Federation Internationale de la Precontrainte (FIP), Proceedings of the 12th Congress, Vol. 1, 1994

Seismic behavior of a six-story precast office building

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Introduction

As a part of the USA-Japan coordinated Precast Seismic Structural Systems (PRESSS) program, a research project is underway to investigate the behavior of a six-storey precast concrete office building under moderate seismicity. The structure is designed as a 'building frame system' in which the gravity loads are supported by a frame and lateral forces are resisted by shear walls. The gravity load resisting system consists of hollow core planks, prestressed wide and shallow beams, and columns. Shear wall and cruciform panels are the main lateral load resisting elements. Because current building codes in the United States do not address seismic design of precast concrete buildings specifically, a design process for the building system is developed to identify areas where research is needed.

Within the program, cyclic lateral load tests of a variety of panel-to-panel connections are conducted to evaluate energy dissipation capacity, stiffness, and shear strength. Beam-to-column joints are also subjected to cyclic loading to verify the ability to withstand large drifts without losing vertical load-carrying capacity.

Design problem

The centerpiece of this research project is a six-storey precast office building with a typical floor-to-floor height of 4.0 m. Plan dimensions are 31.1 m × 68.3 m divided into twenty 9.8 m × 10.4 m bays as shown in Fig. 1A. This building configuration is selected because it represents a commercial building layout commonly used in the United States.

The gravity load resisting system consists of 203 mm thick hollow core planks, 2.4 m wide and 406 mm deep prestressed beams (Fig. 1B), and 508 mm × 508 mm columns. Hollow core planks are supported on beams which, in turn, rest on columns. The beams are continuous while the columns are discontinuous through the beam-column joint. Columns are mechanically spliced at the job site to achieve continuity. The beam cross-section is developed through a study sponsored by the Precast/Prestressed Concrete Institute (PCI) [1]. The lateral load resisting system comprises exterior architectural cruciform panels (Fig. 1C) and interior shear walls in the stairwells and elevator cores.

Several benefits follow from the adoption of this building system. Since only precast structural elements are used, the superiority of precast construction over cast-in-place concrete construction in erection speed, product quality, and span length is retained. In addition, column corbels are not used in the present building, which simplifies column fabrication. Furthermore, studies have shown that the 'building frame system' is an outstanding system for seismic resistance [2].

