Experimental investigation of the cyclic response of reinforced precast concrete framed structures

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The use of precast concrete in framing systems is widespread in many countries, particularly for single-story or low-rise industrial buildings. Rapid and economical construction, high allowance for quality controls, and less labor required on-site have led the prefabrication of reinforced concrete elements to become an established technique in Italy in the past 50 years.

The majority of Italian and European industrial facilities consist of reinforced precast concrete frames comprising continuous monolithic columns and pin-ended beams characterized by high flexibility and low resistance of beam-to-column and panel-to-structure connections. Their seismic response greatly depends on the behavior of the connection system, and the key role played by proper design and detailing of the joints is well established in the literature.1–7 In the past decade, extensive research was undertaken to test traditional structural layouts and connections in quasi-static, pseudodynamic, and dynamic fashion.8–13

Despite significant progress in research, the majority of actual structures have shown inadequate seismic performance14–18 when connections were insufficiently detailed. Major earthquakes in Italy (May 20 and 29, 2012, Emilia seismic sequences)19 resulted in a similar scenario.20–23

In an experimental program examining the seismic response of reinforced precast concrete structures in Italy that were severely damaged by major earthquakes in May 2012, two ¾-scale two-bay, three-story precast concrete frames with and without cladding panels were subjected to a quasi-static cyclic displacement history.

The flexibility of the cantilevered static scheme forced large displacement demand and incompatible transfer of horizontal forces to occur in joints lacking shear and ductility capacity.

Displacement incompatibility between structural elements and cladding panels caused the premature failure of their connections.
To simulate the seismic vulnerabilities observed, the most critical features of current design practice in accordance with the traditional force-based approach prescribed in the national seismic code have been synthesized. Results from representative case studies are discussed as a background to the experimental program conducted. The building stock considered consists of 650 industrial facilities built between 1960 and 2010. The structural layouts, determined on the basis of field surveys of pre-code warehouses located in Tuscany, Emilia Romagna, and Piedmont have been redesigned in accordance with the current seismic code.

Current design practice

Because the Italian market in precast concrete buildings is strongly affected by speed of construction, manufacturers prefer to use dry connections able to transmit only horizontal shear. Therefore, second-order effects and the interstory drift limit, assumed as 1% in accordance with the current Italian code, govern design because stability against horizontal seismic loads is entirely transmitted to the columns of these flexible one- and multistory structures. As a result, precast concrete producers usually adopt large sections of up to $1000 \times 1000 \text{ mm} (39 \times 39 \text{ in.})$ in the case of three-story buildings. These large column sections result in a significant increase in strength, mostly due to the minimum longitudinal and transversal reinforcement requirements. In particular, because the minimum number of stirrups depends on the size of the column, extremely short spacing (50 to 70 mm [2 to 3 in.]) is generally needed. This strength increase implies a reduced effective displacement ductility or even an elastic design. If the connections are not designed according to the modified structural response, equivalent to a reduction of the behavior factor (Fig. 1), they may become the weak point of the system, potentially activating anticipated, not dissipative, collapse mechanisms. Thus overdesigned columns result in overdesigned foundations because the design demand depends on the flexural resistance assigned to the columns both in high and low ductility class or worse, on the actions determined by the elastic seismic demand.

Two further criticalities must be highlighted:

- There are uncertainties related to the real constraint at the base of the columns when precast concrete socket foundations are used, as in the majority of the cases no foundation-to-foundation structural links are provided to inhibit relative ground displacements. In addition, the use of low-strength mortar for the injection grout between the foundation and the column may cause the column to rotate into the pocket foundation up to large drift levels.

![Figure 1. Increase of column cross section and subsequent reduction of behavior factor to limit second-order effects. Note: $g$ = acceleration due to gravity; $q$ = behavior factor; $T$ = fundamental period; $\theta$ = interstory drift sensitivity coefficient or $P-\Delta$ ratio; $\mu$ = displacement ductility. 1 mm = 0.0394 in.](image)
the kinematics of this mechanism. Hence, the joints need to be designed and detailed to guarantee equilibrium under seismic action and relative displacement compatibility of the structural members, considering the concentration of strain and force demand at the connection level due to the greater flexibility of the connection compared with the connected precast concrete elements. This damage pattern and the related design aspects will be described in detail when discussing the experimental response of the two frames investigated.

Experimental program

In light of this scenario, two planar three-story frames were extracted from a reference three-dimensional (3-D) structure designed for medium to high seismicity (that is, peak ground accelerations equal 0.25g, where g is acceleration due to gravity) in accordance with the current Italian seismic code.\textsuperscript{24} Soil class C (180 m/sec < $V_s$ < 360 m/sec [590 ft/sec < $V_s$ < 1180 ft/sec], where $V_s$ is shear wave velocity) was assumed to perform the design by a series of response spectrum analyses on 3-D models, including vertical seismic component, second-order effects, and accidental eccentricity of the seismic masses.

