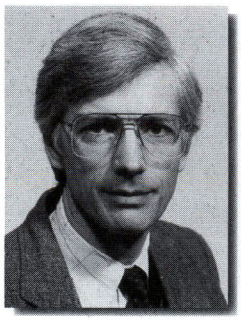
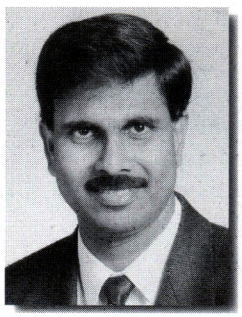
An Overview of the PRESSS Five-Story Precast Test Building

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At the culmination of the PRESSS (Precast Seismic Structural Systems) research program, a 60 percent scale five-story precast/prestressed concrete building will be tested under simulated seismic loading. This paper describes the prototype buildings used for design and the structural features of the test building. The buildings were designed using the direct displacement based approach, which is able to take advantage of the unique properties of precast/prestressed concrete using dry jointed construction. The test building incorporates four different seismic frame systems in one direction, and a jointed shear wall system in the orthogonal direction. Pretopped double tees are used on three floors, while the other two floors are constructed using topped hollow-core slabs. A major objective of the test program is to develop design guidelines for precast/prestressed concrete seismic systems that are appropriate for use in various seismic zones. These design guidelines can then be incorporated into the appropriate building codes.

The Precast Seismic Structural Systems (PRESSS) program has been in progress for ten years, with the final phase of the program well underway. PRESSS, sponsored by the National Science Foundation (NSF), Precast/Prestressed Concrete Institute (PCI) and Precast/Prestressed Concrete Manufacturers Association of California, Inc. (PCMAC), has coordinated the efforts of over a dozen different research teams across the United

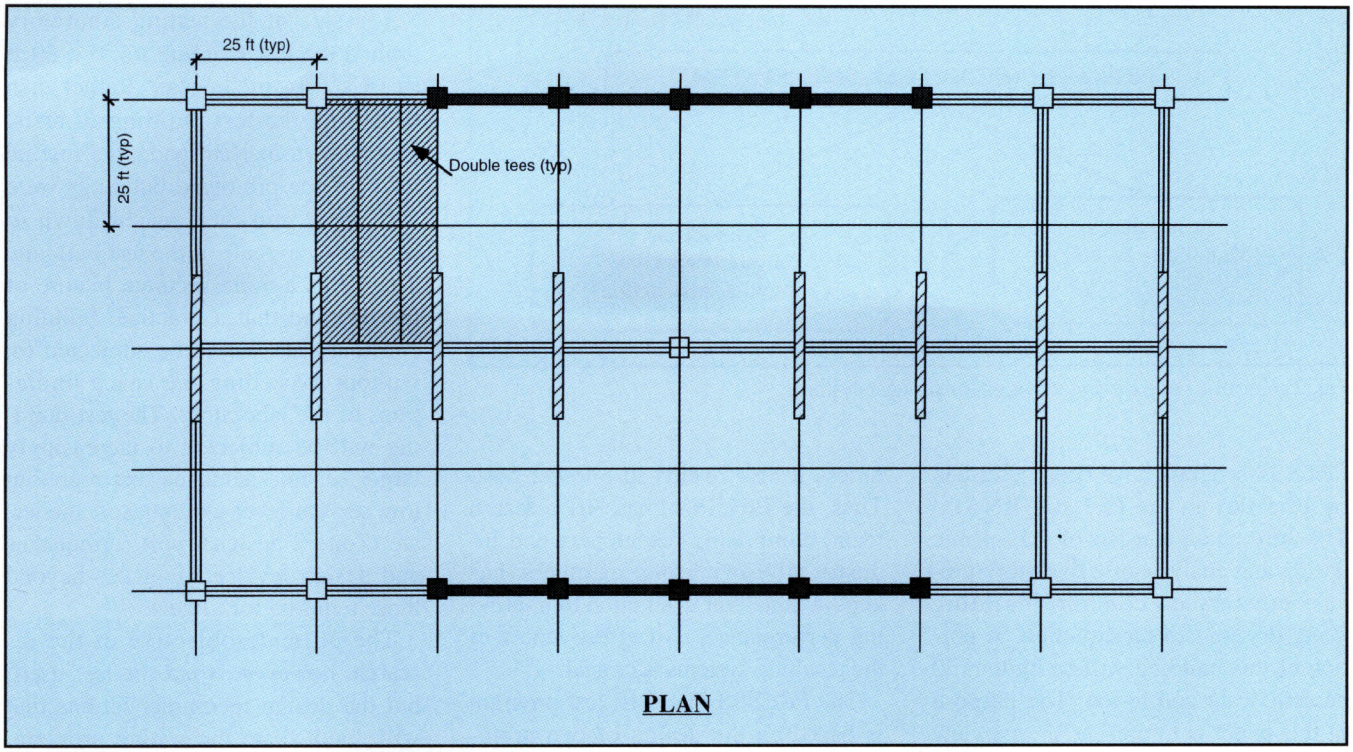


Fig. 1. Prototype building with pretopped double tees. **Note:** 1 ft = 0.3048 m.

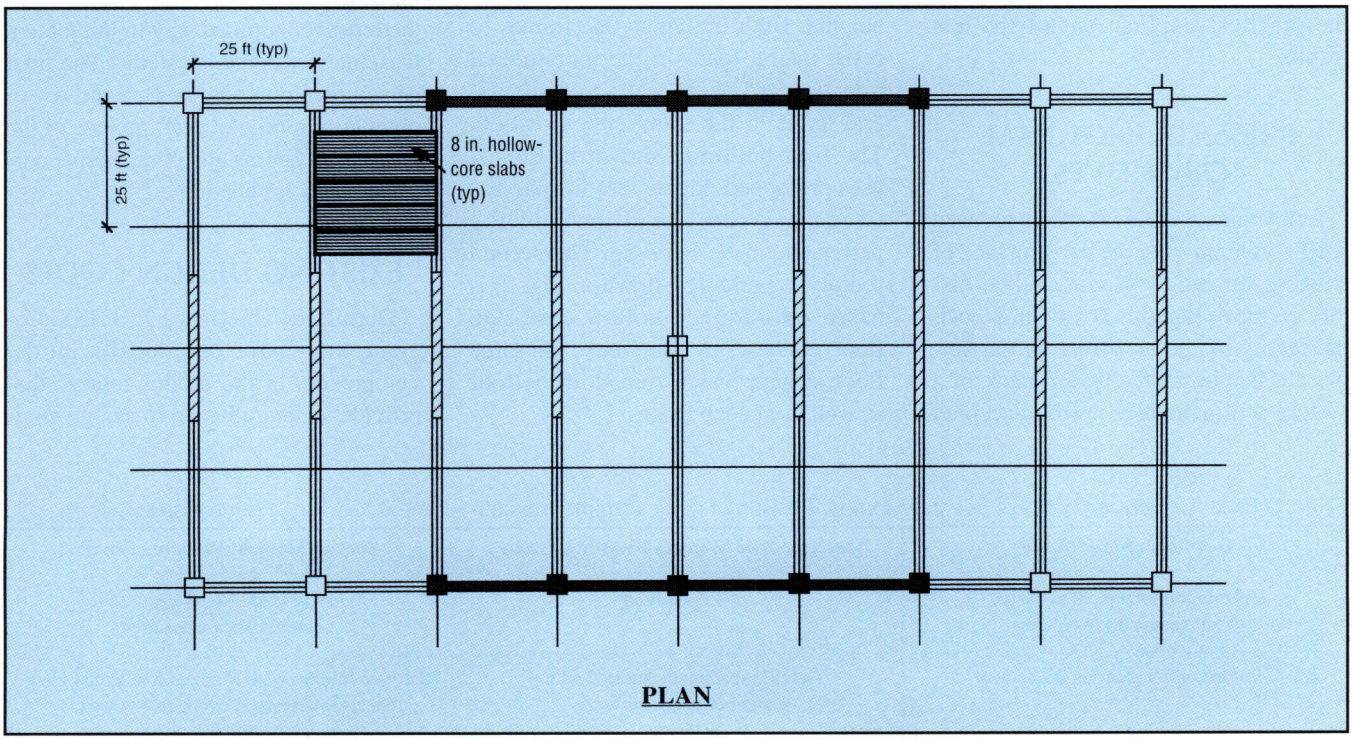


Fig. 2. Prototype building with topped hollow-core slabs. **Note:** 1 ft = 0.3048 m; 1 in. = 25.4 mm.

States to improve the seismic performance of precast/prestressed concrete buildings. In the context of this paper, "buildings" refer to low- and high-rise buildings such as office buildings, parking structures, hotels, hospitals, multi-family housing, and other special structures. However, bridges and transportation structures are excluded.

Since the very beginning of the PRESSS program, all of the research teams involved in the program have focused their sights on two primary objectives:

- 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 first and second phases of the

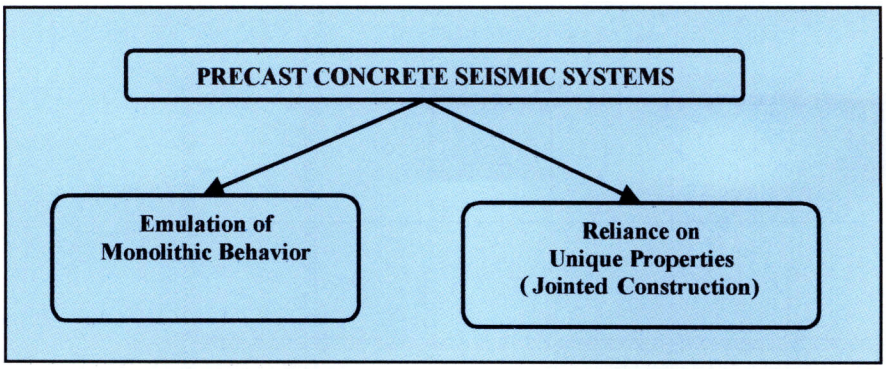


Fig. 3. Current code design choices for precast systems.

