized at a single crack at the column face.

It is evident that the steel ratios of the bottom reinforcement in the core should be higher than used in Unit 2 (it was 1.4 percent of the core bd in Unit 1) in order to improve the plastic hinge behavior. This points to the importance of having sufficient longitudinal reinforcement in the bottom of the cast-in-place concrete core to cause the precast part of the composite beam to crack during positive bending moment if bond exists between the U-beam and the core.

The hoops in the beam-column joint core remained in the elastic range during the tests. Diagonal tension cracking of the joint core occurred only during negative moment loading, since the positive moment joint core shears were too small to cause cracking. The column remained in the elastic range with limited cracking during the tests.

CONCLUSIONS

The theoretical considerations and test results from three full-scale beam-exterior column test units subjected to simulated seismic loading lead to the following conclusions:

1. A convenient building system for seismic resisting frames involves the combination of precast prestressed concrete U-beams acting compositely with cast-in-place reinforced concrete beam cores. The incorporation of precast concrete elements has the advantage of high quality control and speed of construction, and the cast-in-place reinforced concrete provides the structural continuity and ductility necessary for adequate seismic performance. Special attention to detailing in design is necessary to ensure that composite frames do perform in a ductile manner during severe seismic loading.

2. Code provisions do not cover all aspects of the design for ductile behavior under seismic loading for this type of construction. Proposals can be made for additional design recommendations where necessary to take into account the presence and directional influence of the precast prestressed concrete U-beam during severe seismic loading. A summary of design recommendations is given in Appendix A and a design example is provided in Appendix B.

3. When the composite beam is formed from a precast prestressed concrete U-beam and a cast-in-place reinforced concrete core, and when the pretensioned strand in the U-beam is terminated at the beam end and is not anchored in the reinforced concrete column, the following conclusions from the test results with regard to seismic performance were reached:

(a) During seismic load reversals

bond deterioration at the interface of the cast-in-place concrete core and the precast concrete U-beam, along with cracking of the precast concrete U-beam, allow yielding of the longitudinal beam reinforcement to spread along a reasonable length of the cast-in-place beam core. Hence, an undesirable concentration of plastic hinge rotation does not occur at the beam crack at the column face and satisfactory ductile behavior is achieved. However, a sufficient quantity of longitudinal reinforcement is required in the bottom of the cast-in-place beam core in order to ensure that the flexural strength of the beam core is great enough to cause the composite section to crack away from the beam end, thus permitting this desirable spread of yielding during positive bending moment.

(b) Deliberate debonding of the interface of the cast-in-place and precast concrete in the potential plastic hinge region results in a larger potential plastic hinge length in the cast-in-place core and no damage to the precast concrete U-beam during seismic loading. However, since satisfactory ductile behavior can be obtained without debonding the potential plastic hinge region, the extra cost of the debonding operation can only be justified if damage during severe seismic loading is unacceptable.

(c) The dependable flexural strengths of the composite beams for both positive and negative bending moment at the column face were found to be that given by the cast-in-place reinforced concrete beam core alone. The maximum flexural strengths required for beam overstrength considerations, for the determination of the design shear force, were found to be given by the composite beam for negative moment and by the cast-in-place reinforced concrete beam core alone for positive moment.

(d) Close spacing of stirrup ties in the potential plastic hinge region of the beam near the column face was essential to prevent sliding shear of the beam along a vertical crack at the column face. A spacing of stirrup ties in the beams of $0.44h_c$ was too large since it resulted in serious sliding shear along the vertical crack at the column face, whereas a spacing of $0.18h_c$ prevented sliding shear, where h_c is the overall depth of the beam core.

(e) The design of shear in the potential plastic hinge zone of the beam should be based on shear carried by the beam core alone, and the stirrup ties should be capable of resisting the total shear force by truss action.

(f) The shear stresses at the interface between the precast concrete U-beam and cast-in-place concrete core should be checked. In the units designed for seismic loading the calculated imposed interface shear stress, assuming composite action at the ultimate load, was slightly in excess of 0.55 MPa (80 psi). The interior surface of the precast U-beams had been roughened with an amplitude of typically 3 mm (0.12 in.). The application of a limit of 0.55 MPa (80 psi) for the calculated interface shear stress outside the plastic hinge zone for these units appeared to result in satisfactory behavior.

4. A general conclusion from the tests is that Units 1 and 3 which were designed for seismic loading would indeed be satisfactory for use in ductile seismic resisting frames, but that Unit 2 which was designed without the special provisions for seismic loading would be suitable for nonseismic resisting frames in structures where seismic loads are carried by walls or other structural systems.

ACKNOWLEDGMENTS

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REFERENCES

1. "Karioi Mill," *New Zealand Concrete Construction*, V. 24, June 1980, pp. 18-21.
2. Park, R., and Bull, D. K., "Behaviour of Cast in Situ Reinforced Concrete Frames Incorporating Precast Concrete Beam Shells Subjected to Seismic Loading," *Bulletin*, New Zealand National Society for Earthquake Engineering, V. 17, No. 4, December 1984, pp. 223-250.
3. Park, R., and Paulay, T., *Reinforced Concrete Structures*, John Wiley and Sons, New York, N.Y., 1975, p. 769.
4. *Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1984*, Standards Association of New Zealand, Wellington, 1984, p. 100.
5. *Code of Practice for the Design of Concrete Structures, NZS 3101:1982*, Standards Association of New Zealand, Wellington, 1982, p. 127.
6. Park, R., "Ductile Design Approach for Reinforced Concrete Frames," *Earthquake Spectra*, Professional Journal of the Earthquake Engineering Research Institute, V. 2, No. 2, May 1986.
7. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)," American Concrete Institute, Detroit, Michigan, 1983, p. 111.

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NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by March 1, 1987.

APPENDIX A — SEISMIC DESIGN RECOMMENDATIONS

Seismic design provisions for cast-in-place reinforced concrete frames incorporating precast prestressed concrete U-beam shells, subjected to gravity and seismic loading, are not fully covered by design codes in New Zealand[5] or the United States.[7] Supplementary seismic design recommendations are made below where necessary to take into account the presence and directional influence of the precast concrete U-beam during severe seismic loading.

