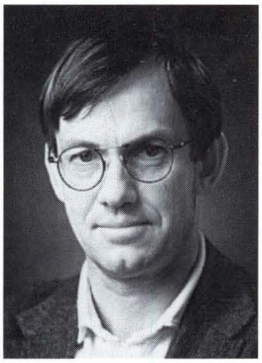
The PRESSS Program — Current Status and Proposed Plans for Phase III



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The current status of the coordinated Precast Seismic Structural Systems (PRESSS) program is summarized and plans for the final phase outlined. PRESSS Phase III is built around two large-scale super-assemblage experiments, one of a frame building and the other of a panel structure. Precast concrete elements will be used to the maximum in both structures. The buildings will be designed using a displacement-based design procedure developed as part of the PRESSS program and the results of experimental programs forming PRESSS Phase II. The purposes of PRESSS Phase III are to demonstrate the viability of precast concrete design for regions of high seismicity, to establish clearly the predictability and dependability of performance of properly designed precast concrete buildings under seismic response, and to emphasize the advantage of precast concrete seismic performance, including reduced damage levels compared with equivalent reinforced concrete or steel structures. A final objective of the PRESSS program will be to develop design guidelines for precast concrete structures in zones of high and moderate seismicity which can be incorporated into the model building codes.

The PRESSS research program is the fourth phase of a joint U.S.-Japan coordinated analytical and large scale testing program for seismic response of buildings. The research program is jointly sponsored by the National Science Foundation (NSF), the Precast/Prestressed Concrete Institute (PCI) and the Precast/Prestressed Concrete Manufacturers Association of California (PCMAC).

The seven-year research program has two fundamental aims:

- To develop comprehensive and rational design recommendations needed for a broader acceptance of precast concrete construction in different seismic zones.

• To develop new materials, concepts and technologies for precast concrete construction in different seismic zones.

The PRESSS research program consists of three phases. Phase I was completed in 1993. Most of the Phase II projects will be completed by the end of 1996. Phase III is scheduled to be initiated in 1997.

PRESSS Phase I consisted of five projects:

1. Concept development

2. Connection classification and modeling

3. Analytical platform development

4. Preliminary design recommendations

5. Research coordination

Several papers associated with the PRESSS Phase I Program and the four PRESSS seismic workshops, conducted in April 1991, were published in the PCI JOURNAL.[1,2,3]

PRESSS Phase II has emphasized experimental and analytical studies of different ductile-connection precast systems and the development of seismic design procedures for precast buildings in various seismic zones.

Four generic types of connections have been considered in PRESSS Phase II, as briefly outlined below:

Non-Linear Elastic (NLE) Connection Systems — These generally involve some form of unbonded post-tensioning to form the connection.[4] As cracking develops in the connection, the stress in the post-tensioning increases slowly, but remains in the elastic range, resulting in largely elastic response with non-linear characteristics, as illustrated in Fig. 1(b). Energy dissipation is larger than for a purely linear elastic system, at about ξ = 10 percent equivalent viscous damping, but is much less than for an equivalent reinforced concrete or steel structure.

Advantages include reduced damage levels, reduced residual drift and retention of high initial stiffness after a major earthquake, and simplified member design, particularly of beam-to-column joints in frame buildings. This improved response is achieved at the expense of slightly increased displacement resulting from the reduced effective damping.

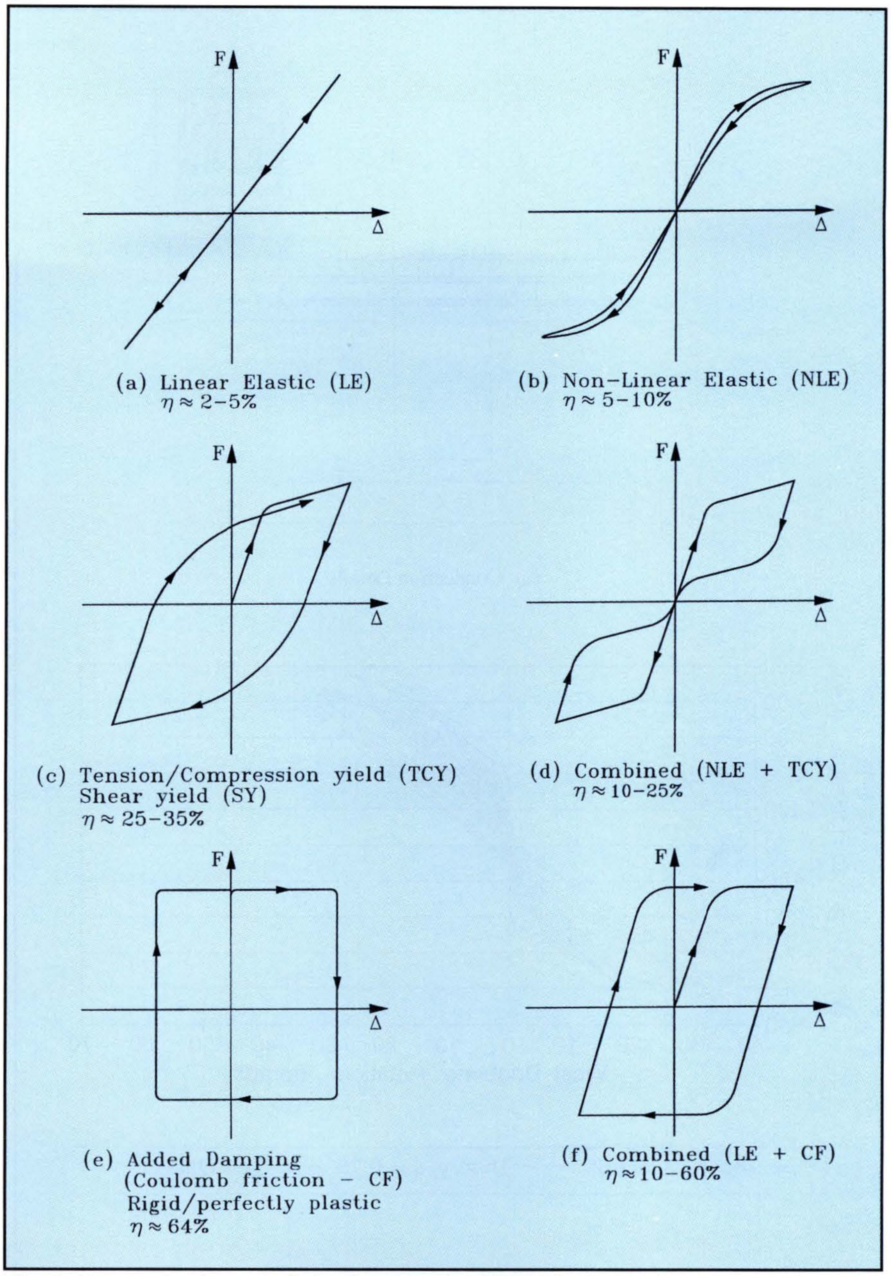
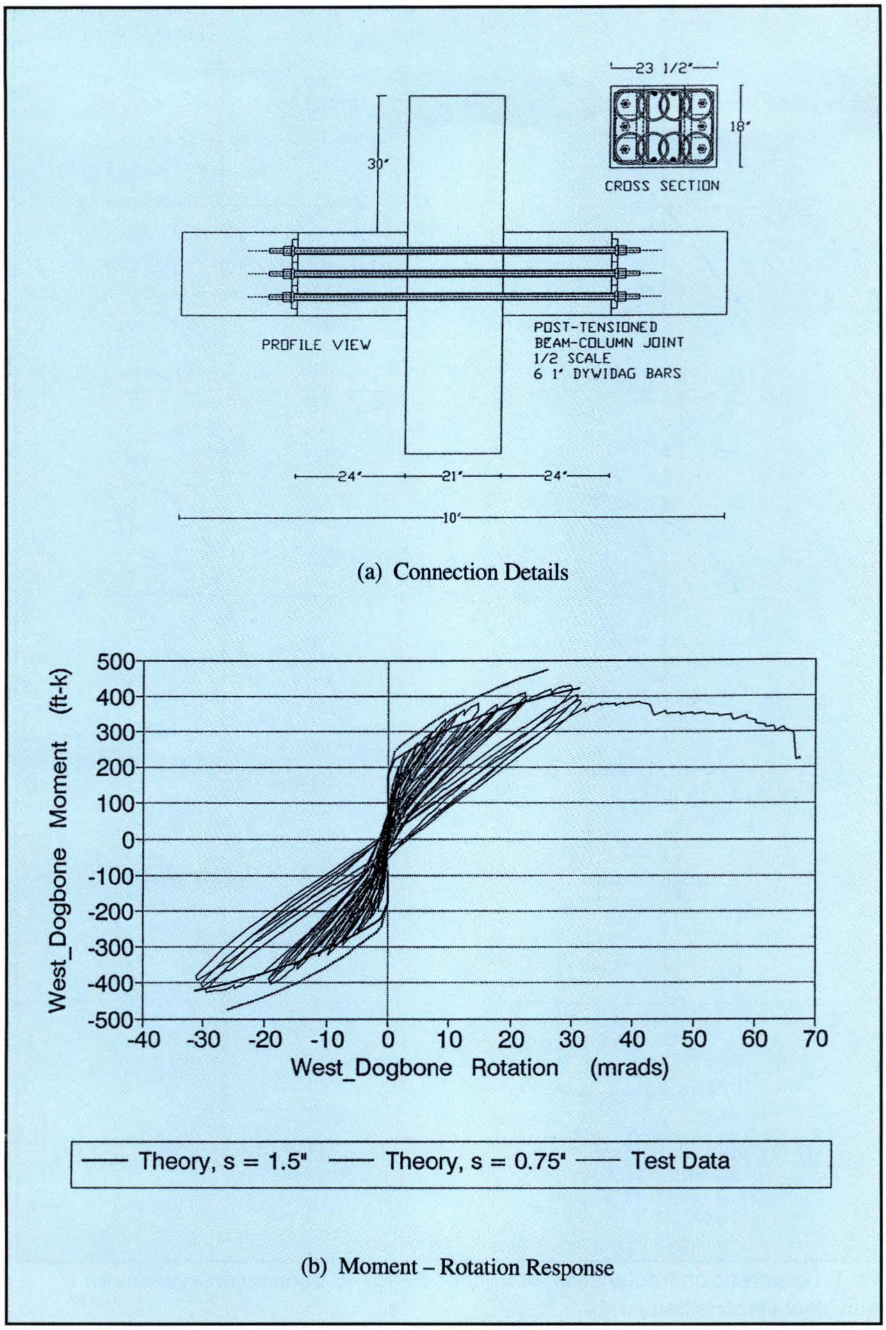


Fig. 1. Hysteretic characteristics for generic PRESSS connection systems (η = equivalent viscous damping).

Tension-Compression Yield (TCY) Connection Systems — Connection elements, normally of medium strength steel, are designed to yield in tension and compression under opposite directions of seismic response. Behavior is thus similar to monolithic reinforced concrete response and is characterized by reasonably high energy dissipation, as illustrated in Fig. 1(c). Equivalent viscous damping at high ductility levels is about ξ = 35 percent. Disadvantages of TCY systems include high residual displacement and low residual stiffness after inelastic seismic response. Special treatment of contact surfaces between precast elements in the form of shear keys or special roughening may be needed to avoid excessive shear deformation at the interface.

An attractive compromise between the NLE and TCY systems can be obtained by a simple combination. Primary connection is provided by unbonded post-tensioning with additional damping provided by special TCY elements. The relative strengths of the NLE and TCY elements can be chosen to ensure significantly increased damping while maintaining the advantages of minimal drift and high resid-



(a) Connection Details

(b) Moment – Rotation Response

Fig. 2. Unit UMn-PTB elastic non-linear subassemblage (University of Minnesota).

ual stiffness of the NLE system, resulting in the "flag" type of hysteretic response illustrated in Fig. 1(d).[5]

Shear Yield (SY) Connection Systems — The NLE and TCY systems are activated by moment at the connection. A different category of ductile connection involves inelastic shear response at the connections. Typically, such connections will occur at midspans of beams using precast tees for frame systems, or at vertical separations between adjacent precast panels in panel systems. Hysteretic response is similar to the TCY systems, shown in Fig. 1(c).

Energy Dissipating/Coulomb Friction (CF) Connection Systems — Special damping devices, typically involving some form of friction sliding, can be placed in the connections between precast frame or panel elements. Hysteretic response of these devices can generally be characterized as rigid-perfectly plastic, as shown in Fig. 1(e). When used in combination with linear elastic connection elements [Fig. 1(a)], bilinear elasto-plastic response shown in Fig. 1(f) results, with equivalent viscous damping in the range $\xi = 20$ to 60 percent, depending on the level of ductility developed in the connection.

The concepts described above are generic and are applicable to both frame and panel structures. Experimental research in Phase II has been directed towards establishing the viability of the generic systems rather than development of the most cost-effective design details utilizing each system; this is felt to be an industry prerogative. It is expected that a number of innovative connection systems can be developed based on each of the generic systems.

