A Hybrid Reinforced Precast Frame for Seismic Regions

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A precast, prestressed concrete framing system for resisting earthquake loads is described. The system uses both unbonded post-tensioned reinforcement and bonded bar reinforcement. Tests have shown that its performance is equal or superior to that of a conventional cast-in-place moment frame. Design criteria and code implications resulting from the test program as well as suggested topics for future research are discussed.

Precast, prestressed concrete has the potential of being widely used in high seismic areas of the United States. Its widespread use, however, has been hampered by the absence of a well documented history of its superior performance both here and abroad as well as a lack of a substantial body of research to support its design and construction practices. Also, there is an absence of prescriptive seismic code provisions for precast and prestressed concrete in local and national building codes. A major challenge has been to find a connection that can reliably undergo nonlinear cyclic loading.

This paper presents the underlying principles of a new precast framing system that uses precast elements, connected by unbonded post-tensioning steel and bonded reinforcing bars. This framing system, which has been developed recently, displays excellent seismic performance. To test the proposed system, an experimental program was conducted at the laboratories of the National Institute of Standards and Technology (NIST) in Gaithersburg, Maryland. In the research program, several variants on the basic theme were tested.

For comparison, a pair of conventional monolithic, cast-in-place frames were built and subjected to the same test

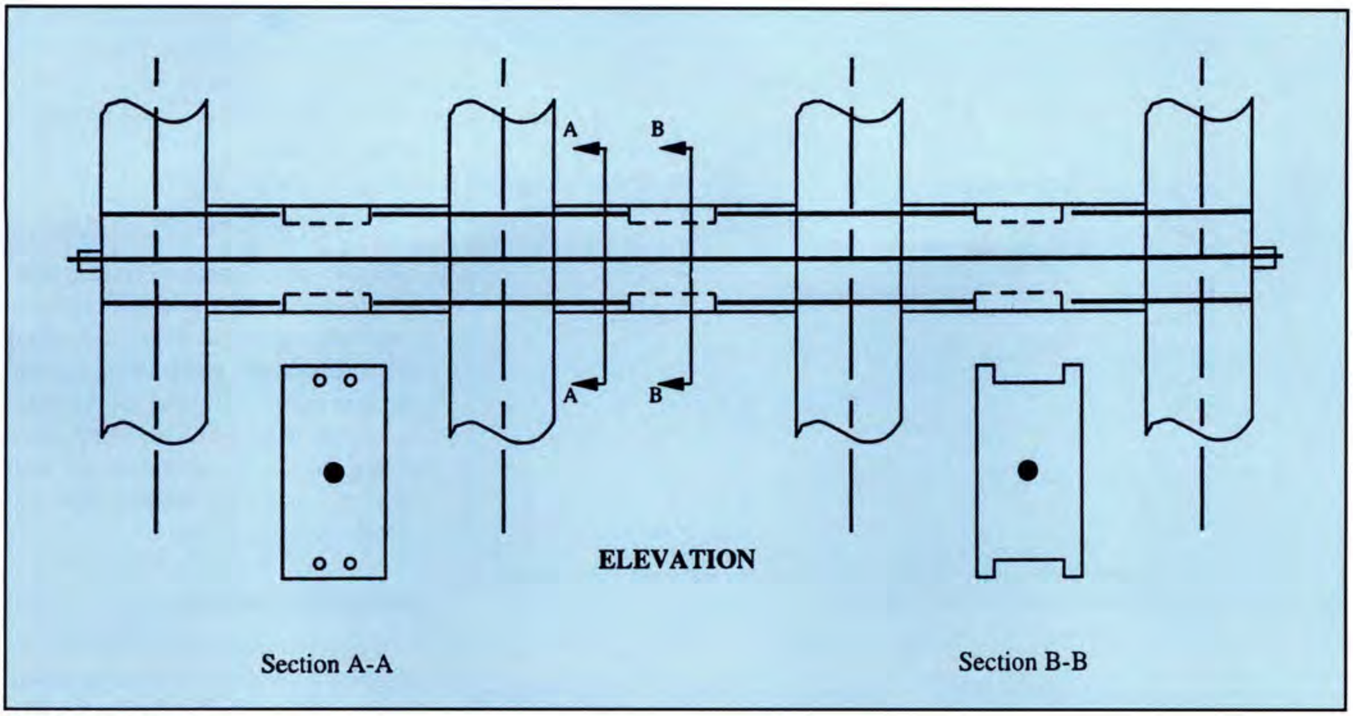

Fig. 1. Framing system.

program. The new precast frame proved to be at least equal to the monolithic frames in almost all respects, and superior in most. Detailed information on the tests can be found in Cheok and Stone.[1] The design method for the new system is treated in detail by Cheok et al.[2]

DESCRIPTION OF SYSTEM

The precast frame system is shown in Fig. 1. The beams and columns are precast and no permanent corbels are necessary. The column is assumed to be a single, multistory unit in order to save the costs of splicing. The beam has a solid rectangular cross section at its ends and, in its central region, it has a trough at the top and bottom. The beam-to-column connection is made with grouted reinforcing bars (referred to here as bar steel) and post-tensioning steel that is either partially or totally unbonded. The bar steel is placed in ducts at the top and bottom of the beam, and the post-tensioning tendon is straight and is located at mid-height of the beam.

In the expected construction sequence, the columns are erected first and are equipped with temporary steel corbels on which the beams will be set. The beams are then erected, the reinforcing bars are placed in the trough and are passed through the ducts in the solid ends of the beam, which line up with matching ducts in the column. The gap between the beam and column is then grouted with a fiber-reinforced grout. The ducts containing the reinforcing bars may be grouted at the same time. When a line of beams is in place and the grout at the beam-to-column interface has gained strength, the post-tensioning steel is installed and stressed. The floor system is then installed and the temporary corbels may be removed.

PRINCIPLE OF OPERATION

The intent of the system is that the beams and columns should act essentially as rigid bodies and that the deformations of the system should be concentrated at the beam-to-column joints. The end of the beam rocks against the column face and a single crack opens there. The post-tensioning steel is designed to remain elastic at all times. This is achieved by debonding it so that the strand extension arising from the crack is distributed over a length that is great enough to keep the total strain below yield.

The prestress is lower (e.g., approximately $0.40f_{pu}$) than is customary in order to reduce the initial strain, and the tendon is placed at mid-height in order to minimize the increase in strain caused by beam rotation. The post-tensioning steel thus provides an elastic restoring force that, after an earthquake, re-centers the structure with no residual displacement and closes any cracks that have opened. It also provides a permanent clamping force across the beam-to-column interface that provides friction which resists the gravity and seismic shear forces.

The post-tensioning tendon is anchored using conventional multi-strand anchors at each end. It cannot be grouted over its full length without preventing the system from working in the intended manner, so it must remain unbonded over most of its length. However, supplementary back-up anchorage may be supplied if desired by partially grouting it at strategic points, as described later in the section on "Design Criteria."

