# Seismic-drift-compatible design of architectural precast concrete cladding: Tieback connections and corner joints

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- Architectural precast concrete cladding is a nonstructural system sensitive to both seismic floor accelerations and story drifts. Cladding panels must be designed to resist forces, particularly in the out-of-plane direction of motion, and accommodate in-plane story drifts.
- Presently, these important issues are addressed rather broadly in design codes, leaving the details to the discretion and experience of the designer.
- This paper summarizes key results obtained from system and component tests on cladding panels and tieback connections and applies these findings to developing guidelines for the drift-compatible design of tieback connections and corner systems.


Architectural precast concrete cladding is a type of building facade composed of concrete panels attached to the building exterior with steel connections. Individual panels are separated from each other by horizontal and vertical joints filled with caulk. This type of facade has been widely used worldwide since the 1960s in a variety of buildings due to its durability, low life-cycle costs, aesthetics, and ability to create many different shapes and textures.[1] Architectural precast concrete also provides superior finish quality and speed of erection compared with cast-in-place concrete. Panels are fabricated at a precast concrete plant, delivered to the site just before installation, and several panels can be installed during a workday. A variety of panel geometries can be constructed, depending on practical issues, such as transportability and architectural constraints. Among the most common are solid wall panels, wall panels with central punched windows, U-shaped wall panels with upper window openings, spandrel panels, and column covers and mullions, which can be combined to emulate a U shape (Fig. 1). This paper focuses on the widely used punched-window wall panels (**Fig. 1**).

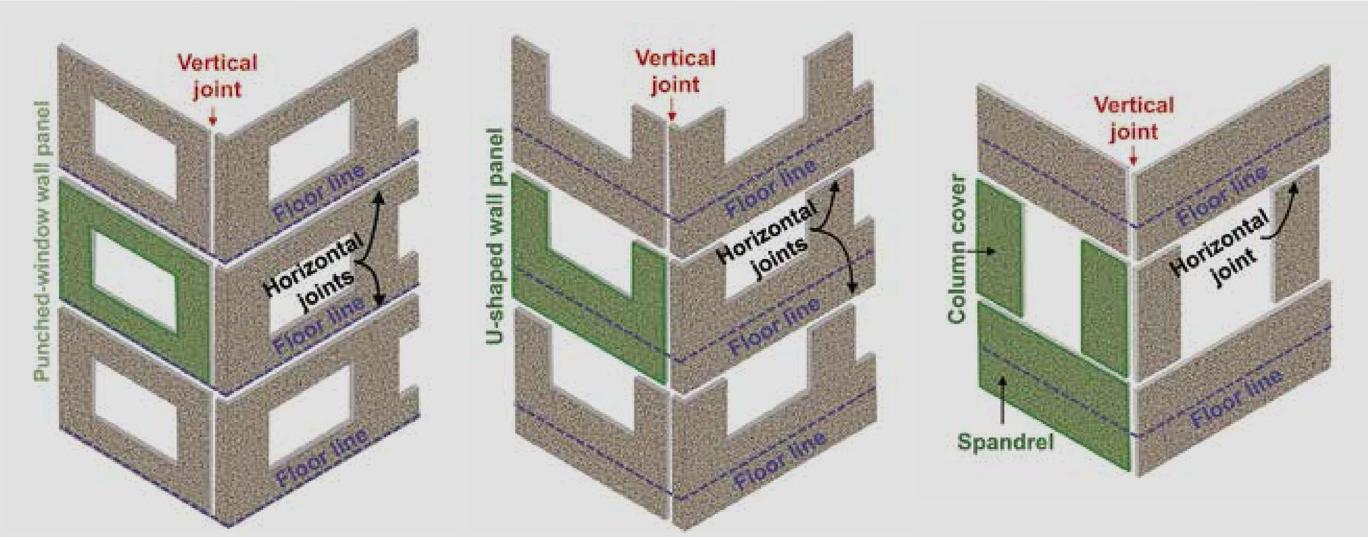



**Figure 1.** Architectural precast concrete panel configurations.

## Seismic protection of architectural precast concrete cladding

Architectural precast concrete cladding is generally designed as a nonstructural component, meaning that during an earthquake it should only transmit its own inertial force to the structure while forces generated elsewhere in the building should not pass through it. The seismic protection of this facade system is provided mainly via its steel connections, which must resist seismic forces created by the panel mass and accommodate structural displacement. Because punched-window panels span from floor to floor, this displacement is equal to the full story drift. Story drift in this context refers to drift that develops between the floors. Because these panels have high in-plane stiffness, the steel connections are used to provide flexibility between the panel and the building in this direction. Absent such a mechanism, panels may act as structural shear walls and attract forces created in the structure while also stiffening it. In typical U.S. practice, uncoupling of the in-plane displacements is achieved for low-aspect-ratio panels by allowing the panels to move with a translation mechanism. In this case, the panel is connected to one floor with a fixed bearing–type connection, while a tieback connection is used to attach the panel to the other floor. Commonly, the bearing connections are at the bottom and the tieback connections are at the top; however, the reverse may occur as well. The tieback connections are intended to restrain the panel in the out-of-plane direction while allowing in-plane translation. The main element of a tieback is a steel rod that can absorb drift by either sliding inside a slot (sliding connections) or bending (flexing connections).

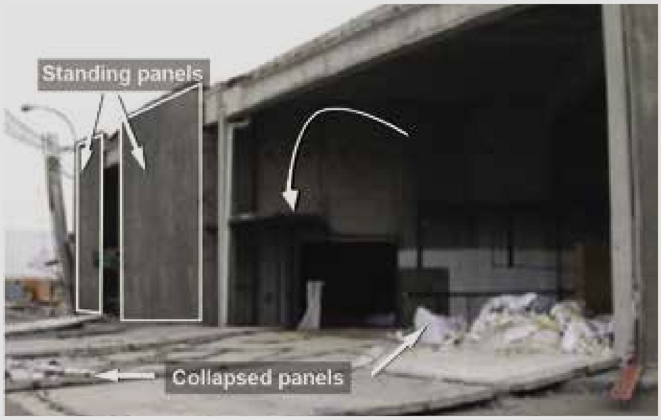
Two panels that meet at a corner experience a differential movement equal to the story drift because the top of the out-of-plane panel moves with the top slab while the in-plane panel moves with the bottom slab. When the vertical joint between panels that intersect at a corner is smaller than this story drift, they will collide, generating large forces and possibly failure in the corner connection. Although drift compatibility of tieback connections and corner joints represents a key issue controlling the architectural precast concrete cladding as a system, current design codes address the issue broadly and leave much to the discretion of the designer.