Description of test specimens

As previously mentioned, poor connection detailing is most likely the reason for the inadequate response observed during past seismic events and is still the major aspect to address in the proper design of earthquake-resistant precast concrete structures. The use of traditional discontinuous dowel connections, assembled without cast-in-place concrete, may noticeably reduce the potential seismic performance of the structure. Cyclic pure shear tests on such solutions\textsuperscript{11} revealed different behavior in the push and pull directions and highlighted the significant shear strength decay induced by the crucial effect of the cyclic reversals. As detailed in the past,\textsuperscript{27,28} the cyclic response of these dry pinned joints shows a resistance more than halved if compared with that observed under monotonic loads. When accounting both for vertical components and dynamic effects, connections can be expected to behave even worse. Therefore, their behavior is still cause for concern and current knowledge in terms of analytical shear strength predictive expressions dates back to the mid-1980s.

As shown in some of the industrial buildings surveyed in the aftermath of the 2012 Emilia earthquakes, beam-column and beam-joist interaction may imply a sort of unexpected restraint at the connection level due to the large relative rotation experienced, resulting in additional actions generally neglected in design (Fig. 2). Figure 3 illustrates the kinematics of this mechanism. Hence, the joints need to be designed and detailed to guarantee equilibrium under seismic action and relative displacement compatibility of the structural members, considering the concentration of strain and force demand at the connection level due to the greater flexibility of the connection compared with the connected precast concrete elements. This damage pattern and the related design aspects will be described in detail when discussing the experimental response of the two frames investigated.

Typical damage pattern of a beam-column connection observed in the aftermath of May 2012 Emilia earthquakes.
loads used in design. Their computation was performed by assuming the slabs to be unidirectional because the most common flooring system in Italian building practice consists of prestressed double-tee joist or hollow-core slabs with or without cast-in-place concrete topping. These single-span systems are usually simply supported on inverted-tee framing beams. Continuity in terms of horizontal seismic load transfer is ensured by cast-in-place concrete topping or shear keys. Two-dimensional representations of two specimens were extracted from a reference 3-D structure; therefore, the slabs were not provided, while their weights were considered and reproduced in the tests.

The specimens were assembled by seating the beams on the 250 mm (10 in.) corbels or on top of the columns. In particular, a 15 mm (0.6 in.) thick rubber pad was provided in combination with two 16 mm diameter (0.63 in.) vertical steel threaded bars for each connection. Two 80 × 50 mm (3 × 2 in.) hollow steel profiles were embedded at both beam ends, permitting the steel dowels to be set in place. After their anchorage into the corbels or directly into

Table 1. Dead, live, and total distributed loads at each floor

<table>
<thead>
<tr>
<th>Floor</th>
<th>Floor height, mm</th>
<th>Beam height, mm</th>
<th>Beam weight, kN</th>
<th>Dead load, kN</th>
<th>Live load, kN</th>
<th>Total distributed, kN/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>7900</td>
<td>450</td>
<td>18.6</td>
<td>20.0</td>
<td>10.0</td>
<td>8.70</td>
</tr>
<tr>
<td>2</td>
<td>5500</td>
<td>450</td>
<td>18.6</td>
<td>27.0</td>
<td>20.0</td>
<td>13.6</td>
</tr>
<tr>
<td>1</td>
<td>3100</td>
<td>350</td>
<td>16.6</td>
<td>42.0</td>
<td>25.0</td>
<td>19.4</td>
</tr>
</tbody>
</table>

Note: 1 mm = 0.0394 in.; 1 m = 3.28 ft; 1 kN = 0.225 kip.
the tops of the columns, steel channels were grouted up to approximately the midheight of the beam to impose a shear failure mechanism on the steel bars. As is commonly done, a 15 mm (0.6 in.) gap was provided between the end of the beam and the internal side of the column. Figure 5 shows a schematic of the connection system used.

The 1.70 m (5.6 ft) high socket foundations were post-tensioned to the strong floor of the lab and overdesigned to minimize their interaction with the frame as the experimental program focused mainly on beam-column and panel-to-structure connections. In current building practice, the height of the socket foundations is commonly assumed to be greater than \( \frac{1}{10} \) of the total height of the structure, in accordance with basic considerations related to second-order effects. To prevent any local undesirable mechanism from taking place in the socket foundations, the height was chosen to be approximately \( \frac{1}{5} \) of the total height of the structure and the reinforcement was overdesigned to ensure an elastic response. For the connection between the column and the socket foundation, the column embedment depth into the foundation was selected to be approximately 1.40 m (4.6 ft) (approximately three times larger than column depth). The base of the column was set into the socket foundation by means of a centering pin grouted into the 300 mm (12 in.) deep plate at the base of the socket foundation. High-strength mortar was injected to fill the 100 to 50 mm (4 to 2 in.) gap between the column and the socket foundation at top and bottom, respectively.