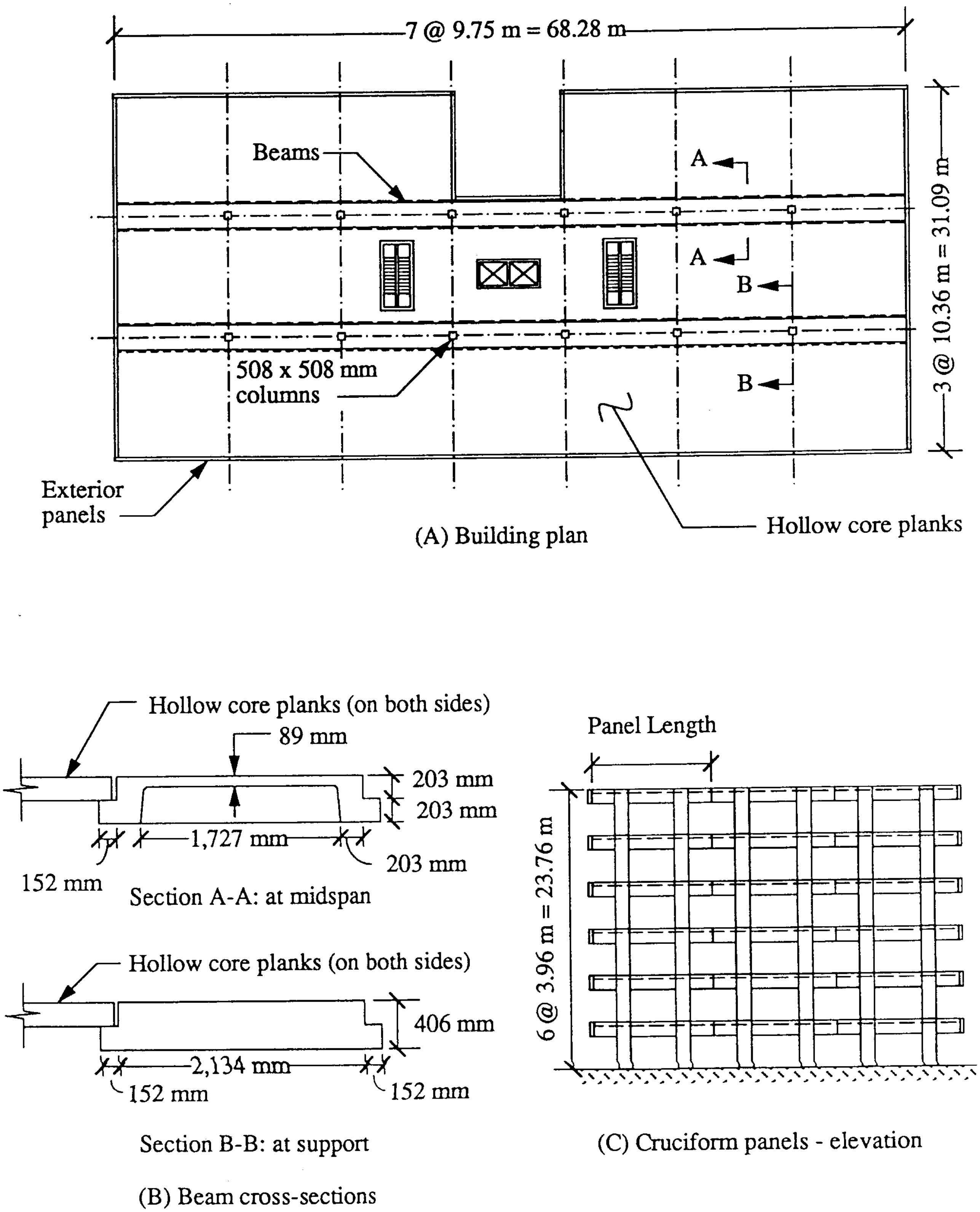
Structural analysis

The interior framing system is analyzed under gravity loads. Floor loads consist of 2.4 kPa live loads, 1.7 kPa superimposed dead loads, a 1.0 kPa uniformly distributed construction load, and a 2.9 kPa hollow core plank weight.

During construction, the hollow core plank weight and other loads are assumed to be applied to the beam after beam continuity is created. Applying the factored loads of 1.4D + 1.7L, where D is the dead load and L is the live load, as suggested by ACI 318-89 Building Code [3], critical bending moments, shear forces, and axial forces are computed.

A two-dimensional, static, lateral load analysis of the building is performed, assuming that the building is subjected to moderate seismicity (Uniform Building Code [4] Seismic Zone 2B). Since there are no standard specifications for the analysis and design of precast concrete structures in seismic zones in current U.S. building codes, earthquake loads and design requirements from the Uniform Building Code (UBC) are used as guidelines. According to the UBC, factored loads are given by (1.05D + 1.40E) or (0.9D + 1.43E), where D and E are dead loads and earthquake loads, respectively.

Total base shear of the building is computed using the UBC equations. The response modification coefficient (R_w), which depends on the type of structure, is assumed to be equal to 8.0. This coefficient needs verification since the UBC does not explicitly address precast concrete structures. At the time of this analysis, the authors reasoned that R_w for a precast concrete frame is less than the value used for a cast-in-place monolithic frame (R_w = 12) since the level of hysteretic energy absorption for precast systems is less than that for cast-in-place frames. The other reason is because the



(A) Building plan

Section A-A: at midspan

Section B-B: at support

(B) Beam cross-sections

(C) Cruciform panels - elevation

Fig. 1 Selected system

cruciform panel (Fig. 1C) is designed on the basis of monolithic emulation, which is considered equivalent to an intermediate moment-resisting frame.