On the design side, two approaches have been simultaneously developed. The first approach involves developing design rules for "strong-connection" systems (i.e., where ductility is not required within the connection) based on reinforced concrete emulation. As a result of PRESSS initiatives, appropriate provisions have already been incorporated in the 1994 National Earthquake Hazards Reduction Program (NEHRP) seismic design recommendation,[6] and PRESSS is actively involved in developing provisions for the 1997 Uniform Building Code (UBC) through an ad hoc subcommittee of the Structural Engineers Association of California (SEAOC) Seismology Committee.

The second approach involves the development of displacement-based design procedures for ductile-connection systems, because the characteristics of these may differ significantly from reinforced concrete or structural steel systems typically envisioned by seismic codes.

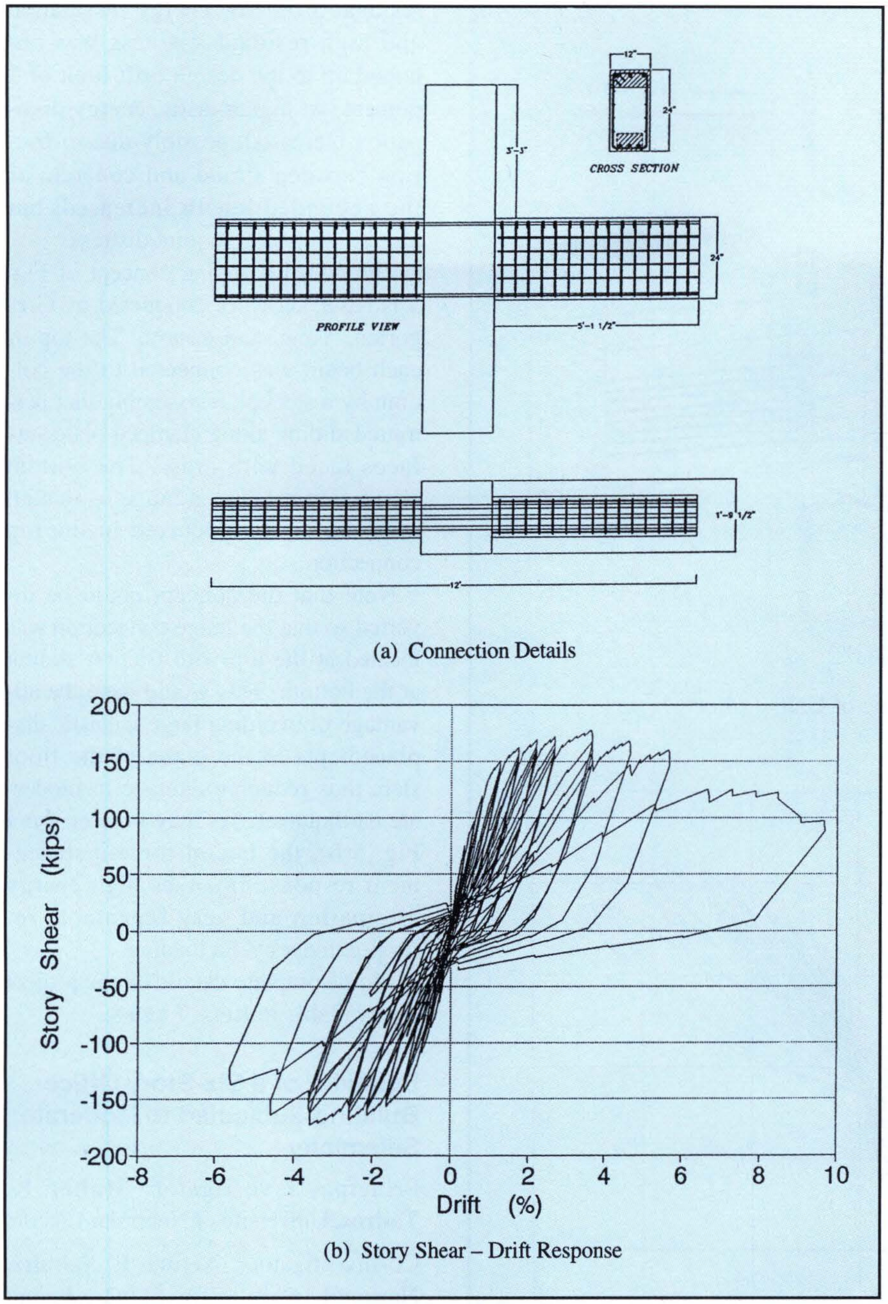
The current status of PRESSS Phase II projects is described briefly in the following sections.

SUMMARIES OF PRESSS PHASE II PROJECTS

Ductile Connections for Precast Concrete Frame Systems (Part I)

Principal Investigator: **Catherine W. French,** University of Minnesota

Concepts for ductile connection between precast beam and column members representing non-linear elastic (NLE) and tension-compression yield



(a) Connection Details

(b) Story Shear – Drift Response

Fig. 3. Unit UMn-TCY tension/compression yield subassemblage (University of Minnesota).

concepts (TCY) (see Fig. 1) have been developed and tested at the University of Minnesota. The connection details were conceived to be appropriate for "up-and-out" construction and, thus, used individual connection details rather than, say, connecting a string of beam-to-column connections with continuous prestressing. Representative examples of the NLE and TCY connections are shown in Figs. 2 and 3, respectively, together with test results in the form of lateral force-drift hysteretic response.

Unit PTB (post-tensioned beam, see Fig. 2) was based on the unbonded post-tensioning concept proposed by Priestley and Tao[4] and utilized Dywidag bars for connection. The bars were designed to remain within the elastic range of response up to the design drift of 2 percent. As a consequence, the connection was capable of accommodating large deformation without loss of load carrying capacity, as shown in Fig. 2(b). The elastic behavior also ensured minimal residual deformation on load removal.

Unit TCY (see Fig. 3) incorporated blockouts through the beams and embedded corrugated pipes in the column to accommodate placement of reinforcement. Continuity reinforcement was placed in the beam blockouts, slid through the column, tied in place and subsequently grouted. The hysteretic response, shown in Fig. 3(b), represents dependable response for drift levels substantially larger than the design drift limit of 2 percent.

Based on the results observed during the laboratory tests, analytical connection models have been developed to be included in the general purpose finite element program DRAIN-2DX. The model behavior is characterized by the moment-rotation curves observed during the laboratory tests. Two connection models have been formulated, representative of the nonlinear-elastic and tension-compression yielding concepts, respectively.

These models have been utilized in the numerical analyses of two complete precast frame systems, one five stories and the other fifteen stories, subjected to a variety of loads simulating available earthquake records. The results from these analyses provide the behavior of the proposed connections when utilized in a multistory structure subjected to earthquake loading. Preliminary results indicate that precast frame systems may experience only slightly larger displacements than those of monolithic concrete frame structures.

More complete information on the project is available in Refs. 7 and 8.

Ductile Connections for Precast Concrete Frame Systems (Part II)

Principal Investigator: **Michael E. Kreger,** University of Texas at Austin

The work at the University of Minneapolis has been complimented by a parallel study at the University of Texas at Austin. Test units based on the NLE, TCY, and CF concepts (see Fig. 1) have been tested. The TCY unit differed from the companion University of Minneapolis unit in that the connecting bars were located above and below the beam rather than on either side. It was found that detailing of force transfer from the beam to the connection was more difficult with this detail, and as a consequence, structural

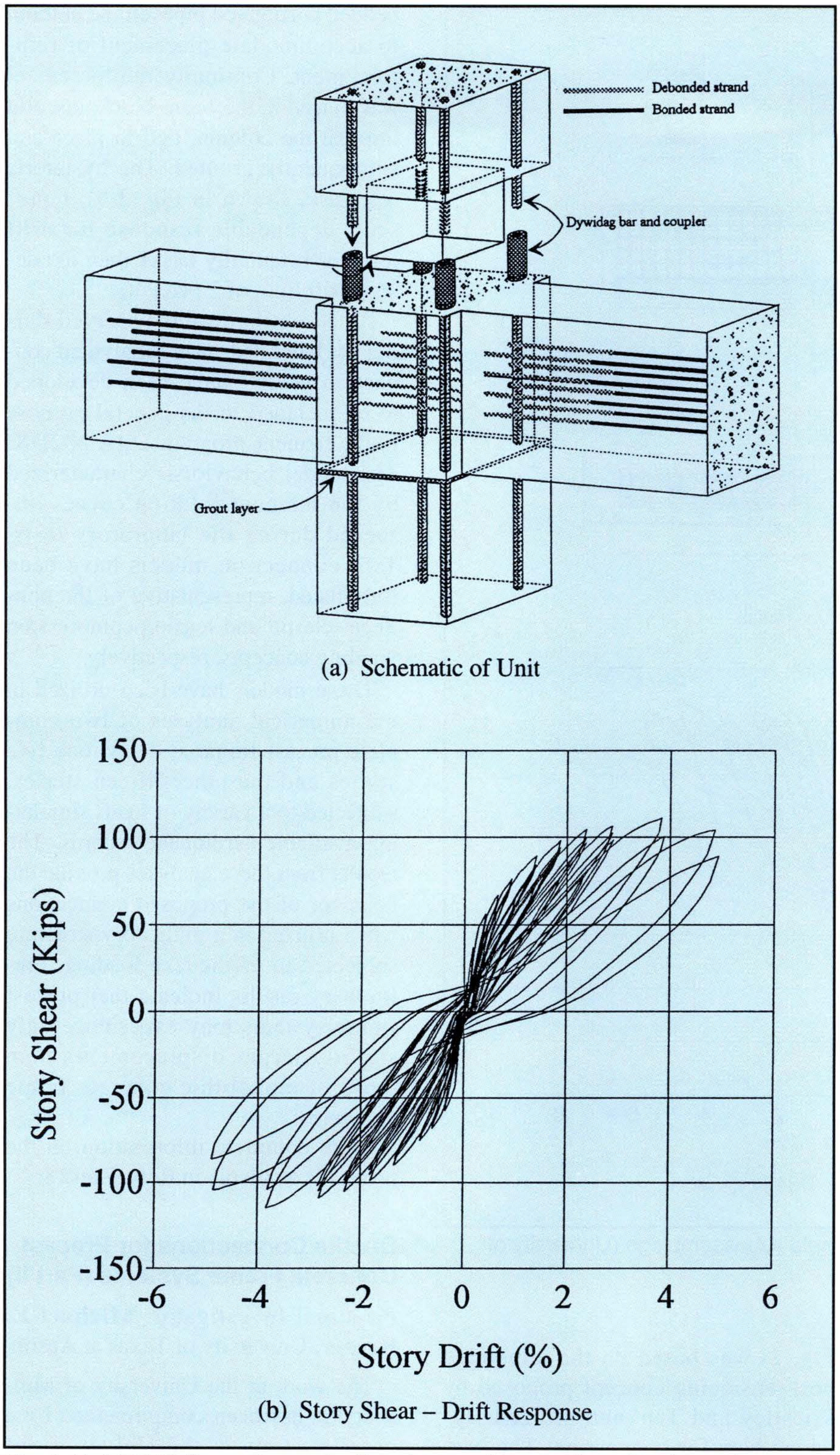


Fig. 4. Unit UT-PTS non-linear elastic subassemblage with pretensioned beam (University of Texas at Austin).

degradation commenced earlier.

Figs. 4 and 5 show concepts tested for the NLE and CF concepts, respectively. The non-linear elastic concept differed from the University of Minneapolis unit and from the tests at the University of California at San Diego (UCSD), described subsequently. In this case, the beam was continuous and pretensioned with strand debonded for a distance on either side of the column [see Fig. 4(a)], with precast columns connected through the beam with couplers.

Lateral force-displacement response, shown in Fig. 4(b), indicated that the expected NLE behavior of low residual drift, low energy dissipation and high residual stiffness was obtained up to the design drift limit of 2 percent. At higher drifts, energy dissipation increased, possibly due to friction between strand and concrete as the debonded length increased, but also possibly due to joint distress.

The added damping concept of Fig. 5 is based on work conducted by Gregorian, Yang, and Popov.[9] The top of each beam was connected to the column by a steel plate assembly that permitted sliding along clamped plate surfaces faced with brass. The bottom connection acted as a hinge so that all inelastic action occurred in the top connection.

Note that the concept could be inverted so that the hinge connection was located at the top with friction sliding at the bottom. This would have the advantage of avoiding large inelastic displacements in the plane of the floor slab, thus reducing damage in moderate earthquakes. As may be seen from Fig. 5(b), the lateral force-displacement response involves high energy dissipation and very repeatable response under cyclic loading.

More complete details on the project are available in Refs. 7 and 8.