The system also contains reinforcing bars that act both as energy dissipators and auxiliary sources of flexural and shear strength. They are placed at the top and bottom of the beams, because there they undergo the greatest strains for a given story drift and thus dissipate the maximum possible energy through cyclic yielding in tension and compression. Yield of the bars in tension depends largely on adequate bond. In compression, it

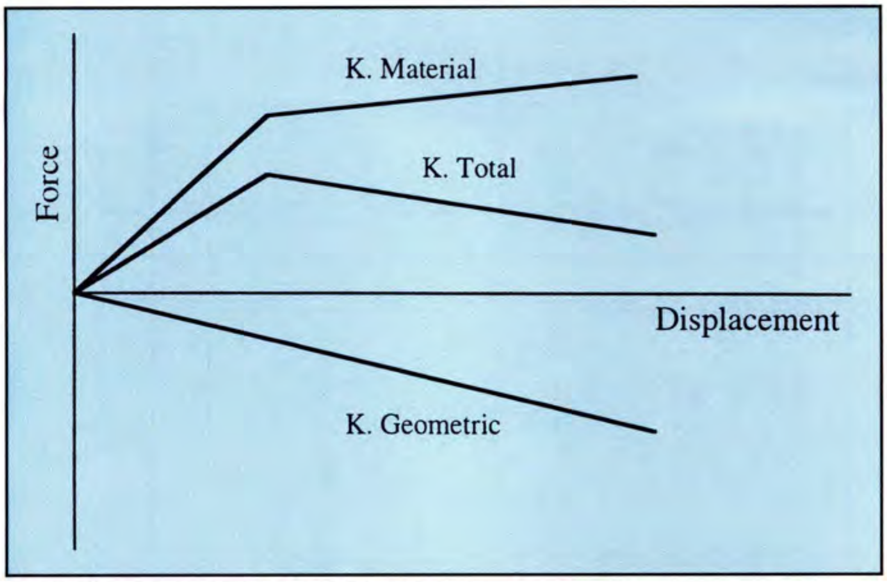

Fig. 2. Material, geometric and total stiffnesses.

is assured by appropriate design of the post-tensioning steel, which must induce a large enough force to yield the bars when the lateral load reverses.

Based on the experimental results, the precast elements suffer relatively little damage after undergoing nonlinear cyclic loading and the subsequent structural repair costs can be expected to be lower than they would be in a conventional monolithic frame.

The use of two different types of steel, each with a different function, distinguishes this framing system from others. In most conventional seismic designs, it is considered desirable to cause all the steel to yield and dissipate energy. Prestressing steel offers little opportunity for energy dissipation, even if it is bonded, so its use is essentially prohibited by the Uniform Building Code.[3] If a strand were to yield in tension, load reversal would cause buckling rather than yielding in compression, so yielding would occur only on the first extension.

In the new system, the strand is specifically designed *not* to yield, because its function is to maintain the clamping force between the elements and to provide a reliable and permanent restoring force, which it can only do if it remains elastic. The energy dissipation is provided separately by the bar reinforcement. The simultaneous use of unbonded post-tensioning steel and bonded bars is referred to here as hybrid reinforcement.

It is worth noting that the use of a single type of reinforcement could not meet the design objectives. For example, high strength bars alone would lose their restoring force if they yielded and would dissipate no energy if they did not.

SYSTEM PROPERTIES

Cast-in-place concrete frames reinforced with mild reinforcing bars (conventional frames) are the usual standard against which other seismic concrete framing systems are presently measured. It is, therefore, worth comparing their seismic properties with those of typical precast structures and with the hybrid system.

Damage to Concrete

In a conventional frame, the bars are designed to yield in order to dissipate energy. This yielding leads unavoidably to damage in the form of significant cracks in the concrete, which in turn can lead to shear failures. In the hybrid frame, the post-tensioning steel remains elastic and undamaged and the reinforcing bars are grouted in ducts that prevent the propagation of damage to the surrounding concrete. The result is very little structural damage, even under high drift demand.

Initial Stiffness

The post-tensioning steel in the hybrid frame confers on the system a higher initial stiffness than would be the case for either a conventional frame or a typical precast frame, as shown by Stanton.[4] This initial stiffness is benefi-

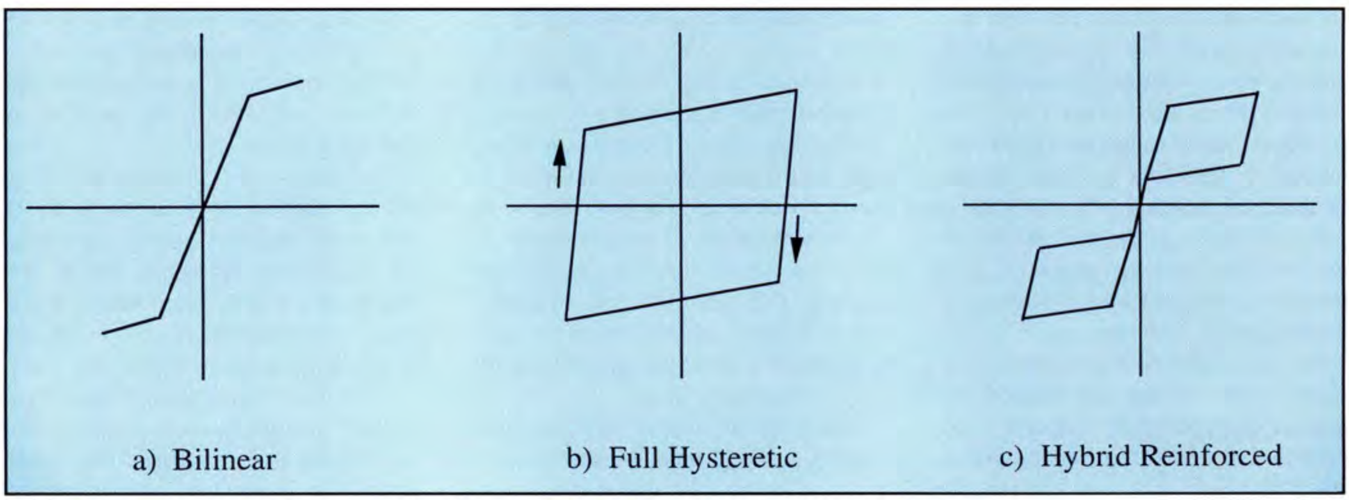

Fig. 3. Idealized load-deflection curves for bilinear elastic, full hysteretic and hybrid systems.

cial for reducing deflections of any type. However, the frame stiffness will be reduced significantly when the initial precompression force in the beam is overcome by the seismic moments and the top or bottom of the beam separates locally from the column. This will occur in a moderate earthquake. The performance of the system is controlled by its post-cracking properties, and is only slightly affected by changes in the initial stiffness. Therefore, it is suggested that the design base shear should be calculated using a period derived from the gross cross section properties of the members, just as is done for a conventional monolithic frame. If displacement based design methods are developed and become accepted by the profession, other more rational design approaches will be possible and the benefits of the behavior available from the hybrid system will be more fully utilized.

Residual Drift and Cracks

In a conventional frame, yielding of the steel will lead to some residual drift. (It is usually longer in steel structures than in reinforced concrete structures, but it exists nonetheless in both.) In the hybrid frame, the elastic post-tensioning steel virtually eliminates residual drift and closes the cracks after an earthquake.

Anchorage

In many precast systems, one of the primary difficulties lies in making connections. At least some of them must carry tension but they must be deformable enough so as not to fracture and anchored well enough so as not to damage the surrounding concrete during an earthquake. These requirements are difficult to meet. In the hybrid system, the problem is avoided by providing a permanent compressive connection between members rather than embedding hardware in them to carry tension.

Shear Forces

Priestley and MacRae[5,6] tested and analyzed an unbonded post-tensioned system that contained no bonded reinforcing bars. They showed that the shear forces in the beam-to-column joint were carried largely by diagonal compression struts and that in such frames less joint reinforcement is needed than is required by the Uniform Building Code.[3] It can be shown that a similar principle holds true for beam shear in unbonded systems, so the tie reinforcement there could also be reduced if the code is modified to recognize this form of behavior. Similar arguments apply to the hybrid frame.

Enough steel should still be provided for confinement to prevent a brittle compressive failure. When the beam-to-column joint undergoes large rotations, the compressive force is highly concentrated near the top or bottom of the beam. Tests by Cheok and Stone[1] have shown that "armor" angles at the top and bottom of the beam are very effective in preventing compression-induced spalling under these circumstances.

Reserve Drift Capacity

At extreme drifts, greater than about 2 percent, monolithic frames start to suffer serious damage and to lose a significant proportion of their lateral capacity. Tests on the hybrid frame have shown that much larger cyclic drifts can be accommodated while more than 50 percent of the flexural resistance remains and shear failure is prevented.