The computed base shear is distributed among cruciform panels and shear walls in proportion to their relative stiffnesses, assuming that floor diaphragms are rigid. A two-dimensional finite element model (Fig. 2) is generated using a general purpose structural analysis program for the analysis. The model comprises cruciform panels and interior shear walls connected with rigid links. The cruciform panels are modelled using beam and column members, the rigid links using truss members, and the shear walls using four-node hybrid plate elements. Spring elements are used to simulate the additional stiffness from panels in the perpendicular direction.

Structural member design

Structural elements of the building system are proportioned and detailed assuming normal weight concrete (unit weight = 2400 kg/m^3) with a 28-day compressive strength $f'_c = 51.7$ MPa for beams and columns, and $f'_c = 34.5$ MPa for wall panels. The concrete strength at prestress release is $f'_{ci} = 24.1$ MPa for the beams. The prestressing steel is assumed to be low-relaxation seven-wire strand with ultimate strength $f_{pu} = 1860$ MPa. Grade 60 reinforcing bars with a minimum yield strength $f_y = 414$ MPa are used for structural members, and A36 structural steel with a minimum $f_y = 248$ MPa is used for connections.

A total of 26-12.7 mm diameter prestressing strands are provided in each beam, 14 for positive moment and 12 for negative moment. These amounts are based on ultimate strength requirements. The level of prestressing, however, is based on working stress requirements. Stresses at prestress release and under full service loads are checked against the limitations specified in the ACI 318-89 Building Code [3]. Shear design of the beam is based on the ACI 318–89 Building Code requirements for prestressed concrete members. Inverted u-stirrups are used in the beam for ease of fabrication. Stirrups are spaced 152 mm center to center.

A 508 mm × 508 mm square column section is found to be adequate for this building system. The cross-section is kept constant throughout the building height. Structural design is performed with the aid of the commercial computer program PCACOL [5]. The design showed that four 28.6 mm diameter (#9) bars are needed. Rectangular 13 mm diameter (#4) closed ties at 51 mm spacing are provided at both ends of the column for a distance of 610 mm as a confinement reinforcement for improved ductility. Outside the confinement regions, the ties are spaced 152 mm center to center.

Details of the shear wall panels are shown in Fig. 3. Each 254 mm thick precast unit is three stories high and 3.05 m wide. Two such panels are erected side-by-side to create a three-storey wall segment. Principal vertical reinforcement is proportioned assuming the shear wall is a flexural member under combined bending and axial compression. Design strength to resist the factored loads in the first storey of the shear wall is provided by five 36 mm diameter (#11) deformed bars along each exterior edge of the shear wall (Fig. 3B). These bars are continuous through the horizontal joints and are spliced using grout-filled splice sleeves.

Continuous confinement is provided by 7 mm diameter wire (W6 or No. 1 Gage) bent into 203 mm diameter spirals with a pitch of 38 mm (Fig. 3B). This reinforcement is provided in the first storey only because the load requirement