Behavior of a Six-Story Office Building Subjected to Moderate Seismicity

Principal Investigator: **Maher K. Tadros,** University of Nebraska-Lincoln

Co-Investigators: **Arturo E. Schultz,** National Institute for Standards and Technology (NIST); **Rafael Magaña,** LEAP Associates International, Inc.

The project has involved two stages of experimental work. The first stage related to the performance of a gravity precast frame system with wide shallow beams tested at the University of Nebraska-Lincoln. Full scale tests were carried out on beam-to-column subassemblages [see Fig. 6(a)] to determine the ability of the gravity frame to sustain the displacement levels imposed by response of the lateral bracing system, without strength degradation. Typical results, shown in Fig. 6(b), indicate that the connections were competent to high drift levels and would also contribute significantly to seismic resistance.

A second experimental program investigating the seismic performance of horizontal and vertical joint connections in precast walls has been carried out at NIST. The connections were designed to be ductile and to be the major location of inelastic response of the structure. Vertical joint connections included different designs of welded loose plates (see Fig. 7) and bolted ductile connections (see Fig. 8) incorporating flexural yield, tension/compression yield (TCY), shear yield (SY) and friction sliding/coulomb friction (CF) concepts.

The most dependable response was obtained from the VJF (vertical joint friction) and UFP (U-shaped flexure plate) connections, as shown in Figs. 8(b) and 8(c), respectively, where dependable hysteretic behavior was obtained up to relative vertical displacements as high as 4 cm (1.6 in.). The force-displacement response for these units is shown in Figs. 9(a) and 9(b), respectively. Note that these units were tested in a horizontal configuration for convenience so the lateral force and lateral displacement shown translate to vertical force and relative vertical slip in the intended configuration. More complete details are available in Refs. 7, 10, and 11.

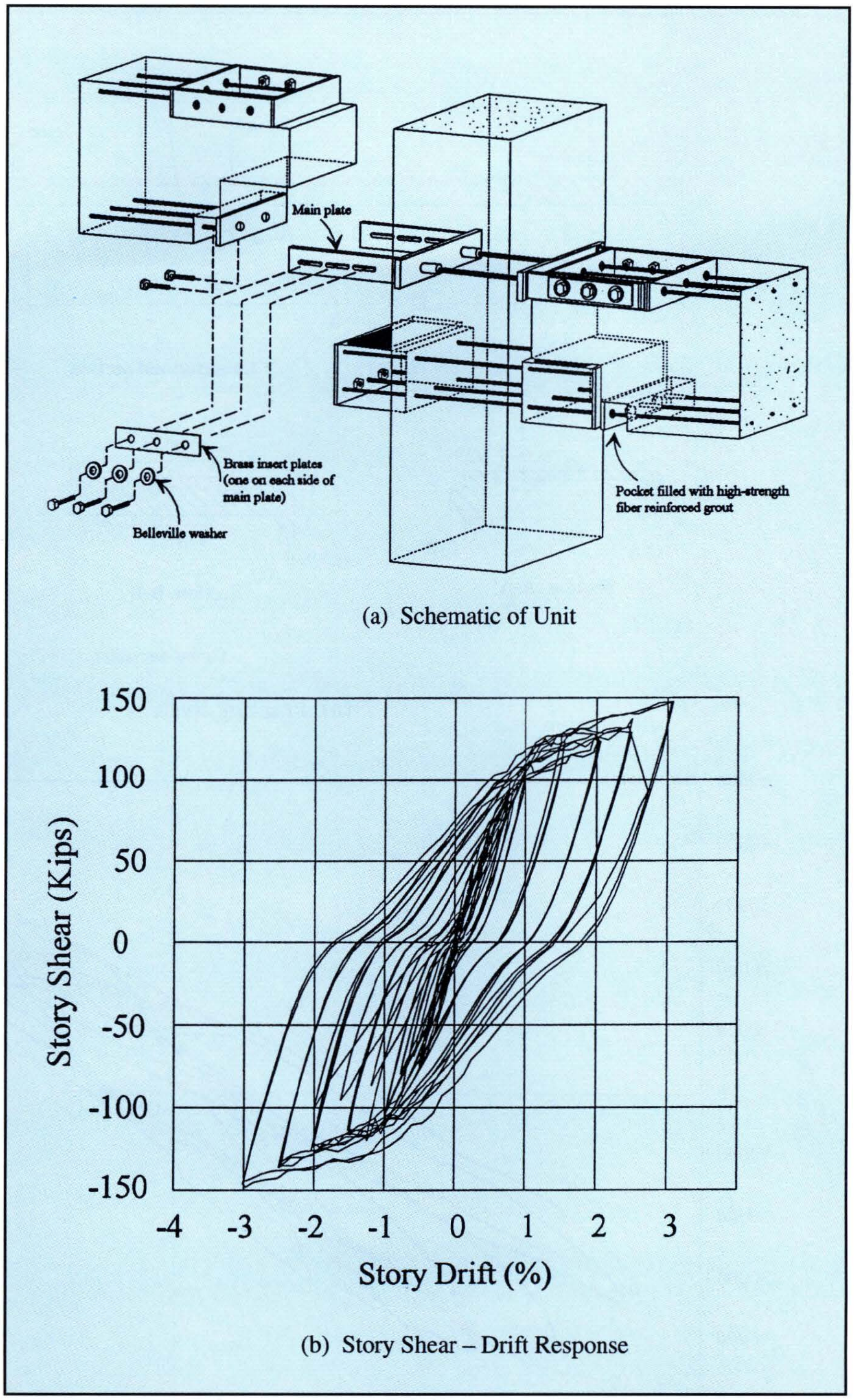


Fig. 5. Unit UT-FR added-damping subassemblage (University of Texas at Austin).

Seismic Response Evaluation of Precast Structural Systems for Various Seismic Zones and Site Characteristics

Principal Investigators: **Stephen P. Pessiki, Richard Sause,** and **Le-Wu Lu,** Lehigh University

This project is based on the response of prototype precast frame and panel structures that have been designed for regions of moderate and high seismicity. Connection response is characterized by fiber beam-to-column elements using the DRAIN-2DX program. Precast systems representing NLE connections have so far been considered, with non-linearity in beam-to-column connections, panel-to-panel connections, and panel-to-foundation connections. Fig. 10(a) shows the analytical model used to represent an NLE post-tensioned beam-to-column connection and an envelope prediction of response compared with a typical experimental result. Fig. 10(b) shows the model representation of a six-story precast wall.

The fiber models have been found to give good predictions of response in PRESSS component tests. Current research involves inelastic time-history analysis parameter studies on both components and prototype frame-wall structures designed for zones of both moderate and high seismicity.[7]

Dynamic Response of Precast Concrete Frames

Principal Investigator: **Daniel P. Abrams,** University of Illinois at Urbana-Champaign

Co-Investigators: **Sharon L. Wood,** University of Texas at Austin and **Neil M. Hawkins,** University of Illinois at Urbana-Champaign

The objective of this research is to

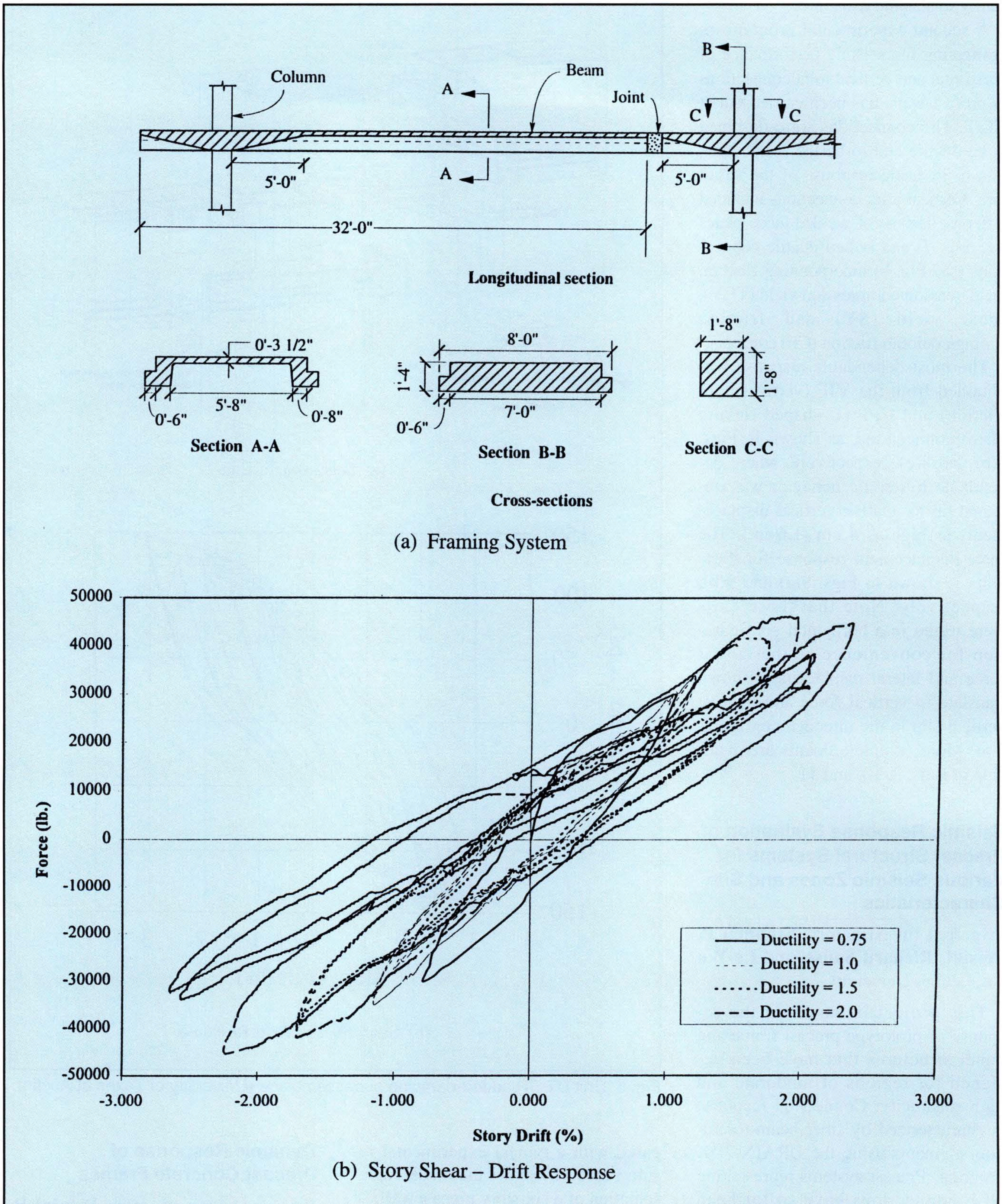


Fig. 6. Lateral response of a gravity-load precast frame for regions of low seismicity (University of Nebraska-Lincoln).

investigate non-linear dynamic response of precast concrete frame systems through a series of shaking table tests on six-story, reduced-scale structures [see Figs. 11(a) and 11(b)]. In particular, differences in response of non-linear, inelastic systems are being contrasted with response of non-linear, elastic systems by testing two different frame types.

Beam and column members of the first frame type (dog-bone connection, DB1), as shown in Fig. 11(c), are joined using steel bolts that are designed to yield in tension and com-

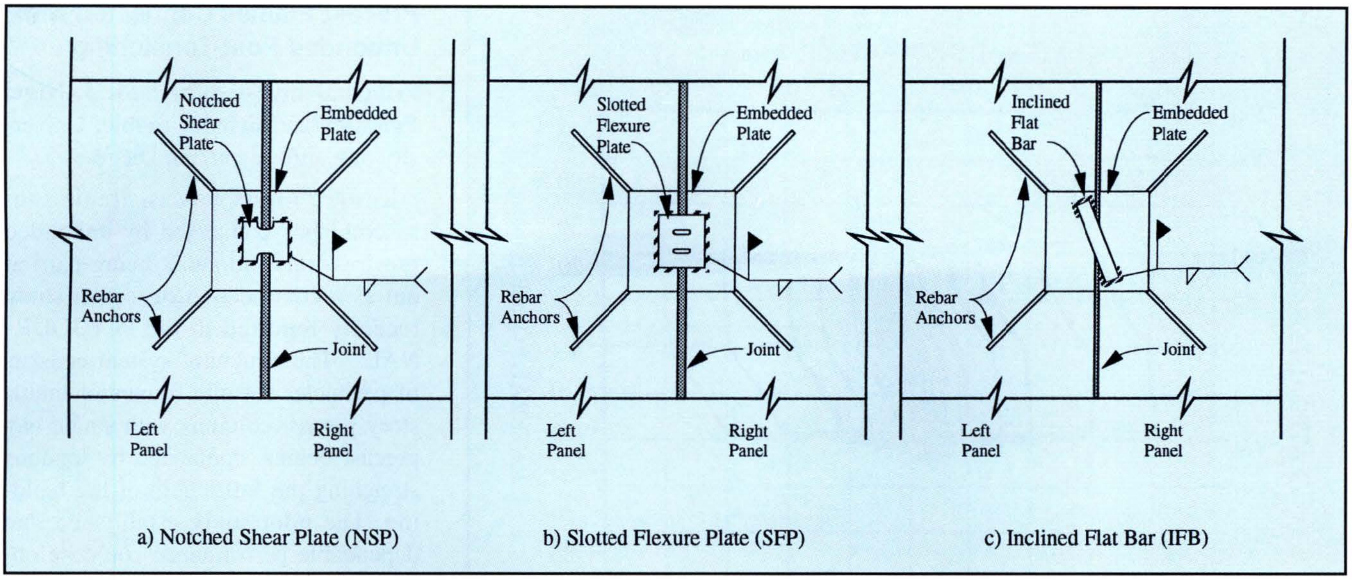


Fig. 7. Welded loose plate connections tested at NIST.