The 200 mm (8 in.) thick lightweight reinforced precast concrete cladding panels were directly supported by the columns by means of top and bottom systems (Fig. 5) to inhibit panel overturning and drop, respectively. Their surface was 4.00 \( \times \) 2.50 m (13 \( \times \) 8.2 ft), and the load per unit area was scaled from the 38 kN/m\(^2\) (794 lb/ft\(^2\)) of the full-scale structure by means of polystyrene blocks.

### Materials and reinforcement layout

Concrete classes C45/55 and C40/50 were used for beams and columns, respectively. The compressive strength of concrete is denoted by concrete strength classes, which relate to the characteristic (5%) cylinder strength, or the cube strength, in accordance with Eurocode 2\(^{23}\) (Table 3.1 of Eurocode 2). Compressive cylinder tests were performed on 100 \( \times \) 200 mm (4 \( \times \) 8 in.) cylinders at 28 days, showing strengths higher than those specified by the supplier. The mean 28-day compressive strength for the cylinders from beams was 56.1 MPa (8100 psi), compared with a mean specified strength of 53 MPa (7900 psi). Similarly, the cylinders representing the column concrete were about 7% stronger than specified.

Longitudinal reinforcement consisting of twelve 20 mm diameter (no. 7) and sixteen 18 mm diameter (no. 6) bars, twelve 20 mm diameter and eight 18 mm diameter bars, and four 20 mm diameter and eight 18 mm diameter bars was provided in the columns of the first, second, and third floors, respectively. The transverse reinforcement was composed of 6 mm diameter (no. 2) stirrups spaced at 200 mm (4 in.). Mild steel with a yield strength of 430 MPa [62 ksi] was used to reinforce both beams and columns. Seven-wire, low-relaxation Grade 270 ksi (1860 MPa) strands with a 0.5 in. (13 mm) diameter were used for all of the beams, and they were initially prestressed at roughly 70% of their nominal yielding resistance. Steel with a yield strength of 560 MPa [81 ksi] was...
used for $80 \times 50$ mm (3 × 2 in.) channels and 16 mm (no. 5) threaded bars.

Figure 6 provides an example of the reinforcement layout of beams, columns, and corbels, along with the node nomenclature used in the upcoming discussion of the experimental results.

**Test setup and procedure**

A quasi-static cyclic displacement history at increasing roof displacements was imposed by actuators in displacement control. In particular, the experimental loading protocol consisted of a series of five symmetric horizontal drift targets (Table 2). Three cycles per amplitude were planned with an additional monotonic ramp up to collapse.

The imposed lateral force distribution was computed as a function of the seismic mass on the beams at the $i$th floor $m_i$ and of the story height at $i$th floor $h_i$, as expressed by Eq. (1). The actuator at the third floor, labeled in the following as 1, was used to impose the target displacement and to control those at the first and second floors (2 and 3, respectively), applying the force ratio needed to hold the distribution constant during the tests.

$$\frac{F_i}{F_{top}} = \frac{h_i m_i}{h_{top} m_{top}} = \begin{cases} 1.00 & F_1 = F_{top} \\ 0.98 & F_2 \\ 0.71 & F_3 \end{cases}$$

**Table 2. Experimental loading protocol: Imposed drift levels for two prototype frames**

<table>
<thead>
<tr>
<th>Specimen F1 drift, %</th>
<th>Specimen F2 drift, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>±0.50</td>
<td>±0.25</td>
</tr>
<tr>
<td>±1.00</td>
<td>±0.50</td>
</tr>
<tr>
<td>±1.50</td>
<td>±0.75</td>
</tr>
<tr>
<td>±2.00</td>
<td>±1.00</td>
</tr>
<tr>
<td>±3.00</td>
<td>±1.50</td>
</tr>
</tbody>
</table>

**Figure 6. Structural configuration of prototype frames: representation of geometry, node nomenclature, and reinforcement layout of beams, columns, and corbels (that is, front and side view). Note: All measurements are in millimeters. 1 mm = 0.0394 in.**
Summary of test results

In the following discussion, the main results obtained from the experimental investigation of the two specimens will be summarized. The bare frame will be identified as specimen F1, while the frame with cladding panels will be named specimen F2. The response of the two frames will be used to describe the global and local behavior of classical multistory precast concrete structures at different limit states. A general overview of their main characteristics is given here, and specific aspects are referenced as needed to explain key points. Particular care is paid to the response of beam-column joints and to the influence of cladding panels.