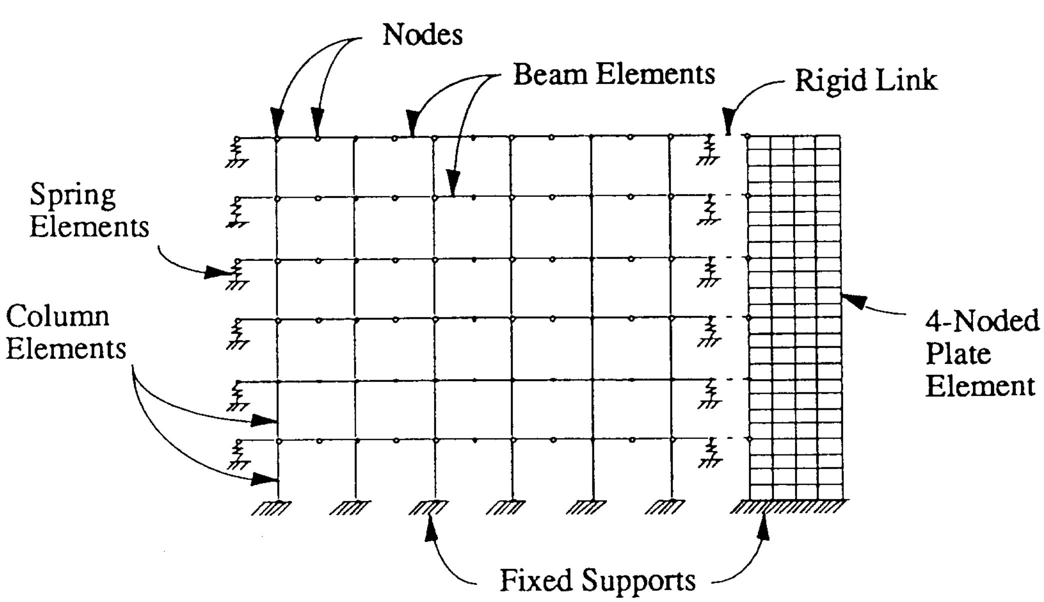


Fig. 2 Mathematical model.

is considerably higher. However, verification of this technique to strengthen the compression region of precast shear walls is identified as one of the issues to be investigated.

In the present design, two layers of wall reinforcement are provided in the form of 152 mm × 152 mm meshes of W8 plain welded wire fabric. The wire has a nominal diameter of 8 mm.

Typical design and detailing of interior frame connections

Details of column-column, beam-beam, beam-hollow core slabs connections are briefly discussed in this section. Columns of adjacent levels are mechanically spliced using commonly available grout-filled splice sleeves. Beam-to-beam connections are achieved using three steel plates. In a typical connection, two of these plates are embedded in one beam and the third is cast in the adjacent beam. Holes are drilled in the plates to allow for a bolted connection. Connection elements are covered with mortar after erection. Each beam-to-hollow core plank connection is provided using looped wire inserts and threaded rods (Fig. 4). The wire inserts are embedded in the beam and the threaded rods are tied to the inserts and placed between hollow core planks.

Typical design and detailing of wall connections

Shear wall panels are connected at horizontal joints using the grout-filled sleeves that serve to splice the vertical reinforcement in the jambs (Fig. 3). Shear strength of horizontal joints at ultimate flexural condition is assumed to arise from friction in the compression zone. Under such conditions, shear strength is estimated as the product of the resultant compressive force in this region and a coefficient of friction of 0.6, as recommended by the ACI 318-89 document [3] for shear friction design. Verification of this mechanism is needed for two reasons.

- If the joints crack due to rocking and/or flexural tension, as it is likely in this flexure-dominated slender wall, the coefficient of friction may decrease under cyclic loading due to grinding damage of the grouted joint.