P-Δ Effects

The P-Δ effect is a negative, geometric component of stiffness that must be added to the positive material stiffness of the members and connec-

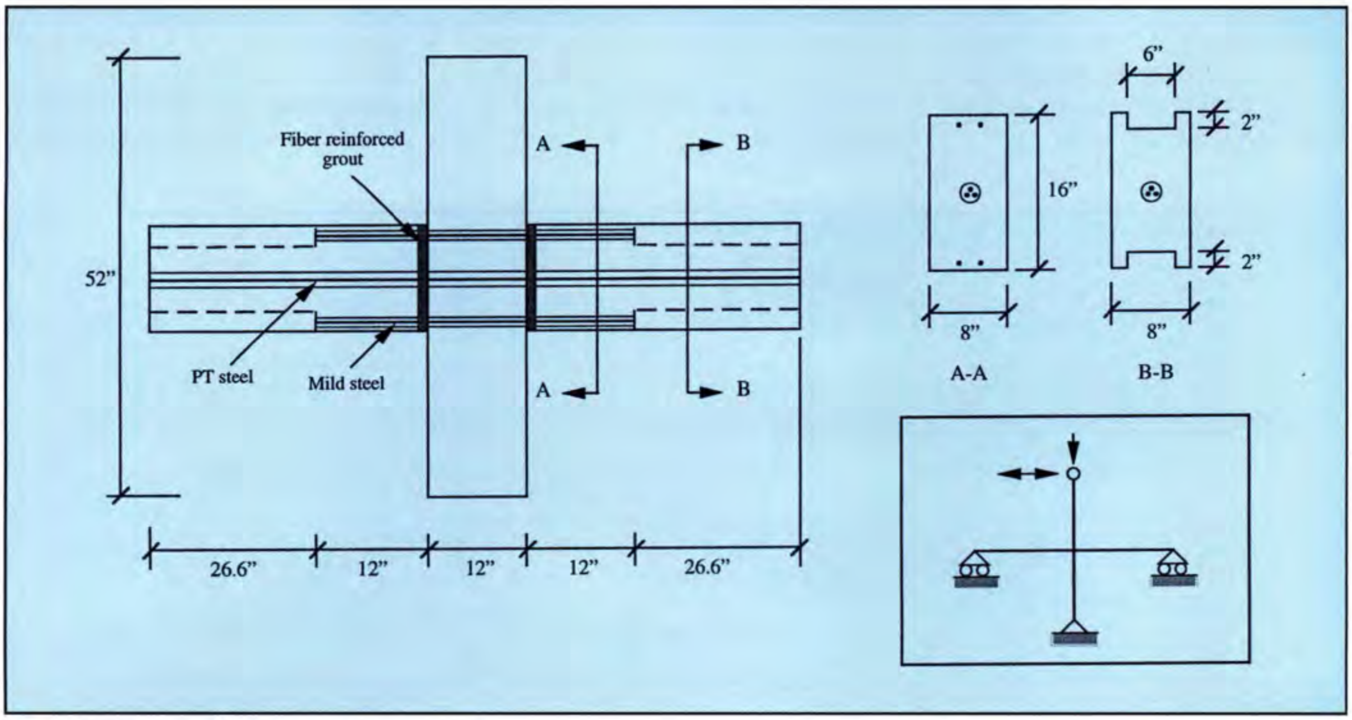

Fig. 4. Dimensions of test specimens.

Table 1. Reinforcing details of test specimens.

Specimen	Bars	$f_{y,nom}$ (ksi)	$f_{y,exp}$ (ksi)	$f_{u,exp}$ (ksi)	Strand	f_{pe} (ksi)
M-P-Z4	2 #3 Grade 60	60	61	98	3 – 1/2 in.	120
N-P-Z4	2 SS 304*	60	75	100	3 – 1/2 in.	120
O-P-Z4	3 #3 Grade 60	60	61	98	3 – 1/2 in.	120
P-P-Z4	3 SS 304*	60	62	101	3 – 1/2 in.	120

Note: 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.
* Stainless steel 304.

tions. When the structure is elastic, its material stiffness significantly exceeds the geometric component so the P-Δ effect is generally negligible. However, after yielding, the material stiffness is reduced. If this reduction were significant, the total stiffness could become negative (as shown in Fig. 2) and the structure would collapse. In a conventional frame, the primary protection against this behavior lies in the strain hardening of the reinforcing bars, but this offers little protection if the concrete is badly damaged. In the hybrid frame, which sustains less damage than the conventional frame, protection is offered by the post-tensioning steel remaining elastic and providing a permanent positive material stiffness.

Hysteresis and Damping

The load-deflection curves for the hybrid system have an appearance that is different from those for conventional systems. Fig. 3 shows idealized hysteresis loops for a pure unbonded post-tensioned system, a pure yielding system and a hybrid system.

In the pure unbonded post-tensioned system, the relationship is elastic but nonlinear, as shown in Fig. 3a; that is, it loads and unloads along the same path so there is no hysteresis and no energy dissipated, but the path is nonlinear because of the opening and closing of the cracks.

The pure yielding system dissipates energy (see Fig. 3b), but will exhibit residual drift because the displacement is not zero when the force drops to zero.

The hybrid system displays curves (see Fig. 3c) that are a combination of the other two. The prestressing force is designed to be large so that enough of the elastic restoring force will overcome the yield strength of the mild reinforcing steel and return the system to zero displacement when the external load is removed. The curve (see Fig. 3c), therefore, goes back through the origin. However, while the bar steel is yielding, energy is dissipated by hysteresis, as shown. The area of the loop and the point at which the unloading curve rejoins the loading curve depend on the relative quantities of the two types of reinforcement.

EXPERIMENTAL PROGRAM

Tests on one-third scale models of the hybrid connection have been conducted at NIST.[1] An earlier three-phase investigation there had concentrated on precast, prestressed joints made from bonded prestressing steel, both alone and in combination with some bar reinforcement,[7-9] so a fourth phase of that testing program was initiated to investigate the hybrid system.

Phase IVa was exploratory and contained several different configurations such as the use of high strength bars in place of strands, placement of the post-tensioned reinforcement at the top and bottom of the beam so that it could be installed on a column-by-column basis, replaceable reinforcing bar systems, and other variables.

Each configuration had advantages and drawbacks as described by Cheok and Stone.[1] The best alternative, shown schematically in Fig. 1, was selected for more detailed study in Phase IVb. A pair of conventional cast-in-place specimens, tested in the first phase, served as a reference for comparison.

The primary lessons learned from the Phase IVa tests were that:

- The hybrid system performed very well. The system maintained its integrity to very large drifts, even after the bar steel had fractured. No slip occurred at any time at the beam-to-column interface, despite the continuous application of a vertical load that simulated gravity.
- Strand, rather than high strength bar, should be used for the post-tension-

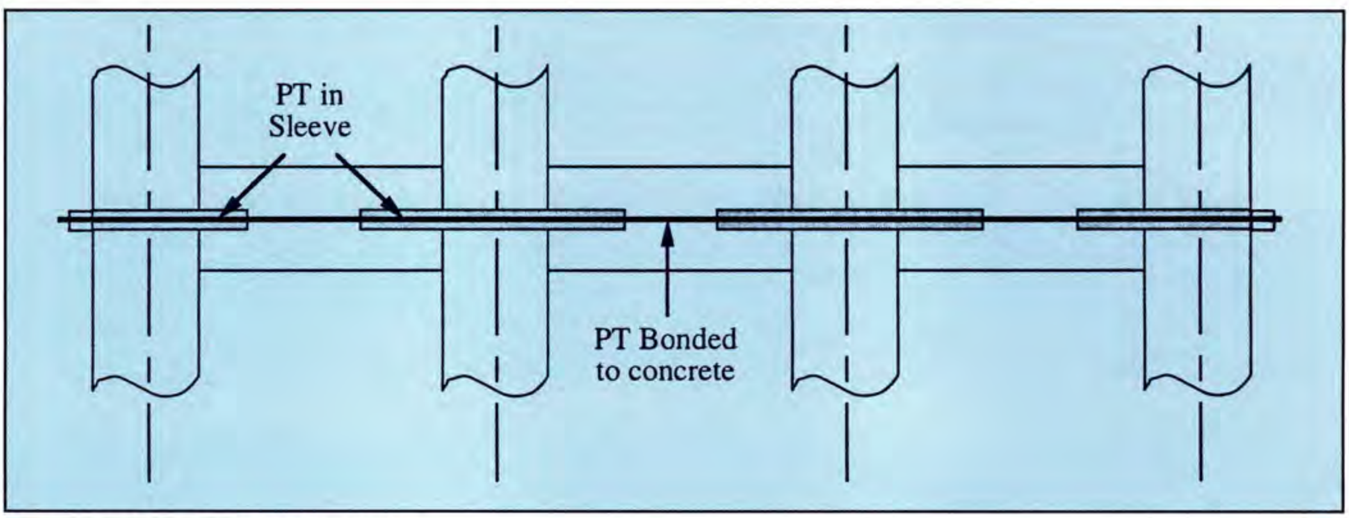

Fig. 5. Partial bonding of post-tensioning.

