Seismic Behavior of a Six-Story Precast Concrete Office Building

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The work described in this paper was undertaken to demonstrate that precast concrete systems can be practically designed to resist earthquakes, to identify areas where code provisions for seismic design need to be re-evaluated to specifically address the intrinsic characteristics of precast concrete, and to recommend improvements to these areas. The research focuses on a six-story precast concrete office building with a size and layout common in the United States. The system consists of an interior gravity load-resisting frame and a “dual system” for lateral load resistance consisting of interior shear walls and exterior spandrel frames. A testing program comprising panel-to-panel and beam-to-column connection tests was conducted, and an inelastic dynamic analysis of the lateral load-resisting system was performed. The study shows that the seismic performance of precast systems in resisting seismic loads can be as good as or better than that of cast-in-place systems.

According to the Uniform Building Code (UBC), provisions for reinforced concrete structures resisting earthquake forces, the design of precast concrete structures in seismic zones must result in buildings that emulate monolithic cast-in-place (CIP) concrete structures. This is an outcome of an incomplete understanding of the behavior of precast concrete structures under seismic loading. As a result, the market share of structural precast concrete in regions of high seismicity is considerably
smaller than that of other building materials.

It is the thrust of the U.S.-Japan coordinated Precast Seismic Structural Systems (PRESSS) research program to investigate the behavior of precast concrete structures subjected to seismic loading. The objective of the program is to develop appropriate structural systems, improve analysis tools, and formulate design recommendations. The PRESSS program also aims to address the unique features of precast concrete structures and to exploit those properties that enhance seismic resistance.

This paper demonstrates how the framework of existing code provisions, which are intended for monolithic reinforced concrete structures, can be adapted to design fully precast concrete structures in areas of moderate seismic risk.

**STRUCTURAL SYSTEM**

The selected building system in this investigation is a highly efficient system that features a very shallow structural floor depth, interior columns at a spacing that allows for space layout flexibility, and spandrel frames that serve both architectural and structural functions. The building configuration represents a commercial building layout commonly used in the United States.

The system is designed as a concrete dual system. In the 1994 UBC, a dual system is defined as “a structural system in which the gravity loads are supported by a space frame, and resistance to the total base shear is provided by moment-resisting frames and shear walls.” The moment frame is required to resist at least 25 percent of the total lateral load, and the base shear is distributed among the moment frames and shear walls in proportion to their relative stiffness.

The structure is a six-story precast office building with a typical floor-to-floor height of 13 ft (4.0 m). Plan dimensions are 102 x 224 ft (31.1 x 68.3 m), divided into twenty 32 x 34 ft (9.8 x 10.4 m) bays, as shown in Fig. 1.

The gravity load-resisting system consists of 4 ft (1.2 m) wide and 8 in. (203 mm) deep hollow-core slabs, 8 ft (2.4 m) wide and 16 in. (406 mm) deep prestressed beams (see Fig. 2), and 20 x 20 in. (508 x 508 mm) columns. Hollow-core slabs are supported on beams which, in turn, rest on the columns. The beams are continuous while the columns are discontinuous through the beam-to-column joint. Column reinforcement is mechanically spliced at the job site to achieve continuity. The beam cross section was developed through a
Fig. 2. Gravity load-resisting system showing elevation and typical cross sections.
study sponsored by the Precast/Pre-stressed Concrete Institute (PCI).

The lateral load-resisting system comprises exterior spandrel frames (see Figs. 3 and 4) and interior shear walls around the stairwells (see Fig. 5).

The perimeter spandrel frames are built using double-cruciform (H-shaped) panels (see Fig. 3). The shaded areas of Fig. 3 show that only five sets of forms are needed to fabricate the spandrel frames for the entire building. This configuration was chosen because it allows for large window openings. A three-dimensional view of a typical spandrel frame is presented in Fig. 4.

The structural components are connected at story midheights and at midspans where the bending moments resulting from seismic forces are small (idealized inflection points). Floor slabs are field-connected to each other and to the top of the spandrel beams to create a continuous diaphragm.

The walls around stairwells and elevator shafts are 10 in. (254 mm) thick and three stories high (see Fig. 5). Walls wider than 10 ft (3.0 m) consist of two pieces that are erected side-by-side and are connected along the vertical joint using shear-resisting connections.