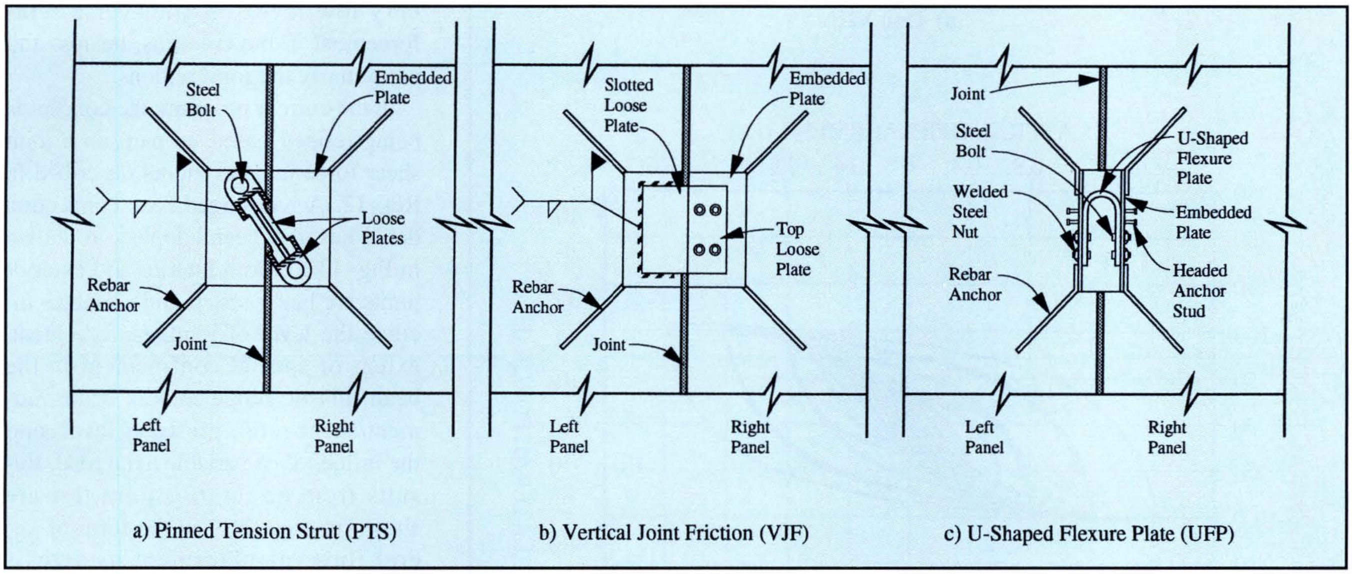


Fig. 8. Bolted ductile connections tested at NIST.

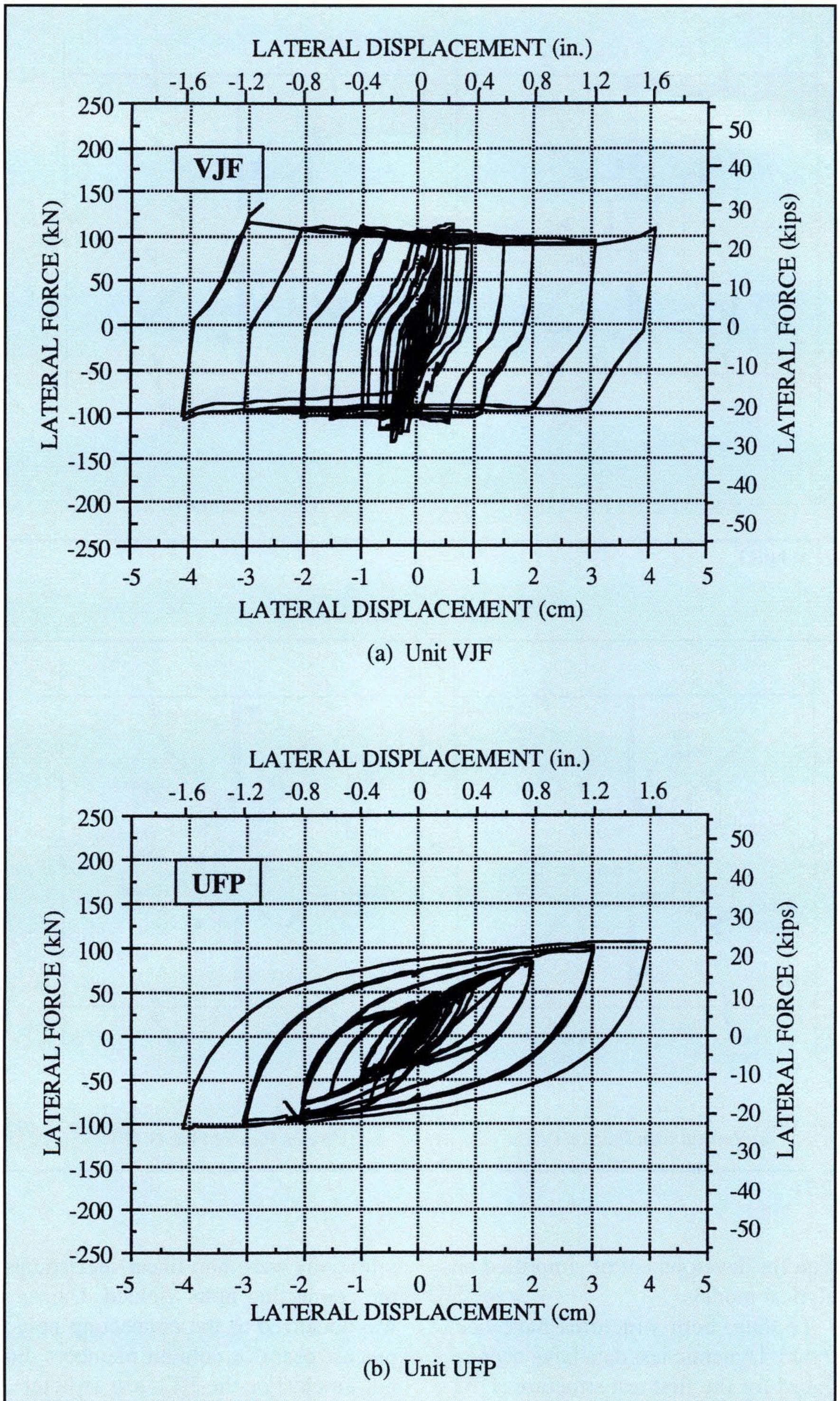
pression and serve as the weak link or fuse element for dissipating inelastic energy. Members of the second frame type (post-tensioned connection, PT1), as shown in Fig. 11(d), are joined by post-tensioning beams to column faces with horizontal unbonded tendons. A series of static load reversal tests are done to define non-linear force-deflection properties of the tension-compression yielding and prestressed connections.

Measured response of the two shaking table test structures provides: (a) a demonstration of seismic performance for precast concrete frame systems; (b) benchmark data for evaluating computational models; and (c) exploratory data for development of simplified analytical models.[7]

To date, both structures have been tested. Dynamic test data have been reduced for the first test structure (DB1) and results have been analyzed. Data from the second structure (PT1) are now being reduced and analyzed. All of the static tests of beam-to-column connections have been completed.

Testing of the two test frames revealed that a precast frame could be designed to respond at desired limit states for various earthquake intensities. Small-amplitude vibrations were linear elastic as the frame system emulated cast-in-place construction. For the DB1 test structure, large-amplitude vibrations were non-linear inelastic as the connecting bolts yielded. Damage was localized at the connecting bolts; precast beam or column members did not crack. For the PT1 test structure, large-amplitude vibrations were non-linear but remained elastic until the post-tensioning tendons yielded. Damage was limited to crushing at the ends of beam members. Little or no cracking was observed.

The period of the DB1 test structure lengthened with progressive damage, and thus lateral inertial forces did not increase proportionately with the intensity of base motion as indicated on spectral response curves computed from measured base



(a) Unit VJF

(b) Unit UFP

Fig. 9. Load-displacement response for Connections VJF and UFP tested at NIST.

motions [see Fig. 12(a)]. The sequence and frequency content of base shear and deflection histories [see Fig. 12(b)] were similar even with non-linear behavior suggesting that a simplified substitute structure approach might be admissible for estimating response maxima.

The measured base shear vs. drift history [see Fig. 12(c)] clearly indicated non-linear action, yet lateral force distributions were triangular and lateral deflected shapes [see Fig. 12(d)] were invariant with the amplitude of vibration. These findings suggest that nonlinear dynamic response could be represented with a single generalized coordinate.

Precast Frames Connected with Unbonded Post-Tensioning

Principal Investigators: **M. J. Nigel Priestley** and **Frieder Seible,** University of California at San Diego

Testing of six precast frame subassemblages connected by unbonded prestressing tendons is being carried out as a continuation of a pilot study recently reported in the PCI JOURNAL.[12] The structural system consists of perimeter seismic frames of multistory precast columns with single bay precast beams, connected by tendons stretching the full length of the building. The pilot study established that dependable performance could be obtained with low residual deformation, low damage, and high residual stiffness up to very large drifts despite very low levels of transverse reinforcement in the columns, beams, and particularly the joint regions.

In the current program, the concept is being refined based, in part, on a joint shear force transfer model described in Ref. 12. A redesigned base connection detail has also been adopted, as shown in Fig. 13(a). Both interior and exterior joints are being tested and variables include the level of joint reinforcement, extent of special confinement in the beam plastic hinge region, beam moment/shear ratio, prestress level, and the influence of variable axial load. Results from an interior joint test are shown in Fig. 13(b) in the form of lateral force-displacement hysteresis loops. Very dependable behavior at high drift levels is apparent.

Results from other tests indicate that the shear strength of the prestressed beams is considerably higher than that predicted by current codes. Dependable beam-to-column joint performance can be obtained with low transverse steel ratios and a coefficient of friction of $\mu = 1$ is appropriate at the beam-to-column interfaces, even without special preparation of the surfaces.

High Performance Fiber Reinforced Concrete (FRC) Energy Absorbing Joints for Precast Concrete Frames

Principal Investigators: **Antoine E. Naaman** and **James K. Wight,** University of Michigan

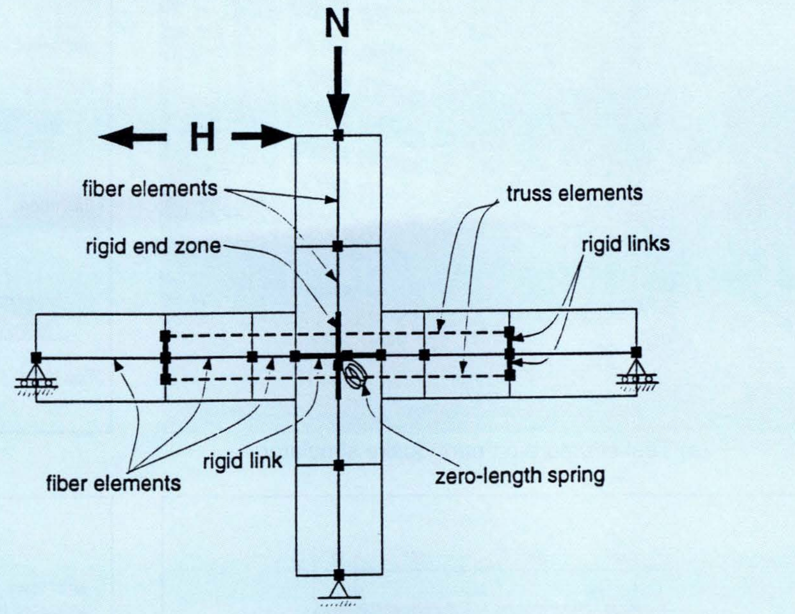
The main goal of this research is to contribute to the development of a high energy absorbing joint (or joints) for precast concrete structures in seismic zones. The joint is made from high performance fiber reinforced cement composites (HPFRCC) such as high fiber content fiber reinforced concrete and SIFCON (slurry infiltrated fiber concrete).

The joint is designed to act as an energy dissipating connector between the beam and the column elements. The immediate objective of the research is to understand the fundamental mechanisms that control the properties of the joint and its response under monotonic and cyclic loads, with particular attention to the issue of bond of reinforcing bars and the shear resistance of the joint material.