## Damage during past earthquakes

Complete or partial separation of architectural precast concrete panels from buildings has been observed after several earthquakes worldwide, such as the 1985 Mexico earthquake[2] and the 1995 Kobe earthquake in Japan.[3] In more recent years, failures of architectural precast concrete cladding were observed following the 2009 L'Aquila earthquake in Italy[4] and the 2010 Maule earthquake in Chile, when several panels collapsed in the out-of-plane direction[5] (**Fig. 2**). During the 2011 Christchurch earthquake in New Zealand, several panels failed due to inadequate detailing.[6] Insufficient displacement capacity and excessive lateral forces in the connections also caused several panel failures during the 2012 Emilia earthquake in Italy.[7,8] Extensive cracking, corner crushing, and residual displacement of the panels were also observed after many of these earthquakes.

Fortunately, collapse of panels in the United States has been observed only in a few cases, namely after the 1964 Alaska earthquake[9,10] and the 1987 Whittier Narrows earthquake in California.[11] The 1989 Loma Prieta earthquake in California did not result in any panel failures; however, reconnaissance teams reported that if the intensity of the shaking had been greater, damage to the panel connections would have been likely.[10] Architectural precast concrete


2009 L'Aquila earthquake
(Miyamoto International [2009])




2010 Maule earthquake
(Ghosh and Cleland [2012])

**Figure 2.** Examples of collapsed panels.

cladding also performed well during the 1994 Northridge earthquake in California.[12] When damage has been observed, the cause has been identified as primarily excessive force at connection locations or excessive drifts, which were not accounted for in the design.

## Previous research

Improved understanding of the seismic behavior of this facade system is of critical importance, not only because of evidence of damage from past earthquakes, but also because damage to architectural precast concrete panels and their connections poses a threat to human safety and their repair is costly and time consuming. In fact, facades are one of the most expensive nonstructural components installed in buildings.[13] Nevertheless, few studies on the seismic behavior of architectural precast concrete cladding have been performed, and thus this knowledge remains rudimentary.

One of the largest of these projects, conducted by Wang, consisted of applying static displacement to a structural frame encased by architectural precast concrete panels installed according to common practice in California and Japan.[14] Lessons learned during this project included the sensitivity of the behavior of sliding connections to installation errors and the importance of providing gaps large enough to avoid contact between panels.

In 1988, Rihal embarked on a project that included static tests of typical connections, cyclic racking tests on full-sized architectural precast concrete panels, and dynamic tests on a reduced-scale model of a two-story frame with and without cladding.[15] Analytical models were developed for each phase of testing. This research revealed that flexible tieback connections might fail at low drift amplitudes if they are too short. Craig et al. also performed tests of flexing rod connections and concluded that they might be subjected to low-cycle fatigue.[16]

In more recent years, Memari et al. studied the influence of vertical ground motion on these panels with the aid of a computer model and concluded that the vertical component of shaking can increase the forces absorbed by the connections.[17] Maneetes and Memari developed a refined model of an architectural precast concrete panel and its connections, which led to an understanding of the different sources of flexibility in the panels.[18]

Efforts by McMullin et al. included vibration tests on panels installed on real buildings,[19] component tests on connections,[20] pushover tests on panels,[21] and a test on two panels installed on a full-scale building that was shake-table tested.[22] Results from tests on panels installed using rocking connections indicate that this protection mechanism is effective for tall and narrow panels.

## Scope of this paper

The severe consequences of seismic damage to architectural precast concrete cladding and the lack of quantitative guidelines for its drift-sensitive design motivated an experimental investigation into its seismic behavior. The focus was placed on the widely used punched-window wall panels as typically designed and installed in highly seismic areas of the United States. This research included a system-level experimental program and companion component tests on both sliding and flexing tieback connections. This paper provides an overview of these tests, highlighting key findings, and provides a synthesis of guidelines, which were informed by the tests. Interested readers may find additional information in a design procedure document published by the authors.[23]

## Current practice

**Figure 3** provides a photograph of a panel during transportation. Although panels typically have two bearing connections, the number of tiebacks depends on the size of the

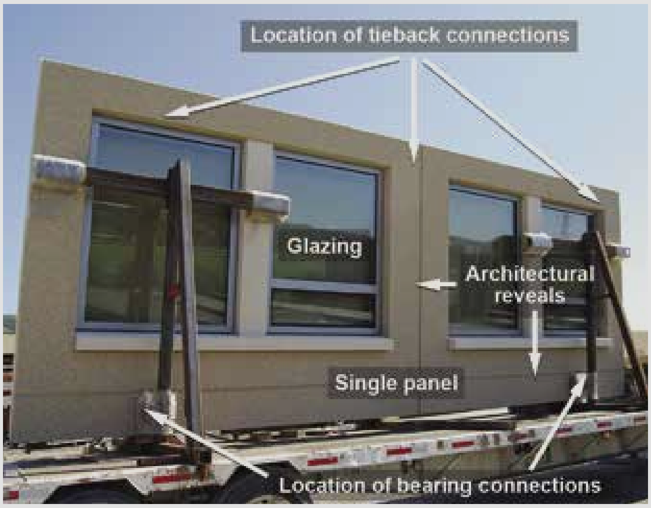



**Figure 3.** Typical punched-window wall panel.

panel. In current practice, the design of tieback connections and corner joints for drift compatibility during seismic loading presents a major challenge for engineers.

## Design of tiebacks

Two types of tieback connections are typically used in U.S. practice: sliding and flexing connections. A typical sliding connection consists of a panel embed, a relatively short threaded rod, a slotted clip welded to the structure or a structural embed, two plate washers, and two nuts. The threaded rod is connected to the panel with a threaded insert in the panel embed. It then extends through the slotted clip and is held in position by a plate washer and nut on each side of the clip (**Fig. 4**).

In-plane displacements are accommodated via the sliding of the threaded rod inside the slotted clip (Fig. 4). To facilitate sliding, it is important for the nuts to be no more than finger tight to avoid the creation of excessive frictional resistance. Out-of-plane forces created in the panel are transmitted to the clip through tension or compression in the rod and then to the structural embed. The two plate washers are denoted tension and compression depending on the type of axial force developed in the rod during their action. When the length of the rod is zero, the configuration is named snug. A typical flexing connection involves components that are similar to those used in sliding connections; however, the rod is longer and the through hole in the clip is oversized only to account for installation tolerance (**Fig. 5**). Flexing connections allow in-plane displacement through elastic and plastic bending of the relatively long threaded rod.

Flexing connections are typically more economical and easier to install than sliding-type tieback connections. However, they require a longer rod that often does not fit because of the presence of architectural finishes and sometimes there are concerns about the potential for buckling of long rods. When this occurs, designers often prefer sliding connections. For this reason, U.S. West Coast precast concrete designers and manufacturers commonly use sliding connections. A survey by the authors of 14 precast concrete producers and designers revealed that 93% use sliding connections, while only 29% use flexing connections.