PRESSS program have been described by Priestley in the PCI JOURNAL.[1] The third phase consists of the seismic design and analysis of a five-story precast/prestressed concrete building using dry jointed construction. A portion of this building will be built at 60 percent scale and tested. The purpose of this paper is to present an overview of the test building, describe the major features of the structural systems investigated and offer some thoughts on the practical implications of the test results.

PRESSS III PROGRAM OBJECTIVES

Academic research is often focused solely on improving the performance of existing structural systems. While history confirms that this is a worthy goal, the reality of the construction marketplace is that improved performance of a system will generally not be accepted unless it also results in a lower cost. Thus, the PRESSS Phase III research team, comprising researchers and industry advisory group members, has kept in mind that in addition to improving performance, cost effectiveness of the resulting systems is crucial.

The PRESSS Phase III test program is based on the design of two prototype five-story precast office buildings, 100 x 200 ft (30.5 x 61 m) in plan, with 12 ft 6 in. (3.81 m) story heights. Both buildings use frames to resist lateral loads in the longitudinal direction and shear walls to resist lateral loads in the transverse direction. The first building, shown in Fig. 1, uses pretopped double tees to span between a central gridline and the perimeter of the building. The second prototype building, shown in Fig. 2, is based on a topped hollow-core slab floor system. For simplicity, the same floor system was assumed at the roof as well as at each floor.

The size of the testing laboratory limited the test building to 30 x 30 ft (9.14 x 9.14 m) in plan. Rather than designing the test building to resist just its own inertial loads, the inertial loads of the prototype buildings were calculated and then scaled down to represent the scale of the test building. This gives a more accurate picture of the demand that a practical building configuration would be subjected to, without exceeding the space limitations of the laboratory. The test building will be subjected to increasingly larger seismic demands that represent low service level earthquakes, moderate (Zone 2 design level) earthquakes and design level earthquakes beyond those required for Zone 4.

The ultimate objective of the research, however, is not the test itself, but the design recommendations that will result from the testing program. Because there are so many different combinations of systems included in the test building, it does not represent the most economical way to implement these new structural systems. The final design recommendations are the key to obtaining improved performance of the proposed systems at a competetive cost in practical applications.

EXISTING DESIGN CODES

During the life of the PRESSS program, there have been significant developments in the model codes[2,3] that provide some guidance to design engi-

Table 1. Detailing requirements of Special Moment Resisting Frame and Intermediate Moment Resisting Frame systems.

Detailing requirements	Intermediate Moment Resisting Frame	Special Moment Resisting Frame
Column reinforcement to ensure weak beam/strong column	Not required	Often a few additional longitudinal column bars are required
Column confinement reinforcement	Tight tie spacing is required on top and bottom of column	Same as Intermediate Moment Resisting Frame except where axial overload is possible (normally at end bays of frames)
Column shear reinforcement	Required	Required
Joint shear stress limitations	No limit; however, joint shear reinforcement is required	Limited; this requires a larger column only where beams are heavily reinforced
Beam shear reinforcement	Required	Required
Positive moment resistance in beam	Required	Required
Design base shear	150 to 160 percent of that required for Special Moment Resisting Frame	60 to 65 percent of that required for Intermediate Moment Resisting Frame

neers wanting to implement precast seismic systems in their buildings. As shown in Fig. 3, current codes allow precast seismic systems that either emulate monolithic concrete or rely on the unique properties of precast concrete (i.e., jointed, dry construction).

While jointed construction is allowed by the code, the focus of the prescriptive code provisions has been on emulation of monolithic concrete, largely because a consistent set of design recommendations for jointed precast systems have not been developed. Jointed systems can only be used if they are justified by test data on a case-by-case basis. The PRESSS program goes a step further by focusing its efforts almost exclusively on systems that rely on and take advantage of the unique properties of precast concrete. The intention is then to develop a consistent set of design recommendations for jointed precast systems that can be used to update existing code provisions.

Force Based Design

Seismic design in current codes is exclusively force based. That is, a designer uses elastic properties to determine an elastic base shear, which is then divided by a force-reduction factor R to obtain the design base shear. The value of R depends largely on the nominal ductility capacity of the system chosen, which is somewhat arbitrary and varies between codes. While maximum structural displacements must satisfy certain limits, they are in most cases based on elastic structural properties and are amplified by factors intended to approximate the post-elastic response. This approach has some significant drawbacks, as discussed by Priestley,[4] especially for precast concrete. Despite these difficulties, it will continue to be the legal design procedure for at least the foreseeable future.

In Force Based Design, there are two main ways that a designer can reduce the cost of a seismic system. Both methods depend on reducing the design loads because for consistent detailing, a lower force results in a lower cost. In the first method, a larger R factor is used to reduce the design base shear. For frames, the R value can be maximized by detailing the structure as a Special Moment-Resisting Frame

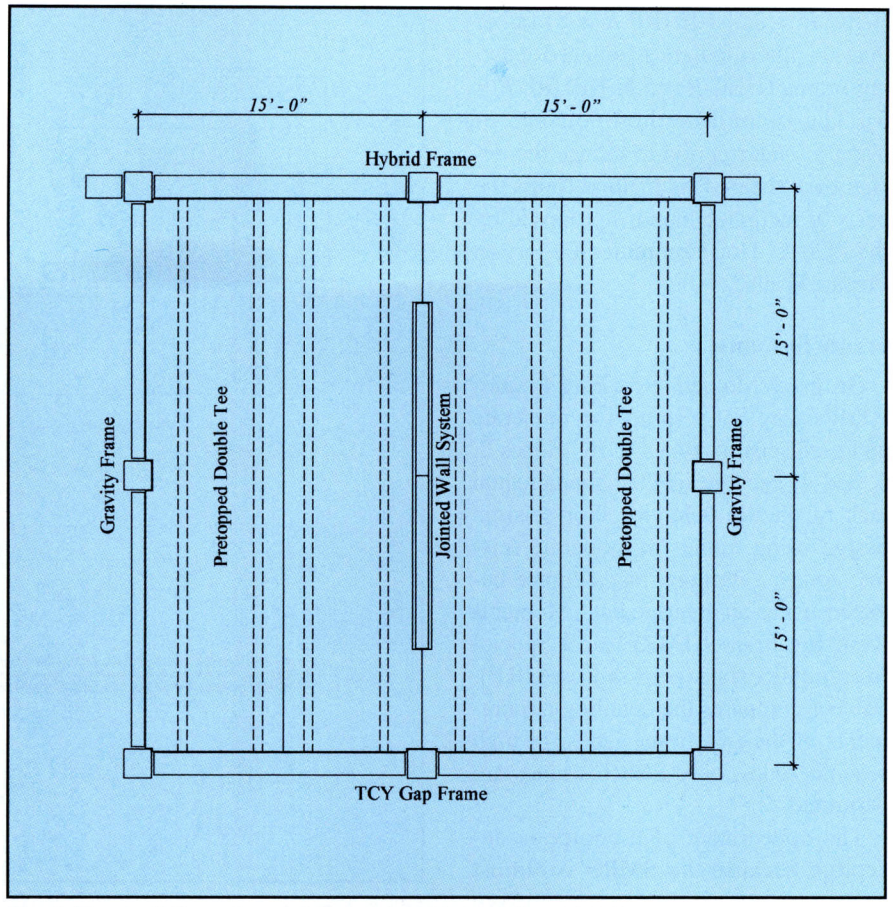


Fig. 4. Test building – Level 1 floor plan. **Note:** 1 ft = 0.3048 m.

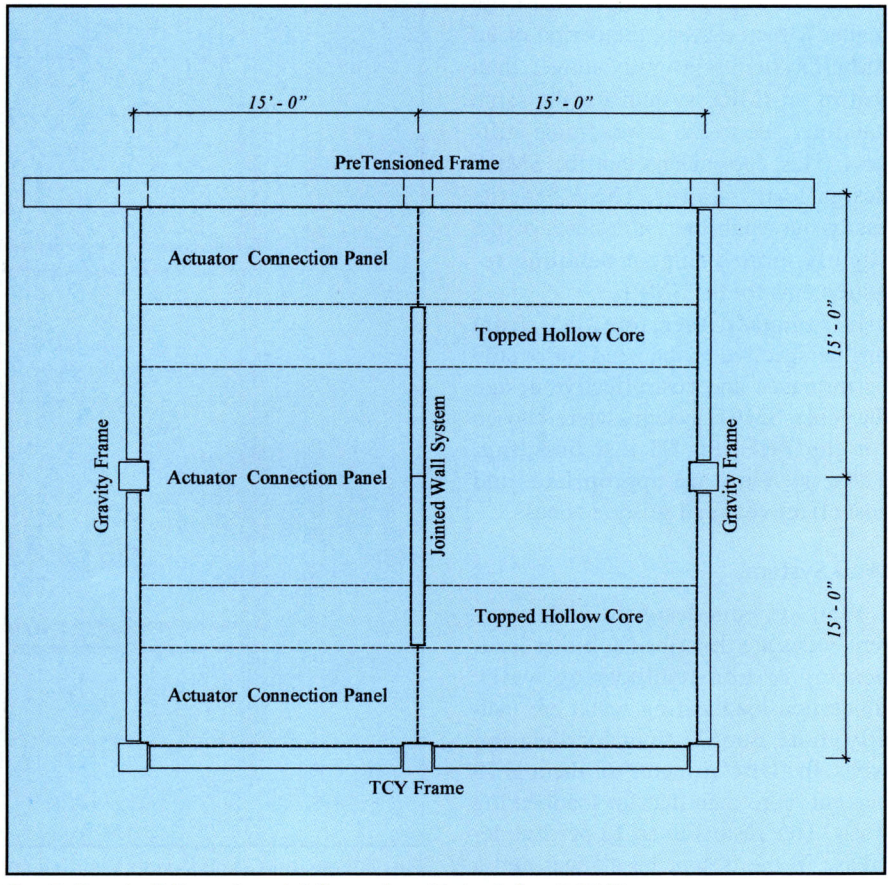


Fig. 5. Test building – Level 4 floor plan. **Note:** 1 ft = 0.3048 m.