In addition to preserving the benefits and quality of precast construction, the building system offers important advantages including reduction of total building height and elimination of column corbels. Perhaps, the most
important feature of the selected system is the use of shear walls in combination with spandrel frames as a lateral load-resisting system. Past experience and research have indicated that buildings with stiff shear walls performed well during severe earthquakes.\textsuperscript{4,5}

**SCOPE OF WORK**

The scope of this study is limited to establishing design recommendations for precast systems subjected to moderate seismicity. This goal is accomplished through two stages of structural analysis and a connection testing program. The main issues investigated are the energy dissipating and ductility factor $R_w$ for precast systems, adequacy of spiral reinforcement in the compression zones, toughness of gravity beam-to-column connections, and development of representative connection models for inelastic dynamic analysis.

This study was performed in three steps. First, a preliminary structural design based on the results of the UBC equivalent static force method was conducted. Second, the areas that require further study were identified. Finally, a test program and inelastic dynamic analysis were carried out to verify the assumptions made in the preliminary design stage.
STRUCTURAL ANALYSIS AND DESIGN

The building is analyzed using a commercially available computer program for planar structures. The UBC provisions for equivalent static force method are used to compute the lateral loads. The objective of this design is two-fold: to size members and connection components for testing and dynamic analysis, and to identify areas where further research is needed.

In the analysis, interior frames (hollow-core slabs, beams, and columns) are assumed to support gravity loads only, and their lateral load resistance is ignored. The floors and roof (hollow-core slabs and beams) are assumed to transfer seismic inertial forces through diaphragm action to the lateral load-resisting system. The diaphragm is assumed rigid as defined by Section 2312(e) C6 of the UBC. For this type of building system in Seismic Zone 2B, with a total height not exceeding 160 ft (48.8 m), the equivalent static force method is applicable.

Loads

Both gravity and lateral loads are established in accordance with the UBC recommendations. Gravity loads include 25 psf (1.2 kPa) of partitions, ceiling, and mechanical utilities; 50 psf (2.4 kPa) of live load; and the self-weight of structural members.

Base shear is determined using Eqs. (34-1), (34-2), and (34-3) of the UBC, which, for easy reference, are given below as Eqs. (1), (2), and (3), respectively:

\[
V = \frac{ZIC}{R_w} W \quad \text{(1)}
\]

\[
C = \frac{1.25S}{T^{0.3}} \quad \text{(2)}
\]

\[
T = C_t(h_n)^{3/4} \quad \text{(3)}
\]

where

- \( V \) = total design lateral force or base shear
- \( Z \) = seismic zone factor
- \( I \) = importance factor
- \( C \) = numerical coefficient with a maximum value of 2.75
- \( W \) = total seismic dead load of the building

\( R_w \) = numerical coefficient which depends on the type of structural system and lateral load-resisting system

\( S \) = site coefficient for soil characteristics

\( T \) = fundamental period of vibration of the structure in the direction under consideration (seconds)

\( C_t \) = 0.02

\( h_n \) = height above the base to level of the building

The parameters \( Z \) and \( I \) are assumed equal to 0.2 and 1.0, respectively. Because the site conditions of the building are not known, a conservative value of 1.5 is assumed for the site coefficient \( S \). The total weight of the building is computed as 19,500 kips (86.7 MN).

Because precast concrete structural systems are not specifically addressed in the UBC, the selection of an appropriate \( R_w \) factor deserves special attention. One of the primary functions of the \( R_w \) factor in Eq. (1) is to reflect the ability of the building system to dissipate energy imparted by earthquakes. Buildings with a lower ability to dissipate energy are designed with lower \( R_w \) values.

Assuming that the spandrel frames are designed as intermediate moment-resisting space frames (IMRSF), the closest lateral load-resisting system addressed in the UBC is that of the monolithic reinforced concrete dual system comprising shear walls and IMRSF, for which \( R_w = 9 \). The ability of this precast concrete building system to dissipate seismic energy is expected to be slightly less than that of similar monolithic concrete structures. Precast concrete joints are somewhat more flexible and deformable than monolithic concrete joints. Thus, the value of \( R_w \) is assumed equal to 8.

Given the values of the parameters above, the base shear from Eq. (1) is

\[ V = 1344 \text{ kips} (6000 \text{ kN}) \].

This base shear is distributed in a triangular pattern over the height of the building in accordance with Eqs. (34-6), (34-7), and (34-8) of the UBC (see Table 1). The story force is the distributed force at each floor level, while the story shear is the accumulated shear force at each level.

<table>
<thead>
<tr>
<th>Floor</th>
<th>Roof</th>
<th>6</th>
<th>5</th>
<th>4</th>
<th>3</th>
<th>2</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lateral force (kips)</td>
<td>384</td>
<td>320</td>
<td>256</td>
<td>192</td>
<td>128</td>
<td>64</td>
<td>0</td>
</tr>
<tr>
<td>Story shear (kips)</td>
<td>—</td>
<td>384</td>
<td>704</td>
<td>960</td>
<td>1152</td>
<td>1280</td>
<td>1344</td>
</tr>
</tbody>
</table>

Note: 1 kip = 4.448 kN.