**Accommodation of seismic forces** When designing the connections, forces created by dead and earthquake loads are combined to produce the worst possible combination. The seismic force $F_p$ can be calculated using Eq. (13.3-1) in the American Society of Civil Engineers' (ASCE's) *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-10:[24]

$$F_p = \frac{0.4 a_p S_{DS} W_p}{\left(\dfrac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right)$$

where

$a_p$ = component amplification factor

$S_{DS}$ = spectral acceleration at short periods

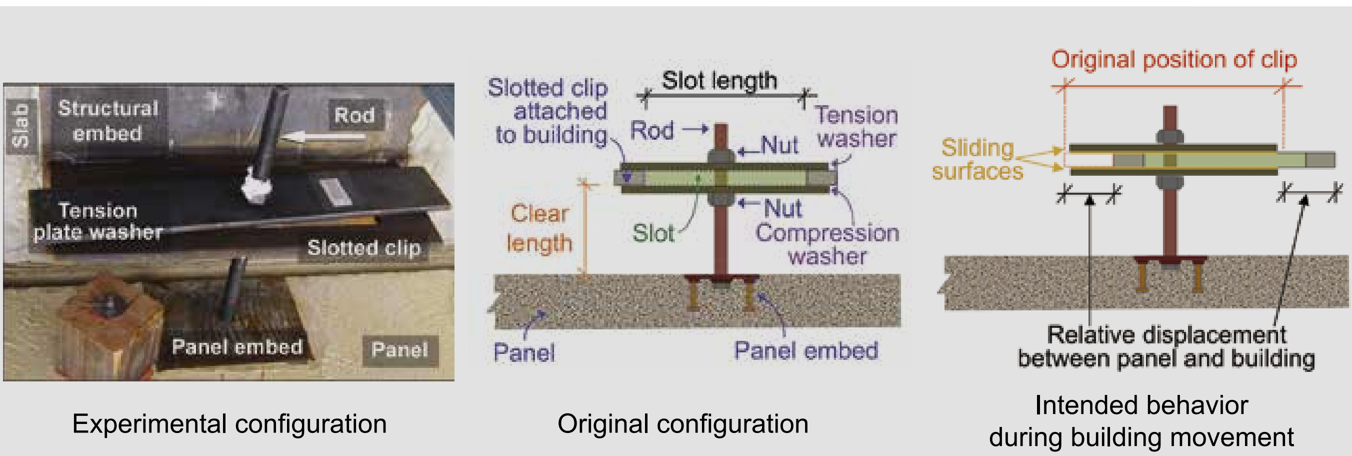



**Figure 4.** Sliding connections.

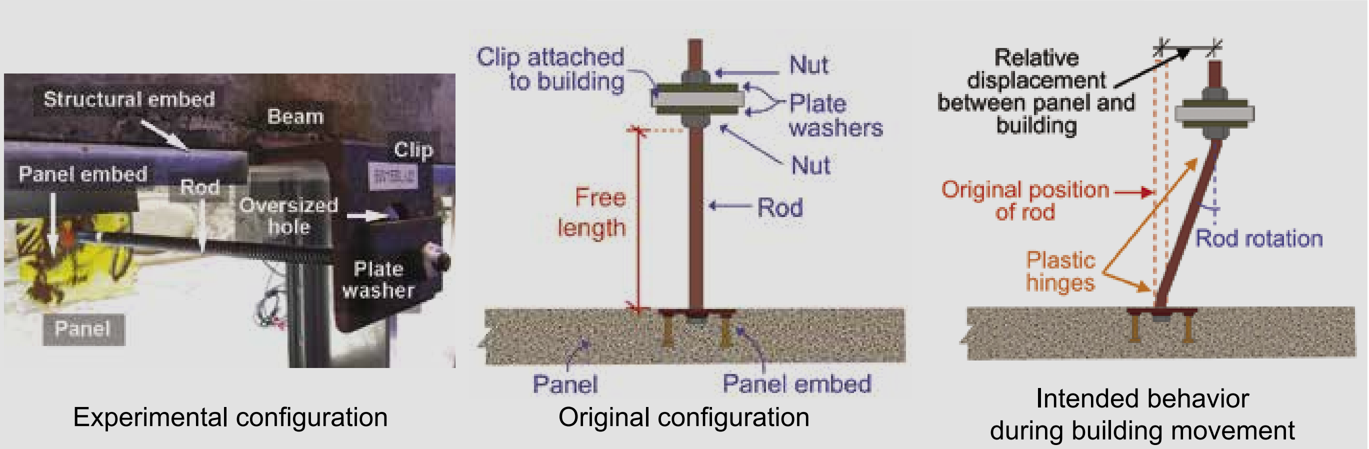



**Figure 5.** Flexing connections.

$W_p$ = weight

$R_p$ = response modification factor

$I_p$ = importance factor

$z$ = height in the structure of the point of attachment of the component

$h$ = average roof height of the structure with respect to the base

The parameters $a_p$ and $R_p$ for the body and its fastener elements of the connection are stipulated in ASCE 7-10 Table 13.5-1. The values of $a_p$ and $R_p$ result in a force more than three times larger for fastener components of connections. This strategy attempts to maintain the response of these elements in the elastic range and prevent brittle failure modes. This amplified force level satisfies the exception to the ductility provisions of D3.3.4.3d and D3.3.5.3c of the American Concrete Institute's (ACI's) *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)*[25] appendix D for anchorage to concrete.

**Accommodation of relative displacements** While load estimates are reasonably discussed in the design section of ASCE 7-10, provisions addressing drift compatibility of cladding panels are either too vague or too qualitative to provide specific guidance to design professionals detailing these systems. Specifically, ASCE 7-10 requires that "the connecting member itself shall have sufficient ductility and rotation capacity to preclude fracture of the concrete or brittle failures at or near welds."

The key drift-sensitive parameters to be designed for in a sliding connection are the slot and rod lengths. The slot length is determined by adding two times the expected story drift to the rod diameter and erection tolerance (which is usually 1 in. [25 mm]). As for the rod length, which is usually considered the clear length $L_c$ from the outer sliding surface to the panel (Fig. 4), a compromise needs to be found among varying issues. Practical considerations, such as allowance for construction tolerances, architectural finishes, and the speed of installation, tend to encourage designers to detail sliding connections with a larger rod length. However, if the rod is excessively long (and therefore flexible) it will have a tendency to bend well before sliding can occur. Thus, the key unresolved issue in the design of sliding connections is the maximum $L_c$ that can be provided to obtain a satisfactory seismic performance.