(UBC R = 8.5, NEHRP R = 8) rather than an Intermediate Moment-Resisting Frame (UBC R = 5.5, NEHRP R = 5). The second method consists of using a longer period to reduce the design base shear. This method forms the basis of recommendations proposed by the PCI Ad Hoc Committee Report on Precast Walls.[5]

Frame Systems

Ordinary Moment-Resisting Frames (OMRF) are not permitted in moderate and high seismic zones (UBC Zones 2, 3, and 4) because of their fundamental lack of ductile behavior. For seismic design using frames in moderate seismic zones, a designer has a choice between using an Intermediate Moment-Resisting Frame (IMRF) or a Special Moment-Resisting Frame (SMRF). Table 1 compares the detailing requirements of the two frame types. In high seismic zones, only SMRF frames are permitted.

The appearance of a choice is deceptive because the SMRF is almost invariably the most cost effective frame solution. This is so because the design loads on an SMRF are 35 to 40 percent lower, primarily due to the higher R factor. Also, the period of an SMRF system is slightly longer than that of an IMRF system for the same building, due to the lower frame stiffness. This, too, means that the SMRF design load is lower. These benefits easily outweigh the extra costs of the slightly more stringent detailing requirements for the SMRF.

In summary, therefore, it is from this perspective of the need for ductile performance and cost effective design that only SMRF systems were chosen for the PRESSS III test building. These systems are appropriate, and cost effective, in all seismic zones.

Wall System

Wall systems designed under current codes are described as either loadbearing or non-loadbearing walls. Since non-loadbearing walls are usually more ductile than loadbearing walls, the UBC R factor for them is 18 percent more than that for loadbearing walls. This results in an 18 percent decrease in the design base shear and a concomitant reduction in the cost for

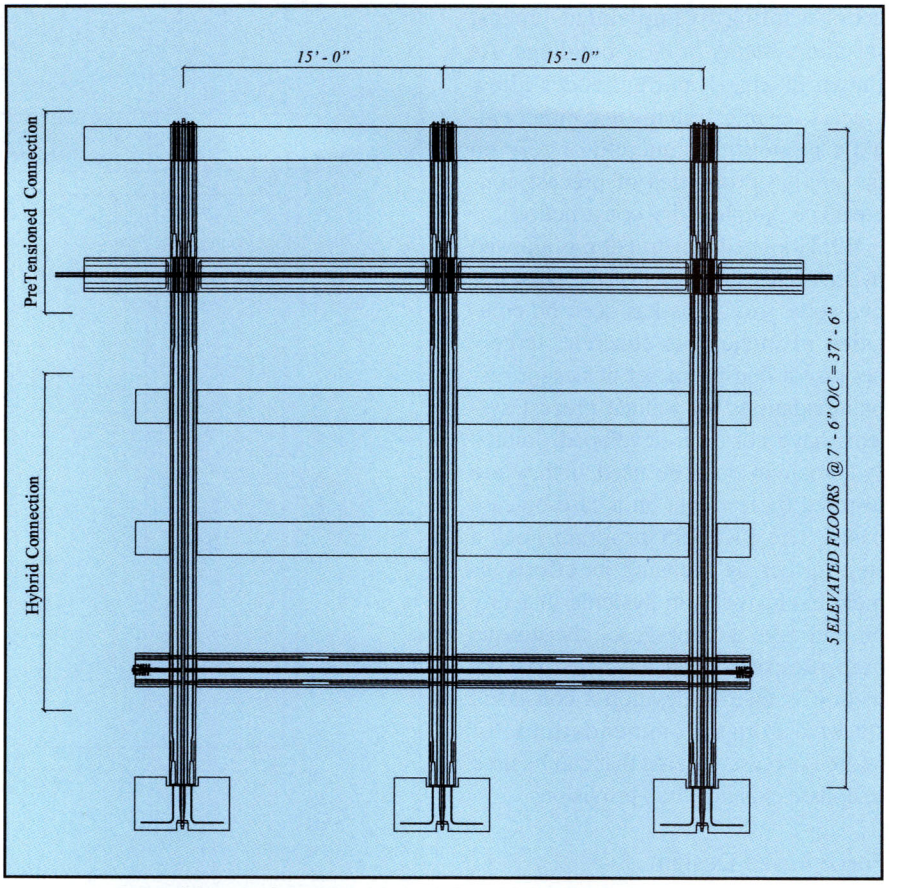


Fig. 6. Prestressed frame elevation. **Note:** 1 ft = 0.3048 m; 1 in. = 25.4 mm.

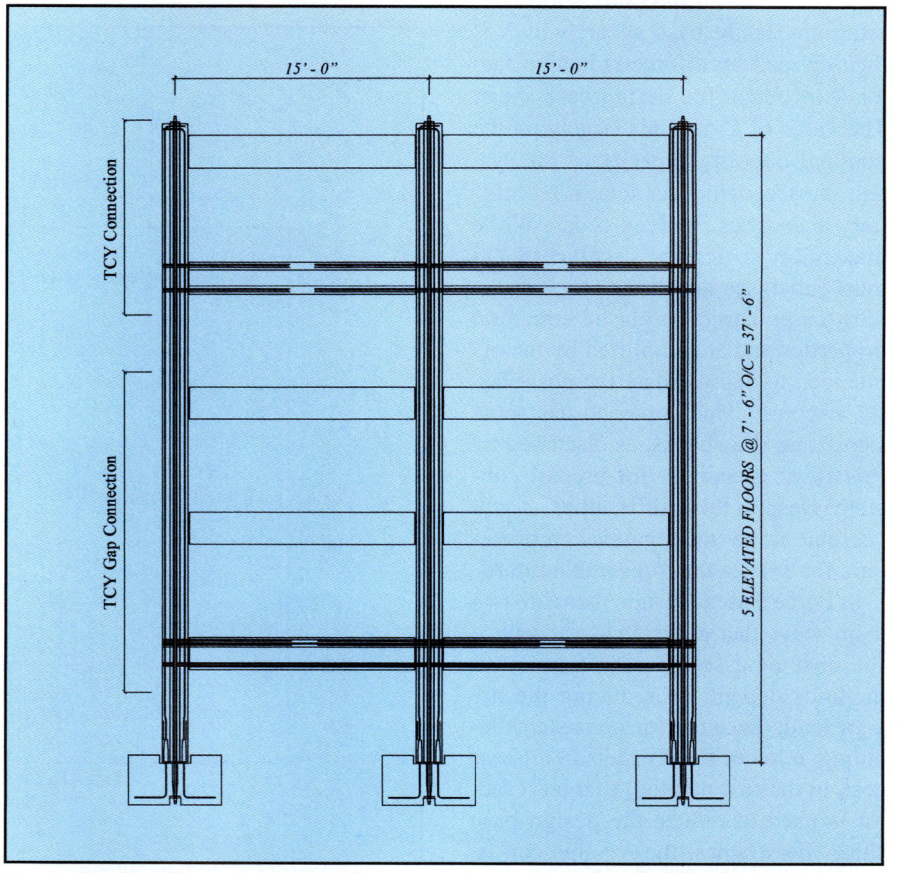


Fig. 7. Tension-Compression Yielding (TCY) frame elevation. **Note:** 1 ft = 0.3048 m; 1 in. = 25.4 mm.