Mathematical Model of the Lateral Load-Resisting System

The lateral load-resisting system is idealized as a two-dimensional frame (see Fig. 6). This procedure is commonly used by designers for symmetric building systems. The building is analyzed for the combination of gravity loads and seismic forces in the critical direction (north-south).

The analysis takes advantage of symmetry and includes only one-half of the structure. The two 20 ft (6.1 m) walls of the stairwell are "lumped" into one wall with twice the thickness. The contribution of the elevator shaft walls is neglected because these walls are narrow and their associated stiffness is small compared with that of the stairwells.

The spandrel frames are modeled using beam-to-column elements while the shear walls are modeled using four-node hybrid plate elements. The gross cross section properties are used throughout. Floor diaphragms are assumed rigid and are modeled using rigid link elements. The stiffness of rigid zones at the intersection of beams and columns of the spandrel frames is incorporated in the analysis.

Spring supports are placed under the exterior beam cantilevers to simulate the stiffness of the spandrel beams from the orthogonal direction. The stiffness of the springs is set equal to the elastic stiffness of the spandrel beams. Base supports are assumed fixed for the spandrel columns and shear walls.

The torsional effect caused by accidental eccentricity of load as specified...
by the UBC is neglected in the analysis because it is estimated to be very small in the direction being analyzed, and the corresponding torsional shear forces are shared by the orthogonal (east-west) lateral load-resistant system.

**Results of Analysis**

Results of the analysis show that the spandrel frames carry about 43 percent of the total base shear (see Table 2). The load combinations specified in Sections 1909.2 and 1921.2 of the UBC are applied, and the maximum member forces are selected for design. These forces (factored) are summarized in Table 3.

**Structural Design**

The following references were generally consulted in the design process: the ACI 318-95 Code for member design, and the PCI Design Handbook and the Manual of Steel Construction for connection design.

Material properties assumed in the design are: compressive strength \( f'_c = 5000 \text{ psi} \) (34.5 MPa) for concrete; yield stress \( f_y = 60 \text{ ksi} \) (414 MPa) for reinforcing bars; yield stress \( f_y = 36 \text{ ksi} \) (248 MPa) for connection elements; and ultimate strength \( = 270 \text{ ksi} \) (1860 MPa) for \( 7/8 \) in. (12.7 mm) diameter seven-wire low-relaxation strands.

**Interior frame elements** — The interior frame elements are designed for gravity loads. The shallow beams are designed using a spreadsheet computer program, and the columns are designed with the aid of a commercial computer program. Reinforcement of hollow-core slabs is obtained from standard design tables. The strong-column, weak-beam concept is not applied to the interior frame because it is designed to resist gravity loads only.

**Interior frame reinforcement** — It is found that four 1.2 in. (30 mm) diameter (9) bars are needed to reinforce the critical interior columns. Rectangular closed ties are used as transverse reinforcement for shear and concrete confinement. As required by Section 21.4.4.4 of the ACI 318-95 Code, confinement reinforcement is needed over a length \( L_o = 24 \text{ in.} \) (610 mm) to improve the ductility of the column. In these regions, #3 closed ties spaced at 2 in. (51 mm) on center are provided to meet the requirement.

Because the beam spans over maximum negative and positive moment regions, both top and bottom prestressed reinforcement is provided

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**Fig. 6. Two-dimensional computer model.**

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**Table 2. Shear force and distribution.**

<table>
<thead>
<tr>
<th>Base shear</th>
<th>Percentage of total shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spandrel frames</td>
<td>584 kips</td>
</tr>
<tr>
<td>Shear walls</td>
<td>43 percent</td>
</tr>
</tbody>
</table>

Note: 1 kip = 4.448 kN.

**Table 3. Member design forces.**

<table>
<thead>
<tr>
<th>Force</th>
<th>Spandrel beam</th>
<th>Spandrel column</th>
<th>Shear wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear (kips)</td>
<td>123*</td>
<td>118*</td>
<td>269†</td>
</tr>
<tr>
<td>Bending moment (kip-ft)</td>
<td>586*</td>
<td>539‡</td>
<td>1200§</td>
</tr>
<tr>
<td>Axial load (kips)</td>
<td>211†</td>
<td>498§</td>
<td>266§</td>
</tr>
</tbody>
</table>

Note: 1 kip = 4.448 kN; 1 kip-ft = 1.356 kN-m.