For flexing connections, the primary displacement-sensitive design parameter is the free rod length $L_f$, corresponding to the total length of the rod that is actually going to bend (Fig. 5). A rod that is too long might not fit due to architectural constrains, but a rod that is too short may fracture due to excessive cycle rotations. The outstanding question for the design of flexing connections is the minimum length $L_f$ that can be used to avoid fracture of the rod during seismic motion.

## Design of corner joints

Vertical corner joints of architectural precast concrete panels constitute a particularly critical location in the system because this is the point where panels moving in plane and out of plane interact. When a panel moves in plane, ideally it displaces rigidly with the lower slab. In the out-of-plane direction, panels are allowed to deform or tilt, thus moving at their bottom with the bottom slab and their top with the top slab. Clearly, in the absence of a gap as large as the story drift, the top of the two corner panels will impact (**Fig. 6**). This collision could result in connection failure and disengagement of the panel from the structure.

Until the 1990s, a ¾ in. (19 mm) joint at this intersection was expected to perform suitably; however, code changes have resulted in seismic joint widths that match the maximum expected inelastic drift of the structure (ASCE 7-10, section 13.5.3, part a). As a result, joint widths of 3½ in. (90 m) are not uncommon. These large joints running vertically at the

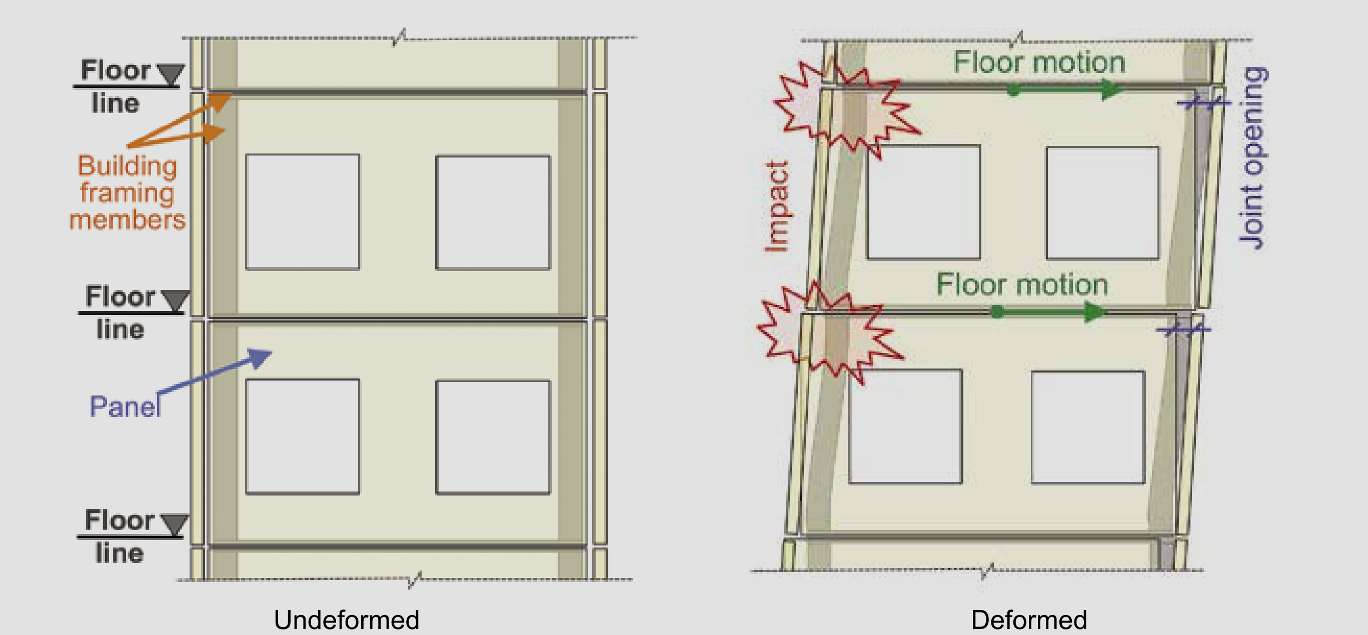



**Figure 6.** Architectural precast concrete cladding attached to a building frame as undeformed and deformed during seismic displacement.

corner of the building are architecturally unappealing (**Fig. 7**) and can make the market for architectural precast concrete cladding less desirable compared with other facades.

## Tests on architectural precast concrete panels and connections

A series of tests to study the seismic behavior of architectural precast concrete panels and connections was completed at two universities in collaboration with two commercial companies. These tests included one system-level experiment and two component test programs on flexing and sliding connections, respectively.