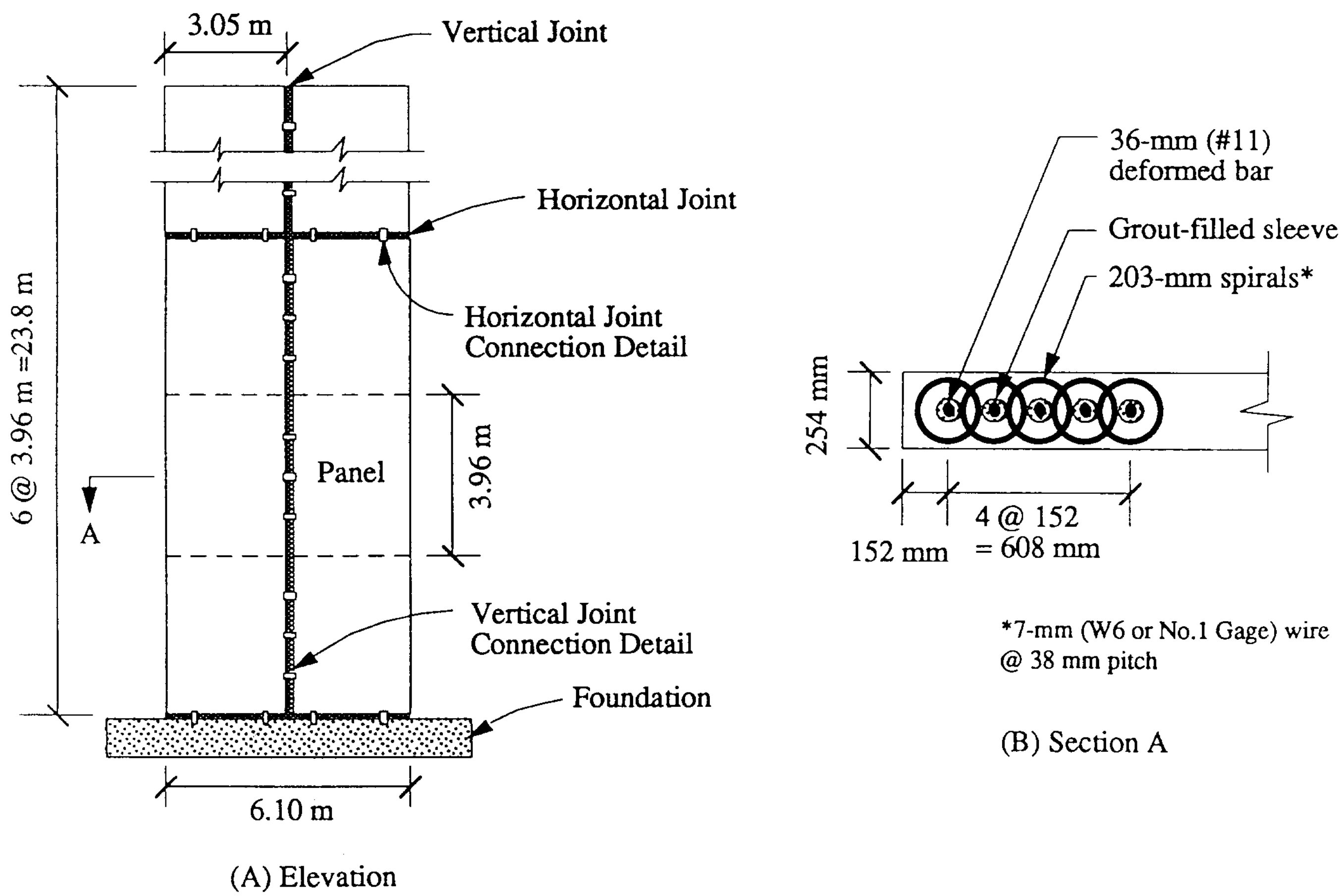


Fig. 3 Shear wall panels

- The coefficient of friction may be different from that observed in shear friction mechanisms because the joint may be pre-cracked at the interface of two different materials (concrete and grout). However, if additional shear resistance is needed, a variety of measures including shear keys and shear reinforcement, such as plates, angles, and bars can be provided.

A typical vertical joint connection detail is shown in Fig. 6. The connection features a notched plate welded to embedded plates in each of the adjacent panels. Fillet welds with returns are provided to ensure strength and stiffness, and the embedded plates are anchored by means of headed anchor studs and reinforcing bar anchors. In addition, it was also assumed that upon reaching design capacity, the notched plates will yield and dissipate energy through non-linear cyclic deformation. Thus, all other component of this ductile connection must be proportioned so as not to yield at loads smaller than the strength of the notched plate. The panels are assumed to remain elastic, up to and beyond yielding of the notched plates.