Figure 7 shows the test setup and instrumentation. Dead and live loads were simulated by means of concrete blocks appropriately scaled and screwed to the framing beams. The mass distribution along the height of the specimens was chosen to represent an actual mass distribution for common actual structures, which are usually characterized by higher loads at the lower floors for building usage purposes. Sixty-four linear variable differential transformers (LVDTs) were used to measure absolute and relative quantities at key locations throughout the frame. In particular, the LVDTs were arranged to obtain the absolute displacements at each floor, the curvatures at the base of each side of each column, and beam-column relative translations and rotations (Fig. 7).
Response of bare frame:
Specimen F1

Figure 8 shows the global response of the bare frame F1 in terms of total base shear versus roof displacement hysteresis loops. Table 3 summarizes the most significant results measured during each cycle of the test, including base shear peaks, maximum story forces, normalized displacement profiles, and rotation/curvature peaks at each significant node. Computations related to the cumulative dissipated energy and the equivalent viscous damping are performed to confirm that the response of this structure was nearly elastic-fragile because of the brittle shear failure of its beam-column connections, where damage and slippage were observed to take place.

Prototype F1 resisted a maximum base shear of 148.9, 237.2, 322.9, 385.3, and 470.6 kN (33.5, 53.3, 72.6, 86.6, and 105.8 kip) (the first cycle of each drift amplitude) at top displacements of up to 37.0, 74.5, 112.0, 149.5, and 224.6 mm (1.46, 2.93, 4.41, and 5.89 in.), respectively. Figure 9 shows peak horizontal displacement profiles during pulling and pushing phases, revealing the high flexibility of the specimen characterized by a typical cantilevered static scheme. In detail, a slight discrepancy between pushing and pulling profiles can be observed because of the combined effects of damage and slip in the components of the connections, as discussed later. The beam-column gap opened up on the bottom face of the beam but closed on its top face. During the cyclic reversal, the opposite beam end reseated itself and began to bear on corbel and column, resulting in an increased relative stiffness and additional demand at the level of the connections, which were observed to be the most damaged zones of the structural system tested. A mechanism similar to those evidenced by direct observation in the aftermath of the 2012 Emilia seismic events (Fig. 2 and 3) was experimentally predicted (Fig. 10). Local quantities will be provided to detail and quantify such behavior when discussing the response of beam-column joints.

Incipient yielding occurred at roughly 1.5% drift for an actuator load of approximately 300 kN (67 kip), as evidenced by a slight deviation of the capacity curve from the initial elastic response. This was a consequence of damage and slippage in the connections of the specimen. Nevertheless, a pseudoelastic response was obtained for this system up to 2% drift cycles, as confirmed by the significantly low residuals experienced. The maximum residual displacement was about 6% of the top displacement peak imposed during the cycle.

Energy dissipation estimates (Fig. 11), in accordance with the traditional area-based approach, reaffirm such a trend. In particular, Fig. 11 indicates the cumulative energy dissipated by cycles at different drift levels, obtained as the area enclosed by a complete hysteresis loop $E_d$, and shows the equivalent viscous damping $\xi$, determined by equating...
the energy absorbed by the hysteretic steady-state cyclic response at a given displacement level, as expressed by Eq. (2):

\[ \xi = \frac{E_p}{2\pi F_m d_m} \]  

(2)

where

- \( F_m \) = maximum force that occurred in the complete cycle
- \( d_m \) = maximum displacement that occurred in the complete cycle

Even though overestimates were expected from this approach, low peaks of up to 8.9% were predicted, as well as a visible decay after the yielding condition was exceeded (that is, 42% if the first 2% drift cycle is considered). Values of about 5% or lower were determined at the second and third cycle of each drift level in accordance with the conventional values suggested by current seismic codes for elastic reinforced concrete structures. The equivalent viscous damping \( \xi \) decreased as the number of constant amplitude cycles at a given drift level increased, particularly between the first and second cycles. The maximum decay (35%) was observed at 0.5% drift and the minimum (18%) at 2% drift.

Global collapse of the system was observed at the second 3% drift cycle due to local shear failures of the vertical steel dowels in three beam-column joints, namely nodes 2, 5, and 9. Figure 12 shows the displacement shape at failure, and an example of this collapse mechanism, which was identified as the ultimate limit state of specimen F1.