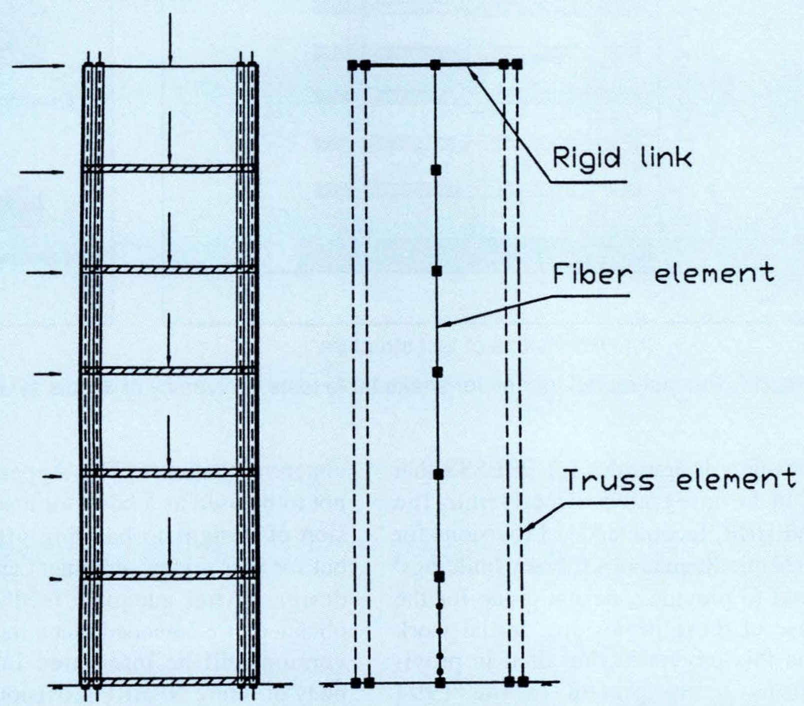
Tests to determine the bond vs. stress slip relationship of the interface between reinforcing bars and HPFRCC under monotonic and cyclic loads have been completed; parameters include four different matrix compositions with target compressive strengths of 5 and 9 ksi (34.5 and 62 MPa), pull-push and pull-pull type of loading, and the use of spiral confinement for comparison.[13]

Fig. 14 describes the bond stress vs. slip response of a reinforcing bar embedded in a slurry infiltrated fiber concrete (SIFCON) composite subjected to reversed cyclic loading. It is observed that significantly high levels of stress can be maintained at relatively high slips. Moreover, no spalling of the concrete cover was observed during the test. These characteristics are important in seismic design for energy absorption and for maintaining the integrity of the structure.

A medium scale test setup to determine the shear response of HPFRCC joints under monotonic and cyclic loading has been built. The setup simulates an exterior beam-to-column subassemblage. It is made out of steel beam and column elements connected to the reinforced concrete "joint" by threaded bars and nuts. Seven joint specimens have been tested so far. The parameters include three different shear spans, a plain concrete matrix, a fiber reinforced concrete matrix, and a confined concrete matrix. Tests are still in progress.



(a) Analytical Model for NIST Frame Subassemblage Test



(b) DRAIN-2DX Model of a 6-Story Precast Wall with Debonded Post-Tensioning

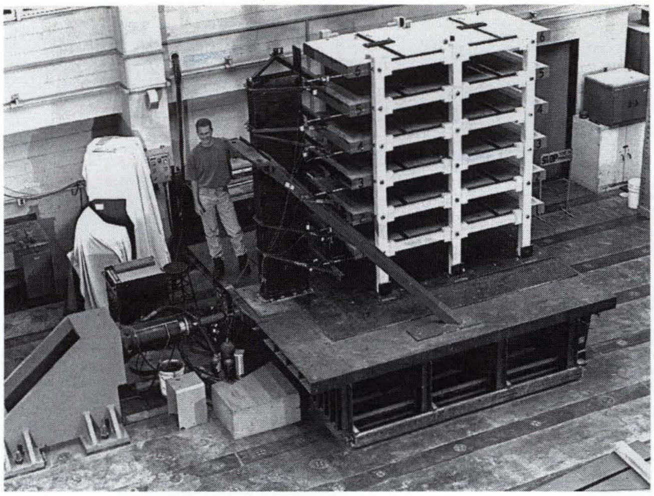
Fig. 10. Analytical models for precast frames and panel structures utilized in Lehigh University studies.

A typical specimen made with fiber reinforced concrete is shown in Fig. 15 where the fine crack distribution and the failure mode can be observed. Analysis of the test results and a related doctoral thesis are planned for completion by December 1996.

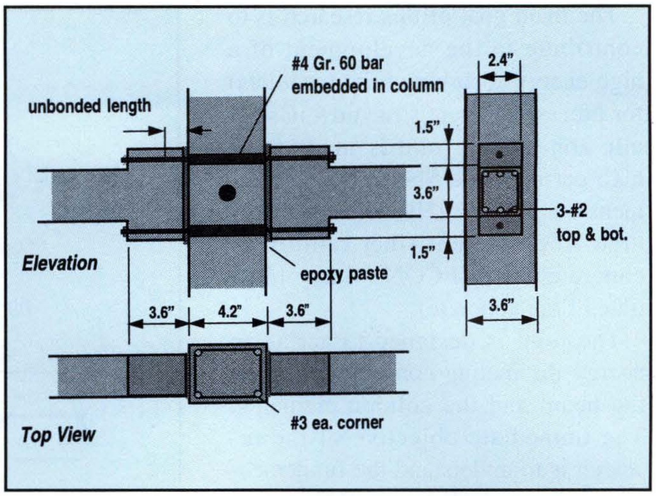
Codified Design Provisions for PRESSS

Principal Investigator: **Neil M. Hawkins,** University of Illinois at Urbana-Champaign

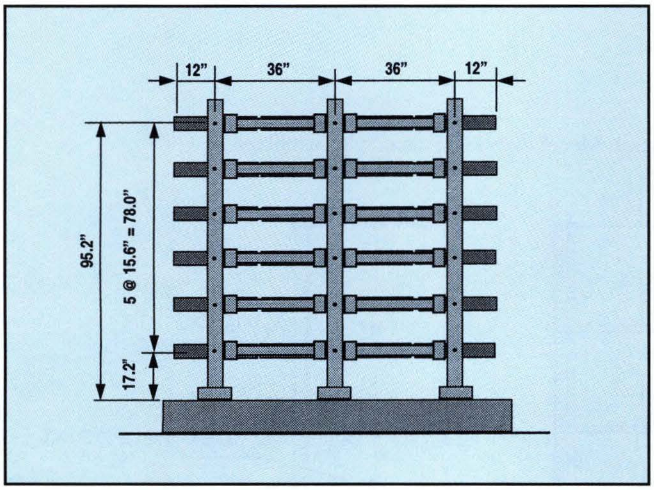
The objective of this project is to



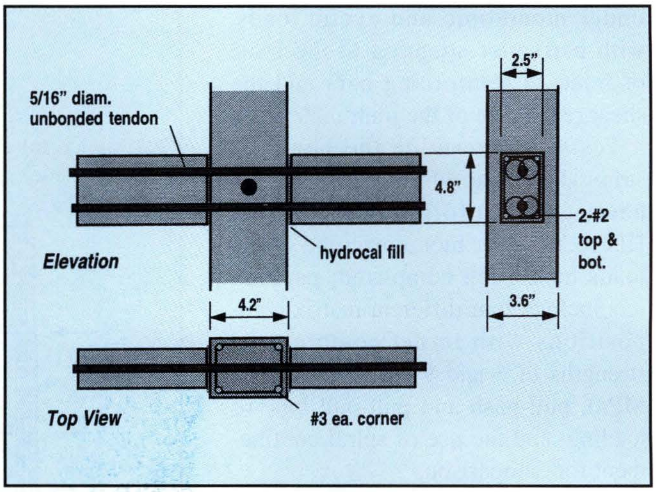
(a) Test structure on earthquake simulator



(c) Detail of dog-bone connection



(b) Dimensions of test structure



(d) Detail of post-tensioned connection

Fig. 11. Precast model frames for shake table tests (University of Illinois at Urbana-Champaign).

develop design rules for PRESSS that can be integrated directly into the NEHRP Recommended Provisions for Seismic Regulations for new buildings[6] and to provide a design guide for the use of those provisions. Initial work on this project has resulted in provisions being placed in the 1994 NEHRP provisions for "strong connection" design, particularly for frame structures where following the provisions will result in structural performance emulating that of an equivalent reinforced concrete structure.

A second design approach, appropriate for "ductile connection" systems, has been incorporated within an Appendix to the 1994 NEHRP provisions. That Appendix represents a compilation of the understanding of how best to proceed with the seismic resistance design of structural systems composed of interconnected precast concrete elements. The Appendix is not to be used as a basis for a submission of designs to building officials, but for user review, comment and trial designs. After adequate feedback is obtained, it is intended that a modified version will be integrated into the body of future NEHRP provisions.

The Appendix contains sections dealing with general provisions, lateral force resisting system requirements, connection performance requirements, and connection design requirements. Acceptance of a new structural system must be based on appropriate experimental evidence of both connection and overall structural competence. Rather severe limitations are currently imposed on many structural systems being investigated in PRESSS Phase II — particularly those of the non-linear elastic category (see Fig. 1).

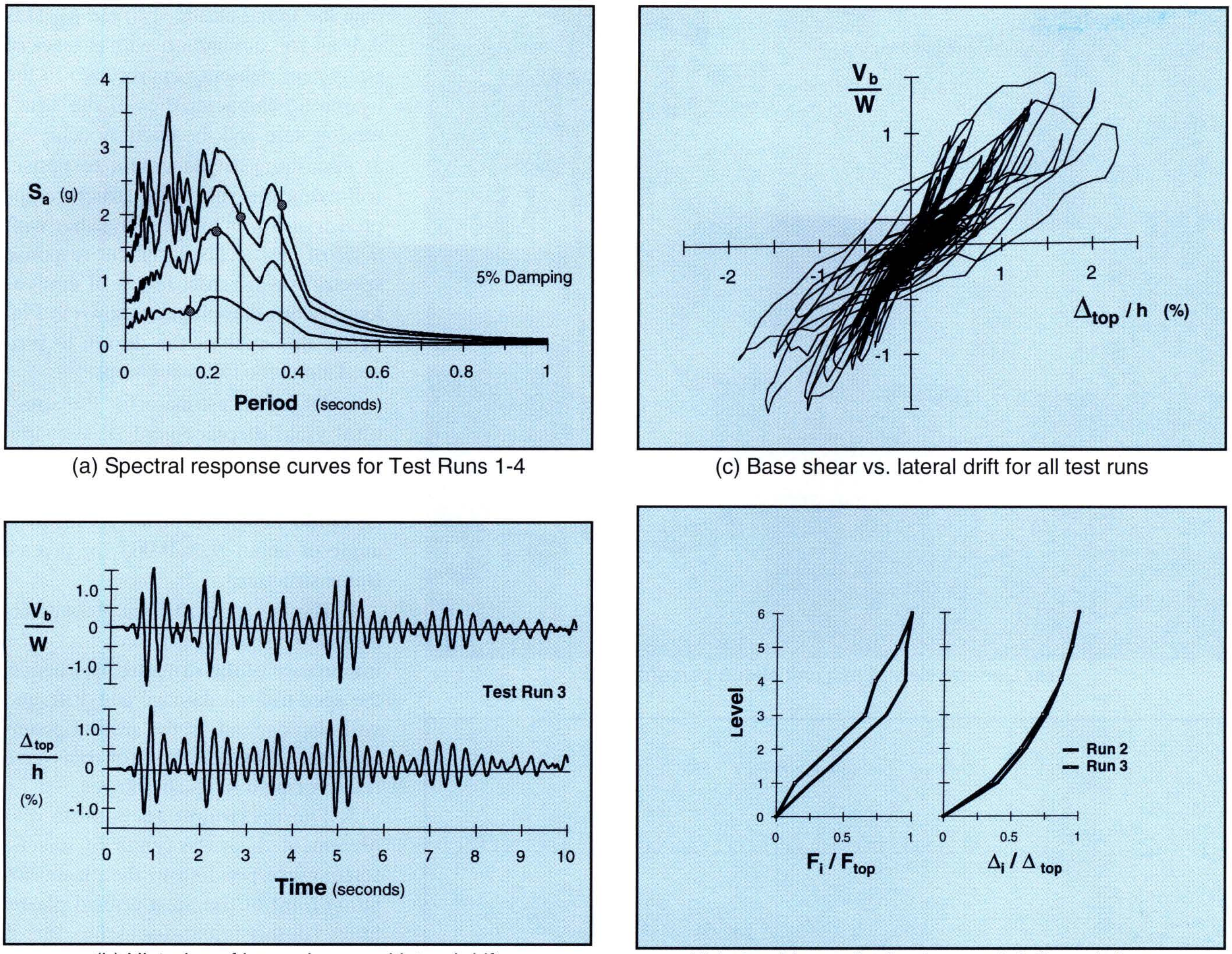
Current activities involve design studies of specific "strong-connection" and "ductile-connection" structures using the 1994 NEHRP provisions. In addition to providing feedback on the practicalities and consequences of those provisions, the information will be used to develop design guides.[7]

Seismic Behavior and Design of Double-Tee Panel Precast Systems

Principal Investigators: **Jose A. Pincheira** and **Michael G. Oliva,** University of Wisconsin at Madison

The purpose of this project is to examine the ability of precast double-tee floor diaphragm and wall systems to perform adequately under in-plane seismic forces. Work on the project is divided into three major phases:

1. Behavior of connections between double-tees



(a) Spectral response curves for Test Runs 1-4

(c) Base shear vs. lateral drift for all test runs

(b) Histories of base shear and lateral drift

(d) Lateral force distributions and deflected shapes

Fig. 12. Selected measured dynamic response of precast model Frame DB1 (University of Illinois at Urbana-Champaign).