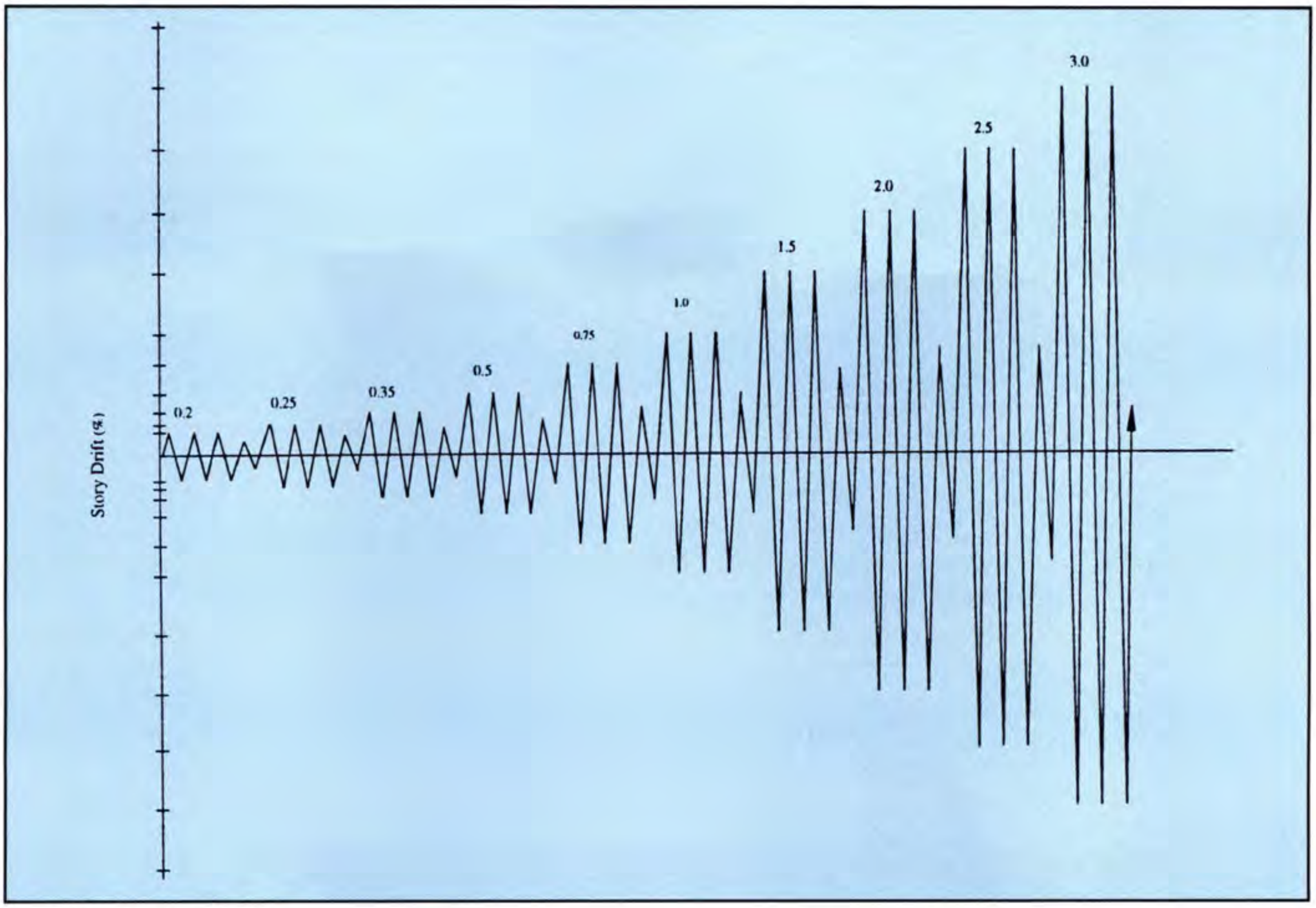

Fig. 6. Displacement history.

ing tendon because of its higher yield strain and ability to remain elastic under higher elongations.

- The strand should be placed at mid-depth of the member and should remain unbonded for most of its length. The energy dissipating mild bar steel should be placed at the top and bottom of the beam.
- The post-tensioning steel should be contained within a single tendon duct running the entire length of the frame. Use of "dogbones" (locally increased beam depth) at each end of the beam and a post-tensioned connection between the beam end and the column gave rise to severe stresses and significant damage in the beam end, despite heavy reinforcement there.
- Detailing of the bar steel is important. The bonded length should be large enough for anchorage, but a short length on both sides of the beam-to-column interface should be debonded in order to avoid strain concentrations and risk of premature fracture there.

Test Arrangement

Full details of all the Phase IVb tests are presented elsewhere.[1] The specimens were beam-column units, illustrated in Fig. 4, subjected to displacement controlled cyclic translations at the top of the column. Reinforcing details of the specimens are given in Table 1.

The specimens in Phases IVa and b were named (I through P)-P-Z4, for consistency with the earlier Phase I-III tests. The first letter identifies the specimen, the second letter, P, signifies precast concrete and the Z4 indicates Seismic Zone 4 design (the two monolithic reference specimens were A-M-Z4 and B-M-Z4).

The concrete in both beams and columns had a nominal compressive strength of 6000 psi (41.4 MPa) at 28 days, although some tests were conducted at greater ages. The grout had a nominal cube strength of 10,000 psi (69 MPa). The grout in the beam-to-column interface contained $^1/_2$ in. (12.7 mm) nylon fibers.

The beams were post-tensioned with three $^1/_2$ in. (12.7 mm) diameter 270K grade strands, bonded over part of their length. The effect of partial bonding was investigated by bonding a short length of the strand at midspan of each bay in the prototype, as shown in Fig. 5. Partial bonding is used to provide back-up protection against progressive collapse in the unlikely event of an anchorage failure.

In the prototype, the grout would be at midspan of the beams so, for complete reproduction in the test specimen, it should have been placed at both cantilever beam ends. However, all the grout was placed at one end, so that load cells could be used at the other end to detect changes in the prestress force throughout the test. The total bonded length was scaled directly from the prototype.