**Table 3. Test results for prototype F1**

<table>
<thead>
<tr>
<th>Drift, %</th>
<th>Cycle</th>
<th>( V_b ), kN</th>
<th>( F_1 ), kN</th>
<th>( F_2 ), kN</th>
<th>( F_3 ), kN</th>
<th>( D/D_{top} )</th>
<th>( R_{\text{max},1} ), mrad</th>
<th>( R_{\text{max},2} ), mrad</th>
<th>( R_{\text{max},4} ), mrad</th>
<th>( R_{\text{max},7} ), mrad</th>
<th>( R_{\text{max},8} ), mrad</th>
<th>( \chi_{\text{max},1} ), ( \mu ) m/m</th>
<th>( \chi_{\text{max},2} ), ( \mu ) m/m</th>
<th>( \chi_{\text{max},3} ), ( \mu ) m/m</th>
</tr>
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<td>1</td>
<td>148.9</td>
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<td>54.2</td>
<td>39.5</td>
<td>1 (1);</td>
<td>4.71</td>
<td>7.10</td>
<td>4.15</td>
<td>7.16</td>
<td>4.16</td>
<td>7.23</td>
<td>5.15</td>
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<td>4.70</td>
<td>7.12</td>
<td>4.16</td>
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<td>7.21</td>
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<td>7.89</td>
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<td>53.1</td>
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<td>0.255;</td>
<td>4.73</td>
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<td>1 (1);</td>
<td>9.62</td>
<td>14.2</td>
<td>8.58</td>
<td>14.0</td>
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<td>117.7</td>
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</table>

Note: \( D \) = peak displacement at \( i \)th floor; \( D_{top} \) = peak displacement at roof; \( F_1 \) = story force imposed at roof; \( F_2 \) = story force imposed at second floor; \( F_3 \) = story force imposed at first floor; n/a = not applicable; \( R_{\text{max},1} \) = peak rotation at node 1; \( R_{\text{max},2} \) = peak rotation at node 2; \( R_{\text{max},4} \) = peak rotation at node 4; \( R_{\text{max},5} \) = peak rotation at node 5; \( R_{\text{max},7} \) = peak rotation at node 7; \( R_{\text{max},8} \) = peak rotation at node 8; \( V_b \) = total base shear; \( \chi_{\text{max},1} \) = peak curvature at base of leftmost ground column; \( \chi_{\text{max},2} \) = peak curvature at base of central ground column; \( \chi_{\text{max},3} \) = peak curvature at base of rightmost ground column. 1 \( \mu \)m = 0.0000394 in.; 1 mm = 0.0394 in.; 1 kN = 0.225 kip.
concentration on the column or an additional demand on the connection, depending on the direction of the loads. In actual structures, this behavior is expected to be worse if diaphragms are grouted with a cast-in-place concrete topping.

Even if in some cases (for example, node 5a [Fig. 14]), load pickup was effective and the hysteretic behavior was characterized by local ductility levels greater than 6, in the majority of the joints (for example, nodes 5b, 2, and 7 [Fig. 14]), the effects of slip were pronounced and the loops were too unstable to develop a rationally controlled, ductile mechanism.

Good design can prevent or partially inhibit the localized damage observed at the corners of the corbel. In particular, the formwork may be properly shaped to remove or smooth its sharp edge, and hence the precast concrete beam is free to deform without causing excessive damage in that zone of the corbel. Alternatively, L-shaped steel profiles may be grouted in correspondence to the upper corner of the corbel, designed along the lines of a conventional strut-and-tie approach, using local reinforcement details (Fig. 6). Closed hoops have been provided as shear and tensile reinforcements, and bushing sleeve couplings are set in place to connect the vertical threaded steel bars protruding into the precast concrete beam, thus avoiding their pullout in tension. Nevertheless, without changing the beam-column connection system and static scheme, none

At 0.5% drift, cracks in the concrete corbels opened and gradually evolved under cyclic reversals. Crack width and length increased with increasing drift amplitudes, propagating from the steel dowels to approximately midheight of the corbel. This mechanism resulted in a partial expulsion of the corner of the corbels at the first two floor levels. A similar damage pattern developed at the top floor (Fig. 10), causing the concrete to crush at the interior side of the columns. A slightly less inclined crack was observed in the latter case because the beams were directly seated on the tops of the columns, and, hence, no restraint was provided between the bottom face of the beam and the edge of the column, thus varying the inclination of the resulting struts. By contrast, in the former case, the beam-column gap had to be taken up before the end of the beam made contact with the side of the column, resulting in a delay in the load pickup. After a relative horizontal displacement of about 15 mm (0.59 in.), the rotation at the end of the beam was restrained by the edge of the column, resulting in a stress concentration on the column or an additional demand on the connection, depending on the direction of the loads.

In actual structures, this behavior is expected to be worse if diaphragms are grouted with a cast-in-place concrete topping.

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of these recommendations is able to prevent the brittle shear failure observed in the threaded bars (Fig. 12).