2. Analytical modeling of connectors, and diaphragm and wall systems

3. Development of design guidelines for double-tee diaphragms and wall systems

This project was initiated later than the other Phase II projects and, as a consequence, is not as far advanced. Current work has primarily been on the first phase of the project where three different systems providing connection between the flanges are being investigated.

Connector Type 1 [see Fig. 16(a)] consists of two #3 reinforcing bars that are fillet ("stick") welded to the steel plate at an angle of 45 degrees in the plane of the double-tee flanges. In a plane perpendicular to the steel plate, reinforcing bars are welded at an angle of about 10 degrees.

Connector Type 2, shown in Fig. 16(b), consists of two $^3/_8$ in. (9.5 mm) diameter deformed anchor studs (hereafter, referred to as studs) that are welded using a stud gun. The studs employed in this first test series are welded at a 90-degree angle in the plane of the flanges and at a 10-degree angle in a plane perpendicular to the steel plate. The third connector to be studied, Connector Type 3, is essentially the same as that of Connector Type 2, except that the welded studs are bent at a 45-degree angle in the plane of the flanges [see Fig. 16(b)].

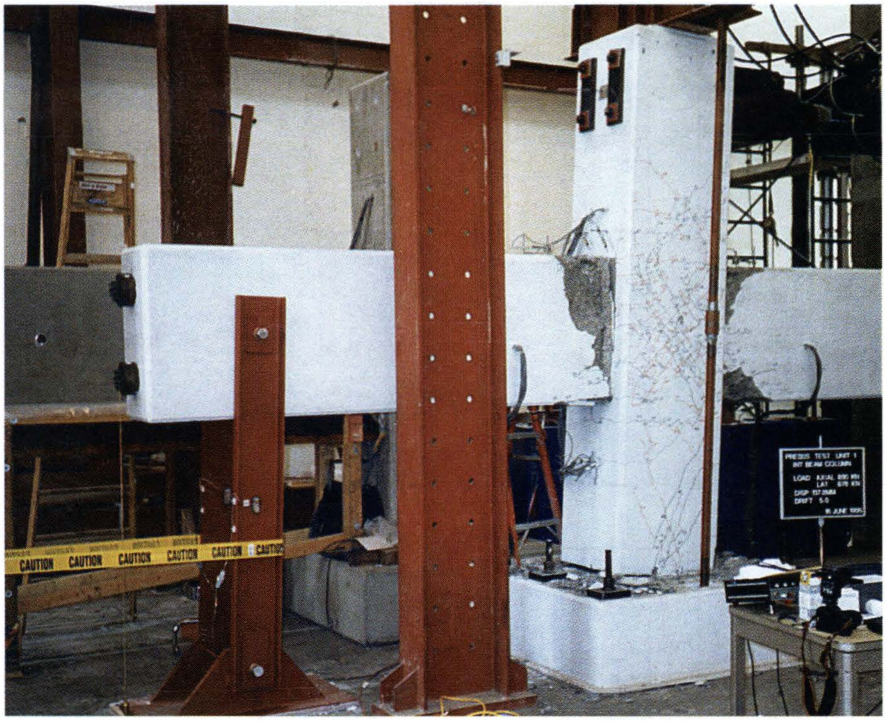
PRESSS Coordination — Displacement-Based Design Procedures

Principal Investigator: **M. J. Nigel Priestley,** University of California at San Diego

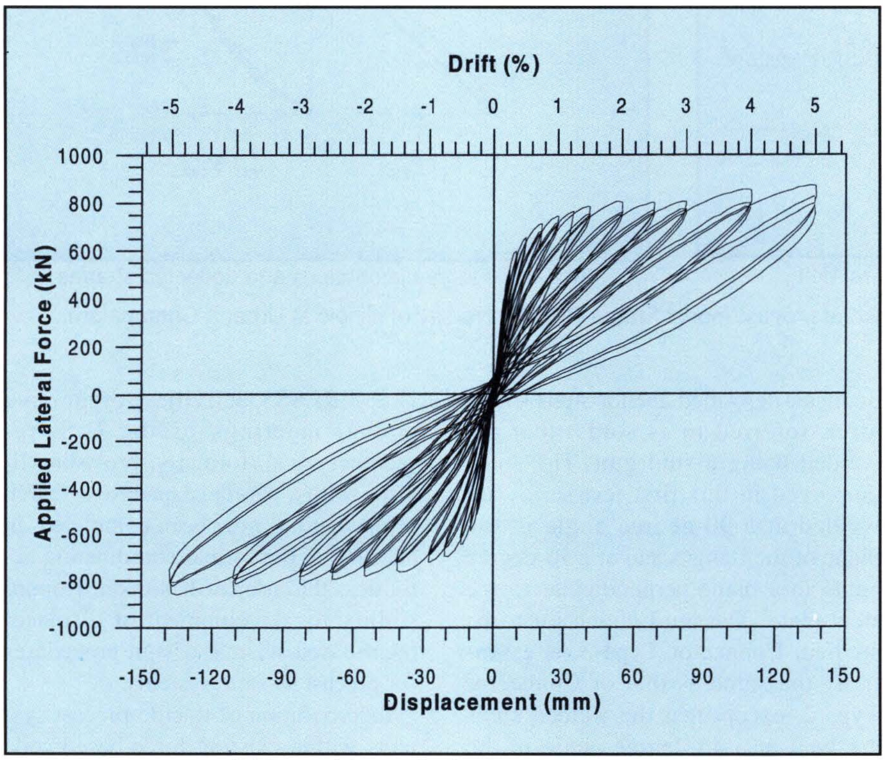
The PRESSS coordination project has responsibility for coordinating U.S. PRESSS activities, organizing annual meetings of the U.S. researchers, and, formerly, providing liaison with a parallel Japanese research effort that has now been completed. In addition to the formal coordination activities, this project has taken responsibility for development of displacement-based seismic design procedures for precast seismic systems.

Incorporation of ductile precast systems within existing force-based code seismic design procedures has problems because energy dissipation characteristics (as illustrated in Fig. 1) and concepts of "yield displacement" differ considerably from the elasto-plastic behavior assumed in developing existing code design approaches.

It appears that a more consistent design approach may be obtained for all structures, and in particular for those with non-standard hysteretic response,



(a) General view of test unit after 5 percent drift



(b) Lateral force-displacement response

Fig. 13. Interior precast beam-to-column unit with unbonded tendons (University of California at San Diego).

by a complete inversion of the design process. In this inversion, the starting point is the specification of design drift and the end products of the design process are the required strength and initial stiffness of the structure.

The procedure is illustrated by reference to a single degree of freedom approximation of a precast structure. Structural stiffness is characterized by the secant stiffness K_{eff} at maximum response, as shown in Fig. 17(a), rather than the initial elastic stiffness K_i. This is used in conjunction with a level of equivalent damping appropriate to the hysteretic characteristics of the structural system and the ductility achieved at maximum displacement response, following the substitute structure approach of Sozen et al.[14,15] Together with a set of elastic displacement response spectra for different levels of equivalent viscous damping, as shown in Fig. 17(b), this enables the design to proceed along the following steps:

1. An initial estimate for the structural yield displacement Δ_y is made. Because final results are not particularly sensitive to the value assumed, Δ_y could be based on a typical drift angle of about $\theta_y = 0.003$ for precast frame structures.

2. The design drift limit θ_u is determined. This will be a function of the importance of the structure, and hence, the need to limit damage and drift, and will also depend on the section geometry and acceptable level of transverse reinforcement in plastic hinges.

3. The maximum acceptable displacement Δ_u at the center of seismic force, corresponding to the plastic rotation limit of the most critical plastic hinge (in this initial discussion, this is the hinge at the base of the vertical cantilever) is found from the structure geometry and drift.

Thus:

$$\Delta_u = \theta_u L \qquad (1)$$

where L is the height from plastic hinge to center of mass of the equivalent vertical cantilever.

4. An estimate of effective structural damping is made based on the implied ductility level $\mu_\Delta = \Delta_u / \Delta_y$ from Fig. 17(c) where curves are given for hysteretic characteristics of different generic connection systems.

5. With reference to the design elastic displacement response spectra for the site [e.g., Fig. 17(b)], the effective response period at maximum displacement response can now be estimated. The effective stiffness K_{eff} of the substitute structure at maximum displacement can thus be found from:

$$T = 2\pi \sqrt{\frac{M}{K_{eff}}}$$

as

$$K_{eff} = \frac{4\pi^2}{T^2} M \quad (2)$$

From Fig. 17(a), the required structure ultimate strength, or base shear capacity is:

$$F_u = K_{eff} \Delta_u$$

6. With a knowledge of the required strength F_u, the member sizes can now be proportioned and an initial estimate of reinforcement made. The elastic stiffness can thus be calculated, and a refined estimate of the yield displacement obtained.

7. The structure ductility, and hence, effective structural damping are thus revised, and Steps 4 to 6 are repeated until a stable and satisfactory solution is obtained. Individual flexural strength requirements for potential plastic hinges are finalized based on statics.

The approach outlined above has considerable flexibility because plastic hinge rotational capacity can be related to transverse detailing (or vice versa) and the design is not dictated by arbitrary decisions about force-reduction factors. The approach is described in greater detail in Ref. 16.

Application of displacement-based design to multistory precast frame buildings requires some additional assumptions to be made. The two critical pieces of information required are: (1) the relationship between maximum interstory drift and structural displacement at the height of the center of seismic force; and (2) the shape of the lateral force vector to be applied.

These aspects are illustrated in Fig. 18 for an idealized frame of n stories each of equal height h. The center of seismic force is approximately two-thirds of the building height and the maximum displacement at this height can thus be expressed as:

$$\Delta_u = \Delta_y + \frac{2}{3} nh\theta_p K \quad (3)$$

where $K \leq 1$ defines the non-uniformity of drift up the building height and θ_p is the maximum acceptable rotation of the plastic hinges and, hence, the maximum story drift angle. For typical reinforced concrete frame structures, a value of $K = 0.75$ appears appropriate.[17] However, for precast panel structures and for frame structures with NLE characteristics, it appears that drift may be rather uniform with height, resulting in $K = 1.0$.

Having determined the required strength of the ductile connections by the above approach, a capacity design approach is used to ensure that inelastic action occurs only within the connections in the anticipated mechanism.[18] Finally, the design is checked by inelastic time-history analysis, or by elastic analysis of a substitute structure[15] where stiffness of members containing hinges is reduced in proportion to their expected ductility.

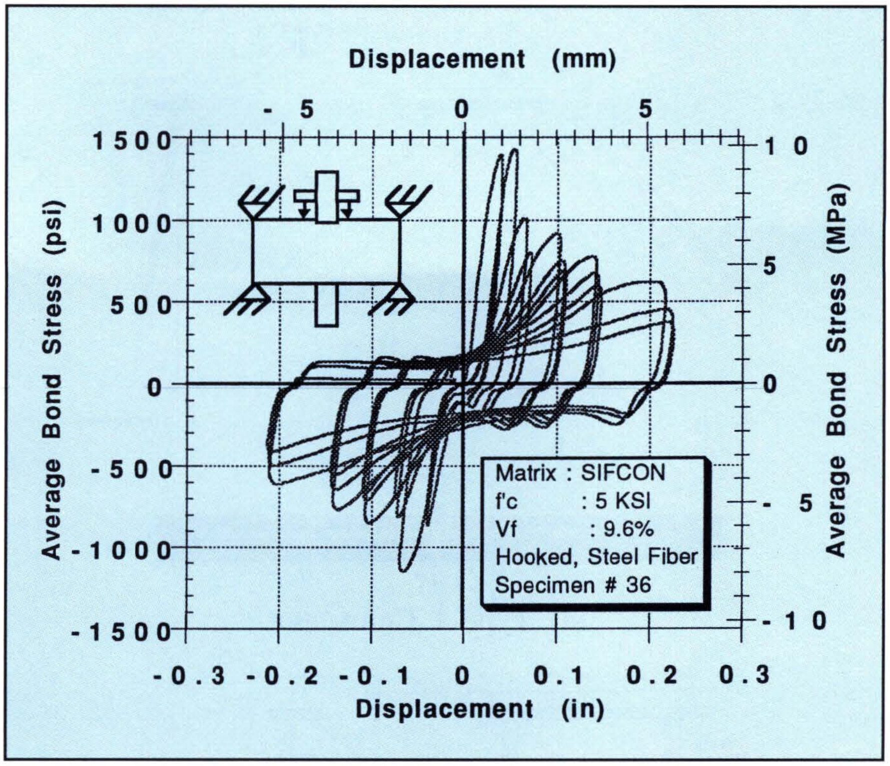


Fig. 14. Bond stress-slip response for a reinforcing bar in SIFCON concrete (University of Michigan).