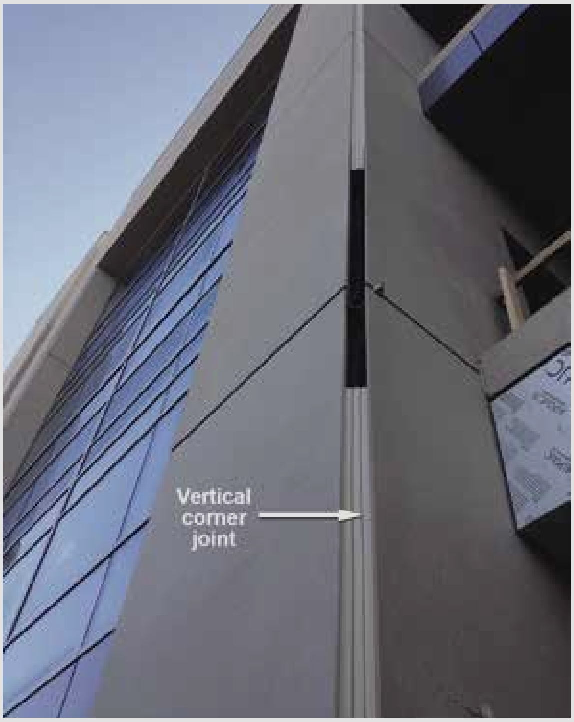


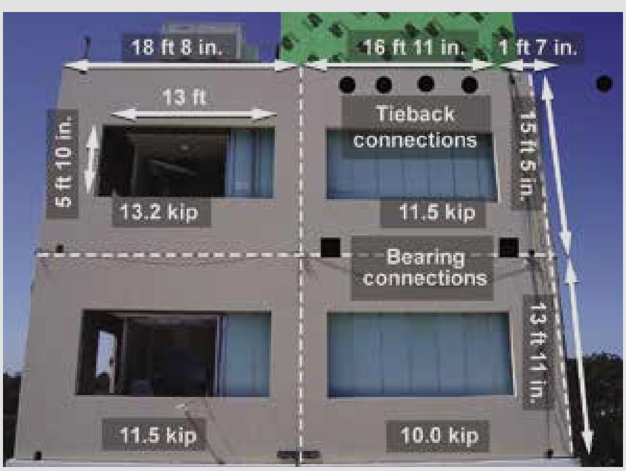

**Figure 7.** Oversized miter corner joint.

### System-level tests: Building nonstructural components and systems project

A five-story, full-scale, reinforced concrete building fully equipped with a large variety of nonstructural components and systems underwent a series of seismic tests on the large high-performance outdoor unidirectional shake table. **Figure 8** shows a photograph of the five-story, cast-in-place concrete, frame-braced building specimen, which was 75 ft (23 m) tall with uniform floor-to-floor heights of 14 ft (4.3 m). Including the installed nonstructural components and systems, the building weighed approximately 1400 kip (6200 kN), whereas its bare weight was 1100 kip (4900 kN). The test building had a fixed-base predominant period in the longitudinal (shaking axis) direction of testing of 0.9 seconds prior to significant damage accumulation.[26] The building was also tested while supported on base isolators; however, seismic demands within the structure were inconsequential to the architectural precast concrete system and are therefore not discussed in this paper. For brevity, only essential aspects of the test building and seismic test results are provided; additional details are presented in a series of technical reports and papers.[27–32]

The two upper floors of this building were completely encased by architectural precast concrete cladding. The design, construction, installation, and instrumentation of this facade system was undertaken by a team of researchers and precast concrete producers with the support of an


Whole building from the northwest corner




Architectural precast concrete panels installed on the south face

**Figure 8.** Building-nonstructural component specimen and systems building specimen. Note: 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

industry advisory board. Two punched-window wall panels per side of the building were installed at each floor, resulting in a total of 16 panels mounted on the test building. Figure 8 shows the geometry of the panels installed on the southern face of the building. Several variables were considered in this test program:

- Flexing and sliding tieback connections had three different lengths. All rods installed in the in-plane panels had a diameter $d$ of 0.75 in. (19 mm). The lengths, however, varied with $L_f$ for flexing connections and $L_c$ for sliding connections (**Tables 1** and **2**, respectively). All rods were ASTM A36 steel.

**Table 1.** Summary of $L_f$ and $L_f/d$ for flexing tieback connections during the building nonstructural components and systems experiment

| Short rod | | Medium rod | | Long rod | |
|---|---|---|---|---|---|
| $L_f$, in. | $L_f/d$ | $L_f$, in. | $L_f/d$ | $L_f$, in. | $L_f/d$ |
| 10.9 | 14.5 | 14.9 | 19.8 | 18.9 | 25.2 |

Note: $d$ = diameter; $L_f$ = free length. 1 in. = 25.4 mm.

**Table 2.** Summary of $L_c$ and ($L_c/d$) for sliding tieback connections during the building nonstructural components and systems experiment

| Short rod | | Medium rod | | Long rod | |
|---|---|---|---|---|---|
| $L_c$, in. | $L_c/d$ | $L_c$, in. | $L_c/d$ | $L_c$, in. | $L_c/d$ |
| 0 | 0 | 3.5 | 4.7 | 7 | 9.3 |

Note: $d$ = diameter; $L_c$ = clear length. 1 in. = 25.4 mm.

- A new type of corner connection allowed for smaller vertical corner joints.

- Miter and butt-return corner joints were considered.

**Test protocol** The building-nonstructural component specimen was subjected to 13 seismic input motion tests, first in a base isolated and then in a fixed-base configuration. The input motions applied by the shake table were unidirectional. Therefore, half of the panels experienced mainly in-plane motion, while the other half were predominantly subjected to out-of-plane motion. During the base-isolated phase of testing, a peak story drift $PD$ of 0.54 in. (14 mm), which corresponded to a peak story drift ratio of 0.32%, was measured at the second level, while a $PD$ of 0.25 in. (6.4 mm) (0.15%) was measured at the upper levels where the concrete cladding was installed. During the second phase of testing, six seismic input motion tests, denoted FB1 through FB6, were applied to the test building while it was fixed to the shake table.

**Figure 9** summarizes the absolute value of the achieved peak story drifts and drift ratios (drift normalized by floor-to-floor height) for these motions. Achieved story drifts were nominally small during the initial motions, while they reached peaks of 4.62 in. (117 mm) (2.75%) at the lower levels during FB5. Test FB5 achieved approximately the design-level earthquake demands in the building (target ~2.5%). Due to the mechanism forming in the building, smaller story drifts were measured at the upper levels, where the $PD$ reached 1.83 in. (46.5 mm) (1.09%) and 0.90 in. (23 mm) (0.54%) at the fourth and fifth levels, respectively. During the final test, a soft-story mechanism developed at the lower levels, with a $PD$ of 10.06 in. (255.5 mm) at the second level, corresponding to a drift ratio of 5.99%. As a consequence of this mechanism, the

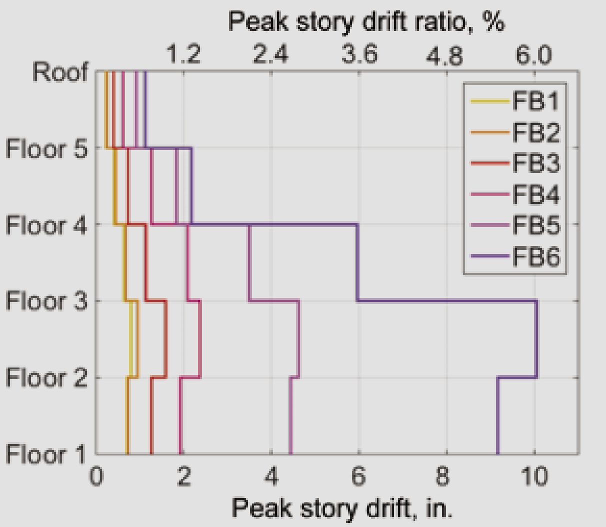



**Figure 9.** Peak story drift and peak story drift ratios measured during the building nonstructural components and systems fixed-base tests. Note: 1 in. = 25.4 mm.

upper levels experienced drifts lower than expected, with the *PD* reaching 2.17 in. (55.1 mm) (1.29%) and 1.10 in. (28 mm) (0.66%) at the fourth and fifth levels, respectively.

### Behavior of tieback connections (in-plane panels)

Following each seismic test, panels and connections were inspected. Typical damage to tieback connections installed in the in-plane panels corresponded to visible plastic bending of the steel rod. Because such damage is repairable and did not pose a threat to the safety of the occupants or function of the cladding, it was classified as moderate. In general, good performance of sliding connections with short and medium-length rods was observed. In fact, short connections did not show evidence of damage, and medium-length connections underwent moderate damage only on one occasion. Alternatively, long sliding connections performed poorly, with rods initially experiencing moderate damage during test FB1, where the peak story drift *PD* was 0.43 in (11 mm). Long sliding connections subsequently displayed plastic deformation during most tests, even when the compression plate was removed to minimize the potential for binding. In some instances, permanent deformations in these connections were larger than the *PD* achieved. This was likely due to the ratcheting of the connection, which developed during asymmetric connection binding and progressive unidirectional sliding upon repeated cycles of loading. **Figure 10** shows an example of moderate damage to long sliding connections.