The strands were temporarily jacked to a stress of $0.8f_{pu}$ in order to seat the chucks and thereby to prevent additional anchor slip when the specimen displacements and strand stresses became large. The tendon was then released and re-jacked to a stress of $0.44f_{pu}$, at which point shims were

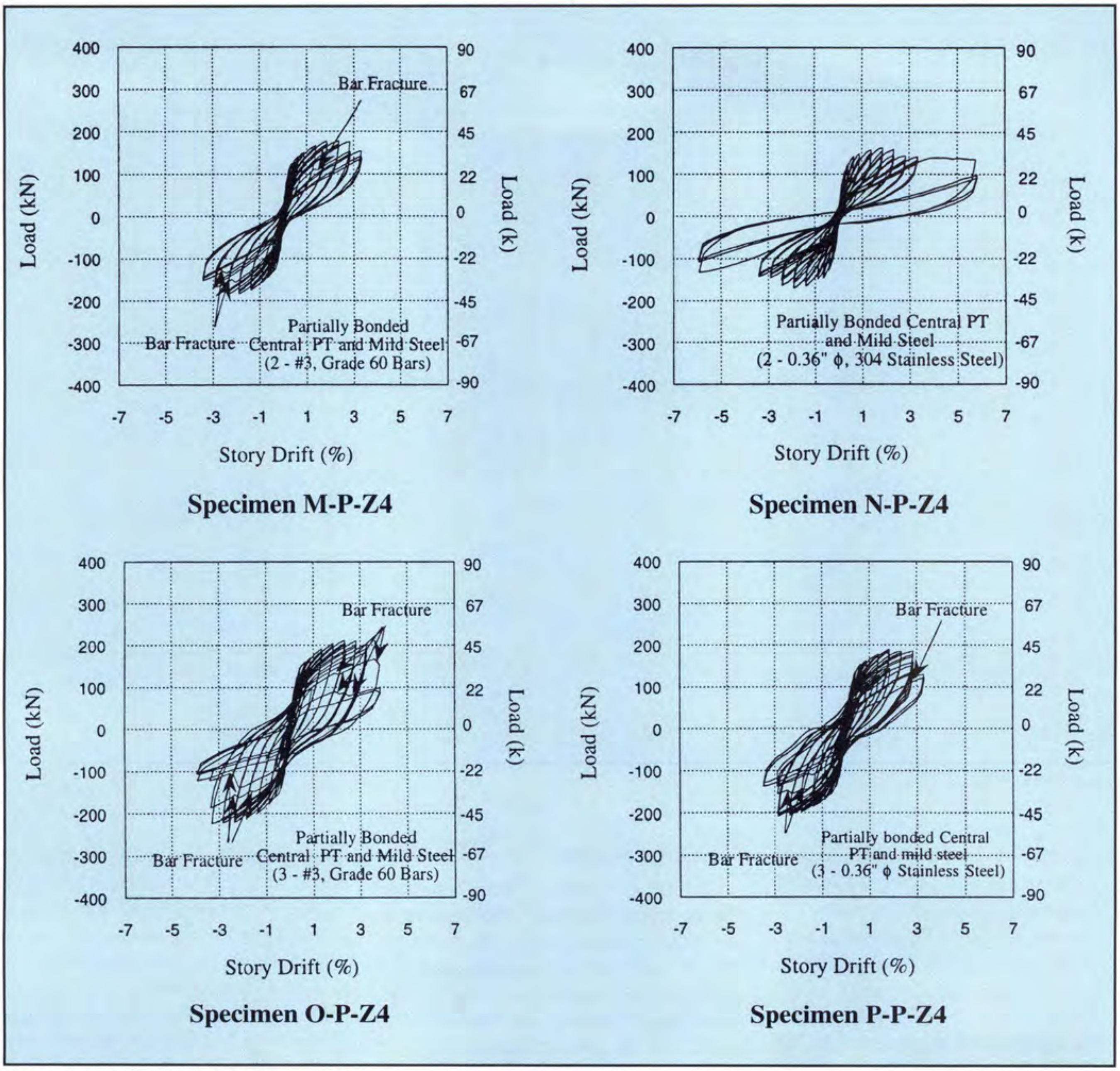

Fig. 7. Experimental load-displacement curves.

placed under the chucks. This stress level was selected as a compromise between a low stress, which would permit a large drift before the strand yielded, and a high stress, which would permit the necessary clamping force to be achieved with a smaller number of strands.

The four specimens differed in the beam reinforcement details, which are summarized in Table 1. They contained either two (Specimens M and N-P-Z4) or three bars (Specimens O and P-P-Z4) in the top and bottom. In Specimens N and P-P-Z4, they were ASTM A240 Type 304 3/8 in. (9 mm) diameter bar stainless steel, into which lugs were machined, whereas the others were standard ASTM A615 Grade 60 #3 bars. The former were used because the material possesses a high elongation at break and some A615 bars in previous specimens had failed prematurely by fracturing. The beam and column ties were made from welded reinforcement grids (WRGs), which made fabrication much easier.

Each specimen was loaded by applying an axial load to the column of approximately $0.4f_c'A_g$. A vertical load, simulating gravity, was applied to the beams adjacent to the column and was maintained throughout each test. The beam ends were supported on straps that permitted horizontal but not vertical displacement; the column base was mounted on a hinge; and the column top, which was also hinged, was subjected to the specified displacement history shown in Fig. 6.

The displacements were specified in terms of drift ratio rather than ductility ratios, which had been used in earlier phases of the program. After application of the initial drifts from 0.2 to 3.0 percent, Specimen N-P-Z4 was subjected to three cycles of 6 percent drift (the test rig limit), and Specimen P-P-Z4 was subjected to an additional series of cyclic drifts at increasing amplitudes, starting at 1 percent, to simulate an aftershock.

Results and Discussion

Each specimen was cycled until it failed. Failure was defined arbitrarily as the point at which the strength dropped to 80 percent of the peak resistance. Failure was caused by debonding of the stainless steel bars in Specimen N-P-Z4, and by bar fracture in the other three cases. Hysteresis loops for all four precast specimens and the monolithic reference specimen are shown in Fig. 7. The strand stress history for Specimens M-P-Z4 (typical) and N-P-Z4 (extreme) are shown in Fig. 8.

The precast specimens failed at drifts between 2.9 and 3.4 percent, which is slightly less than the average of the two monolithic specimens (3.6 percent). (Uncertainties of measured values such as drift and moments are not reported in this paper because the results are obtained from tests of unique specimens with no duplicates.)

This suggests that the precast specimens have less drift capacity than the monolithic ones, but in fact the reverse is true. The reason lies in the definition of failure. The monolithic specimens suffered extensive cracking and loss of shear strength and, after failure, provided very little resistance because they were damaged beyond repair. By contrast, at failure, the precast specimens retained an average of 55 percent of their peak strength because the prestressing strand was undamaged. They also displayed minimal cracking, no loss of shear strength and no slip at the beam-to-column interface.

The cycles to 6 percent drift that were applied to Specimen N-P-Z4 demonstrated the ability of the precast system to maintain a significant proportion of its strength at very large drifts. The calculated drift corresponding to UBC Zone IV loading on the specimens is between 1 and 1.5 percent. Hence, the drift capacity of the system exceeds the potential demand by a factor of at least 4 to 6. This reserve drift capacity is a valuable asset in light of the difficulties in predicting real drifts as shown by Uang and Maarouf.[10] Furthermore, the drift capacity of the hybrid system can be in-