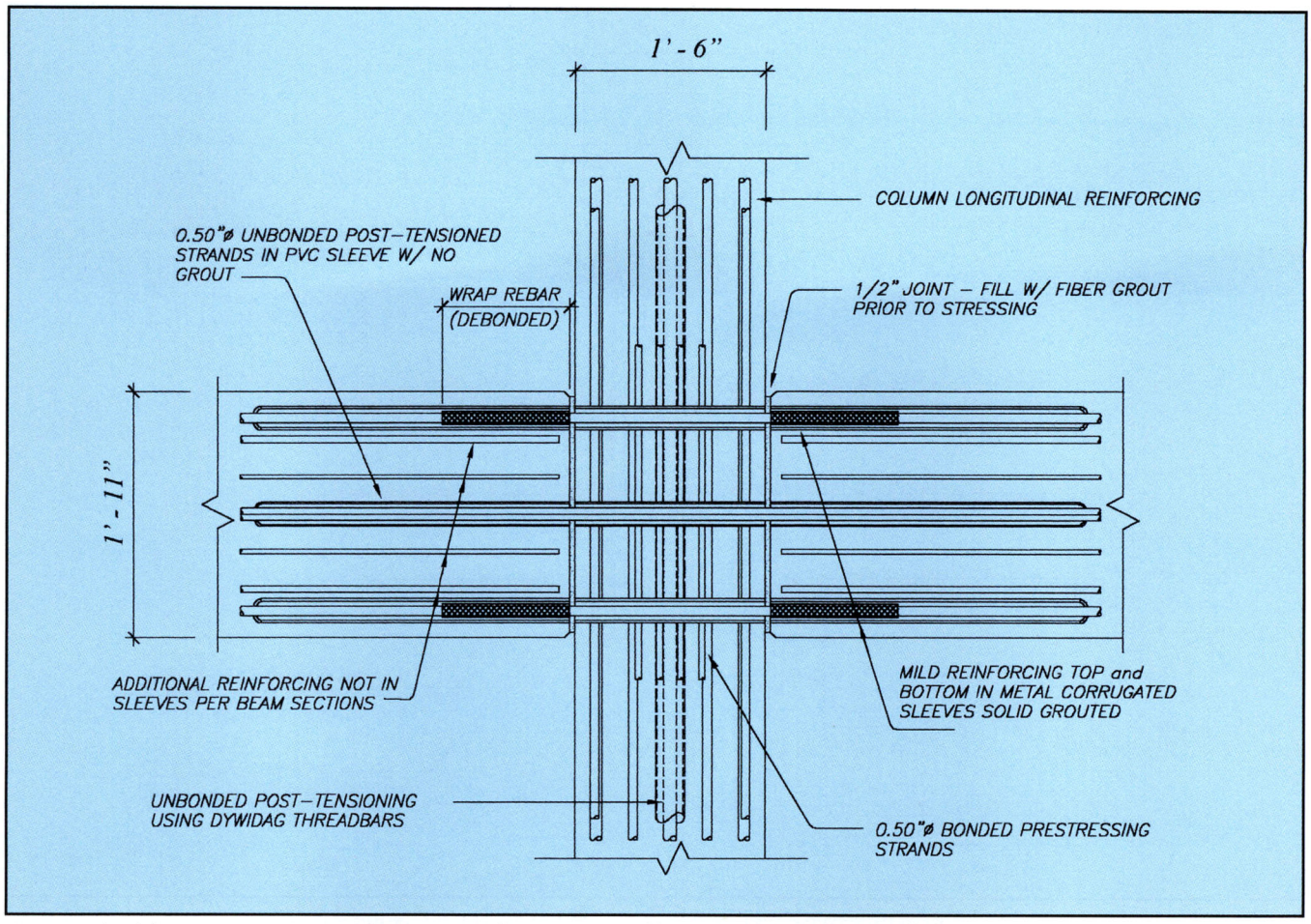


Fig. 8. Hybrid frame interior joint (transverse reinforcement not shown for clarity). **Note:** 1 ft = 0.3048 m; 1 in. = 25.4 mm.

non-loadbearing wall systems that are otherwise identical to their loadbearing wall counterparts.

It is fairly straightforward to lengthen the building period in a precast shear wall system by providing vertical joints between the panels that make up a wall (see PCI Ad Hoc Committee Report on Precast Walls[5]). Thus, by providing a jointed shear wall, the design forces are reduced, resulting in a reduced building cost.

Results of Force Based Design

It should be noted that, although improving ductility and lengthening the system period reward buildings with lower design loads, the magnitude of the reduction reflects only poorly the true advantages that well-designed precast systems offer. For example, the design base shear for the prototype building using force based design in accordance with the 1997 UBC (Zone 4) is as follows:

Frame direction (T_n = 0.67 seconds)
Design base shear = 2248 kips (10000 kN)
Wall direction (T_n = 0.48 seconds)
Design base shear = 4889 kips (21746 kN)

These values reflect the advantages of a ductile system (i.e., R = 8.5 for frames) and a lengthened period for the shear wall building which is comprised of jointed wall panels. The design base shear for an equivalent cast-in-place frame system would be identical, since the elastic stiffnesses of a precast frame and a cast-in-place frame are similar. However, the elastic

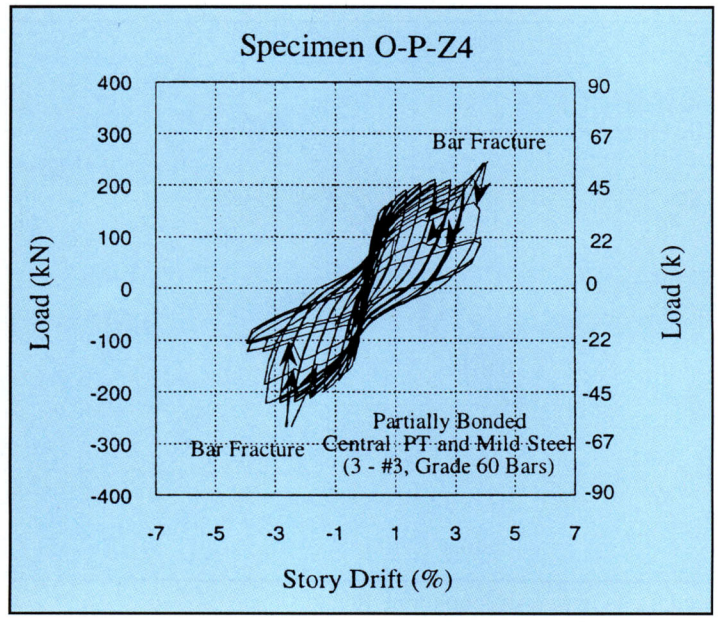


Fig. 9. Hybrid frame hysteresis loop (from Ref. 6).

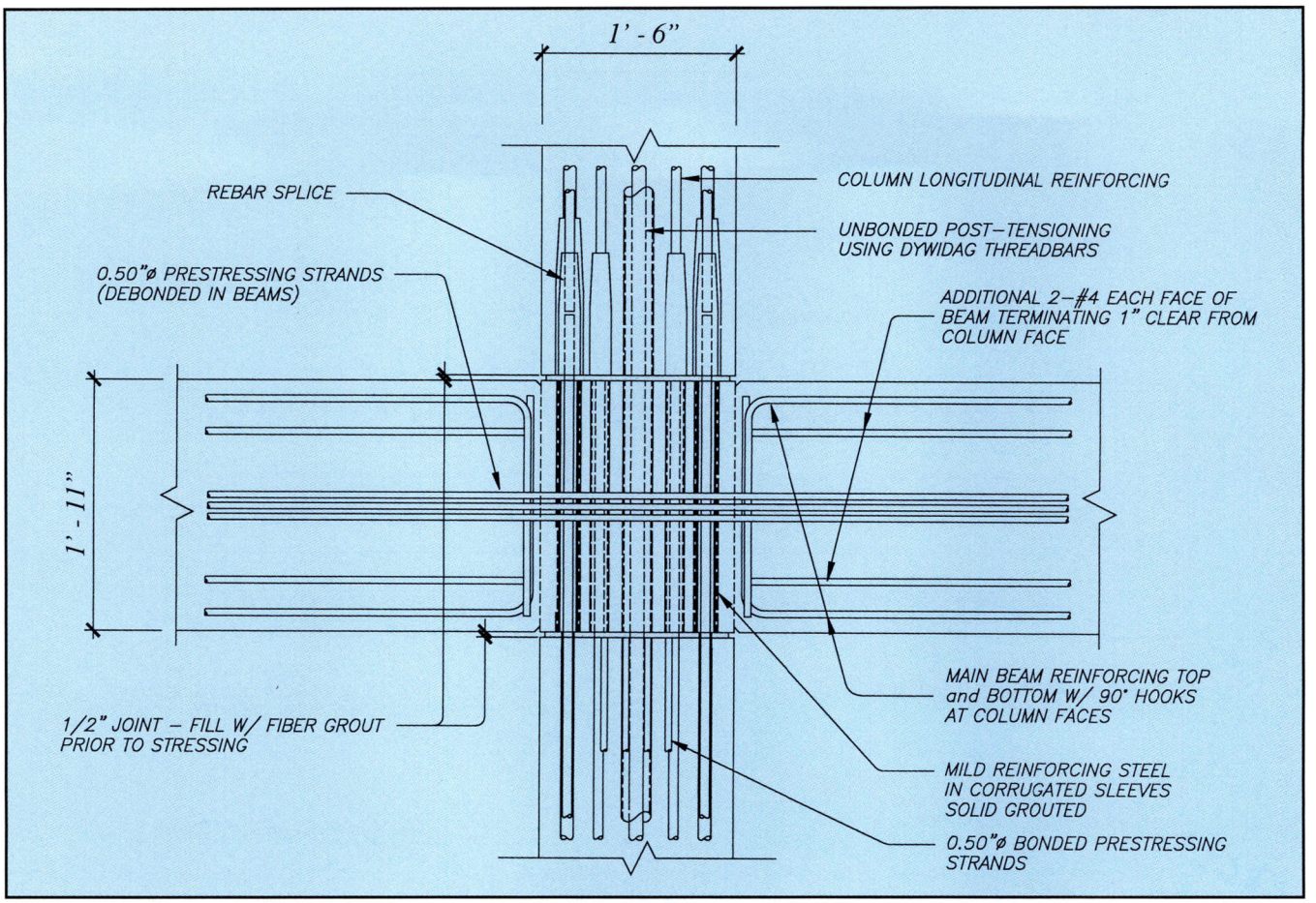


Fig. 10. Pretensioned frame interior joint (transverse reinforcement not shown for clarity). **Note:** 1 ft = 0.3048 m; 1 in. = 25.4 mm.

period for an equivalent, non-jointed, cast-in-place wall would be substantially shorter than the jointed wall period. Except in cases where the maximum base shear governs, a shorter period would result in a higher base shear.

While the systems included in the test building are expected to be cost effective even using force based design, the PRESSS III test building adopts an alternative design procedure that more efficiently incorporates the advantages of well-designed precast systems. As will be discussed below, a further reduction to design base shear is achieved, providing substantial cost savings for precast buildings in all seismic zones.