Flexing connections generally showed much better performance than sliding connections. In the case of flexing connections, plastic deformation was caused by the bending of the rod and sliding of the rod inside the oversized installation hole, which minimized permanent displacement from developing in the panels. These observations led to the conclusion that for both types of connections the actual working mechanism involved both sliding and bending. Long flexing rods achieved moderate damage only after the strongest motions, when a *PD* of 2.17 in. (55.1 mm) was achieved, corresponding to an $(L_f/d)/PD$ of 11.6 in.$^{-1}$ (294 mm$^{-1}$). Medium-length rods reached plastic deformation in several instances, for *PD* as low as 1.10 in. (28 mm) and $(L_f/d)/PD$ equal to 18.0 in.$^{-1}$ (457 mm$^{-1}$). Figure 10 shows an example of moderate damage to medium-length rods. No damage was observed in the short flexing connections. Fracture of the rod was not observed, even in the strongest motion, where the *PD* was 2.17 in., corresponding to $(L_f/d)/PD$ of 11.6, 9.1, and 6.7 in.$^{-1}$ (294, 231, and 170 mm$^{-1}$) for long, medium-length, and short rods, respectively. Further information about observed damage in the in-plane panels can be found in Pantoli et al.[33]

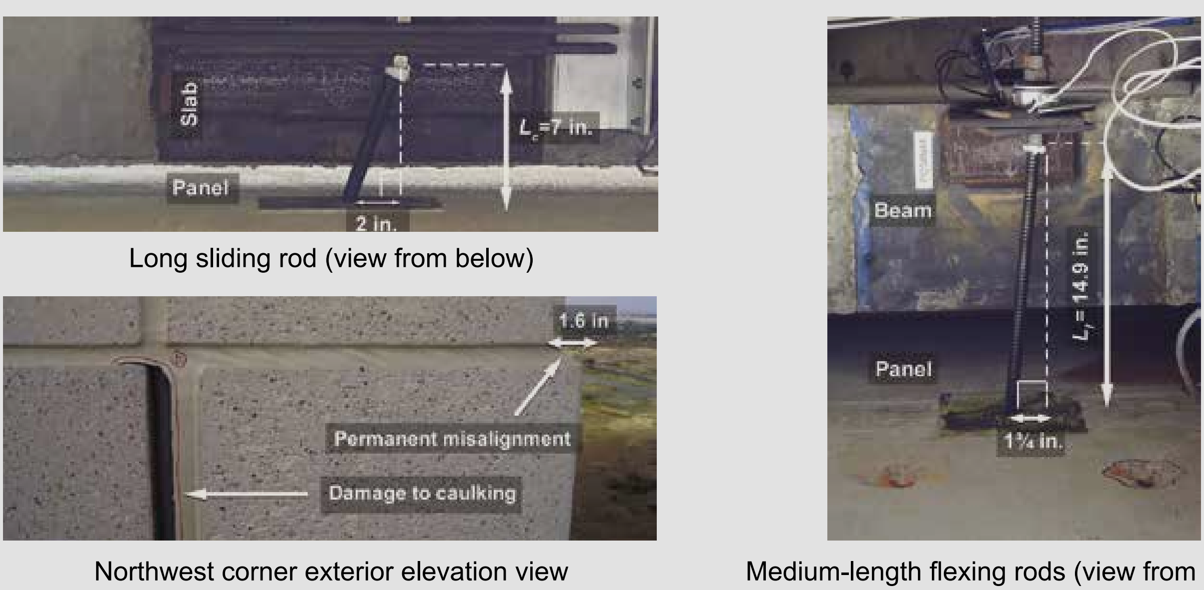
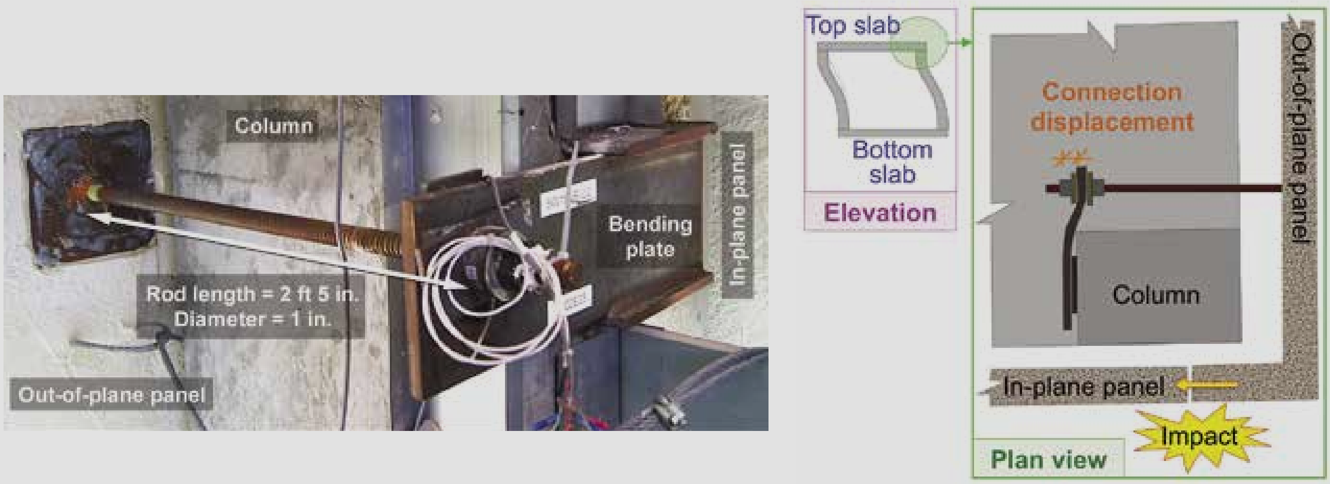



**Figure 10.** Example of damage to architectural precast concrete cladding. Note: $L_c$ = clear length; $L_f$= free length. 1 in. = 25.4 mm.