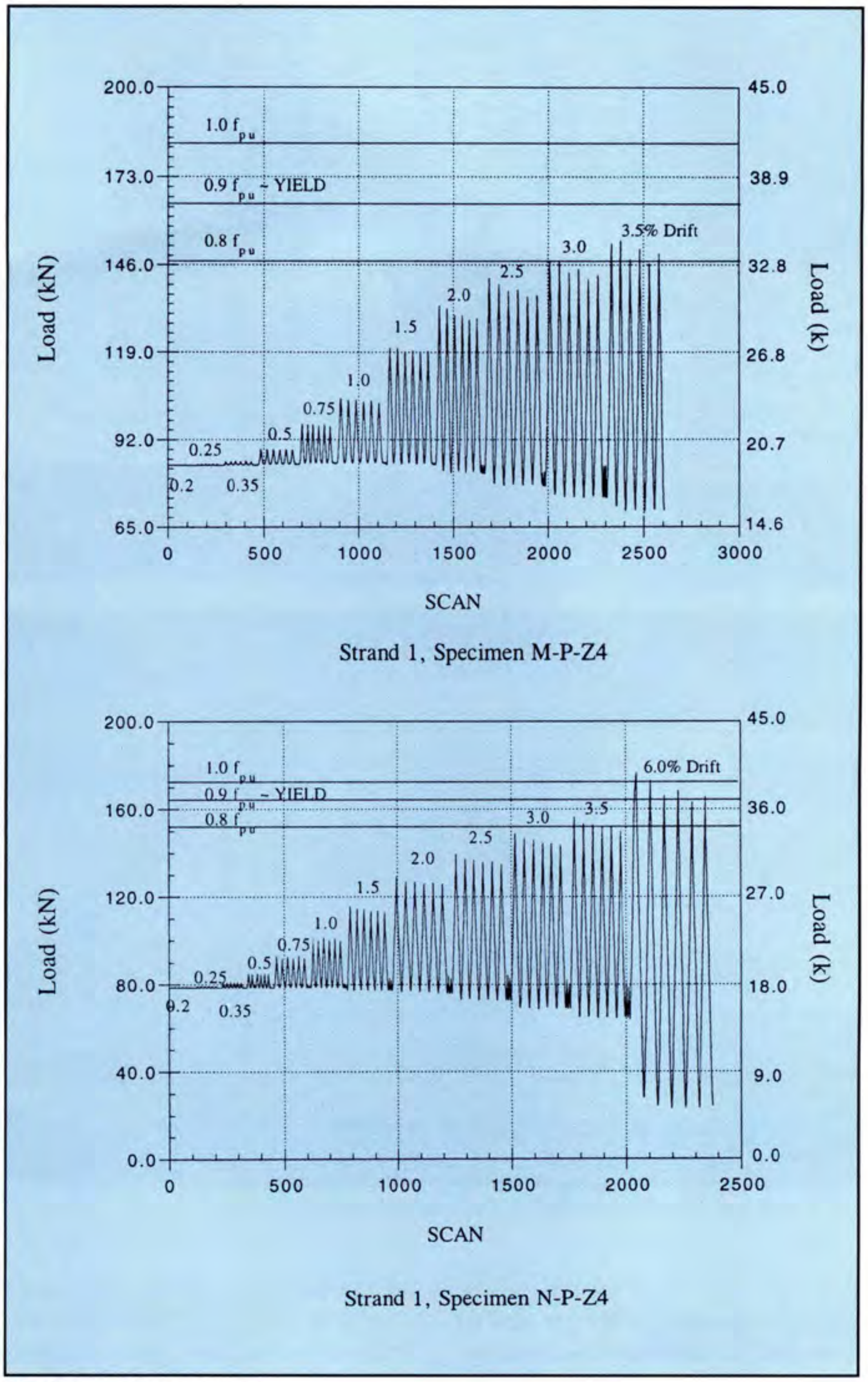

Fig. 8. Strand stress histories.

Table 2. Strengths and drifts achieved in test specimens.

Specimen	Failure drift (percent)	Peak load drift (percent)	M_{exp} (in.-kips)	M_{pred} (in.-kips)	Number of cycles	Failure mode
M-P-Z4	3.4	3.4	1054	966	42	Bar fracture
N-P-Z4	2.9	5.9	1028	1028	38	Debonding
O-P-Z4	3.4	3.9	1231	1116	43	Bar fracture
P-P-Z4	2.9	3.4	1134	1098	57	Bar fracture

Note: 1 in.-kip = 0.113 kN-m.

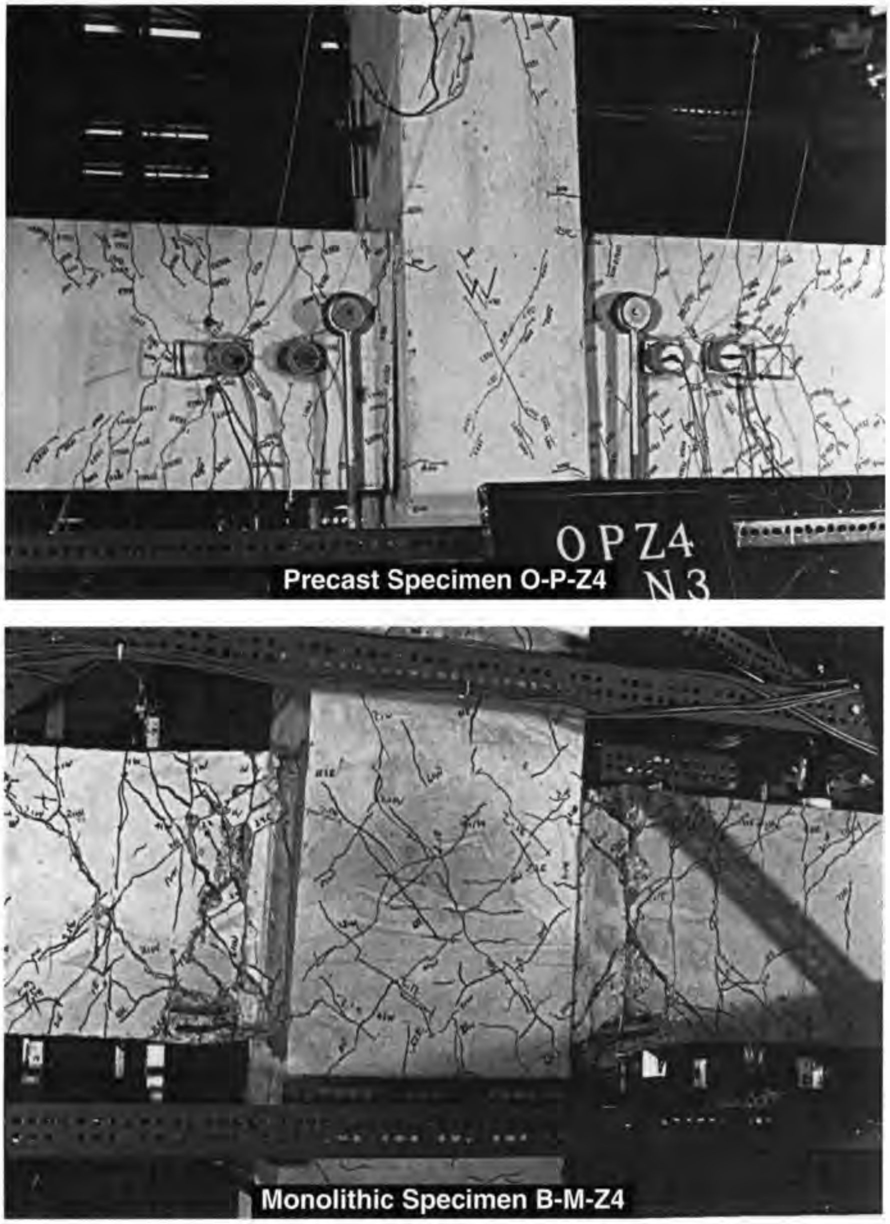

Fig. 9. Precast (top) and monolithic (bottom) specimens at failure.

creased to almost any level desired by selective debonding of the bar steel on both sides of the beam-to-column interface. This feature is not available in conventional, monolithic, non-prestressed designs.

The predicted and measured strengths of the specimens are listed in Table 2. The predicted strengths were calculated based on strain compatibility and the procedure is described by Cheok et al.[2] The ratio M_{exp}/M_{pred} averaged 1.07 for the precast specimens and 1.14 for the monolithic ones. The difference is explained partly by the fact that Specimen N-P-Z4 failed prematurely by debonding of the bar reinforcement, thereby reducing the measured strength, and partly by the fact that a significant proportion of the strength of the precast specimens was derived from the post-tensioning steel. The properties of strand are much more closely controlled than are those of bar reinforcement and it possesses less strain hardening, so a more accurate prediction of the strength should be expected.

The precast and monolithic specimens after failure are shown in Fig. 9. They show that the monolithic specimens suffered severe damage but in the precast specimens, the cracking in both the beams and column was minimal. No crack was wider than 0.025 in. (1 mm.) at full load and all cracks closed completely when the load was removed. The strains in the transverse WRG steel in the beams were on the order of 10 percent of yield. This finding is in keeping with the level of cracking observed and supports the view that the shear reinforcement was supplying little strength and could be reduced significantly. In the precast specimens, the displacement returned almost to zero when the load was removed.