A1 Flexural Strength of Beams

The dependable flexural strength of the beams should be at least equal to that required by the factored (ultimate) gravity and seismic loads:

$$\phi M_i \geqslant M_u \qquad \text{(A1)}$$

where the strength reduction factor ϕ for flexure is 0.9, and the ideal (nominal) flexural strength should be calculated as follows:

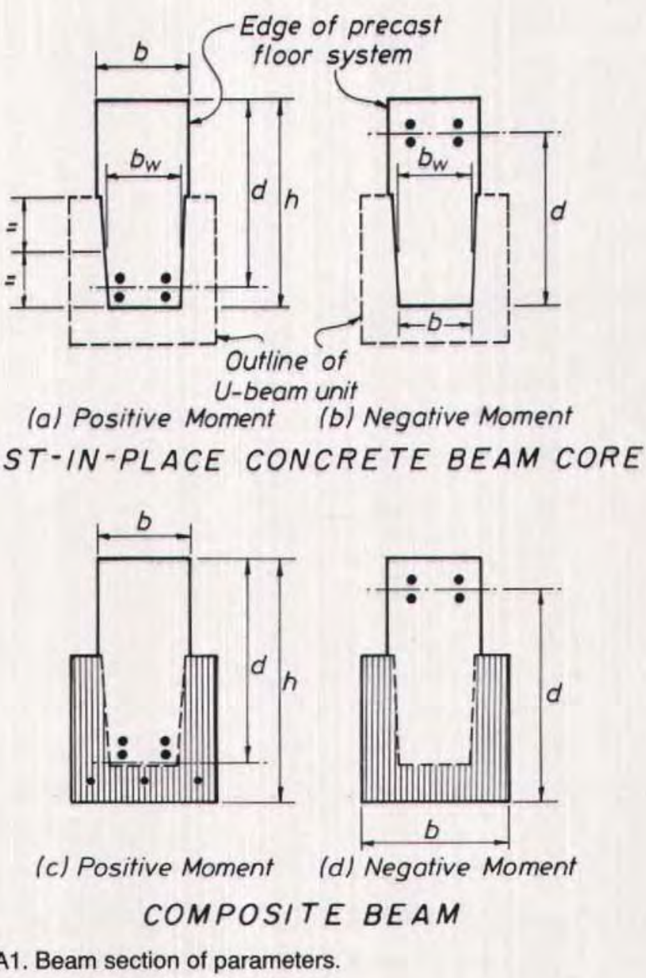

Fig. A1. Beam section of parameters.

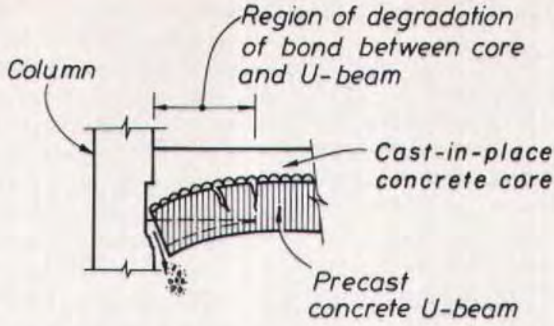

Fig. A2. Possible failure of precast concrete U-beam in debonded plastic hinge region of composite beam after loss of seating at cover.

(a) At or near the beam ends, M_i is based on the cast-in-place concrete beam core alone.

(b) Away from the beam ends, M_i is based on the composite section where the interface shear between the precast and cast-in-place concrete can be transferred satisfactorily and development length requirements of the prestressing strand are satisfied. The normal development length of the prestressing strand (approximately 150 strand diameters) should be increased by 200 mm (7.9 in.) to allow for some degradation of bond during reversals of severe seismic loading.

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. A1.

The precast U-beam should contain some top longitudinal steel so that it has adequate negative moment strength at the ends of the beam. This is to avoid possible failure of the type shown in Fig. A2 if the bond between the precast U-beam and the cast-in-place beam core, and the seating of the precast beam at the column face are lost.

A2 Length of Potential Plastic Region in Beams

In the design of beams that may form plastic hinges during seismic loading, the special detailing requirements for ductility specified by the code should extend over a potential plastic hinge region of length equal to twice the beam depth. This length 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.

A3 Shear Strength of Beams

The dependable shear strength of the beams should be at least equal to that required by the factored (ultimate) gravity loads:

$$\phi\, v_i\, b_w d \geqslant V_u \qquad \text{(A2)}$$

The strength reduction factor ϕ for shear is 0.85. The shear stress v_i is the total ideal shear stress resisted by the concrete mechanisms (v_c) and the truss mechanism of the shear reinforcement.

A further seismic design requirement is that the ideal shear strength should be at least equal to that associated with the overstrength beam moments and the factored gravity loads:[5]

$$v_i b_w d \geqslant V_u^o \tag{A3}$$

In the potential plastic hinge regions at the ends of the beams only the cast-in-place 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-place core and the dimension d is as defined in Fig. A1a and b.

In the potential plastic hinge region the shear reinforcement in the cast-in-place concrete beam core should be designed to resist all the shear force. That is, $v_c = 0$ should be assumed in this region, due to degradation of the concrete shear resisting mechanisms during reversals of severe seismic loading.

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. A1c and d could be used, and b_w can be taken as the width of the upper half of the cast-in-place beam core.

The determination of the shear force V_u^o is illustrated in Fig. A3 for the case where the plastic hinges form at the ends of the beam. The overstrength beam moments should be taken as:

$$M_u^o = \alpha M_i \tag{A4}$$

where $\alpha = 1.25$ includes the effect of strain hardening of reinforcement and the possibility that the actual yield strength is higher than specified, and M_i = ideal (nominal) flexural strength is based on either the cast-in-place concrete core for positive moment or the full composite section for negative moment. The use of these overstrength moments will ensure that the probability of shear failure is sufficiently low.

The transverse reinforcement in the potential plastic hinge regions should also be adequate to confine the compressed concrete and to prevent premature buckling of the compression reinforcement. In potential plastic hinge regions where moment reversal can cause the nonprestressed longitudinal bars to yield both in tension and compression at the top and bottom of the beam core, the maximum permitted center to center spacing of stirrup ties is the smaller of 150 mm (5.9 in.) or six longitudinal bar diameters or $d/4$, where conservatively d is the effective depth of beam core.[5]

Also, the yield force of the leg of the stirrup tie should be at least equal to one-sixteenth of the yield force of the compressed longitudinal bar or bars it is to laterally restrain multiplied by $(s/100)$, where s is the spacing of stirrup ties in mm (1 mm = 0.039 in.).

A4 Interface Shear Stresses of Beams

The composite action of the section is ignored in the potential plastic hinge region due to degradation of bond between the precast and in-place concrete interfaces during reversals of severe seismic loading. Nevertheless, the interface shear stresses should be checked to ensure that they are not excessive. The interface shear stress has components from horizontal and vertical shear stresses.

In general design the interface horizontal shear stress v_{dh} can be found from the factored (ultimate) shear force at the section using:

$$v_{dh} = \frac{V_u}{\phi b_v d} \tag{A5}$$

where d is the effective depth of the composite section and b_v is the total "width" of the interface.