* Due to load combination \( 0.75(1.4D + 1.7L + 1.87E) \)
† Due to load combination \( 0.9D + 1.43E \)
‡ Due to load combination \( 0.9D + 1.43E \)
§ Due to load combination \( 1.4D + 1.7L \).
over the entire length of the beam sections to satisfy serviceability and ultimate strength requirements. For ease of fabrication, the top and bottom reinforcement are prestressed to the same stress level, and some of the strands are debonded at strategic locations to meet the allowable stress limits at pre-stressing release and service conditions. Reinforcement details of the columns and beams are shown in Fig. 7.

**Interior frame elements connections** — The interior columns are connected at each floor level by splicing the main column reinforcing bars using grout-filled splice sleeves (see Fig. 8). High strength, non-shrink grout is used to fill the grout joints and the splice sleeves.

The beam-to-beam connection utilized three steel plates and two bolts (see Fig. 9). Two of these plates (labeled as Plate B) are embedded in one
Fig. 8. An interior column-to-column connection detail.

Fig. 9. An interior beam-to-beam connection detail.
Spandrel frames reinforcement —
The spandrel frames are designed to resist a part of the gravity load and a part of the lateral load. The frame is assumed to behave as an intermediate moment-resisting space frame (IMRSF) and is designed to meet the requirements set forth in the UBC for these types of members. The spandrel beams and columns are proportioned to achieve a strong-column, weak-beam behavior. Energy dissipation of the frame is assumed to occur through the hysteretic behavior of the monolithic spandrel beam-to-column joint.

Dimensions and typical reinforcement details of the spandrel frame are shown in Fig. 10. A total of eight 1.2 in. (30 mm) diameter (#9) bars are used in each of the columns. Closed ties made of 0.4 in. (10 mm) diameter (#3) bars are provided as transverse reinforcement. Four 1.2 in. (30 mm) (#9) bars at the top and three 1.0 in. (25 mm) (#8) bars in the bottom of the beam are used to resist flexure. The beam reinforcement is continuous through the monolithic beam-to-column joint and hooked from both ends for bar development.

Spandrel frames connections —
The spandrel frame units are field-connected to each other and to the foundations by means of commonly used connection details. In this paper, the spandrel beam-to-beam connections will be referred to as vertical joint connections, and the column-to-column connections of these units will be referred to as horizontal joint connections. The basic components of a vertical joint connection are two embedded angles, four loose steel plates, and deformed bar anchors (see Fig. 11). These connections are designed to resist moment in the beam.
because two connections are needed for shear resistance.

During erection, two adjacent spandrel beams are aligned and connected together by welding the loose plates. The joint is grouted after all components are welded. This connection detail is relatively simple to construct, and it provides ample construction tolerance.

Horizontal joint connections of the spandrel frame units are accomplished by connecting the main reinforcement of the columns using grout-filled splice sleeves embedded in the bottom of the upper spandrel frame units (see Fig. 12). Columns of the spandrel frames are connected to the foundation using the same details as that of the horizontal joint connection (see Fig. 13).

**Shear wall panels reinforcement** — The shear walls, comprising the staircase walls in the north-south direction, are assumed to behave like cantilevers fixed at the foundation. They are subjected to the axial compression load due to self weight of the walls, a small share of the gravity floor and roof loads, and shear and flexural moment due to seismic loads. The required reinforcement for the combined axial and flexural loads for each wall is five 1.4 in. (36 mm) diameter (#11) bars on each side (see Fig. 14). These bars are continuous over the full height of the panels and are spliced at the horizontal joints between panels using grout-filled splice sleeves.
The analysis showed that the extreme fiber compression stress in the shear wall due to axial compression and moment exceeds the limits recommended by ACI 318-95 Code, Chapter 21 (Seismic Design) for walls with no boundary elements. To avoid the use of boundary elements, spiral reinforcement is used to strengthen the jambs of the panels. Spirals made of 7 mm diameter (No. 1 gauge) wire with a 8 in. (203 mm) diameter and 1 1/2 in. (38 mm) pitch are required (see Fig. 14).

Each spiral is placed concentrically around a vertical reinforcing bar, and they are overlapped. Spiral reinforcement is only required for the first and second stories because the maximum compressive stress in the upper stories is below the recommended limits. Two layers of welded wire fabric are provided to meet the requirements of the ACI 318-95 Code for

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Fig. 13. Spandrel frame-to-foundation connection detail.

Fig. 14. Shear wall panel reinforcement details.
Shear walls subjected to earthquake loading.