Minor spalling of the surface concrete occurred at the beam corners under the steel armor angles, which were highly effective in confining the concrete beneath them. The grout in the beam-to-column interface crushed during the later load stages, but the fibers prevented it from falling out. Keeping the grout in place is essential for maintaining the clamping force at the beam-to-column interface. Instrumentation showed that no slip occurred there at any time.

When the precast specimens failed at approximately 3 percent drift, the strand had not yielded. Fig. 8 shows that there was a slight stress loss at about 3.5 percent drift, attributed to local crushing of the grout at the beam-to-column interface. Specimen N-P-Z4 was subsequently cycled to 6 percent drift, at which point the strands yielded. However, even then, when the load was removed approximately 30 percent of the initial prestress remained and no slip occurred at the beam-to-column interface.

Specimen N-P-Z4 failed by debonding of the stainless bar reinforcement. Lugs had to be made by machining deformations onto a plain bar. The pattern had been selected for ease of machining, so its bond characteristics were relatively poor. The lug pattern was changed for Specimen P-P-Z4, which suffered no bond problems.

The other specimens failed by fracture of the bars, which was caused by the fact that the bond of the bars in the grouted duct was exceptionally good, so the majority of the strain was concentrated over a short distance at the beam-to-column interface. The fracture did not signal a complete loss of strength in the beam and it occurred at about the drift at which the monolithic specimens failed, so the consequences of it should be evaluated with that in mind. Furthermore, the drift at fracture could be increased by debonding the bar more than the 2 in. [51 mm, 6 in. (152 mm) full-scale] used in these tests.

The energy dissipated by the precast specimens compares quite favorably with that of the monolithic ones. Except for Specimen N-P-Z4, the precast specimens dissipated at least as much energy per cycle as the monolithic specimens up to 1.5 percent drift. At higher drifts, they dissipated an average of 75 percent as much.

This information must be tempered by the fact that the precast specimens underwent at least four times as many cycles as the monolithic specimens. Furthermore, dynamic analyses (Priestley and Rao,[11] Mole[12]) have shown that seismic displacement is much more strongly influenced by the individual characteristics of the ground motion than by modest changes in damping, so the slight difference in energy dissipation at large drifts may not be very important.

DESIGN CRITERIA

The seismic design procedure in the Uniform Building Code[3] is expressed in terms of applied loads and strengths. It is an empirical procedure that fails to recognize the dynamic characteristics of the system being designed. Because the hybrid system differs significantly from the yielding model that underlies the development of the UBC procedure, the UBC method is not a particularly appropriate design tool for the hybrid system. Displacement-based design procedures (Kowalsky et al.[13]) may be more rational and may be adopted in the future, but at present the force-based approach is embodied in codes of practice and so must be used.

Several design issues arise. The first issue is how to establish the design moment strength of the beams. In reinforced concrete and in partially prestressed beams, the beam is considered to have failed when the concrete strain reaches 0.003 in. per in., but that criterion is based on the assumption that the rotation is distributed over a finite length of the beam. This criterion cannot be applied at the beam-to-column interface of the hybrid system because the strain field is not even defined.

Therefore, one alternative approach would be to use the moment at first yield of the reinforcing bar steel, but this would be more conservative than the strain compatibility approach used in prestressed systems because the stress in the strand would be unrealistically low, especially considering the low level of initial prestress used. A compromise is suggested here in which the nominal flexural strength, M_n, is said to be reached when the bar steel reaches the start of the strain hardening, which for many bars is at a strain of approximately $5\varepsilon_y$.

This strain is approximately double the lower limit for tension controlled behavior suggested by Mast[14] and now embodied in Appendix B of ACI 318-95.[15] The strains in the concrete and steel are then established by compatibility of deformations, and the stress in the post-tensioning steel is calculated.

The plastic moment capacity, M_p, is calculated separately as the moment corresponding to ultimate stress in the reinforcing bars, where once again the strand stress is obtained using strain compatibility. The plastic moment, M_p, is used to define the largest possible moment that could exist, and is needed for the shear design.

For any given beam size, the required flexural strength can be satisfied by any one of an infinite number of combinations of strand and bar reinforcement. The selection of a suitable arrangement is influenced by the following criteria:

- The tension force in the post-tensioning steel must be able to force the bars to yield in compression in order to return the system to zero displacement when the lateral load is removed.
- The tension force in the post-tensioning steel must generate sufficient clamping force between the beam and column to resist the vertical shear by friction.
- The initial tension stress in the post-tensioning steel must be low enough that the elongations caused by earthquake motions do not cause it to yield.
- The total amount of the reinforcing steel must be sufficient to provide the required damping by hysteresis.
- Sufficient space should be available for the post-tensioning anchors in the beam-to-column joints at the end of the building.

In general, the displacement response is smaller if more energy is dissipated, and this is especially true if the input motion is nearly periodic (Mole[12]). Thus, unless special circumstances dictate otherwise, the ratio of bar steel to post-tensioning steel should be maximized subject to the limit imposed by the clamping force requirement. In general, this may be achieved when the post-tensioning and the bar steel each provide approximately 50 percent of the resisting moment, M_n.

The effective stress in the post-tensioning tendon is important and is limited by the target drift and the beam geometry. If the prestress is too high, the tendon will yield at an unacceptably low drift. If it is too low, an uneconomically large area of strand will be needed to provide the required clamping force. If the beam itself acts as a rigid body, the strain increase due to earthquake drift can be shown (Stanton[16]) to be:

$$\Delta\varepsilon = \frac{\theta h}{L_{beam}} \tag{1}$$

where

θ = drift angle

h = beam depth

L_{beam} = clear span of beam

In practice, local deformations of the grout joint or the concrete in the beam end will reduce the true strain change to a somewhat smaller value. Eq. (1) allows the prestress level to be selected so as to ensure that the post-tensioning steel remains elastic at the expected drift.

In the simplest version of the system, the post-tensioning steel is unbonded and the stress is developed by mechanical anchorages. Supplementary anchorage by local bonding is not necessary but may be incorporated if desired without detracting from the functioning of the system.

Even if the post-tensioning tendon is unbonded, it does not slip relative to the concrete at midspan of each beam because this is a point of symmetry. Thus, the post-tensioning steel could be bonded at midspan and unbonded

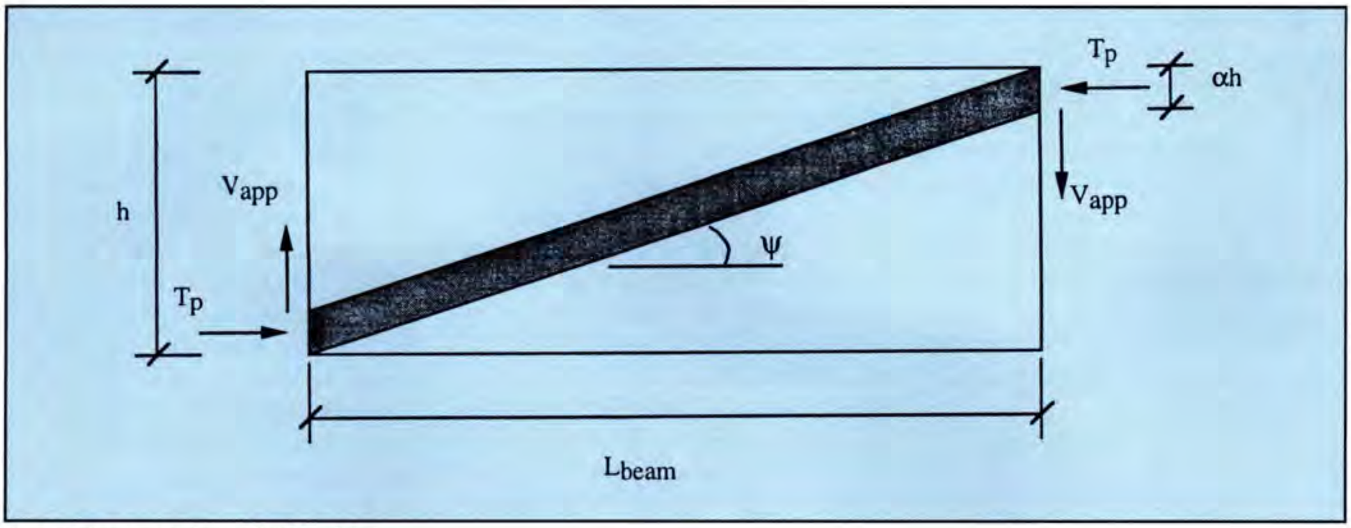

Fig. 10. Strut and tie model for shear.

elsewhere, as shown in Fig. 5, so the relative slip could still occur where it is needed at the column face. By this means, back-up protection against progressive collapse in the unlikely event of anchorage failure could be obtained on a bay-by-bay basis.