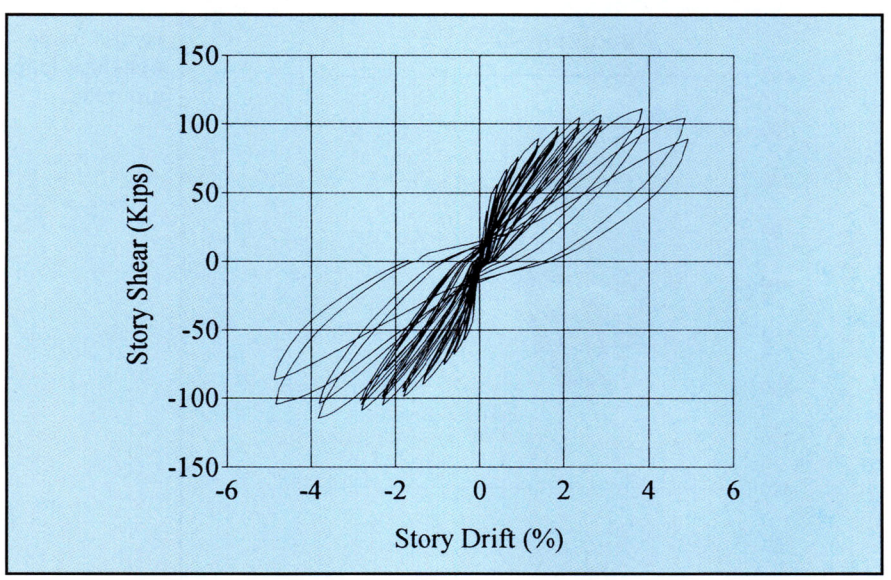


Fig. 11. Pretensioned frame hysteresis loop (from Ref. 7).

DESIGN OF PRESSS III TEST BUILDING

The PRESSS Phase III test building is not intended to create new design concepts, but rather to examine the suitability of design concepts created in earlier phases of the PRESSS program or other precast concrete research. One criterion used in determining which systems would be included in the test building was that the concept had to have been experimentally validated through component tests.

The complete building test is important because it addresses many questions of design and constructability, which do not arise in component tests. Also, the behavior of a complete, statically indeterminate system involves many features, including verification

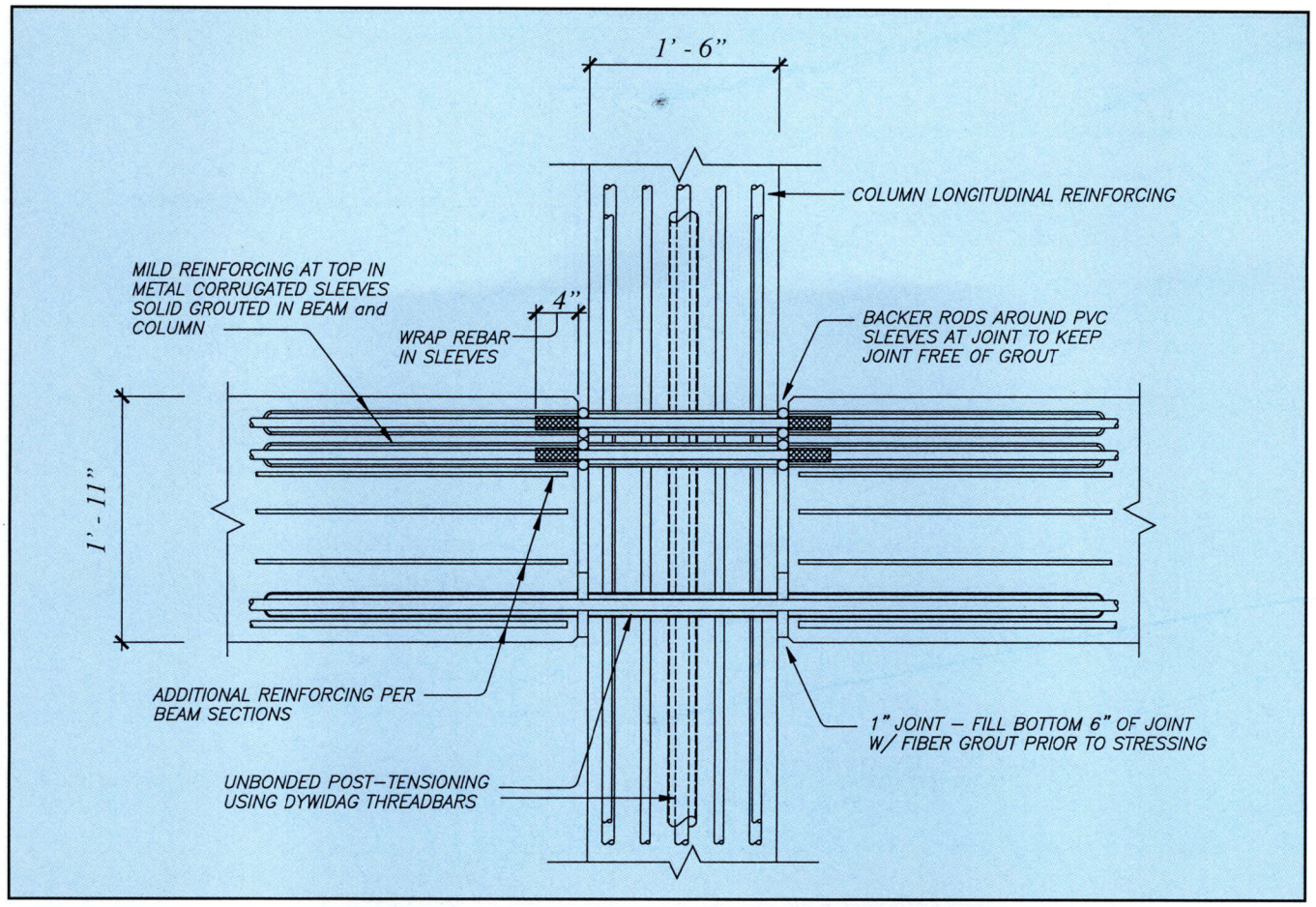


Fig. 12. TCY gap frame interior joint (transverse reinforcement not shown for clarity). **Note:** 1 ft = 0.3048 m; 1 in. = 25.4 mm.

of seismic design methods that do not occur in statically determinate component tests.

The specific objectives of the test are to:

- Validate a rational design procedure for precast seismic structural systems.
- Provide acceptance of prestressing/post-tensioning of precast seismic systems.
- Provide experimental proof of overall building performance under seismic excitation.
- Establish a consistent set of design recommendations for precast seismic structural systems.

The PRESSS III test building consists of frames in one direction and a shear wall in the other, as shown in Figs. 4 and 5. The floor system used in the first three levels is pretopped double tees, and the top two levels consist of topped hollow-core slabs. Those choices were made in order to include the two major structural framing systems commonly used in precast construction today.

The building will be tested in both the frame and wall directions independently under simulated seismic loads that represent earthquakes up to 50 percent stronger than Zone 4 design level earthquakes recognized in codes. During the loading in each direction, two independently controlled actuators at each floor level will prevent torsion.

Frame Connection Systems

Four different types of ductile connection systems are used in the PRESSS III test building frames. They are:

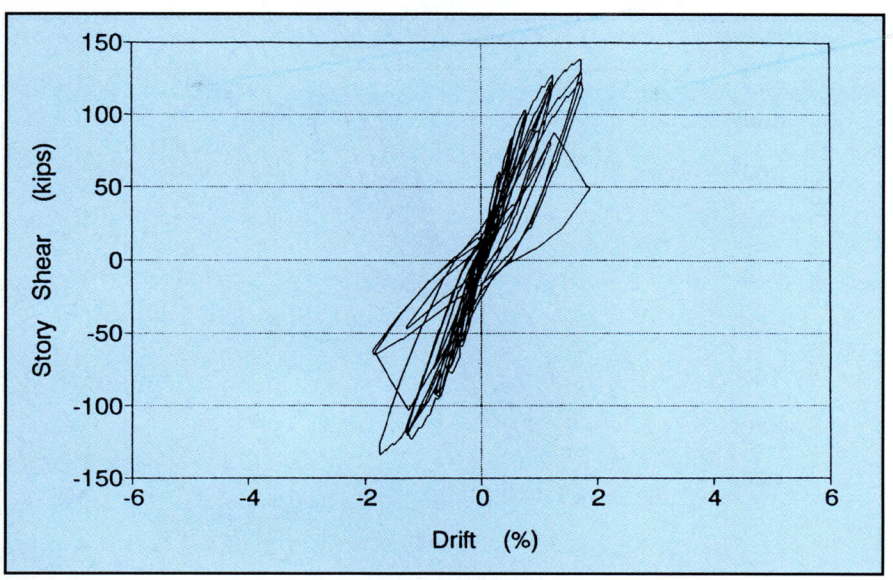


Fig. 13. TCY gap frame hysteresis loop (from Ref. 7).