Installed condition

Conceptual schematic showing the desired behavior of a corner during impact

**Figure 11.** Corner connection with ductile fuse. Note: 1 in. = 25.4 mm.

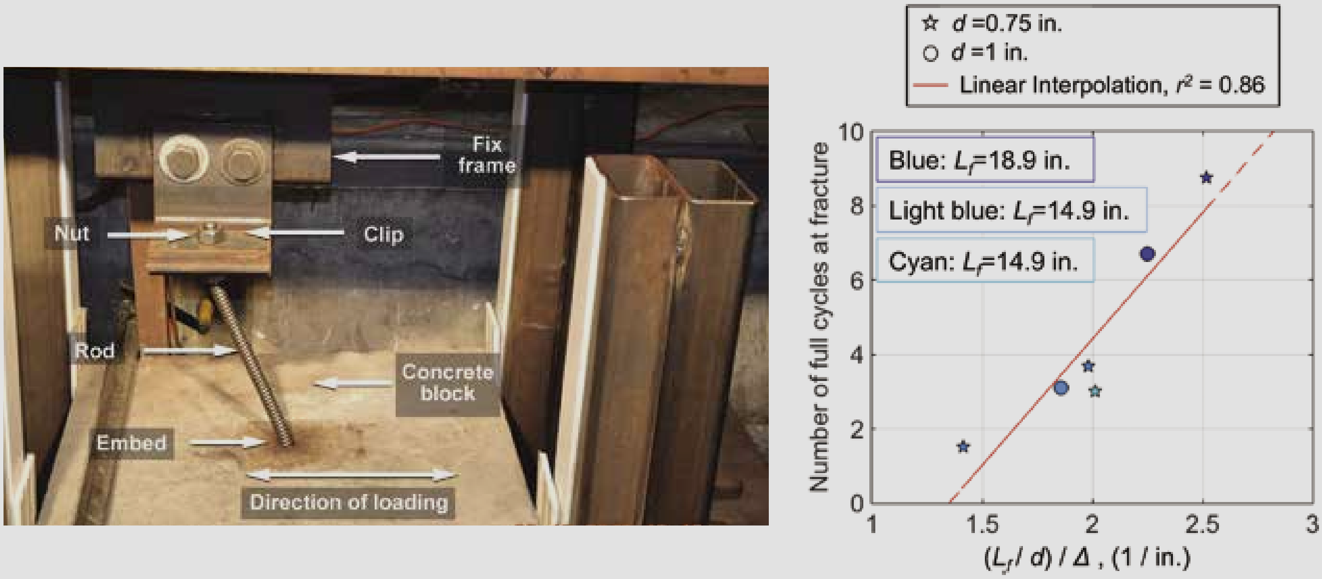
**Behavior at corner panel intersections** Current design imposes the use of corner-joint widths compatible with design-earthquake anticipated inelastic story drifts. As a result of the present practice, impact of panels at corner joints is avoided. However, large joint widths are unappealing for architectural reasons. As an alternative, a new corner connection allowing for smaller corner joint widths was designed and tested during this project. By using this new corner connection, the vertical joints can be sized to match the predicted elastic story drift of the building rather than the inelastic story drift. When the system achieves design-earthquake inelastic demands, the associated larger-story-drift amplitudes will cause corner panels to collide. At that point, the body of the corner connection will yield and deform, allowing the panels to remain in contact while the drift magnitudes increase. This ductile fuse mechanism was detailed using a cantilever plate designed to yield under bending with a flexural capacity selected to prevent overload of other parts of the connection. **Figure 11** shows a photograph of this connection and a schematic of the corner system as it is intended to behave during inelastic drift demands.

These tests demonstrated the generally good behavior of the newly proposed corner joint configuration. However, the response of the system was dependent on the type of corner joint. Panels that intersect at butt-return joints activated plastic bending of the ductile plate, while panels intersecting at miter joints developed cracks in the corner area of the architectural precast concrete panels and the ductile plate did not achieve visible plastic deformation. It is likely that this behavior is attributed to the additional stiffness created by the return, which allowed the panel to transmit the forces to the connection without cracking. Both yielding of the ductile fuse and cracking of the panel created tearing of the caulking and permanent misalignment of the cladding (Fig. 10). Further information about the behavior of the corner system can be found in Pantoli et al.[34]

## Component tests on flexing connections

Component tests on flexing connections were performed.[35] For these tests, typical flexing connections were replicated in the laboratory and rotated 90 degrees (**Fig. 12**). Using this configuration, a concrete block was hung from the rod to create a predefined tension, thus simulating field loading conditions. Tests were conducted using commercially available ASTM A36 steel coil rods of either ¾ or 1 in. (19 or 25 mm) diameter with free lengths $L_f$ varying from 10.9 to 18.9 in. (48.5 to 84.1 mm) ($L_f/d$ between 10.9 and 25.2). During each test, a pseudostatic cyclical displacement targeted to achieve either a constant or increasing amplitude was applied to the rod until it fractured. The primary goal of this testing program was to determine the number of cycles resisted by the rod as a function of the displacement experienced. Key observations from these tests include the following:

- Good correlation between the number of cycles of constant displacement applied at fracture $\Delta$ and the ratio $(L_f/d)/\Delta$ was observed, with fewer than two cycles associated with fracture for $(L_f/d)/\Delta$ equal to 1.4 in.$^{-1}$ (35 mm$^{-1}$) and roughly nine cycles needed to achieve fracture for $(L_f/d)/\Delta$ equal to 2.5 in.$^{-1}$ (63 mm$^{-1}$) (Fig. 12). These results were created by applying a constant (target) amplitude displacement, which is clearly a simplification compared with real earthquake histories, as was applied during the system-level test program.
- The ability to resist several cycles of large displacement is related to prior load history.
- The fracture zone was restricted to a small length of the rod.




Test setup

Number of cycles to fracture versus $(L_f/d)/\Delta$

**Figure 12.** Flexing connection component test setup. Note: $d$ = diameter; $L_f$ = free length; $r^2$ = coefficient of determination; $\Delta$ = constant displacement amplitude applied in the component tests on flexing connections. 1 in. = 25.4 mm.

- The rods continued to resist several cycles of large-amplitude inelastic displacement, indicating good ductility of the material.

### Component tests on sliding connections

Component tests on sliding connections were performed. Similar to the flexing connection test configurations, typical sliding connections, a pair in each test setup, were reproduced and rotated 90 degrees to apply axial tension to the rods (**Fig. 13**). Rods were ASTM A36 steel coil rods with a diameter of either ¾ or 1 in. (19 or 25 mm); an $L_c$ varying from 2.5 to 7.3 in. (64 to 185 mm); and $L_c/d$ ratios of 3.3, 5.3, and 7.3. Dynamic displacements were applied to the connections via a shake table that was fixed to the concrete block with a wooden frame. The objective of these tests was to evaluate the influence of various parameters on the performance of sliding connections.

During the first phase of the experiment, two short rods were tested and the force applied to the sliding surfaces was progressively increased by tightening the connection nuts. It was found that even a slight increase (~150 lb [667 N]) of compression force created binding of the rods. To prevent this, during the second phase of testing, the compression plate washer and nut were eliminated. In this phase of the experiment, six different rod configurations characterized by different lengths and diameters were tested using five seismic motions (denoted M1 through M5). Figure 13 shows a summary of the damage states observed for each configuration, classifying residual bending of the rod as minor damage if revealed only during postprocessing of the data and moderate if clearly visible.