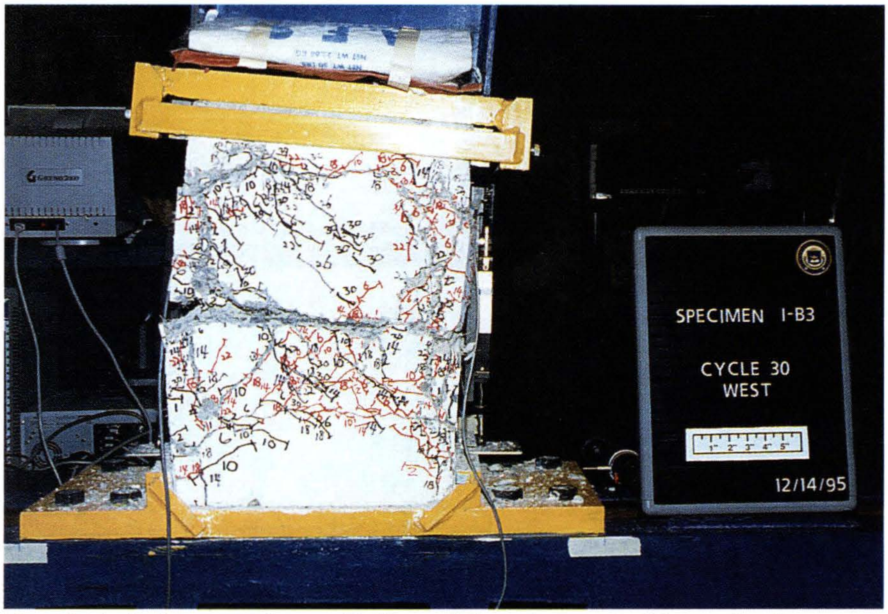


Fig. 15. Typical test specimen with fiber reinforced concrete (University of Michigan).

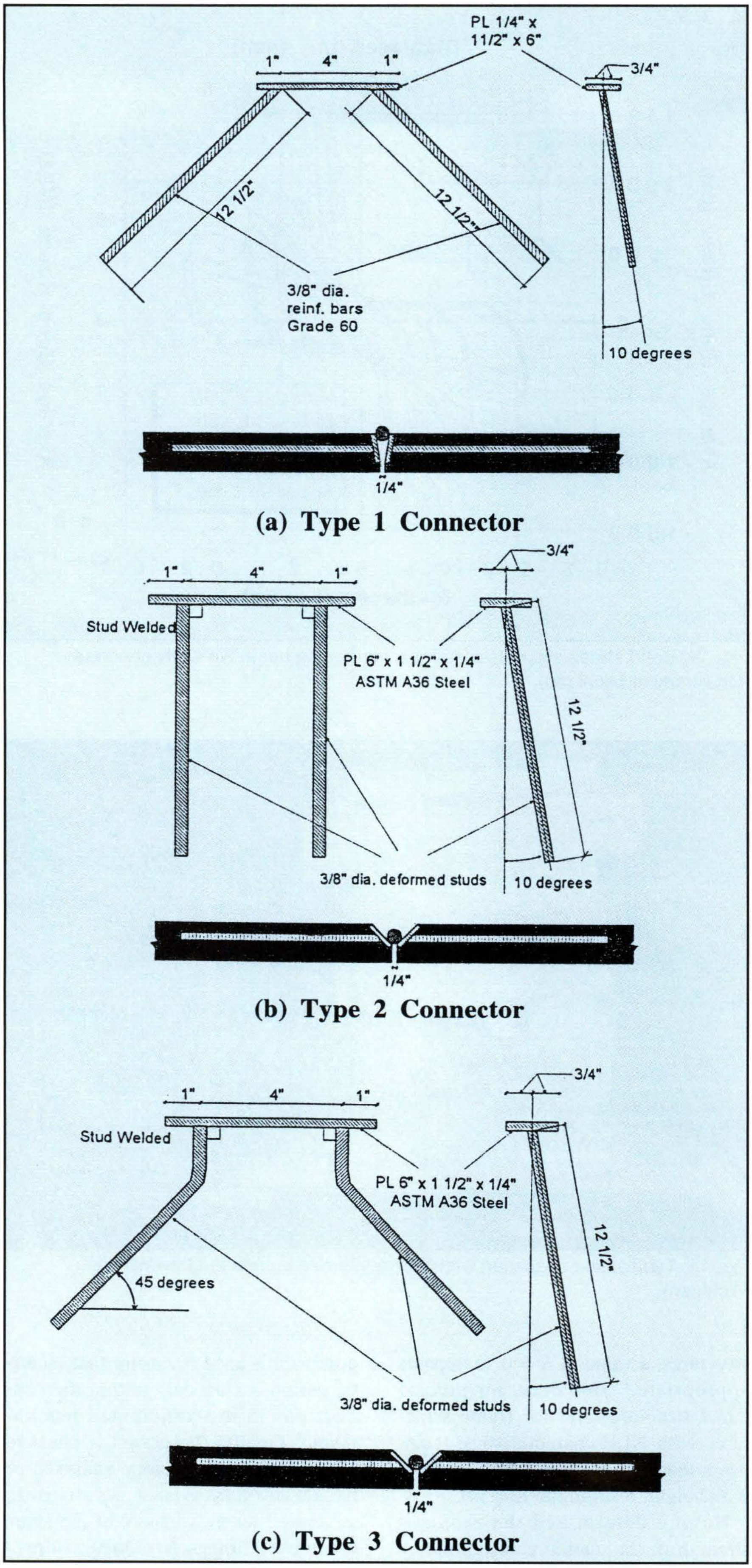


Fig. 16. Flange connectors for double-tee units (University of Wisconsin-Madison).

Hence, if beam hinges are expected to have rotational ductilities of $\mu_\theta = 7$ (which might correspond to a structure displacement ductility of $\mu_\Delta = 4$), then the appropriate stiffness for the beams in the elastic analysis would be:

$$K_\mu = \frac{K_e}{\mu_\theta} = 0.14 K_e$$

The adequacy of the design can thus be checked by a lateral elastic analysis of the substitute structure.

If the inelastic displaced shape can be approximated by Fig. 18, it follows that the vector of lateral inertial forces to be applied to the structure should also take the same shape.

The design approach outlined above appears to have advantages when design of precast systems, whose hysteretic characteristics do not replicate monolithic reinforced concrete, is considered. In these systems, the concepts of a "yield displacement" and "initial stiffness," may be inappropriate, but based on experimental results, relationships between drift and resisting force and between drift and equivalent viscous damping can be determined. Although there may be doubts about the use of the conventional force-based design procedure for these systems, the displacement procedure outlined above can be readily adapted for any hysteretic characteristics.

PRESSS PHASE III PROPOSED PLAN

The PRESSS research project was conceived as a three-phase program with Phase III integrating the components of experimental and analytical research developed in Phases I and II. The basis for this is intended to be studies built around the performance of two structural systems: (1) a precast frame building; and (2) a precast panel structure. For each building, the studies will consist of the following:

- Design studies
- Analytical studies
- Large-scale super-assemblage test
- Final design recommendations

The components of the two studies interact as illustrated schematically in Fig. 19. It is intended that design, analysis and super-assemblage experiment tasks be undertaken by separate research teams. A key element of each study will be the input from an industry advisory group whose tasks will include assisting in translating the generic connection systems selected for study into technically and economically feasible design details, assistance with design details of the super-assemblage experiment, as well as providing a general technical and economic overview of the studies.

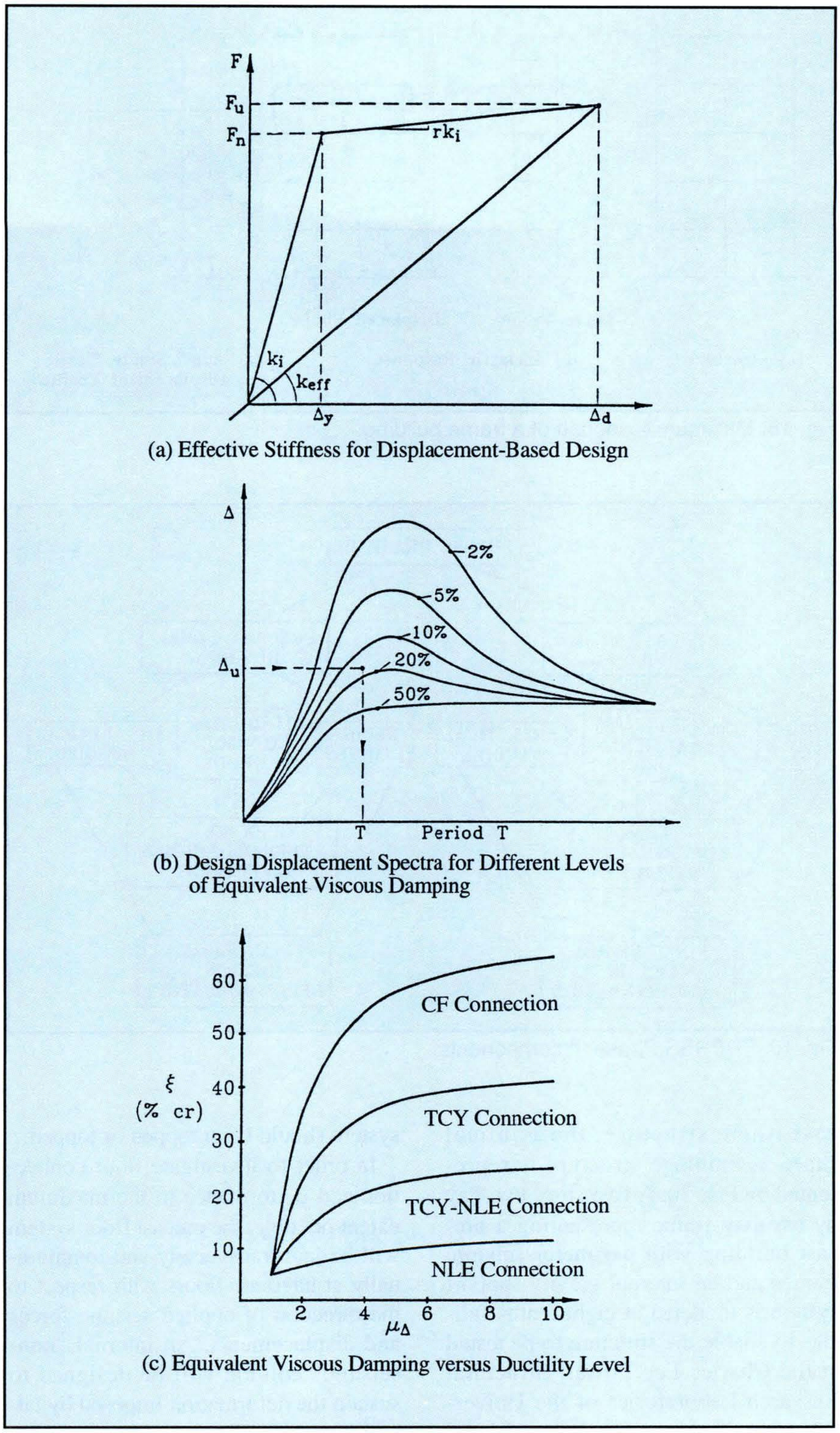
Precast Frame Project

Precast frame studies for ductile-connection systems have been sufficiently developed in Phases I and II for detailed plans for Phase III to be initiated. An overview of the planned project follows.

Design studies — Two buildings, one five stories tall and the other fifteen stories tall, will be designed for two different sites, representing UBC Zone 2 and Zone 4 seismicity, respectively. At least two of the generic connection systems discussed above will be adopted for the structural system. Based on the initial results from these designs, the structural system for the super-assemblage experiment will be selected by the design teams with input from the industry advisory group.