As an approximation $b_v = 2h_i + b_i$, where h_i is the depth of near vertical inside face of the U-section and b_i is the width of horizontal inside face of the U-section. Note that this is a simplistic

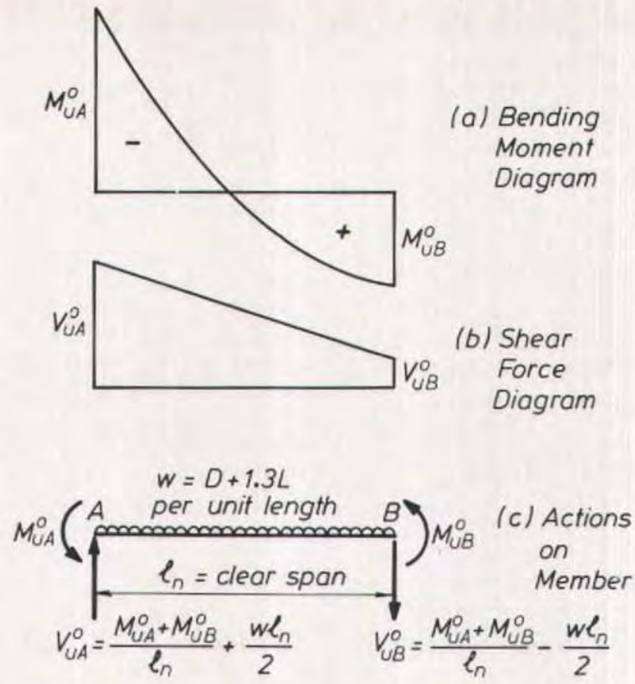

Fig. A3. Calculation of shear force with gravity and seismic loading.

approach to the more complex real behavior of the U-shaped interface. The strength reduction factor ϕ is 0.85.

In seismic design the interface horizontal shear stress can be found from:

$$v_{dh} = \frac{V_u^o}{b_v d} \tag{A6}$$

where V_u^o is the vertical shear force associated with the overstrength beam moments and the factored gravity loads.

The vertical shear stress at the interface, v_{dv}, originating from the U-beam and floor system dead weight and the imposed live loading supported by the floor system, should also be considered. 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-place concrete beam core (and then out to the columns supporting the beam core). In many cases the vertical shear stress due to gravity loading v_{dv} is small enough to be ignored.

The vector sum of the average horizontal shear stress v_{dh} and the vertical shear stress v_{dv} should not exceed the permitted value. For interfaces which have no cross ties but have the contact surfaces cleaned, are free of laitance, and are intentionally roughened to a full amplitude of 5 mm (0.2 in.), the permitted shear stress could be taken as 0.55 MPa (80 psi).

Then the shear stress requirement is:

$$\sqrt{v_{dh} + v_{dv}} \leq 0.55 \text{ MPa (80 psi)} \tag{A7}$$

A5 Columns and Beam-Column Joint Cores

The seismic design procedures used for ductile moment resisting reinforced concrete frames can be used for columns and beam-column joint cores.

APPENDIX B — DESIGN EXAMPLE

This Appendix contains a design example showing how the design recommendations can be used to design a typical beam in a cast-in-place moment resisting reinforced concrete frame incorporating precast prestressed concrete U-beam shells, subjected to gravity and seismic loading.

B1 The Structure

The beam to be designed is shown in Fig. B1. It is an interior beam of a one-way moment resisting frame with columns at 6.0 m (19.7 ft) centers in the direction of the frame. The one-way frames are placed at 7 m (23.0 ft) centers in the building. A typical section of the floor illustrating the concrete dimensions is shown in Figs. B2 and B3.

Concrete Strengths:

Precast U-beam and precast floor system: $f_c' = 40$ MPa (5.80 ksi).
Cast-in-place beam core and slab topping: $f_c' = 25$ MPa (3.63 ksi).

Steel Strengths:

Nonprestressed reinforcing steel:
$f_y = 275$ MPa (40 ksi).

Seven-wire prestressing strand of 12.5 mm (½ in.) diameter: f_{pu} = 1774 MPa (257 ksi).

Service Dead Load:

Element	Resulting load on beam
U-beam	3.1 kN/m (0.21 kips/ft)
Cast-in-place beam core	4.3 kN/m (0.29 kips/ft)
Precast floor plus cast-in-place topping, 4.1 kPa (85.2 lb/ft²)	27.7 kN/m (1.90 kips/ft)
Services and ceiling, 0.75 kPa (15.6 lb/ft²)	5.1 kN/m (0.35 kips/ft)
Total	40.2 kN/m (2.76 kips/ft)

Service Live Load

1.9 kPa (39.5 lb/ft²)	13.3 kN/m (0.91 kips/ft)

Seismic Load

From the results of elastic frame analysis, for the horizontal design seis-

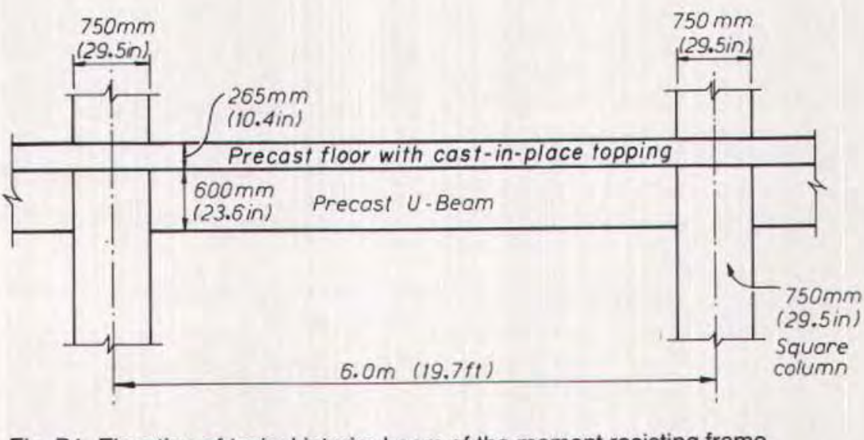

Fig. B1. Elevation of typical interior beam of the moment resisting frame.

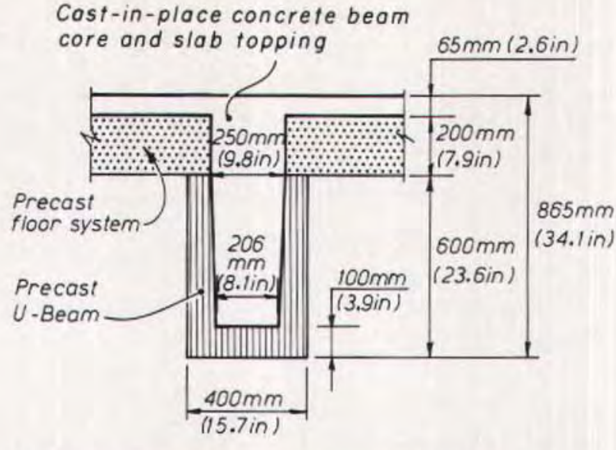

Fig. B2. Typical section of composite beam and floor (reinforcement not shown).