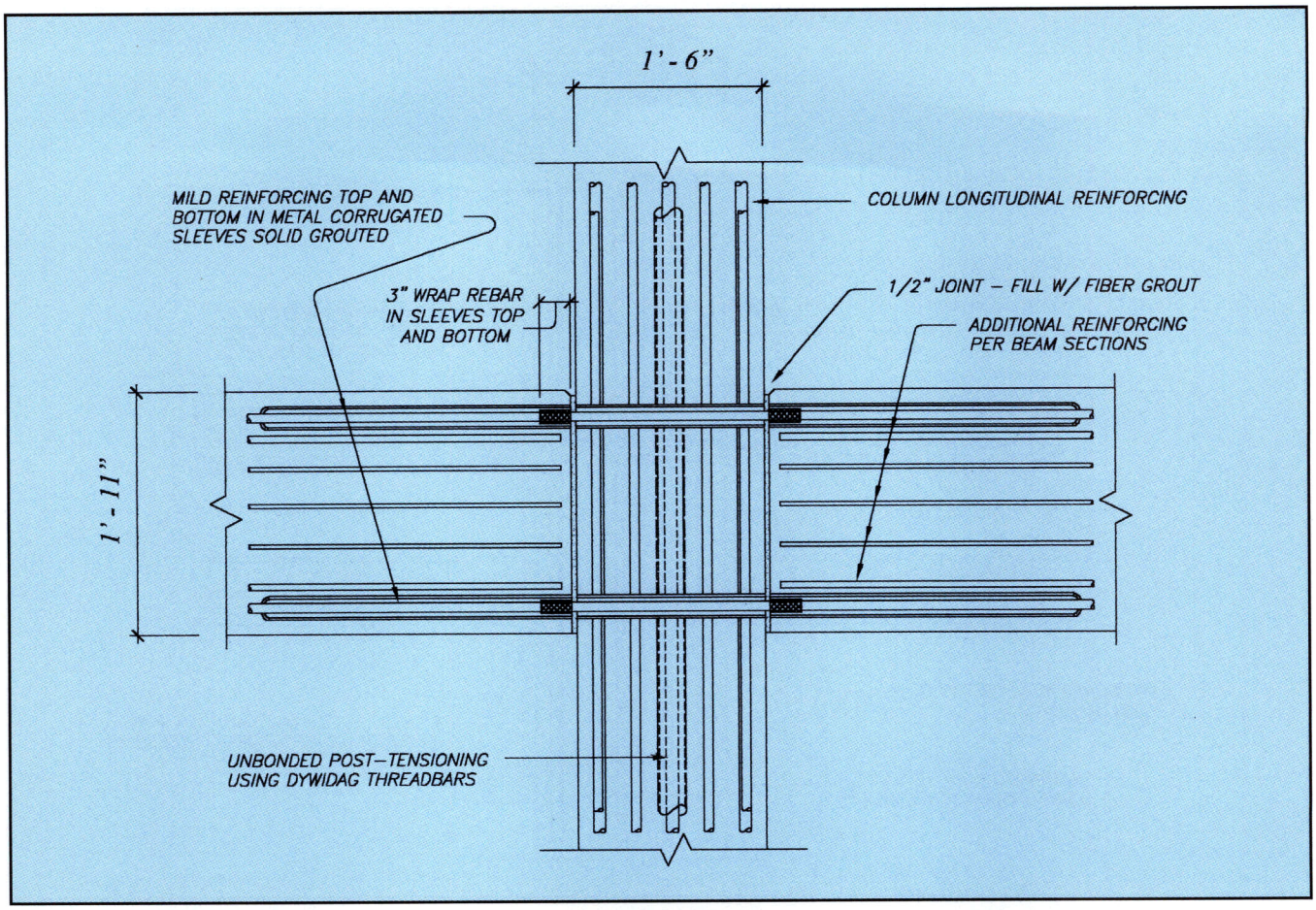


Fig. 14. TCY frame interior joint (transverse reinforcement not shown for clarity). **Note:** 1 ft = 0.3048 m; 1 in. = 25.4 mm.

- Tension-Compression Yielding (TCY) gap connection
- TCY connection
- Hybrid connection
- Pretensioned connection

The first three types of connections consist of multistory columns and single-bay beams, and are appropriate for floor-by-floor construction. The pretensioned connection uses multibay beams and single-story columns and is appropriate for "up-and-out" construction.

The hybrid connection and pretensioned connection are used in one seismic frame, referred to as the Pre-Tensioned Frame, and the remaining two connections are adopted in the other seismic frame, known as the Tension-Compression Yielding Frame. These two frame elevations are shown in Figs. 6 and 7, respectively. The amounts of energy dissipation and residual displacement vary among the four connections, allowing a designer to control seismic behavior of the structure with an appropriate choice of connection system.

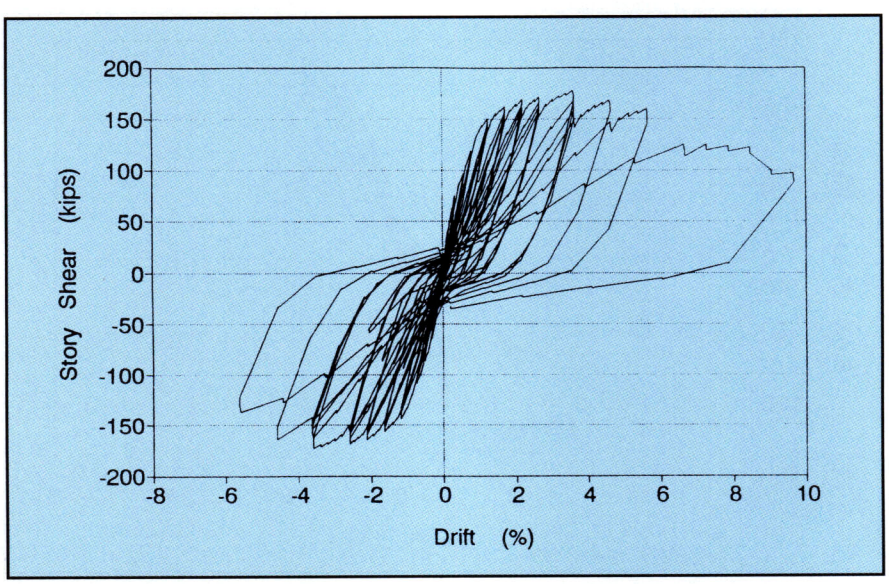


Fig. 15. TCY frame hysteresis loop (from Ref. 7).

Hybrid Frame

The hybrid connection was developed during the last phase of a multiyear project at the National Institute of Standards and Technology (NIST).[6] The hybrid frame interior joint is shown in Fig. 8. The beams are connected to multistory columns by unbonded post-tensioning strands that run through a duct in the center of the beam and through the columns. Mild steel reinforcement is placed in ducts at the top and bottom of the beam,

through the column, and is grouted. It yields alternately in tension and compression and provides energy dissipation (see Fig. 9). The amount of mild steel reinforcement and post-tensioning steel are balanced so that the frame re-centers after a major seismic event.

The exterior joint of the Hybrid Frame uses a "stub" beam that contains the multistrand anchor. This is only required due to the scale of the test building. Research[8] indicates that anchors located within the joint may actually improve joint performance.

PreTensioned Frame

The PreTensioned frame, named so as to differentiate it from just any frame constructed with pretensioned members, is intended to be used for construction where the most economical method consists of using one-story columns with multi-span beams. Long, multi-span beams are cast in normal pretensioned casting beds, with specified lengths of the pretensioning strand debonded.

These beams are then set on one-story columns with the column reinforcing steel extending through sleeves in the beams. Reinforcing bar splices ensure the continuity of the column above the beam, as shown in Fig. 10. As the frame displaces laterally, the debonded strand remains elastic. While the system dissipates relatively less energy than other systems[7,9,10] (see Fig. 11), it re-centers the structure after a major seismic event.

TCY Gap Frame

The TCY gap frame addresses the problem of frame beam elongation in an innovative way. The beams are erected between columns leaving a small gap between the end of the beam and the face of the column. Only the bottom portion of this gap is grouted to provide contact between the beam and column (see Fig. 12). Centered on this bottom grout region, post-tensioning bars clamp the frame together. At the top of the beam, mild steel reinforcement is grouted into sleeves that extend the length of the beam and through the column.

The reinforcing steel is carefully debonded for a specified length at the gap so that it can yield alternately in tension and compression without fracture. Since the gap opens on one side of the column as it closes on the other side by an equal amount, the length of the frame does not change, even as the connection yields. The TCY gap connection tested in a PRESSS Phase II research program[7] used a coupler to splice the reinforcing steel through the column, rather than the sleeve through the column shown in Fig. 12.

The hysteresis loop obtained in PRESSS Phase II shows that this system[7] was performing as expected, and dissipated significant energy, until premature failure of the reinforcing bar couplers at the top of the beam failed the connection (see Fig. 13). The possibility of a premature failure of this type is eliminated by the sleeved connection.

TCY Frame

The TCY frame connection attempts to model a traditional tension/compression yielding connection, similar to what is used in cast-in-place construction. However, rather than distributed yielding over a finite plastic hinge length, yielding is concentrated at the connection. To ensure that the beam reinforcement that provides moment strength and energy dissipation does not fracture prematurely at this concentrated yielding location, it is debonded over a short length at the beam-to-column interface (see Fig. 14).

This type of connection was also tested in PRESSS Phase II research program,[7,9,10] where it showed slightly pinched hysteretic behavior due to vertical slip at the beam-to-column interface (see Fig. 15). Although this type of behavior may also occur in the PRESSS III test building, the connection has been included since it is conceptually very similar to traditional methods of construction. If vertical slip starts to occur at the ends of these beams, steel corbels will be installed during the test so that slip does not adversely affect the overall test results.

Frame Columns

The frame columns used for all systems contain both mild steel reinforcement and post-tensioning bars (see Figs. 8, 10, 12 and 14). The post-tensioning bars are intended to represent the equivalent dead loads based on the prototype structure, but their inclusion in the test will also validate that this method of adding vertical load to a precast column is an effective way to influence system performance.

In addition, the columns in the prestressed concrete frame are pretensioned up to the fourth level of the building. This bonded prestressing economically adds strength to the columns, which are prevented from yielding using capacity based design. These details will validate the performance of pretensioned frame columns.

Building Frame Choices

While it was never intended that multiple connection types would be used on different floors or in different frames of the same building in practice, the PRESSS research team and industry advisors felt strongly that several different frame systems should be included in the test building. The objective was to provide designers with several alternatives using precast concrete; not just different ways of building conceptually similar systems (e.g., structural steel in Table 2), but systems with

Table 2. Frame system alternatives.