**Shear wall panels horizontal joints** — Continuity of the main reinforcement of the shear wall panels is achieved through grout-filled splice sleeves (see Fig. 15). A similar detail is used for connecting the shear walls to the foundation. Shear strength of horizontal joints among shear wall panels and between the wall and the foundation is accomplished by shear friction. The shear friction strength of these joints is, therefore, computed as the product of the compressive force, due to axial and flexural loads, and the coefficient of friction $\mu = 0.6$, as recommended by the ACI 318-95 Code.6

**Shear wall panels vertical joints** — In analyzing the building system under the above-mentioned seismic loads, the shear wall connection along vertical joints is assumed stiff enough for strong coupling of the panels. Thus, the 20 ft (6.1 m) wide shear wall is assumed to behave like a monolithic
shear wall for simplicity of calculations. To validate this assumption, a series of shear-resisting connections are provided along the vertical joints between the panels (see Fig. 5). The primary element of the connection is a notched plate welded to embedded plates in each of the adjacent panels (see Fig. 16). 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.

Other connection details — Additional connection details required for a complete design of this building, including slab-to-slab, slab-to-spandrel beam, interior beam-to-spandrel beam, and slab-to-shear wall connections are available in Ref. 10.

TESTING PROGRAM

Described below are the panel-to-panel and beam-to-column connection tests.

Panel-to-Panel Connection Tests

A total of thirteen 1/4-scale connection details for precast shear walls, six for vertical joints and seven for horizontal joints, were tested under simulated seismic loads at the National Institute of Standards and Technology. The primary purpose of the testing program is to help understand and assess the behavior of such connections under reversed cyclic loading. One of the important benefits expected from this program is the measurement of strength and stiffness of the connections, in addition to the ability of the connection to dissipate seismic energy. This information will be beneficial in evaluating the adequacy of precast concrete structural design and increase the level of confidence of the designers. Detailed descriptions of the vertical joint connection testing are presented in Refs. 11 and 12.

Beam-to-Column Connection Tests

Two beam-to-column joint tests were included in the research program and were conducted at the University of Nebraska. The objective of these tests was to evaluate the toughness of the joint, i.e., the ability of the joint to support gravity loads under large lateral displacement due to earthquakes. Only highlights of the tests are presented in this paper. Details of the tests can be found in Refs. 13 and 16.

A schematic view of a beam-to-column connection subassembly test is shown in Fig. 17. The test specimen was a typical full-scale interior beam-to-column connection. The column height was chosen to represent the distance between the midheights of two consecutive stories (points of inflection). Electrical resistance strain gauges and Linear Variant Displacement Transformers (LVDT) were used to measure strain in the reinforcing.

Fig. 17. Beam-to-column connection subassembly tests, specimen and boundary conditions. Note: 1 ft = 0.305 m; 1 in. = 25.4 mm.
bars and displacement of the specimen, respectively. Quasi-static cyclic lateral load was applied to the specimen at the bottom of the lower column to simulate earthquake loading. The loading pattern recommended by the PRESSS coordinator is used.

Hysteresis curves for Specimens BC-1 and BC-2 are plotted in Figs. 18 and 19, respectively. The story drift in these figures represents the net displacement of the joint divided by a story height equal to 13 ft (3.96 m). The net displacement was obtained by subtracting the lateral displacement at the beam level from the displacement at the loading point.

As can be seen from Fig. 18, the joint dissipated very little energy and exhibited a large reduction in stiffness after the first cycle. Hysteresis curves of the joint are pinched. The pinched region of the hysteresis loops was believed to be the outcome of opening and closing of the interface between the beam and the column, as well as the slippage of column longitudinal reinforcing bars. During the test, slippage of the column reinforcing bars was observed after the mortar joint crushed, chipped, and exposed the bars.

Specimen BC-2, which differs from BC-1 by having an axial compression force of 100 kips (445 kN) applied using a 1 in. (25 mm) diameter prestressing bar, showed significant improvement over Specimen BC-1. As shown in the hysteretic plot in Fig. 19, the joint was able to sustain higher lateral load and displacement and exhibited a smaller reduction in stiffness compared with Specimen BC-1; moreover, the joint did not indicate any strength degradation at a ductility displacement of 2. More importantly, the pinching behavior was completely eliminated in Specimen BC-2 due to the presence of the axial load.

These tests showed that the joint provides adequate toughness to support the gravity loads while the lateral load is resisted by the lateral load-resisting system. This type of connection is suitable for use in the gravity load-resisting system located in regions of moderate seismicity.