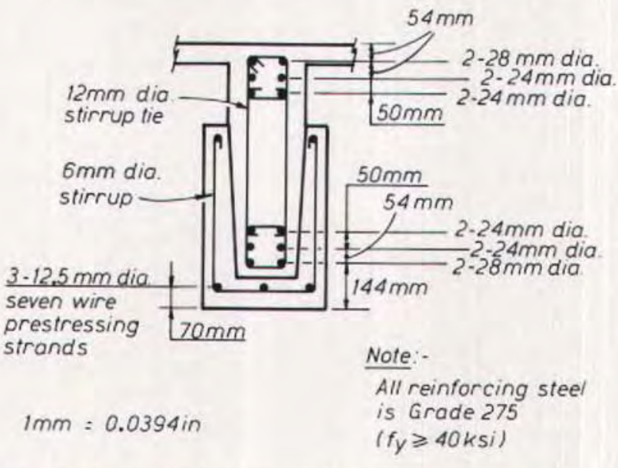

Fig. B3. Details of reinforcement and prestressing steel in precast U-beam and cast-in-place concrete beam core.

mic forces applied to the frames, the bending moments in the interior beam at the column centerlines are found to be ± 477 kN-m (352 kip-ft).

B2 U-Beam Supporting Dead Loads

The precast U-beam during the construction of the structure supports its own dead load plus the dead load of the precast floor system plus the dead load of the fresh cast-in-place concrete during construction, totalling:

$D = 3.1 + 4.3 + 27.7$
$= 35.1$ kN/m (2.40 kips/ft)

The span will be propped during construction (see Fig. B4). The span between props is:

$l = 6.00 - 2 \times 0.625$
$= 4.75$ m (15.6 ft)

The maximum service load moment at

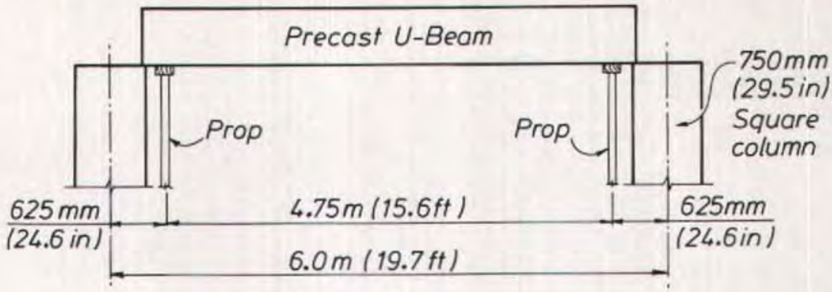

Fig. B4. Elevation showing precast U-beam positioned on columns before precast floor and cast-in-place concrete is poured.

midspan of the simply supported span is:

$M_{max} = Dl^2/8$
$= 35.1 \times 4.75^2/8$
$= 99.0$ kN-m (73.1 kip-ft)

The minimum service load moment is zero near the ends of the span.

The gross section properties of the 600 x 400 mm (23.6 x 15.7 in.) precast U-beam (see Figs. B2 and B3) are:

Distance of centroidal axis xx from bottom fiber = 247 mm (9.72 in.)

$A = 0.126$ m^2 (195 in.^{2})
$I_{xx} = 4.102 \times 10^{-3}$ m^4 (9855 in.^{4})
$Z_t = 0.0116$ m^3 (708 in.^{3})
$Z_b = 0.0166$ m^3 (1013 in.^{3})
$e = 177$ mm (6.97 in.), using three 12.5 mm (½ in.) diameter strands.

Stress in strand after losses:

$0.6f_{pu} = 0.6 \times 1774 = 1064$ MPa (154 ksi)

Total force in strands:

$F = 3 \times 93 \times 1064$ N = 297 kN (66.7 kips)

Check Longitudinal Stresses With Maximum Service Load:

Top fiber, compression:

$$f_t = \frac{F}{A} - \frac{Fe}{Z_t} + \frac{M_{max}}{Z_t}$$

$$= \frac{297{,}000}{126{,}000} - \frac{297{,}000 \times 177}{0.0116 \times 10^9} + \frac{99{,}000{,}000}{0.0116 \times 10^9}$$

$= 6.36$ MPa (922 psi)

which is less than $0.45f_c' = 18$ MPa (2610 psi).

Therefore, top fiber stress is satisfactory.

Bottom fiber, tension:

$$f_b = \frac{297{,}000}{126{,}000} + \frac{297{,}000 \times 177}{0.0166 \times 10^9} - \frac{99{,}000{,}000}{0.0166 \times 10^9}$$

$= -0.44$ MPa (-64 psi)

which is greater than $-0.5\sqrt{f_c'} = -3.16$ MPa (-458 psi).

Therefore, bottom fiber stress is satisfactory.

Check Longitudinal Stresses With Minimum Service Load:

Top fiber, tension:

$$f_t = \frac{297{,}000}{126{,}000} - \frac{297{,}000 \times 177}{0.0116 \times 10^9}$$

$= -2.17$ MPa (-315 psi)

which is greater than $-0.5\sqrt{f_c'} = -3.16$ MPa (-458 psi). Therefore, top fiber stress is satisfactory.

Bottom fiber, compression:

$$f_b = \frac{297{,}000}{126{,}000} + \frac{297{,}000 \times 177}{0.0166 \times 10^9}$$

$= 5.53$ MPa (802 psi)

which is less than $0.45f_c' = 18$ MPa (2610 psi).

Therefore, bottom fiber stress is satisfactory.

Check Shear Strength

At $h/2$ from the prop supports, shear force due to $1.4D$:

$$V_u = 1.4 \times 35.1 \times \left(\frac{4.75}{2} - \frac{0.6}{2}\right)$$
$$= 102 \text{ kN (22.9 kips)}$$

Imposed nominal shear stress at ultimate:

$$v_i = \frac{V_u}{\phi b_w d}$$

where

d = 530 mm (20.9 in.)
b_w = 400 − (250 + 206)/2
= 172 mm (6.8 in.)
ϕ = 0.85

Therefore,

$$v_i = \frac{102{,}000}{0.85 \times 172 \times 530}$$
$$= 1.32 \text{ MPa (191 psi)}$$

Since the V_u/M_u ratio is high near the ends of the beam, the nominal shear stress resisted by concrete is:

$$v_c = 0.3(\sqrt{f_c'} + f_{pc})$$
$$= 0.3(\sqrt{40} + 2.36)$$
$$= 2.61 \text{ MPa (378 psi)}$$

Since v_i is less than v_c, use minimum shear reinforcement: $A_v = 0.35 b_w s/f_y$.