Using a shear yield stress equal to 0.6 F_y and Load and Resistance Factor Design (LRFD) procedure recommended by AISC [6], it is found that three connections using notched plates with an effective cross-section of 108 mm x 16 mm are needed for each storey. The shear demand on the welds was determined assuming that the notched portion of the plate achieves ultimate capacity in shear, and a weld leg dimension of 13 mm and 76 mm long returns are needed. Four 17 mm studs with a length of 102 mm are used as headed anchor studs to resist the vertical shear force required for the notched plate to achieve shear strength. Two 22 mm diameter deformed bars (#7) with a length of 533 mm are needed at corners of the embedded plates to resist the torque associated with the shear strength of the notched plates (Fig. 6).

Beam-column joint test

A beam-column joint test is conducted to investigate the toughness of the connection under reversed cyclic loading. A schematic drawing of the test setup is shown in Fig. 5. The ends of the beam are restrained from vertical translation, and the upper end of the top column is restrained from horizontal movement. The loading pattern suggested by the PRESSS Coordinator is used for the test [7]. In the first cycle, the specimen is loaded to 75% of the column's theoretical capacity. The displacement corresponding to this load level is then divided by 0.75 to obtain the yield displacement of the joint. Loads are applied to the joint with three cycles at 1.0, 1.5, and 2.0 of the yield displacement (Δy).

Hysteresis curves for the joint are plotted in Fig. 7. As can be noted from the graph, the joint dissipated energy and exhibited large reductions in stiffness as shown by the form of pinched hysteresis loops. The pinched region of the hysteresis loops is the outcome of opening and closing of the interface between the beam and the column, as well as slip of the column longitudinal reinforcing bar. The reduced stiffness regime and the rather small energy dissipation capacity of this assemblage are not considered detrimental to overall building performance because the beam-column assemblage is part of the gravity load resisting system. Even up to a displacement ductility of 2, the joint showed a trend to increase in the load capacity. It is also observed that the storey drift of the joint at 2 Δy is about 3%.

Conclusion

The information presented in this paper is based on the design requirements adopted from building codes which do not specifically address issues unique to precast concrete buildings. This research program seeks to overcome these deficiencies. Cyclic testing of representative specimens of structural members and subassemblages are in progress to verify the adequacy of such members and connections to withstand applied gravity and seismic forces. The testing program includes full scale beam-column connections, twelve specimens which represent horizontal or vertical connections in precast shear walls, and a number of bar splice specimens. A full scale beam-column connection has been tested and test results are presented in this article.

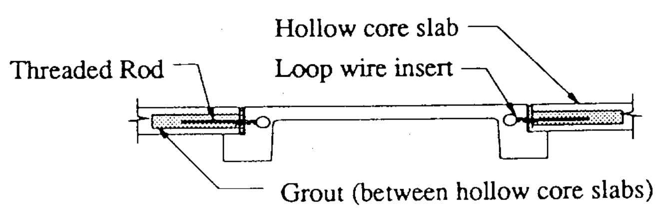


Fig. 4 Beam-to-hollow core plank connection.