**INELASTIC DYNAMIC ANALYSIS**

A parametric inelastic dynamic analysis using the DRAIN-2DX computer program was conducted to study the seismic behavior of the six-story building and to investigate the effects of variations of connection characteristics on its behavior. The analysis was performed using a time-history procedure. Four representative earthquake records were selected and used as dynamic loading.

The study covered finite element models of a monolithic cast-in-place system and four precast systems. The monolithic and precast models contained elements that respond inelastically beyond their proportionality limits.

The characteristics of the connection elements used for the various precast models were varied to monitor the corresponding changes in the overall behavior of the models. Each connection element was described by
its initial stiffness, ultimate capacity, and hysteretic behavior. The properties of the connection elements for Precast System 1 were estimated based on published test results. This system was used as the baseline for comparison.

Precast System 2 was similar to Precast System 1, but the strength of the panel horizontal connection element was reduced by 50 percent. Strength and stiffness of the frame joints in Precast System 3 were increased by about 400 percent, while the strength and stiffness of the shear wall connections were reduced to 50 percent of Precast System 1.

Precast System 4 was identical to System 1 with the exception that two elements with different behavioral models were used for each joint to more accurately simulate the joint behavior assumed in Precast System 4. Only the properties of the joints in Precast System 1 are presented in Table 4. The properties of joints in the other precast systems are presented in Refs. 13, 15 and 16.

Assumptions

Stiffness of the element in rigid end zones of the frames and second-order (P-Δ) effects were taken into account in the analysis. Gravity loads were applied to the model as nodal forces. The inertial mass of the structure was lumped at the nodes, and the total seismic weight at each floor was distributed equally to all nodes in that floor. A 5 percent viscous damping ratio was assumed for beam-to-column elements to account for miscellaneous energy losses. Connections and shear wall panels were assumed to have

Table 4. Properties of joints in Precast System 1.

<table>
<thead>
<tr>
<th>Joint Type</th>
<th>Direction</th>
<th>Initial Stiffness (kip per kip-ft)</th>
<th>Yield Force per Moment (kip kip-ft)</th>
<th>Elasitic Code†</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame joints</td>
<td>X</td>
<td>1110</td>
<td>585/585</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Y</td>
<td>29,000</td>
<td>620(T)/3710(C)†</td>
<td>2</td>
</tr>
<tr>
<td>Column-to-column and</td>
<td>Y</td>
<td>67,300</td>
<td>10,550/10,550</td>
<td>2</td>
</tr>
<tr>
<td>panel-to-foundation</td>
<td>Y</td>
<td>190</td>
<td>20/20</td>
<td>0</td>
</tr>
<tr>
<td>Vertical joint</td>
<td>Y</td>
<td>29,000</td>
<td>250(T)/250(C)‡</td>
<td>2</td>
</tr>
<tr>
<td>Horizontal joint</td>
<td>Y</td>
<td>3370</td>
<td>150/150</td>
<td>2</td>
</tr>
<tr>
<td>panel-to-foundation</td>
<td>X</td>
<td>29,000</td>
<td>210(T)/1540(C)‡</td>
<td>1</td>
</tr>
</tbody>
</table>

Note: 1 kip per in. = 175 kN/m; 1 kip = 4.448 kN; 1 kip-ft = 1.36 kN-m; 1 kip per radian = 4.448 kN rad.
* kips per in. for force and kips per radian for moment.
† 0 = unload elastically; 1 = unload elastically; 2 = unload elastically with gap.
‡ (T) = tension; (C) = compression.
negligible viscous damping. Effects of vertical acceleration were not accounted for in the analysis, and the capacity of the foundation was assumed adequate to support the loads.

**Column-to-column and column-to-foundation joints** — All column-to-column and column-to-foundation joints were assumed to have the same properties, and each joint consisted of three elements: X-translational, Y-translational, and a rotational element (Type 4 elements), see Fig 20.

The initial stiffness of the X translational element was extrapolated from a test report presented in Ref. 18. Its ultimate capacity was determined using the equation proposed by Foerster. The assumed hysteretic behavior of column-to-column and column-to-foundation joints is shown in Fig. 20. These elements were assumed to un-
load inelastically with a gap and dissipate some energy in the process.

**Panel-to-panel and panel-to-foundation joints** — The panel-to-panel joints were assumed to be the same as the panel-to-foundation joints. Two types of connection elements were used to model each connection along the horizontal and vertical joints between the wall panels: X-translational and Y-translational elements. The initial stiffness and strength of the Y-translational element in the vertical joint were obtained from the test results reported in Ref. 11. The initial stiffness of the X-translational element was assumed to be very large, and the yield force was estimated based on the behavior of the test specimens of Ref. 11.