Use 6 mm (0.24 in.) diameter Grade 275 (f_y = 40 ksi) stirrups.

$$s = \frac{2 \times \pi \times 3^2 \times 275}{0.35 \times 172}$$
$$= 258 \text{ mm (10.1 in.)}$$
$$s_{max} \leq 0.75h = 0.75 \times 600$$
$$= 450 \text{ mm (17.7 in.)}$$

or ≤ 600 mm (23.6 in.).

Therefore, use 6 mm (0.24 in.) diameter Grade 275 stirrups at 250 mm (9.8 in.) centers.

Check Dependable Flexural Strength

$$f_{ps} = f_{pu}\left[1 - 0.5\,\frac{\rho_p f_{pu}}{f_c'}\right]$$
$$= 1774\left[1 - 0.5\,\frac{3 \times 93}{150 \times 530} \times \frac{1774}{40}\right]$$
$$= 1636 \text{ MPa (237 ksi)}$$

Therefore:

$$\phi M_i = \phi f_{ps} A_p \left[d - 0.59\,\frac{f_{ps} A_p}{f_c' b}\right]$$

where ϕ = 0.9.

$$\phi M_i = 0.9 \times 1636 \times 3 \times 93 \times \left[530 - 0.59\,\frac{1636 \times 3 \times 93}{40 \times 150}\right]$$
$$= 199 \text{ kN-m}$$

which is greater than $1.4M_{max}$ = 139 kN-m (102 kip-ft). Therefore, flexural strength is satisfactory.

B3 Completed Beam Supporting Gravity Plus Seismic Loads

Loading Cases

Loading cases for the factored (ultimate) loads are:[4]

$$U = 1.4D + 1.7L \qquad \text{(B1)}$$
$$U = 1.0D + 1.3L + E \qquad \text{(B2)}$$
$$U = 0.9D + E \qquad \text{(B3)}$$

where

D = service dead load

L = service live load and

E = earthquake loading

Note that Eq. (B3) will not be as critical for the beams as Eq. (B2), and hence Eq. (B3) will not be considered further.

Bending Moments

It will be assumed that the dead load is carried by the beam as if it is simply supported at the column centerlines, and that the live load and earthquake load are carried by the beam as part of the continuous frame. The bending moments for the loading case $U = 1.4D + 1.7L$ will be found with live load on all spans.

This assumes that sufficient moment redistribution can occur for the pattern loading case (that is, the live load present on alternate spans only) to be disre-

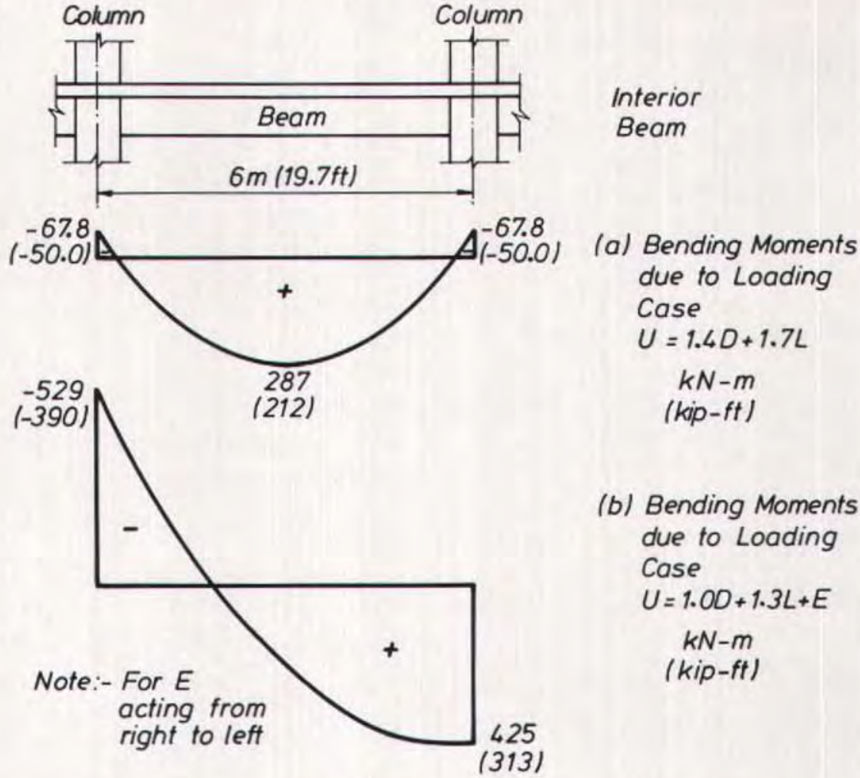

Fig. B5. Bending moments for interior composite beam due to the factored loading cases.

garded. This is a reasonable assumption since the sections are designed for ductility, and in any case the moments due to seismic loading are much greater than the moments due to gravity loading.

For load case $U = 1.4D + 1.7L$ (see Fig. B5a):

Negative moment at supports

$= -(1.7L)\,l^2/12$

$= -(1.7 \times 13.3) \times 6^2/12$

$= -67.8$ kN-m (-50.0 kip-ft)

Positive moment at midspan

$= (1.4D)\,l^2/8 + (1.7L)\,l^2/24$

$= +(1.4 \times 40.2) \times 6^2/8 + (1.7 \times 13.3) \times 6^2/24$

$= 287$ kN-m (212 kip-ft)

For load case $U = 1.0D + 1.3L + E$ (see Fig. B5b):

Seismic load moment at the supports are ± 477 kN-m (352 kip-ft).