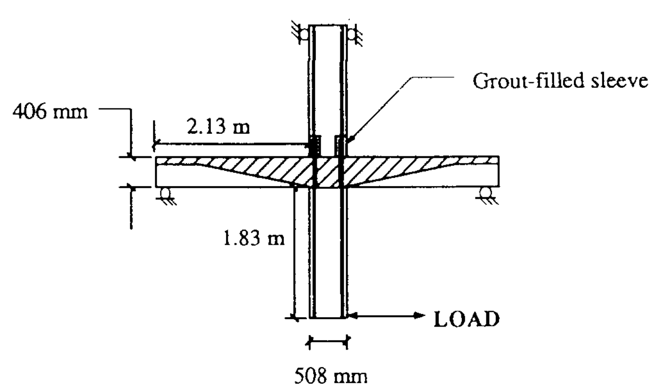
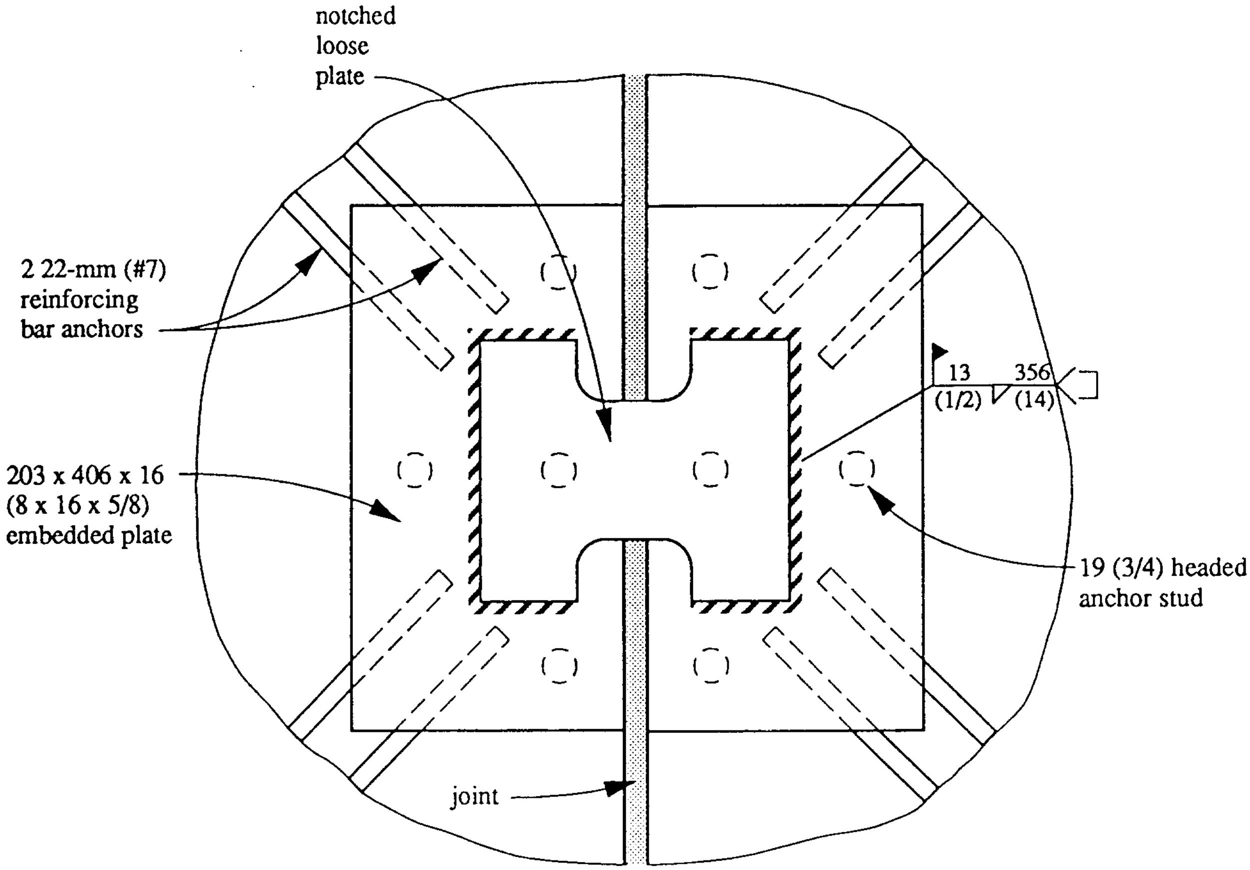
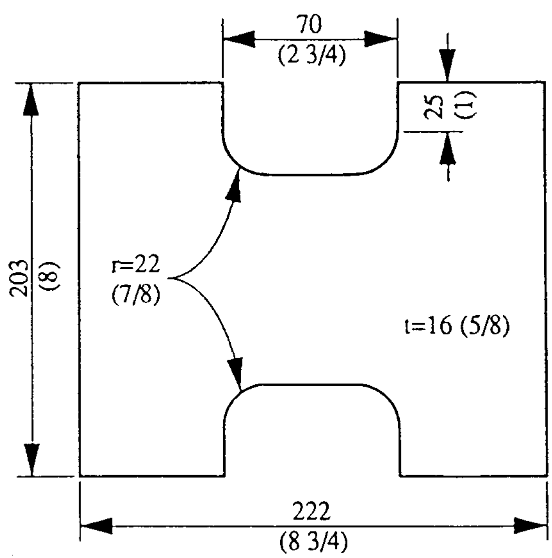


Fig. 5 Frame test.