The initial stiffness of the panel connection elements in the horizontal joint was estimated according to the results presented in Ref. 18. The initial stiffness was assumed to be directly proportional to the concrete area. Yield strength of the joint was computed using the equation proposed by Foerster. Yield force in tension and in compression of the element was obtained by treating the wall panel as a compression member.

### Table 5. Fundamental periods.

<table>
<thead>
<tr>
<th>System</th>
<th>Eq. (3)</th>
<th>Monolithic</th>
<th>Precast 1</th>
<th>Precast 2</th>
<th>Precast 3</th>
<th>Precast 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Period (seconds)</td>
<td>0.4598</td>
<td>0.4682</td>
<td>0.7503</td>
<td>0.7503</td>
<td>0.6756</td>
<td>0.7387</td>
</tr>
</tbody>
</table>

### Table 6. Base shear for all systems.

<table>
<thead>
<tr>
<th>System</th>
<th>Eq. (3)</th>
<th>Monolithic</th>
<th>Precast 1</th>
<th>Precast 2</th>
<th>Precast 3</th>
<th>Precast 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total (kip)</td>
<td>560*</td>
<td>2260†</td>
<td>1820‡</td>
<td>1300‡</td>
<td>1400‡</td>
<td>2190‡</td>
</tr>
<tr>
<td>Percent resisted by frames</td>
<td>55</td>
<td>58</td>
<td>16</td>
<td>42</td>
<td>46</td>
<td>11</td>
</tr>
<tr>
<td>Percent resisted by panels</td>
<td>45</td>
<td>42</td>
<td>84</td>
<td>58</td>
<td>54</td>
<td>89</td>
</tr>
</tbody>
</table>

*Note: 1 kip = 4.448 kN.
† This value is calculated assuming $R_e = 8$ and $S = 1.0$.
‡ Subjected to El Centro 1940 NS record.

### Time-History Analysis

**Earthquake records** — The ground acceleration records used for this study were 1940 El Centro NS, 1952 Taft S69E, 1966 Parkfield N65E, and 1986 San Salvador (CIG Station). These records were scaled to 0.2g peak ground acceleration to represent Zone 2B events. The time increment for all records was 0.02 seconds.

**Results**

**Fundamental periods** — Fundamental periods of all models were computed prior to time-history analysis. In general, the periods of precast models were longer than that computed from Eq. (3) and that of the Monolithic System model (see Table 5). A longer period indicates a reduction in stiffness due to the presence of the joints.

![Fig. 21. Base shear response to El Centro earthquake. Note: 1 kip = 4.45 kN.](image-url)
The values presented in Table 6 are those of only half of the system. The mathematical model represents half of the structure. The noticeable difference between the base shear computed by Eq. (1) and that obtained from the monolithic model is due to the reduction factor $R_w$ in Eq. (1).

The ratio between the two base shear values, however, is 1:4, which is half the $1:R_w$ ratio. This difference indicates that the El Centro acceleration record is not the most severe earthquake for this building. It does not excite the model to generate maximum base shear. In addition, the base shear values for the precast models are considerably lower than that of the monolithic system in all cases. This is because the stiffness of the precast models is less than that of the monolithic system. A system with greater stiffness attracts more inertial force.

Varying the properties of the connection elements results in the following behavioral differences:

1. Reduction of the strength of the panel-to-panel horizontal joint decreases the total base shear and increases the percentage of shear resisted by the frames (see Precast 2).

2. Appropriate adjustment in the properties of the panel-to-panel and column-to-column joints can redistribute the base shear between the frames and the shear walls (see Precast 3).

3. Variation of the behavioral model of the panel-to-panel horizontal joints can significantly affect the response of the system (see Precast 4).

In general, the magnitude of the base shear is significantly affected by the strength and stiffness of the panel-to-panel horizontal joint connections.

Time-history plots of the base shear due to the El Centro acceleration record for the Monolithic System and Precast Systems 1 and 4 are shown in Fig. 21. The Monolithic System experienced a peak base shear at about 2 seconds into the record followed by smaller shear. On the other hand, the precast systems experienced a lower base shear, but at a relatively large magnitude, for three consecutive cycles. This might cause greater damage in the precast systems than the monolithic systems.

**Roof displacement and drift ratio** — Table 7 shows the roof displacement and the maximum inter-story drift ratio of all systems. The roof displacement of the Monolithic System is about 2.8 times greater than that calculated by the UBC equation, and the maximum drift ratio is approximately 2.6 times greater.