Negative and positive moments at supports

$= -(1.3L)\,l^2/12 \mp 477$

$= -(1.3 \times 13.3) \times 6^2/12 \pm 477$

$= -529$ kN-m (-390 kip-ft)

and 425 kN-m (313 kip-ft)

Positive moment at midspan

$= (1.0D)\,l^2/8 + (1.3L)\,l^2/24$

$= (1.0 \times 40.2) \times 6^2/8 + (1.3 \times 13.3) \times 6^2/24$

$= 207$ kN-m (153 kip-ft)

The bending moment envelopes for the loading cases for the design moments at the column faces are shown in Fig. B5c. It is apparent that the maximum positive and negative moments due to $1.0D + 1.3L + E$ occur at the column faces. Hence, the potential plastic hinge regions of the beam are the

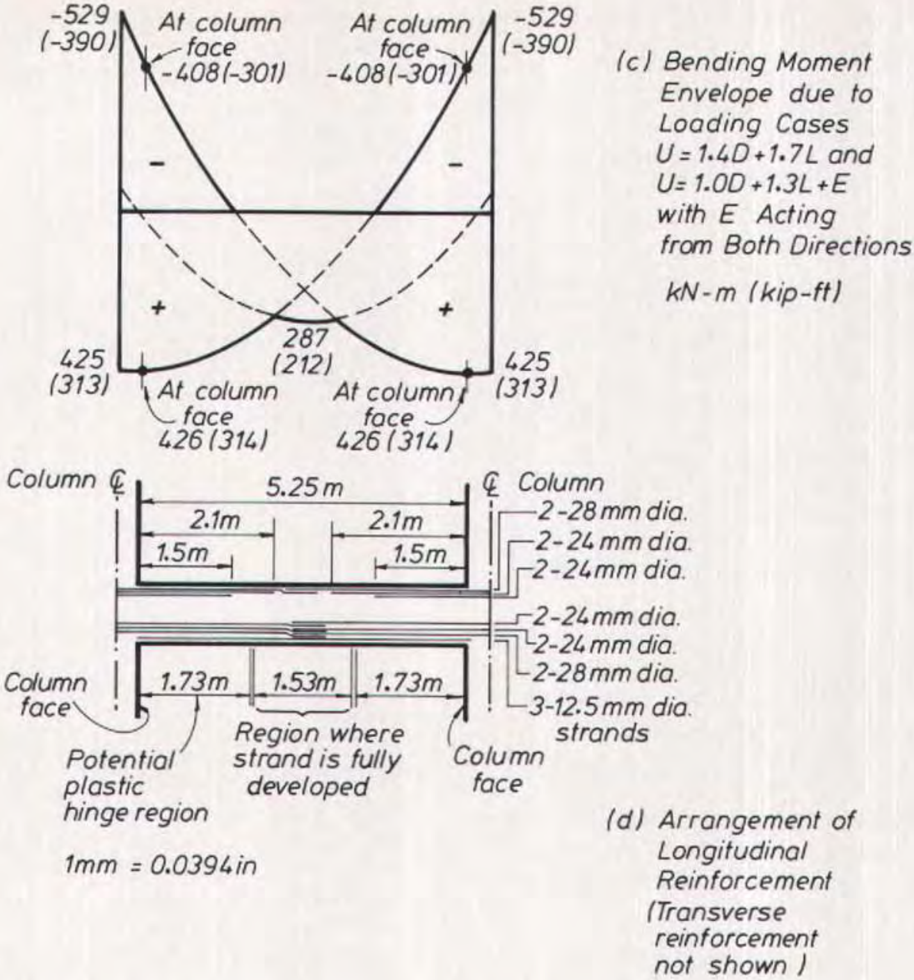

Fig. B5 (cont.). Bending moments for interior composite beam due to the factored loading cases.

end $2h$ of beam, that is the end 2 x 865 mm = 1.73 m (5.67 ft).

Design for Flexure

The maximum moments at the column faces of 426 kN-m (314 kip-ft) and −408 kN-m (−301 kip-ft) are to be resisted by the in-place concrete beam core alone. Use equal areas of top and bottom longitudinal nonprestressed steel, and assume a steel couple to provide the moment capacity, then:

$$A_s = A_s' = \frac{M_u}{\phi f_y (d - d')}$$

where

M_u = 426 kN-m (314 kip-ft)
ϕ = 0.9 and
f_y = 275 MPa (40 ksi)

If the steel arrangement shown in Fig. B3 is used, comprising two 28 mm (1.10 in.) diameter and four 24 mm (0.94 in.) diameter bars in the top and in the bottom, then with 40 mm (1.57 in.) cover at top and 30 mm (1.18 in.) cover at bottom:

$d' = 101$ mm (4.0 in.) and

$d = 674$ mm (26.5 in.)

Hence, the area of the longitudinal reinforcement is:

$$A_s = A_s' = \frac{426{,}000{,}000}{0.9 \times 275 \times (674 - 101)}$$
$$= 3004 \text{ mm}^2 \text{ (4.66 in.}^2\text{)}$$

The available $A_s = A_s'$ provided by the two 28 mm plus four 24 mm diameter bars is:

$(2 \times 615) + (4 \times 452) = 3038$ mm^2 (4.71 in.^{2})

Therefore, the steel area furnished is satisfactory.

Note that the contribution of the steel area in the slab topping is conservatively neglected.

Checks of Flexural Steel Content of Beam:

(a) the maximum diameter of beam longitudinal bar permitted to pass through beam-column joint core to satisfy bond requirements[5] is:

$$d_b \leq \frac{h_{col}}{25} = \frac{750}{25} = 30 \text{ mm (1.18 in.)}$$

which is greater than 28 mm (1.10 in.). Thus, the bar diameter is satisfactory.

(b) The tension steel content at the column face is:

$$\rho = \frac{A_s}{bd} = \frac{3038}{228 \times 674} = 0.020$$

and the compression steel content $\rho' = 0.020$, where b here is taken at the midheight of the concrete core of the U-beam. Hence as required[5] $\rho' > 0.5\,\rho$. Therefore, ρ' is satisfactory.

$$\rho_{max} = \frac{1 + 0.17\left(\frac{f_c'}{7} - 3\right)}{100}\left(1 + \frac{\rho'}{\rho}\right)$$
$$= 0.022 > 0.02, \text{ therefore, satisfactory.}$$

$$\rho_{max} = 7/f_y = 7/275 = 0.025 > 0.02 \text{ therefore, satisfactory.}$$

(c) The maximum moment at midspan of + 287 kN-m (212 kip-ft) is to be resisted by the composite section.

The dependable positive moment flexural strength of the composite section at midspan due to prestressing steel alone (that is, not including the contribution of the bottom nonprestressed reinforcing bars in the core) is:

$b = 250$ mm (9.8 in.) ignoring slab topping, and $f_c' = 25$ MPa (3.63 ksi)

$d = 865 - 70 = 795$ mm (31.3 in.)

$$f_{ps} = f_{pu}\left(1 - 0.5\frac{\rho_p f_{pu}}{f_c'}\right)$$
$$= 1774\left(1 - 0.5\frac{3 \times 93}{250 \times 795} \times \frac{1774}{25}\right)$$
$$= 1686 \text{ MPa (244 ksi)}$$

Therefore:

$$\phi M_t = \phi f_{ps} A_p \left(d - 0.59\,\frac{f_{ps} A_p}{f_c' b}\right)$$
$$= 0.9 \times 1686 \times 3 \times 93 \times \left(865 - 70 - 0.59\frac{1686 \times 3 \times 93}{25 \times 250}\right)$$
$$= 321 \text{ kN-m (237 kip-ft)} > 287 \text{ kN-m (212 kip-ft)}$$

Hence, the prestressing tendons alone can carry the positive moment at midspan.