One way of achieving the partial bonding would be to place a continuous duct in the beam, but to debond it (perhaps by placing it inside a sleeve) at the beam ends. At midspan, the duct would be embedded directly in, and bonded to, the concrete. The duct could then be filled with grout along its full length, which would be a relatively simple operation on site and would provide the additional benefit of extra corrosion protection.

Shear-friction resistance to seismic shears at the beam-to-column interface can be shown to be a function of the span-to-depth ratio, if the beam contains only unbonded post-tensioning steel (Stanton[16]). Specifically, slip will not occur provided that:

$$\mu \geq \frac{(1-\alpha)h}{L_{beam}} + \frac{wL_{beam}}{2T_p} \qquad (2)$$

where

μ = coefficient of friction

αh = depth of compressed concrete ($\approx 0.1h$)

w = gravity load on beam

T_p = instantaneous tension in post-tensioning steel

If the gravity load is ignored for simplicity, μ is conservatively taken as 0.7 and $\alpha \approx 0.1$ then, for no slip, the beam must have $L_{beam}/h \geq 1.3$. In practice, all beams in a frame will easily satisfy this criterion. When gravity load is included and some of the flexural strength is derived from reinforcing bars, the minimum L/h ratio to prevent slip rises somewhat, but the criterion can still be met without difficulty. This analysis is supported by the experimental evidence. In the tests, the ratio was 4.84 and no slip occurred.

Shear reinforcement requirements in both the beam and the beam-to-column joint of the hybrid system are significantly lower than in a conventional frame. Not only would the reduction in tie steel save material, but it significantly simplifies the task of bar placement and thereby offers potential labor cost savings as well.

For the idealized case of a beam reinforced only with unbonded post-tensioning steel and subject only to seismic shear, the shear can be examined from a strut-and-tie perspective as illustrated in Fig. 10. The shear force is introduced by a single diagonal force at one corner of the beam. Its angle of application is defined by:

$$\Psi = \frac{V_{app}}{T_p} = \frac{(1-\alpha)h}{L_{beam}} \qquad (3)$$

The angle represented by Eq. (3) coincides exactly with the orientation of a single strut running approximately between opposite corners of the beam. The strut thus carries all the shear and no reinforcement is necessary for shear. Some tie reinforcement would be needed to confine the compressed concrete at the beam ends and to hold the cage together during casting, so in practice some stirrups would be used.

In a real hybrid system, some of the end moment is introduced by bonded bar steel and some gravity shear must also be carried, so some shear reinforcement would be needed. However, the quantity would be significantly smaller than that required in a non-prestressed frame.

Joint shear strength in the beam-to-column joint often dictates the column size in a cast-in-place frame. Priestley and MacRae[5,6] have shown by the use of strut-and-tie models that the required joint shear reinforcement in a purely unbonded post-tensioned frame is less than in a conventional frame.

The reasoning is similar to that for the beam steel, i.e., the joint forces are introduced as external diagonal compression forces, rather than as bond forces from bars passing through the joint, so they can be largely carried by a single diagonal compression strut through the joint. The limitation on this behavior appears to be development of the vertical component of the diagonal force, which depends on bond to the vertical column bars.

In a hybrid frame only part of the benefit would be gained because some of the joint shear is introduced by bond from the bar steel. Thus, physical arguments suggest that the column size could be reduced without jeopardizing the joint shear strength, although ACI 318-95 does not yet permit such a change. Any reduction in column size would reduce the column weight and facilitate transportation and erection.

CODE IMPLICATIONS

The Uniform Building Code[3] expressly sanctions the use of only certain building systems. Precast, prestressed concrete is not among them. The closest possibility is a reinforced concrete special moment-resisting frame, but the hybrid system violates several requirements for that category, including:

- Only bar reinforcement with an actual yield stress less than 78 ksi (538 MPa) may be used. The hybrid system uses some prestressing strand.
- Beams may not contain reinforcing bar splices closer than twice the effective depth of the section (2*d*) to the beam end. The hybrid system uses bars bonded in ducts at the beam ends.
- The transverse reinforcement in the beams and columns may not be less than that specified by code. (Full transverse reinforcement could be placed in the hybrid members, but it would be unnecessary and wasteful.)

Thus, the only category under which the hybrid system may be used is the "undefined structural system," which "shall be shown by technical and test data which establish the dynamic characteristics and demonstrate the lateral force resistance and energy absorption capacity to be equivalent to systems listed in Table 16-N for equivalent R_w values."

The test data reported here and in Stone et al.[17] demonstrate without question that those criteria are satisfied up to a drift of 1.5 percent. At higher drifts, the hybrid system displays approximately 75 percent of the energy dissipation of a conventional system, and all other characteristics, such as strength and stiffness retention, shear strength and resistance to structural damage and residual drift, are superior.

Some codes (e.g., NEHRP,[18] BOCA[19] and SBC[20]) now permit the use of some prestressing steel for seismic resistance. However, the hybrid system would still be non-compliant because, for example, NEHRP specifications limit the prestress to 350 psi (2.41 MPa) on the concrete and the contribution of the post-tensioning steel towards M_n to 25 percent of the total, both of which would prevent the hybrid system from working in the intended fashion. Further code changes are needed to allow innovative systems such as the hybrid frame to be used on a regular basis.

CONCLUSIONS

Based on the results of this test program, the following conclusions can be made:

1. Precast, prestressed concrete has unique properties that can offer advantages for seismic resistance over bar-reinforced cast-in-place concrete. Precast concrete should be viewed as an individual construction system and should not be relegated to mimicking cast-in-place technology, whose seismic behavior it is capable of surpassing.

2. A hybrid system can be designed to have the same flexural strength as a conventionally reinforced system with members of the same size.

3. The shear resistance of the hybrid system is superior to that of a conventionally reinforced frame. No degradation whatsoever of the shear strength was observed in the tests, and the stresses in the shear reinforcement never exceeded $0.15f_y$.

4. The use of WRG (welded reinforcement grids) for transverse reinforcement significantly simplified the assembly of the reinforcement cages.

5. The hybrid system is self-centering and displays essentially no residual drift.

6. The hybrid system has a very large drift capacity. In tests, it withstood cyclic drifts of ±6 percent while maintaining at least 55 percent of its strength.

7. The hybrid system dissipates more energy per cycle than a conventionally reinforced frame, up to a drift of 1.5 percent. As the drift rises above that level, the energy dissipation gradually drops to approximately 75 percent of that of the reinforced frame.

8. The damage suffered by the concrete in the hybrid frame was minimal. No cracks were visible after the load was removed, even in the test to 6 percent drift. The comparable reinforced frame specimens were damaged beyond repair after approximately one quarter of the number of cycles to a maximum of 3.5 percent drift. The low level of damage in the hybrid frame is expected to minimize downtime and repair costs after a severe earthquake.

9. The precast system requires no corbels and offers clean, economical architectural expression.

RECOMMENDATIONS FOR FURTHER RESEARCH

Further research is desirable in the following areas:

1. Nonlinear dynamic analyses using analytical models that reflect as accurately as possible the properties of the physical system.

2. Shaking table tests to verify dynamic behavior.

3. Cyclic load tests on reinforcing bars grouted into ducts to establish their bond properties.

4. Tests on specimens with different levels of initial prestress.

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