**Response of clad frame: Specimen F2**

As highlighted in Table 2, specimen F2 was subjected to halved drift amplitude cycles because lower displacement capacity was expected in the frame due to the response of panel-to-structure connections. A total base shear of about 157.6, 226.2, 325.1, and 429.6 kN (35.4, 50.9, 63.2, 73.1, 96.6 kip) was resisted at roof displacements of up to 18.3, 37.3, 56.3, 75.3, and 113.0 mm (0.72, 1.47, 2.22, 2.96, and 4.45 in.), respectively. Figure 15 provides the cyclic base shear–roof displacement capacity curve and compares the hysteresis loops obtained for the two frame structures. Horizontal displacement profiles (Fig. 16) were characterized by lower peaks but almost identical shapes, demonstrating the inability of the panels to completely restrain the panel-to-panel and panel-to-column relative displacements, as discussed later. Finally, Table 4 summarizes base shear peaks, maximum story forces, normalized displacement profiles, and rotation/curvature peaks at each critical node, measured at each cycle of the loading history.

In terms of initial stiffness, specimen F2 was approximately six times higher than F1, thus implying an anticipated incipient yielding, which occurred at 0.75% drift cycles for lower actuator loads of about 270 kN (60.7 kip) as a consequence of major damage in the cladding system. The narrow thin-shaped cycles obtained in the case of specimen F1 are opposed to apparently more dissipative loops predicted for specimen F2. As in the case of specimen F1, energy dissipation estimates were conducted. Figure 17 shows both equivalent viscous damping and cumulative dissipated energy, respectively. Significantly higher values of $\xi$, ranging from approximately 14% to 20%, were computed and much less decay was observed between the first and second cycles. A maximum decrease of 16%—three times lower than...
force-resisting system of the frame, the precast concrete cladding panels were observed to produce only a contraction of the hysteresis loops, largely decreasing the displacement capacity of the structure for roughly constant loads.

Finally, the global collapse of specimen F2 was observed at drift levels slightly higher than those demanded by conventional damage limit state requirements. The mechanism, which occurred at 1.5% drift, was induced by local failures of panel-to-structure connections.

Figure 11. Dissipation energy estimates for specimen F1.

that obtained for specimen F1—was determined at 0.5% drift. Nevertheless, the cumulative dissipated energy roughly coincides if the last cycle is considered, thus revealing that the total energy absorption capabilities of the two structures are fairly constant. In particular, 88.1 and 84.8 kN-m (65.0 and 62.5 kip ft) were obtained at failure for specimens F1 and F2, respectively. Neither specimen F1 nor F2 developed rationally controlled dissipation mechanisms because the equivalent viscous damping essentially decreased with increasing drift amplitudes. Therefore, when included in the lateral-
Influence of cladding panels

The overall cyclic response and related damage pattern of specimen F2 depended greatly on the behavior of panel-to-structure connections. These connections, implicitly designed to be the weakest of the system, began failing at 0.5% drift, with the concrete crushing at the top and bottom corners of the cladding panels as a result of the internal strut developed. As the drift amplitude increased, this mechanism evolved and damage progressed to failure of the upper C-shaped steel profiles by fracture of their bolts, which occurred at 1% drift. Finally, 1.5% drift cycles caused three panels to lose their bottom vertical supports, and this mechanism was conventionally identified as the ultimate limit state of specimen F2. Figure 18 depicts damage of the cladding horizontal panels and their connections at the end of the test.

As mentioned, panels were unable to modify the static scheme of the bare frame. Conversely, displacement incompatibility between structural and nonstructural members was crucial in the response of specimen F2 because the structural skeleton forced the cladding panels and their connections to work for unfeasible strain and load demands.

Panels appeared only to be able to inhibit or partially reduce the slip of beam-column connections, improving their load pickup compared with those shown by specimen F1, in turn causing the connections to fail at lower displacement demands. Figure 19 provides an example of such behavior, where the local response of the same beam-column joints, examined for specimen F1, is shown, in terms of interstory force versus relative horizontal displacement curves. The shape of the hysteresis loops was significantly different, particularly for nodes 2 and 5b, and, and more than halved relative horizontal displacements were measured for slightly lower interstory forces. As a result, even if overall global response of the frame and local response of the joints appears more stable, the interaction between the cladding panels and the structure must be accounted for in the design process of these early structures, still common in Italian and European scenarios, as cause of anticipated collapse, rather than as a latent source for stiffening and strengthening.

A conventional approach used in current building practice to minimize panel-structure interaction is to slacken its connections to make the panel slide relative to the structural skeleton, rather than to force excessive and unfeasible load transfer in the connections. Nevertheless, this solution is effective only for relatively small values of drifts of about 1% because of the small strokes provided by these connection devices that, in the majority of the cases, are not specifically designed to accommodate large seismic displacement demands. Even if different cladding solutions may require different design recommendations on the basis of different capacities of their connection components and of different levels of interaction with the supporting structure, the safest design methodology is to impose the 1% interstory drift as a conventional limit to design the cladding system and skeletal frame, as prescribed by the current Italian seismic code and shown by the experimen-
concrete structures common in past and current Italian building practice and severely damaged by 2012 Emilia seismic sequences. Two 3:4 scaled two-bay, three-story precast concrete frames typical of the damaged buildings were tested under pseudostatic cyclic loading. The responses of the specimens with and without precast concrete panels were compared so as to quantify the interaction of the cladding panels with respect to the global program conducted, thus implicitly accepting a pseudo-elastic structural response for this type of precast concrete building.