Cast-in-place concrete	Masonry	Structural steel	Precast concrete
Special Moment Resisting Frame per code	Frame per code	Dog bone	Hybrid
—	—	Cover plates	Pretensioned
—	—	Meyers Nelson Houghton connection	Tension/compression yielding gap
—	—	Others	Tension/compression yielding
—	—	—	Others

fundamentally different types of behavior that might be appropriate for different situations. This, as shown in Table 2, will provide versatility using precast concrete that is not currently available using any other building material.

In addition, by validating several different frame types, it is hoped that future innovations can fit into the framework developed by the PRESSS research program, through component testing rather than requiring additional large-scale building tests.

Wall System

For the past several years, the PCI Ad Hoc Committee on Precast Walls has been promoting precast shear walls as seismic resisting systems for all seismic zones.[5] This work has focused on "tuning" jointed walls to lengthen the structural period and reduce the design base shear forces. The focus was on evaluating elastic stiffness, without explicit consideration of ductility. Elastic forces were distributed so that sufficient resistance to overturning was provided by the gravity loads on the system.

The PRESSS test building takes this concept one step further by considering the behavior of the jointed shear wall system when the wall lifts off and rocks, together with its effect on design forces. An appropriate level of hysteretic damping is added to the wall system through the connection devices located at the vertical joint between the wall panels.

Due to limitations on the building size, imposed by the dimensions of the testing laboratory, only one jointed wall system is incorporated in the test building. Instead of limiting the lateral loads to those that could be resisted by the inherent gravity loads in the system, vertical unbonded post-tensioning is used to resist overturning in this wall system.

U-shaped flexure plates (UFP), as tested in PRESSS Phase II,[11] are used for vertical joint connection devices where damping is achieved by means of flexural yielding of the plates. The unbonded post-tensioning is designed to re-center the wall system when the load is removed so there will be no residual drift after a design-level earthquake. Re-centering is ensured by relating the elastic capacity of the post-tensioning system to the yield strength of the panel-to-panel connections.[12]

Fig. 16 shows the shear wall elevation, with unbonded post-tensioning located at the center of each panel. The shear wall is expected to displace laterally to approximately 2 percent story drift under a design-level earthquake. This is consistent with the drift limits specified in both the UBC[2] and NEHRP provisions.[3]

This lateral displacement requires a vertical panel-to-panel displacement of about 2 in. (51 mm) for the 9 ft (2.74 m) panel. Thus, the UFP connection shown in Fig. 17 was chosen for its ability to retain its force capacity through this large displacement. The post-tensioning was designed to be just at the point of yielding at 2 percent drift. Should the designer desire a smaller design story drift, or less energy dissipation, simpler panel connections could be used.

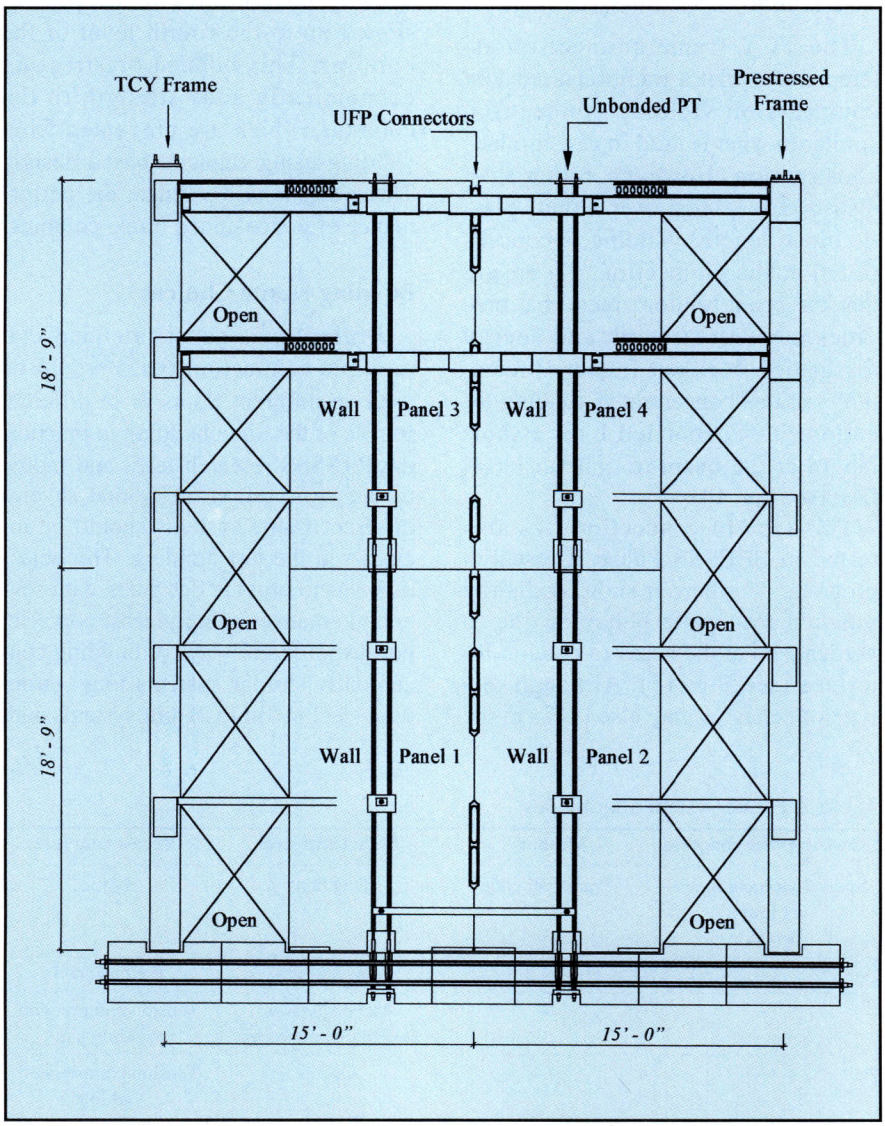


Fig. 16. Elevation of jointed shear wall system. **Note:** 1 ft = 0.3048 m; 1 in.= 25.4 mm.

DIRECT DISPLACEMENT BASED DESIGN

As noted previously, Force Based Design represents the behavior of jointed precast systems poorly. The method relies on an initial elastic period, which is not only difficult to compute in a system whose flexibility resides largely in the connections, but also has little influence on the post-elastic behavior of the structure. The *R* factors included in design codes are also not intended to be applied to systems, such as some of those used here,

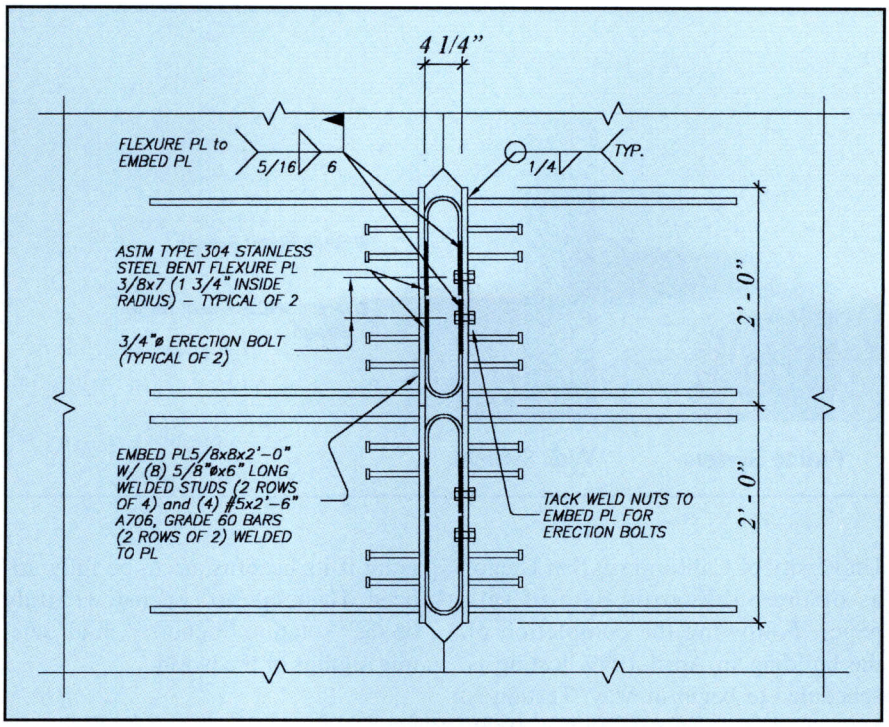


Fig. 17. Detail of U-shaped flexure plate. **Note:** 1 ft =0.3048 m; 1 in. = 25.4 mm.