A detailed design for the super-assemblage structure will then be prepared using the displacement-based design approach outlined above. It is expected that this task will be carried out by a consortium of researchers from Nakaki and Englekirk and the University of Washington, with input from PRESSS Phase II researchers.

Analytical studies — The analytical group will take the designs prepared by the design group and subject them to dynamic inelastic time-history analyses using the DRAIN family of computer codes developed in Phase I of the PRESSS program. Seismic input will be artificial or "massaged" accelerograms matched to the Zone 2 or Zone 4 design spectra. Larger-than-design level earthquakes will also be run to determine response and damage levels under extreme events, such as ground motion recorded in the 1994 Northridge earthquake, which in the near-field region significantly exceeded that implied by UBC Zone 4.



(a) Effective Stiffness for Displacement-Based Design

(b) Design Displacement Spectra for Different Levels of Equivalent Viscous Damping

(c) Equivalent Viscous Damping versus Ductility Level

Fig. 17. Elements of displacement-based design procedure.

Further tasks will involve development of the input motion for the super-assemblage experiments and a prediction of the experimental response. This will be carried out before the experiment (i.e., a true prediction not a "post-diction," as is more common). This task is scheduled to be carried out by Lehigh University.

Super-assemblage experiment — The purpose of the super-assemblage experiment is to provide experimental confirmation of design and analyses studies using a physical model of sufficient size and complexity to represent all the typical connections and interactions of a typical structure. For a pre-

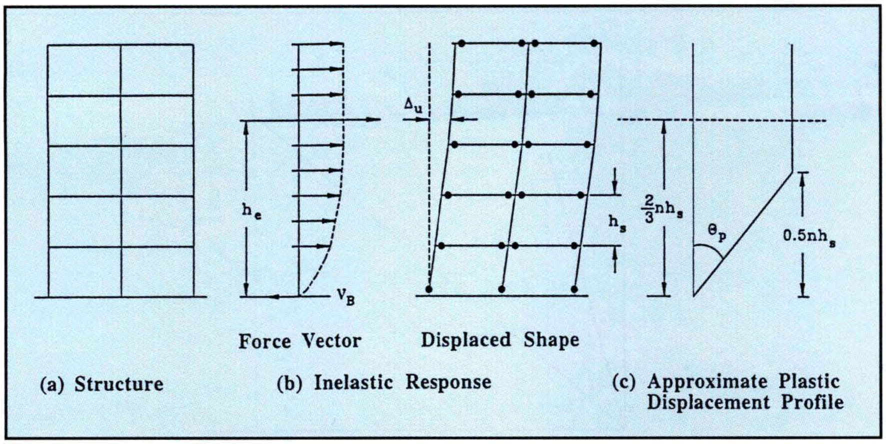


Fig. 18. Maximum response of a frame building.

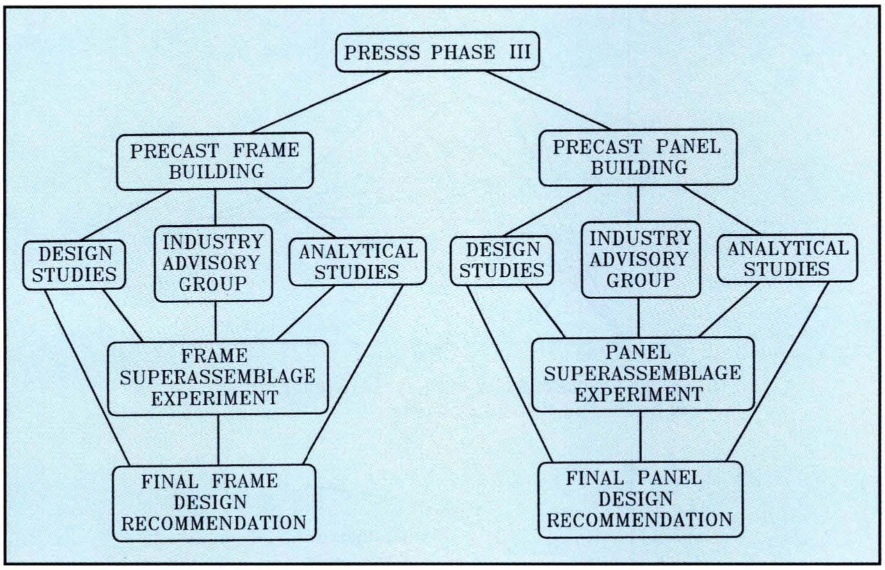


Fig. 19. PRESSS Phase III components.

cast frame structure, the minimal super-assemblage structure is represented by Fig. 20. A five-story two-bay by two-bay frame representing a precast building with perimeter seismic frames and an internal gravity support system is modeled at eight-tenths full-size to enable the structure to be tested at the Charles Lee Powell Structural Research Laboratories of the University of California at San Diego. Model story height will be 10 ft (3 m), with a bay length of 20 ft (6 m).

In addition to providing experimental verification of the design and analysis procedures developed in the PRESSS program, particular interest in the testing will be focused on the performance of the precast floor system and the gravity support system. The design and industry advisory group will have to decide whether the floor system should be untopped or topped.

In order to investigate floor connection and performance to the maximum extent possible, the precast floor system will be laid transversely and longitudinally at alternate floors with respect to the direction of applied seismic forces and displacements. An internal "non-seismic" column will be designed to sustain the deformation imposed by lateral response of the seismic-resisting frames. Inadequate design of non-seismic columns was a problem in the performance of precast buildings in the Northridge earthquake,[19] and confirmation by this test that such details can be made to perform well will be a valuable addition to the program.

Consideration is being given to model construction using different generic structural systems for the two seismic frames parallel to the loading direction. Although this would add complexity to the test program, it would provide valuable additional data. An alternative approach would be to use a slightly smaller scale (approximately six-tenths full-size) and to use different structural systems for the longitudinal and transverse seismic frames. After initial testing of one structural system, the model could be jacked up off the floor, rotated 90 degrees and retested in the orthogonal direction. The reduced scale would be necessary to accommodate the diagonal dimension of the building within the test hall dimensions during rotation.

The structure will be subjected to four successive levels of seismic excitation, using multi-degree-of-freedom pseudo-dynamic test techniques, which have previously been successfully developed at UCSD for a full-size five-story masonry super-assemblage.[20] The levels will represent: (1) elastic response at close to "yield" level; (2) serviceability limit state; (3) design level excitation; and (4) "extreme" seismic event, taken as 1.5 times the design level excitation.

Existing hydraulic equipment, data acquisition systems, test and reference frames, control and data recording computers available from the earlier five-story masonry building test and subsequent large scale experiments make the super-assemblage test relatively inexpensive in terms of hardware acquisition. It is expected that the industry will provide the precast elements to further reduce the costs to the extent that the cost can be met by the PRESSS financial sponsors.

Final design recommendations — Much of the design recommendation work has been carried out in PRESSS Phases I and II. However, the design and analytical studies carried out in Phase III will be essential to provide final calibration of the proposed design procedures. In particular, refinement is required of information on the inelastic deflection profiles developed at maximum response of the generic connection systems for use in the displacement-based design approach, and capacity-design factors relating required strength of elastically responding elements and connections is required for the different systems for both force-based and displacement-based design approaches.

The ultimate goal will be to develop design guidelines for precast/prestressed concrete structures in various seismic zones that can be incorporated into the model building codes.

Precast Panel Project

The precast panel building project is less well developed at this stage than that for the frame building project. This is a consequence of the reduced level of funding that has thus far been available from NSF for panel structures. It is intended that proposals will be submitted to NSF in the near future to seek funding for additional research on precast panel structures. Following this research, the Phase III panel project will be initiated. Thus, it is expected that the frame and panel components of PRESSS Phase III will be undertaken sequentially, rather than in parallel, although it is likely that some overlap will occur. Financial constraints also point to the need for sequential phasing of the research in Phase III.

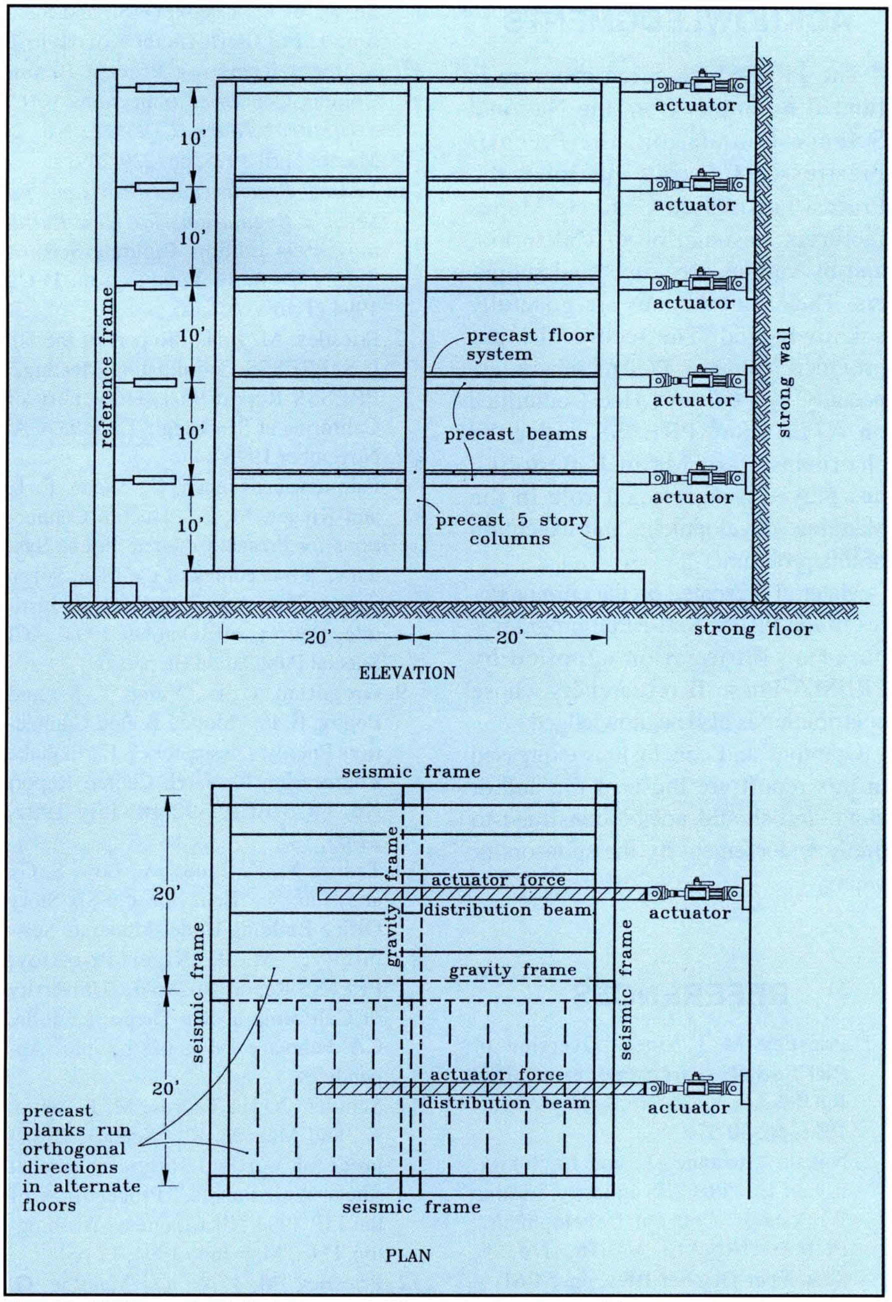


Fig. 20. Eight-tenths full-size precast frame super-assemblage.

CONCLUDING REMARKS

Viable connection systems and design methodologies have been developed in PRESSS Phases I and II for precast frame and panel structures. Particular emphasis has been given to the determination of the hysteretic characteristics of different generic connection systems, rather than of specific design, to maximize the utility of the information provided by the PRESSS program. The anticipation that the PRESSS research would be converted into innovative, specific, technically and economically feasible design details has already started with the development of the Pankow system[21] and the Englekirk/Nakaki system.[22]

Plans are being prepared for PRESSS Phase III, which will consist of design and analysis studies of complete precast frame and panel structures. Of particular emphasis will be the displacement-based design approach, which appears particularly suitable for precast systems whose inelastic deformation characteristics differ significantly from reinforced concrete or structural steel systems.

The Phase III studies will be anchored by near full-scale experiments on super-assemblages that model the important interactions between orthogonal seismic-resistant bracing elements, between seismic and gravity supporting systems, and between floor systems and seismic bracing systems. The super-assemblages will provide verification of the design and analysis procedures developed in PRESSS and give valuable confirmation of the excellent seismic performance and low damage possible with properly designed precast systems.

The ultimate goal is to develop design guidelines that emphasize the inherent advantages of precast systems for seismic resistance. These guidelines will be prepared for precast/prestressed concrete structures in various seismic zones that can then be incorporated into the model building codes.

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Opinions and conclusions expressed in this report are those of the author alone and should not be construed to imply endorsement by the sponsors or others.

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