**Conclusion**

The experimental research described here was conducted to investigate the seismic behavior of reinforced precast concrete structures common in past and current Italian building practice and severely damaged by 2012 Emilia seismic sequences. Two 3:4 scaled two-bay, three-story precast concrete frames typical of the damaged buildings were tested under pseudostatic cyclic loading. The responses of the specimens with and without precast concrete panels were compared so as to quantify the interaction of the cladding panels with respect to the global program conducted, thus implicitly accepting a pseudo-elastic structural response for this type of precast concrete building.

**Figure 13.** Damage in corbels of first two floors (specimen F1).
response of the reference bare frame and with respect to the performance of the beam-column joints. The prevailing observations and conclusions, drawn from the experimental study, are summarized as follows:

- The bare frame specimen F1 provided a flexible pseudoelastic response governed by strength rather than ductility and mainly ascribed to the behavior of its beam-column connections consisting of vertical steel dowels and rubber pads. Both low residuals up to 2% drift levels and energy dissipation estimates confirmed this trend. Global collapse occurred at 3% drift cycles as a consequence of local failures of the vertical steel dowels in three beam-column joints after a significant propagation of damage in the corbels. A similar mechanism was identified in the aftermath of the 2012 Emilia earthquakes.

- Except for visible concrete crushing in the corner of the corbels, no damage was observed in the other reinforced precast concrete members. The only source of nonlinear behavior was in the response of the joints due to a combination of slip and damage to their components. A delay in the load pickup was obtained because the end of the beam made contact with the side of the column, only after the beam-column gap was taken up. In some cases, the beams provided a response close to that of a double bending deformation, bearing on corbel and column. As a result, increased relative stiffness and additional demand at the level of the connection were determined. By contrast, in the majority of the cases, load pickup was ineffective and the effects of slip were pronounced throughout the loading histories. Therefore, a hysteretic behavior characterized by local ductility levels greater than 6, in the former case, contrasted with a response where the loops were too unstable to develop a well-established mechanism, in the latter case. In both cases the rotational capacity was limited, and the simply supported configuration, commonly assumed at the design stage, did not represent the behavior observed.

- The clad frame specimen F2 resisted a roughly 10% lower total base shear at an approximately halved roof displacement. Precast concrete cladding panels were
unable to modify the cantilevered static scheme observed for specimen F1 because the peak horizontal displacement profiles showed almost identical shapes, demonstrating their inability to completely restrain the panel-to-panel and panel-to-column relative displacements.

- In the plots of base shear versus roof displacement, the narrow hysteresis loops of specimen F1 contrast with the apparently more dissipative response of specimen F2, as confirmed by the comparison in terms of cumulative dissipated energy. Therefore, panels caused only a contraction of the hysteresis loops, significantly decreasing the displacement capacity of the bare structure for similar maximum loads. A similar trend was observed in terms of local response of beam-column joints because panels partially inhibited the relative

Figure 15. Total base shear versus roof displacement hysteresis loops of prototype F2 (top). Comparison between experimental capacity curves of specimens F2 and F1 (bottom). Note: 1 mm = 0.0394 in.; 1 kN = 0.225 kip.
Table 4. Test results for prototype F2

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Note: $D$ = peak displacement at 4th floor; $D_{top}$ = peak displacement at roof; $F_1$ = story force imposed at roof; $F_2$ = story force imposed at second floor; $F_3$ = story force imposed at first floor; $R_{max,1}$ = peak rotation at node 1; $R_{max,2}$ = peak rotation at node 2; $R_{max,4}$ = peak rotation at node 4; $R_{max,5}$ = peak rotation at node 5; $R_{max,7}$ = peak rotation at node 7; $R_{max,8}$ = peak rotation at node 8; $V_b$ = total base shear; $\chi_{max,1}$ = peak curvature at base of leftmost ground column; $\chi_{max,2}$ = peak curvature at base of central ground column; $\chi_{max,3}$ = peak curvature at base of rightmost ground column.

1 $\mu m = 0.0000394$ in.; 1 mm = 0.0394 in.; 1 kN = 0.225 kip.