Curtailment of Longitudinal Steel

For 24 mm (0.94 in.) diameter reinforcing bars, if l_d = development length as specified by code[5] and $d = 674$ mm (26.5 in.), then:

For top bars $l_d + d = 1.41$ m (4.63 ft)

For bottom bars $l_d + d = 1.24$ m (4.07 ft)

For 12.5 mm (½ in.) diameter seven-wire strand:

$$l_d = (f_{ps} - 2f_{pe}/3)\, d_b/7$$
$$= (1636 - 2 \times 1064/3) \times 12.5/7$$
$$= 1.66 \text{ m (5.45 ft)}$$

Allowing 200 mm (7.9 in.) for bond degradation:

$l_d + 0.2$ m $= 1.86$ m (6.10 ft)

The top 24 mm (0.94 in.) diameter longitudinal bars can be cut off within the span as shown in Fig. B5d, according to the bending envelope and the required development lengths. A minimum of two bars must extend through the span.[5] The two 28 mm (1.10 in.) bars extending through the span will need to be lap spliced at midspan.

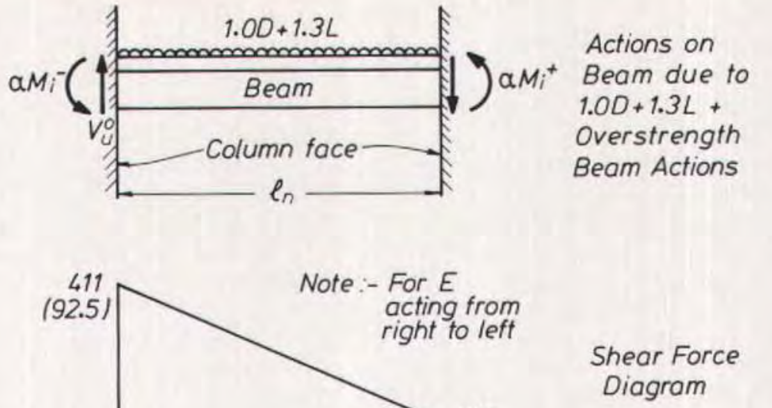

Fig. B6. Shear forces for interior composite beam due to factored gravity loading plus overstrength beam actions.

Curtailment of the bottom bars is not possible. Over the end potential plastic hinge regions of $2h$ = 1.73 m (5.67 ft) the cast-in-place beam core carries the moment alone and the positive moment does not diminish sufficiently to permit curtailment of bottom bars in those regions. The prestressing strand develops its strength f_{ps} at 1.86 m (6.10 ft) from each column face and the strand alone is capable of providing the required beam flexural strength over the central 1.53 m (5.02 ft). However, since $l_d + d$ for the bottom bars is 1.24 m (4.07 ft), there is insufficient length in this central region to consider curtailment. Therefore, all bottom bars are extended through the span with a lap splice at midspan (see Fig. B5d).

Shear Design for Overstrength Beam Moments

The maximum design shear force for load case involving seismic loading (see Fig. A3) is:

$$V_u^o = \frac{\alpha\,(|M_i^-| + M_i^+)}{l_n} + \frac{(1.0D + 1.3L)l_n}{2}$$

where

α = 1.25 = overstrength factor for flexure

l_n = clear span = 6 − 0.75 = 5.25 m (17.2 ft)

D = service dead load = 40.2 kN/m (2.76 kip/ft)

L = service live load = 13.3 kN/m (0.91 kip/ft)

M_i^+ = positive moment ideal flexural strength of cast-in-place concrete core alone

$= A_s f_y (d - d')$, since $A_s = A'_s$

$= 3038 \times 275\,(674 - 101)$ N-mm

= 479 kN-m (353 kip-ft)

M_i^- = negative moment ideal flexural strength of composite section at face of column

$= A_s f_y\,(d - 0.5a)$, ignoring compression steel

$$\text{where } a = \frac{A_s f_y}{0.85 f'_c\, b} = \frac{3038 \times 275}{0.85 \times 40 \times 400}$$

= 61 mm (2.4 in.)

< 100 mm (3.9 in.)

Therefore, the neutral axis lies in the flange of the U-beam.

Therefore:

$$M_i^- = 3038 \times 275\left(865 - 101 - \frac{61}{2}\right)$$

= 613 kN-m (452 kip-ft)

and

$$V_u^\circ = \frac{1.25\,(479 + 613)}{5.25} + \frac{(40.2 + 1.3 \times 13.3)\,5.25}{2}$$

= 260 + 151 = 411 kN (92.5 kips)

The minimum shear force is given by 260 − 151 = 109 kN (24.5 kN). The shear force diagram is shown in Fig. B6.

Transverse Reinforcement in Potential Plastic Hinge Regions

In each end 1.73 m (5.67 ft) region of the member, the shear is resisted by the beam core alone and $v_c = 0$ is assumed. The spacing of 12 mm (0.47 in.) diameter stirrup ties required for shear is:

$s = A_v f_y\, d / V_u^\circ$

$= (2 \times \pi \times 6^2) \times 275 \times 674/411{,}000 =$ 102 mm (4.0 in.)

Use 12 mm (0.47 in) diameter Grade 275 stirrup ties at 100 mm (3.9 in.) centers. Checking spacing required for concrete confinement and to prevent buckling of longitudinal steel:

(a) 150 mm (5.9 in.) > s = 100 mm and, therefore, satisfactory

(b) Six longitudinal bar diameters = 6 x 24 = 144 mm (5.7 in.) > s = 100 mm, and, therefore, satisfactory

(c) $d/4$ = 674/4 = 169 mm (6.6 in.) > s = 100 mm and, therefore, satisfactory

(d) Tie force = $\pi \times 6^2 \times 275\,N$ = 31.1 kN (7.0 kips)

$$> \frac{1}{16} \times \text{longitudinal bar force} \times \frac{100}{100}$$

$$= \frac{1}{16} \times \pi \times 14^2 \times 275\,N = 10.6\text{ kN}$$

(2.38 kips), and, therefore, satisfactory

$$\frac{V_u^\circ}{b_w d} = \frac{411{,}000}{(400 - 172) \times 674} = 2.67\text{ MPa}$$

$= 0.53\sqrt{25} = 0.53\sqrt{f'_c}$ MPa

($6.36\sqrt{f'_c}$ psi)

Therefore, satisfactory since shear stress is less than the maximum value permitted.