Due to the reduction in stiffness, the roof displacement and story drift of...
precast systems are greater than those of the Monolithic System; however, the maximum drift ratio is well below the recommended limits for precast systems (1.5 percent for shear walls and 2 percent for moment frames). The roof displacement vs. time of the Monolithic System and Precast Systems 1 and 4 is depicted in Fig. 22.

**System response** — Roof displacement vs. base shear relationships of the Monolithic, Precast 1, and Precast 4 models when subjected to the El Centro earthquake record are shown in Figs. 23, 24 and 25, respectively. Between the two systems, the Monolithic System exhibits greater strength and stiffness; however, the energy dissipation of the Monolithic System is less than that of the Precast System, as can be seen from the total area within the hysteresis loops.

Because a greater amount of the base shear is resisted by the walls, the behavior of Precast System 1 is identical to the behavior of the X-translational horizontal panel joint element. The hysteresis loops are pinched during inelastic loading.

The results of the analysis of the monolithic and precast systems show that the base shear of a monolithic system is higher and the maximum inter-story drift ratio is lower than that of precast systems. Changing the properties of the panel horizontal joints can affect the behavior of the system.

Field joints in precast concrete buildings reduce the buildings’ overall stiffness compared with that of cast-in-place monolithic buildings. Thus, precast building systems will experience greater inter-story drift and lower base shear than do monolithic buildings. The increased inter-story drift, however, can still be within the limits acceptable according to the building codes and practical recommendations, and the lower base shear allows these buildings to be designed for and, consequently, behave satisfactorily in earthquakes.

This study also showed that the behavior of a precast horizontal panel-to-panel joint connection is influenced by the behavior of the vertical joint. In one of the analyses, it was assumed that the vertical joint connection was made of elements that are extremely stiff and strong. Under earthquake loading, these elements behave like rigid links connecting two adjacent panels together. These types of elements have resulted in a very large force demand in the horizontal joint. Because of the inter-relation between horizontal and vertical joints, strong vertical joint connections can increase the force demand of the horizontal joint significantly.

Further details of the dynamic analysis can be found in Refs. 13, 15 and 16.
CONCLUSIONS

Based on the research summarized above, the following conclusions are drawn:

1. Practical and economical precast concrete systems can be designed for areas of low to moderate seismicity using existing code provisions and exercising sound engineering judgment.

2. The equivalent static force method outlined in the UBC can be used to determine the earthquake induced lateral forces if an appropriate $R_w$ value is used.

3. Tests of two full-scale interior beam-to-column joint specimens for cyclic loading showed that the joint is capable of sustaining at least 2 percent drift without a significant decrease in strength and stiffness. The performance, in terms of strength, stiffness, and energy dissipating characteristics, is improved with the existence of axial load.

4. Inelastic dynamic analysis, time-history analysis, showed that the base shear of a precast system can be 50 percent lower than that of a monolithic system; however, the inter-story drift ratio of a precast system can be ten times higher than the drift of a monolithic system.

5. Lower base shear and greater inter-story drift as indicated in Conclusion 4 are results of a considerable reduction in stiffness of the precast system due to the presence of the joints. However, the maximum drift due to the applied earthquake records is still well below the design limits recommended by PRESSS researchers of 1.5 percent for panel systems and 2 percent for frame systems.

RECOMMENDATIONS

The following recommendations are developed based on the results of this study:

1. Provisions for “Static Force Procedure” in the UBC that are specified for cast-in-place concrete systems are recommended, with minor modifications, for the seismic design of mid-rise [78 ft (23.8 m) or less] precast concrete systems subjected to moderate seismicity (Zone 2B or less) and without significant vertical and horizontal irregularities.

2. The factor $R_w = 8$ is recommended for mid-rise [78 ft (23.8 m) or less] precast dual systems subjected to moderate seismicity (Zone 2B or less), provided that the wall-to-wall connections are designed and detailed to have sufficient ductility and energy dissipation characteristics.

3. Interior beam-to-column connections utilizing mechanical connectors are recommended for use in a precast system subjected to moderate seismic risk.

4. A precast concrete “Dual System” can be used for mid-rise buildings located in Seismic Zone 2B or less where large openings are needed in the perimeter for architectural purposes. Solid precast shear walls around the elevator shafts and stairwells can be designed to resist a large portion of the seismic shear force.

5. Additional analytical and experimental studies are recommended for precast concrete systems similar to the one studied in this project. Inelastic dynamic analysis of different connection models under representative earthquake records should be performed in order to better understand the behavior of these structures and to optimize their design.

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REFERENCES