Figure 16. Experimental displacement profiles of specimen F2; peaks at each drift amplitude in pulling and pushing directions. Note: 1 mm = 0.0394 in.; 1 m = 3.28 ft.
upper C-shaped steel profiles. Global collapse was experienced at 1.5% drift due to the loss of support in three horizontal cladding panels, which identifies the ultimate limit state of specimen F2.

- The damage pattern was governed by the local responses of panel-to-structure connections, which began failing at 0.5% drift, with the concrete crushing at the top and bottom corners of the cladding panels as a consequence of excessive force demand in the panel-to-structure connections. One-percent drift cycles caused bolt fracture in correspondence to the slip of the connection system, causing the frame to fail at much lower displacements because of the local failure of its panel-to-structure joints.

- Although good design practice may partially mitigate the localized damage observed in the cladding connections and at the corner of the corbels, the results found by the experimental program confirmed the inadequate seismic behavior of this frame typology. It was characterized by a nearly elastic-fragile global response as

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**Figure 17.** Dissipation energy estimates for prototype frame F2.
The findings of this research may be directly applicable to the design, assessment, and strengthening of such a vulnerable structural typology, as well as to the implementation and validation of analytical and numerical models capable of accounting for the re-

a consequence of structural deficiencies intrinsic to static scheme and connection systems. Therefore, the application of this precast concrete solution for earthquake resistance is not recommended in zones of high seismic hazard.

Figure 18. Damage of cladding panels (that is, top and bottom panel-to-structure connections) at end of test: significant concrete crushing at the corners and clean fractures in bolts and plates.

- The findings of this research may be directly applicable to the design, assessment, and strengthening of such a vulnerable structural typology, as well as to the implementation and validation of analytical and numerical models capable of accounting for the re-
sponse of this beam-column joint system in which the joints are assumed to be pinned, whether interacting with panels or not and, hence, with panel-to-structure connections.

**Acknowledgments**

The authors wish to express their gratitude to the Italian Department of Civil Protection for the financial support received through a three-year framework program established with the European Centre for Training and Research in Earthquake Engineering. In-kind support was also provided by the Italian company Cielle Prefabbricati, which constructed and provided the two specimens tested.

**References**


Notation

\( D_i \) = peak displacement at \( i \)th floor

\( d_m \) = maximum displacement occurring in a complete hysteresis cycle

\( D_{top} \) = peak displacement at roof

\( E_D \) = cumulative dissipated energy obtained as the area of a complete loop

\( F_i \) = story force imposed at \( i \)th floor

\( F_{top} \) = \( F_1 \) = story force imposed at roof

\( F_2 \) = story force imposed at second floor

\( F_3 \) = story force imposed at first floor

\( F_m \) = maximum force occurring in a complete hysteresis cycle

\( g \) = acceleration due to gravity

\( h_i \) = story height at \( i \)th floor

\( h_{top} \) = total height

\( m_i \) = seismic mass at \( i \)th floor

\( m_{top} \) = seismic mass at roof

\( q \) = behavior factor

\( R_{max,1} \) = peak rotation at node 1

\( R_{max,2} \) = peak rotation at node 2

\( R_{max,4} \) = peak rotation at node 4

\( R_{max,5} \) = peak rotation at node 5

\( R_{max,7} \) = peak rotation at node 7

\( R_{max,8} \) = peak rotation at node 8

\( T \) = fundamental period

\( V_b \) = total base shear

\( V_s \) = shear wave velocity

\( \xi \) = equivalent viscous damping

\( \theta \) = interstory drift sensitivity coefficient or \( P-\Delta \) ratio

\( \mu \) = displacement ductility

\( \chi_{max,1} \) = peak curvature at base of leftmost ground column

\( \chi_{max,2} \) = peak curvature at base of central ground column

\( \chi_{max,3} \) = peak curvature at base of rightmost ground column
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Abstract

This paper summarizes the results of an experimental program devoted to assessing the main structural characteristics and criticalities in the seismic response of typical reinforced precast concrete structures in Italy that were severely damaged by major earthquakes in May 2012. Two ¾ scale two-bay, three-story precast concrete frames with and without cladding panels were subjected to a quasi-static cyclic top displacement history consisting of symmetric stepwise increasing drift cycles. Pinned beam-column connections consisted of neoprene pads with vertical steel dowels working in shear. The flexibility of the cantilevered static scheme forced large displacement demand and incompatible transfer of the horizontal forces to occur in joints lacking shear and ductility capacity. In addition, displacement incompatibility between structural elements and cladding panels caused the premature failure of their connections. The results demonstrate the inadequate performance of this structural solution because of its beam-to-column and panel-to-structure connection systems, the responses of which reaffirm that this frame typology should not be used in areas of high seismic hazard because of its nearly elastic-fragile behavior.

Keywords

Beam-column connection, cyclic loading, experimental test, multistory frame, seismic response.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute’s peer-review process.

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