Transverse Reinforcement Outside the Plastic Hinge Regions

In the central 1.79 m (5.87 ft) region of the member, the maximum shear force is 311 kN (70.0 kips) and is predominantly in the positive moment region. It will be assumed, as in Fig. Alc, that the shear is resisted by a composite beam of width b_w = 250 mm (9.8 in.) and effective depth d = 684 mm (26.9 in.) (i.e., the d to the centroid of the bottom nonprestressed and prestressed steel) with $v_c = 0.17\sqrt{f'_c}$ MPa ($2\sqrt{f'_c}$ psi), and the stirrups in the precast U-beam will be ignored. The spacing of 12 mm (0.47 in.) diameter stirrup ties in the core required for shear on this basis is:

$311{,}000 = v_c\sqrt{f'_c}\,bd + A_v f_y\, d/s$

$= (0.17\sqrt{25} \times 250 \times 684) + (2 \times \pi \times 6^2 \times 275 \times 674/s)$

from which $s = 257$ mm (10.1 in.).

The spacing is not to exceed $d/2 = 684/2 = 342$ mm (13.5 in.). Therefore, use 12 mm (0.94 in.) diameter Grade 275 stirrups at 250 mm (9.8 in.) centers.

Note: Shear design for loading case $U = 1.4D + 1.7L$ will not be more critical.

Check of Interface Shear Stress

Assume that the contact surface is clean, free of laitance and intentionally roughened to a full amplitude of 5 mm (0.2 in.) and that the permitted interface shear stress when the beam over-strength moments develop is 0.55 MPa (80 psi). The shear force diagram is shown in Fig. B6.

Horizontal interface shear stress is found from Eq. (A6), where

b_v = 500 + 206 + 500 = 1206 mm (47.5 in.)

d = 674 mm (26.5 in.), i.e., for core section, conservatively

V_u° = 411 kN (92.5 kips) as in Fig. B6 at the column face.

Therefore:

$$v_{dh} = \frac{411{,}000}{1206 \times 674} = 0.506 \text{ MPa (73 psi)}$$

Vertical interface shear stress is found from loading case $1.0D + 1.3L$. The vertical interface shear force is

$35.9 + 1.3 \times 13.3 = 53.2$ kN/m (3.65 kip/ft).

Each side face is 500 mm (19.7 in.) deep.

Therefore:

$$v_{dv} = \frac{53{,}200}{(500 + 500) \times 1000} = \begin{matrix} 0.053 \text{ MPa} \\ (8 \text{ psi}) \end{matrix}$$

and

$$\begin{aligned} \sqrt{v_{dh}^2 + v_{dv}^2} &= \sqrt{0.506^2 + 0.053^2} \\ &= 0.509 \text{ MPa (74 psi)} \\ &< 0.55 \text{ MPa (80 psi)} \end{aligned}$$

Therefore, the total imposed shear stress at interface is satisfactory.

Note that with regard to the question of interface shear in seismic over-strength design where the shell beam in the plastic hinge areas is ignored, it could be said that interface shear capacity has degraded to zero and cannot be relied upon. Therefore, the maximum shear force for the interface shear stress check might be considered as the maximum shear of the composite beam shear design that is at the end of the plastic hinge region adjacent to the composite midspan region.

In seismic design where there is a significant shear gradient along the beam it would be more economic and justifiably safe to approach the interface check in this manner. In pure gravity load design the interface shear check reverts back to the shear force at d from the column face. In the above example then the horizontal interface shear would be 311 kN (70.0 kips) rather than 411 kN (92.5 kips) (Fig. B6), resulting in an interface stress of 0.38 MPa (55 psi).

Cast-in-Place Topping Slab

The cast-in-place topping slab needs to act as a diaphragm to transfer to the frame system the seismic forces originating from the mass of the floor and the loads placed on it. The reinforcement placed in the two directions in the topping slab should not be less than that required by the code for shrinkage and temperature reinforcement in slabs.

* * *

APPENDIX C — NOTATION

A = area of precast U-beam section
A_g = gross area of column section
A_p = area of prestressing steel
A_s = area of top longitudinal reinforcement in cast-in-place beam core
A_s' = area of bottom longitudinal reinforcement in cast-in-place beam core
A_v = area of shear reinforcement at spacing s
b = width of compression face of beam
b_i = width of precast U-beam measured at bottom of core
b_v = width of interface of section being considered for horizontal shear
b_w = width of web of beam
d = distance from extreme compression fiber of concrete to centroid of tension reinforcement
d' = distance from extreme compression fiber of concrete to centroid of compression reinforcement
D = service dead load
e = eccentricity of prestressing force in precast U-beam section
E = earthquake loading
f_b = longitudinal concrete stress in bottom fiber
f_c' = compressive cylinder strength of concrete
f_{pc} = longitudinal stress in concrete due to prestress at centroid of beam section
f_{pe} = stress in prestressing steel after losses
f_{ps} = stress in prestressing steel at flexural strength
f_{pu} = ultimate tensile strength of prestressing steel
f_{su} = ultimate tensile strength of reinforcing steel
f_t = longitudinal concrete stress in top fiber
f_y = yield strength of reinforcing steel
F = total force in prestressing strand
h = depth of section
h_{col} = depth of column
h_i = depth of core of precast U-beam measured from the top of U-beam
I_{xx} = moment of inertia of precast U-beam about horizontal centroidal axis
l = span of beam between column centers
l_n = clear span of beam
L = service live load
M = bending moment
M_i = ideal (nominal and theoretical) flexural strength of cast-in-place concrete core alone calculated assuming $\phi = 1$
M_i' = ideal (nominal and theoretical) flexural strength of composite section calculated assuming $\phi = 1$
M_{max} = maximum measured moment
M_u = design bending moment from factored (ultimate) loading
$+P_i$ = beam end load to cause positive moment M_i to be reached
$-P_i$ = beam end load to cause negative moment M_i to be reached
$-P_i'$ = beam end load to cause negative moment M_i' to be reached
s = spacing of shear reinforcement
U = design load combinations
v_c = shear stress resisted by concrete mechanisms
v_{dv} = total imposed vertical shear stress at interface
v_{dh} = total imposed horizontal shear stress at interface
v_i = total ideal shear stress resisted by concrete mechanisms and shear reinforcement
V = shear force
V_c = shear force resisted by concrete shear resisting mechanisms
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

Z_b = section modulus to bottom of precast U-beam section

Z_t = section modulus to top of precast U-beam section

α = overstrength factor for beam flexural strength

Δ = beam end deflection

Δ_y = beam end deflection at first yield

ρ = A_s/bd

ρ' = A_s'/bd

ρ_p = A_p/bd

ϕ = strength reduction factor

μ = Δ/Δ_y = displacement ductility factor

* * *