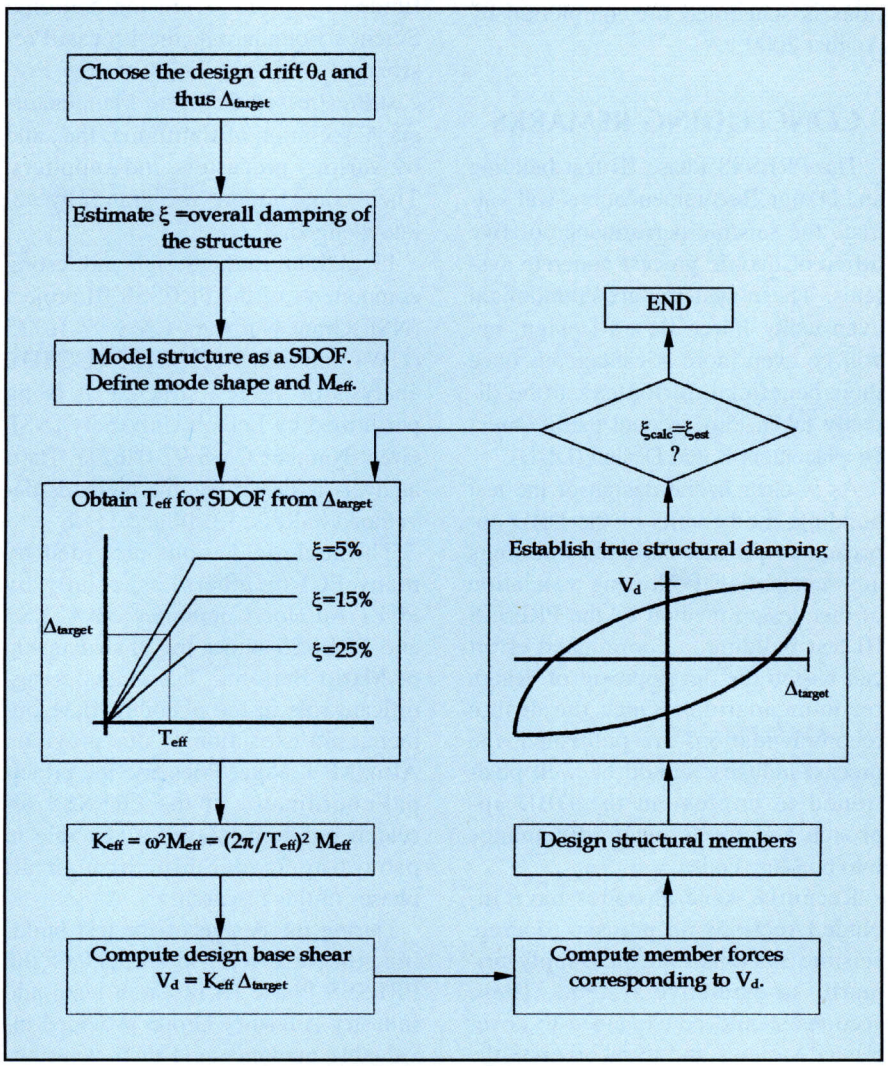


Fig. 18. Flowchart showing Direct-Displacement Based Design method.

which do not emulate monolithic concrete structures. Thus, the results obtained by representing the seismic performance of precast systems using a Force Based Design approach are questionable.

For this reason, the test building was designed using a more consistent Direct Displacement Based Design (DBD) procedure,[4] in which the design is based directly on an inelastic target displacement and effective stiffness. The target structural displacement is determined from an allowable interstory drift permitted by design codes while the effective stiffness is approximated to the secant stiffness of the building corresponding to its expected fundamental mode of response. Use of both the elastic stiffness for determining inelastic structural displacements and arbitrary reduction factors, as in Force Based Design, are completely eliminated in this design approach.

Direct Displacement Based Design Procedure

Direct Displacement Based Design (DBD) is a process that is intended to ensure that the structure reaches, but does not exceed, a target displacement selected by the designer, in response to a given ground motion. In this method the true hysteretic behavior is replaced by a linear system in which the stiffness is equal to the true secant stiffness and the viscous damping provides the same energy dissipation per cycle.

The DBD design procedure, as adopted in the test building, is illustrated in Fig. 18. Once the target drift is chosen, the damping is estimated for the building using prior component test results. Representing the building with a SDOF system, the fundamental period corresponding to the target displacement is found from the displacement spectrum. The effective stiffness is computed from the known mass and the estimated period.

The design base shear is then obtained from the effective stiffness and target displacement. Member sizes and reinforcement are chosen to resist this base shear. The true physical properties of the members are used to generate a more refined, hysteretic, force-displacement curve. The effective damping is calculated from the hys-

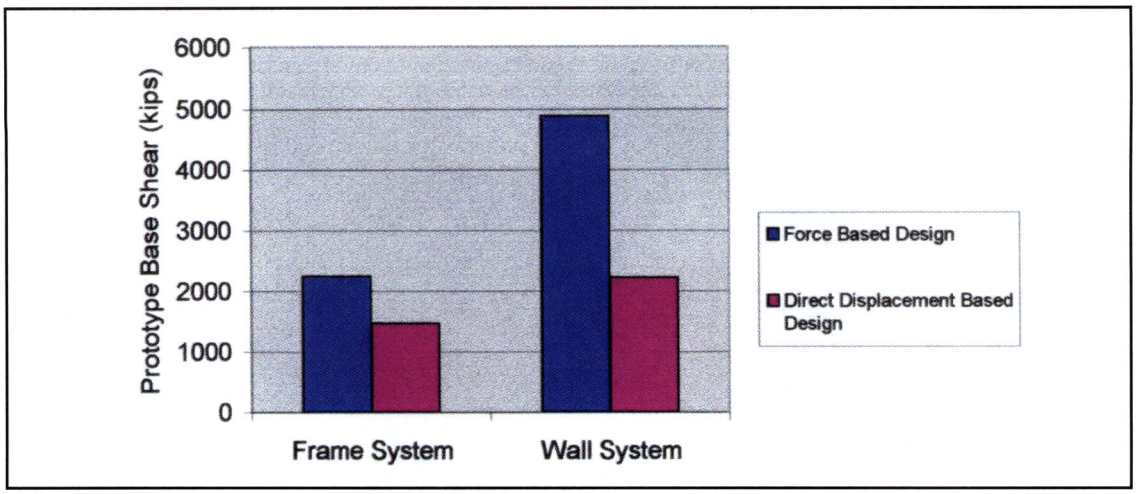


Fig. 19. Comparison of design base shears.

teresis loop area and is checked against the assumed value. If they differ significantly, the process is repeated with a new value of assumed damping. This final step is only necessary because of the lack of information on global hesteretic damping for the systems used in the test building.

Results of Direct Displacement Based Design

For the PRESSS III prototype building, Direct Displacement Based Design resulted in a design base shear noticeably lower than would be used for force based design. For the prototype building, the design base shears are as follows:

Frame direction:
Design base shear = 1467 kips (6525 kN)

Wall direction:
Design base shear = 2223 kips (9888 kN)

In the frame direction, this is 65 percent of the equivalent Force Based Design base shear, resulting in a substantial cost savings. In the wall direction, the savings are similar, even if the lengthened period is used in Force Based Design. The wall direction DBD base shear is just 45 percent of the equivalent Force Based Design value (see Fig. 19). Clearly, the improved performance of these systems can also result in substantial cost savings over traditional structural systems.

TESTING SCHEDULE

The PRESSS III test building is under construction in the Charles Lee Powell Structural Laboratory of the University of California at San Diego, as of the publication date of this paper. Following the completion of the building in April 1999, testing is scheduled to begin in May. Testing is expected to be complete by July 1999, with analysis and reports to follow. The report on design recommendations is scheduled for completion by August 2000.

CONCLUDING REMARKS

The PRESSS Phase III test building and Design Recommendations will validate the seismic performance of five different ductile precast concrete systems. These systems are economical even using Force Based Design, but will be even more advantageous once their beneficial attributes can be directly taken into account using Direct Displacement Based Design (DBD).

As is clear in the design of the test building, the benefits of the DBD approach to precast concrete buildings are substantial. Following validation of this design method by the PRESSS III test building, a coordinated effort can hasten the development of design recommendations. Once the design recommendations are published, the precast industry should be well positioned to implement the DBD approach and facilitate its acceptance into building codes.

Recently, several codes have included sections on precast concrete seismic systems, but they apply primarily to emulative systems. These sections should be expanded to cover jointed systems and to incorporate the results of the PRESSS research program if its benefits are to be fully utilized. Then, precast concrete will truly be the "solution of choice" in all seismic regions of the world.

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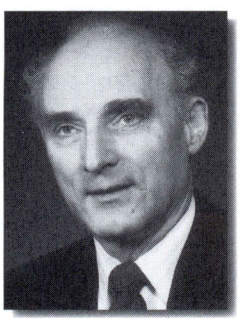
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Design of Prestressed Girder Bridges Using High Performance Concrete — An Optimization Approach



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The objective of this paper is to assess the potential economic benefits from the use of high performance concrete (HPC) for continuous precast, prestressed I-girder highway bridges. Unlike the traditional approach of trial and error, this approach uses mathematical optimization techniques to provide systematic procedures for obtaining optimal designs. The analytical results illustrate the cost effectiveness of HPC for bridge girders. Several practical advantages come from the use of HPC. Among the benefits is that because of increased girder capacity, the increased cost of HPC is fully offset by the reduced number of girders.

A common application of high performance concrete (HPC) has been in building structures in which strengths of 100 MPa (14.5 ksi) and more have been used in the lower columns of high-rise buildings. In comparison, according to a 1990 survey by the PCI High Strength Concrete Committee,[1] the average 28-day concrete strength used in the precast, prestressed concrete industry at that time in the United States was 41 MPa (6 ksi). The maximum strength producers felt they could reliably supply was 59 MPa (8.5 ksi). Similar strengths have been the norm in Canada.

Although some producers are now comfortable achieving strengths of 70 MPa (10 ksi) or more, there is a need to explore the benefits of higher concrete strengths. Precasters and contractors still unfamiliar with HPC assume that the use of HPC involves higher risk. This may result in higher costs for HPC bridges than equivalent designs with normal strength concrete.[2]

The use of HPC in bridges is unlikely to advance quickly without a clear economic incentive. The overall objective of this paper is to assess the potential economic benefits from the use of concrete with compressive strengths of up to 100 MPa (14.5 ksi) for continuous, precast, pretensioned I-girder highway bridges.

These bridges with cast-in-place, reinforced concrete decks on precast, prestressed concrete I-girders are commonly known as slab-on-girder bridges (see Fig. 1). These were chosen because they represent the most common type of prestressed concrete bridge constructed during the last 50 years in North America.[3]