(a) Connection Detail



(b) Notched Loose Plate

Fig. 6 Vertical joint connection

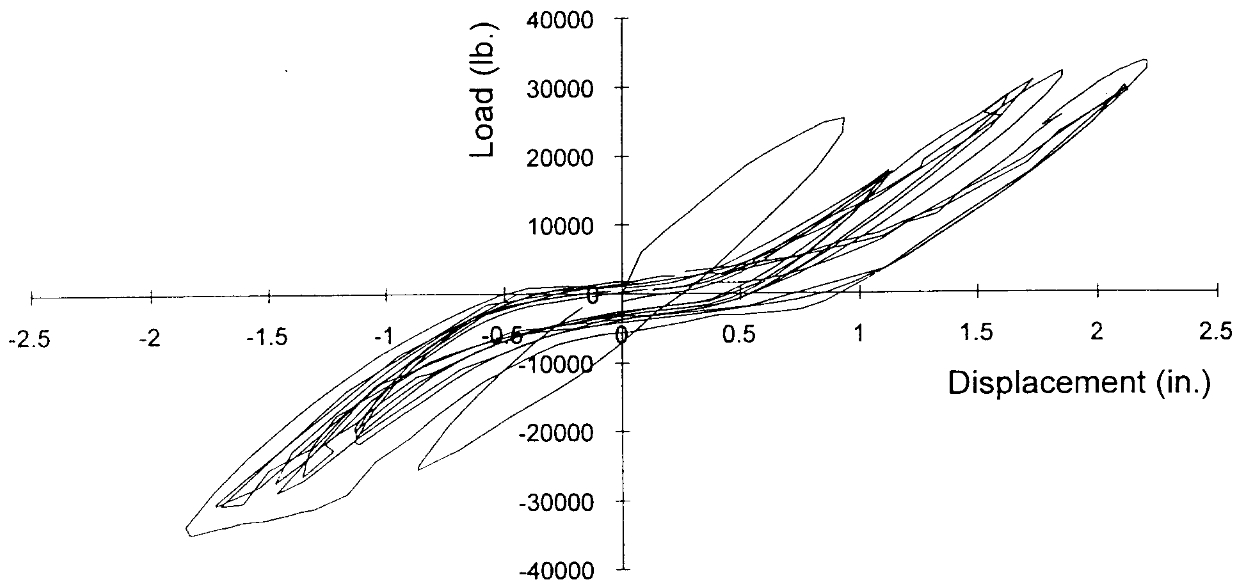


Fig. 7 Hysteresis curves for beam-column joint.

Acknowledgments

This research was carried out as a part of the U.S.-Japan PRESSS Program, Nigel Priestley, Coordinator. Financial support of the National Science Foundation (NSF), Mahendra P. Singh and Shih-Chi Liu, Program Directors, Grant BCS 91-23015, Precast/Prestressed Concrete Institute (PCI), Paul Johal, Research Director, Precast/Prestressed Concrete Manufacturers Association of California, Inc. (PC-MAC), Precast Concrete Association of Nebraska (PCAN), and Center for Infrastructure Research of the University of Nebraska - Lincoln (CIR-UNL) is gratefully acknowledged. The Industry Advisory Committee guiding this project, especially Jagdish Nijhawan, Vilas Mujumdar and Simon Harton, has provided valuable input. Interaction with other researchers within the PRESSS program, especially Cathy French, Mike Kreger, John Stanton, and Susie Nakaki has been very valuable to this study. The findings in this paper are those of the authors and do not necessarily represent the view of the NSF, PCI, PCMAC, PCAN, or PRESSS.

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