Key observations from these tests include the following:

- Sliding connections are highly sensitive to the normal force applied to the sliding surfaces. Such forces are difficult to control during dynamic horizontal loading, such as is imposed during an earthquake. When this normal force becomes too large, connections tend to bind and bend rather than slide. The bending of the rod creates a further normal force between the compression plate washer and the clip, producing a clamping mechanism that renders the sliding even more unlikely. In some cases, these connections can also exhibit a ratcheting behavior.

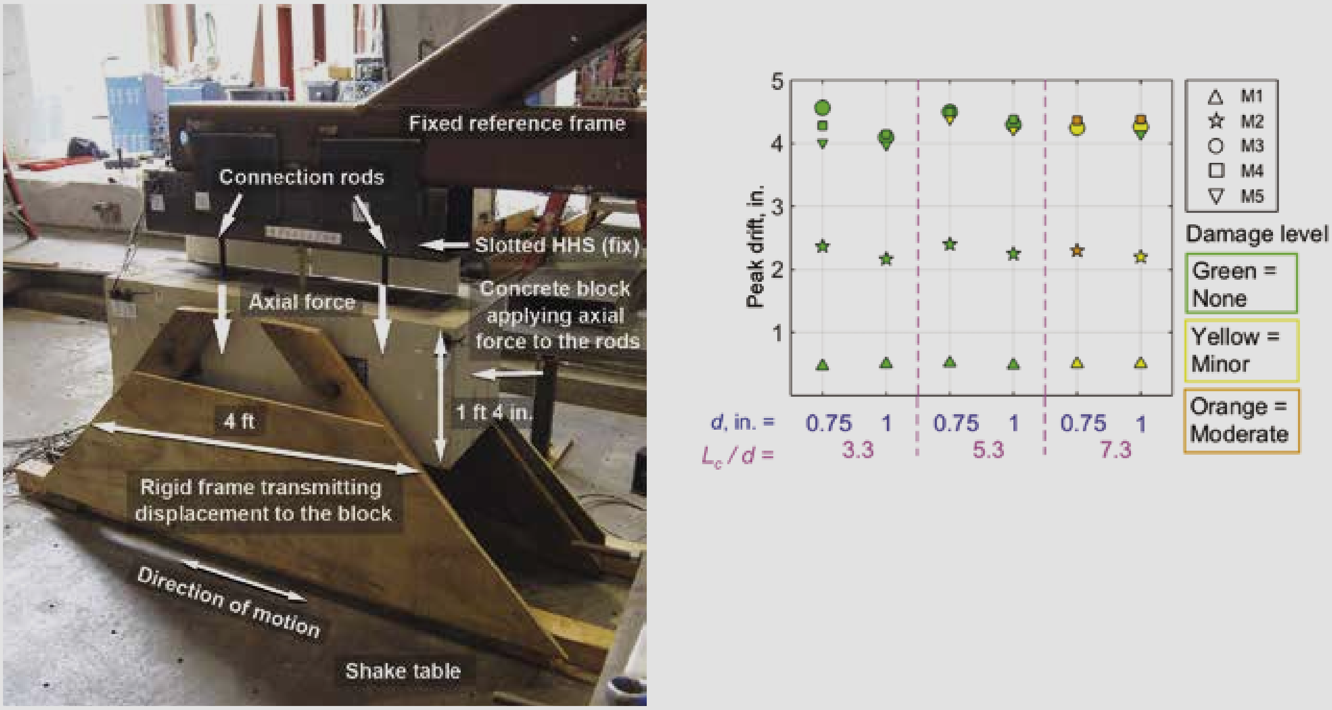
- Removing the compression plate can improve behavior, decreasing the possibility of binding the connection.

- Even when the compression plate washer is removed, plastic deformation of the rod after the motion starts observed for $L_c/d$ was larger than 5.3.

Additional information regarding results from these tests may be found in Pantoli et al.[36]

## Conclusion

Architectural precast concrete cladding is a common type of building facade used worldwide. Its seismic protection is based on the decoupling of the panels from the building in the in-plane direction, which is achieved by allowing the panels to translate or rotate rigidly with the bottom floor through the use of tieback connections at the tops of the panels. These connections are designed to absorb forces




Test setup

Damage states observed after the second phase of testing

**Figure 13.** Experiment on sliding connections. Note: $d$ = diameter; $L_c$ = clear length. 1 in = 25.4 mm; 1 ft = 0.305 m.

in the out-of-plane direction while allowing in-plane drifts by either the sliding or the flexing of a steel rod. While their design for seismic forces is well addressed in present design codes, the design of tieback connections and corner joints to accommodate drift is addressed only in a qualitative form.

With the goal of providing practical guidance to architectural precast concrete cladding designers, a group of researchers and precast concrete industry practitioners supported by an industry technical advisory board embarked on an experimental program focused on assessing the behavior of architectural precast concrete panels and their connections. This test program included a system-level experimental phase in which 16 architectural precast concrete panels were installed on a full-scale, five-story building shake table tested on a large high-performance outdoor shake table and companion component tests on tieback connections. This paper highlights each of these test programs and synthesizes the key findings that influenced the subsequent development of a design procedure document for practitioners.[23]

- **Flexing tieback connections.** During the building-nonstructural components and systems project, it was observed that flexing rods benefit from combined modes of bending and sliding. Where sliding is facilitated within the oversized installation hole, which is incorporated to facilitate construction tolerances at the connection clip. When sliding is activated, it can result in a permanent deformation of the rod even when the panel does not have any residual displacement. Both the building nonstructural components and system tests and the component tests of flexing connections indicate that the free length of the rod $L_f$, its diameter $d$, and the relative displacement absorbed by the connection are key variables in determining the performance of the connection. To harmonize the latter with the present design code, the seismic relative displacement is defined as $D_{pl}$. Therefore, these tests indicate that good performance of flexing connections can be achieved by providing a $(L_f/d)/D_{pl}$ larger than 6.0 in.$^{-1}$ (152 mm$^{-1}$). The rod material must also be ductile—for example, a low-carbon, mild steel of ASTM A36 grade.

- **Sliding tieback connections.** During both the building nonstructural components and system project and the component tests, it was observed that when sliding rods are excessively flexible or the frictional resistance on the sliding surfaces is too high, rods tend to bend rather than slide. As the rod bends, its rotation creates a clamping force between the compression plate washer and the slotted clip, which makes sliding less likely. Sliding connections can also exhibit ratcheting behavior, accumulating deformations in one direction while sliding in the other, and thus undergoing a plastic deformation much larger than the maximum drift experienced by the connection. Rods that are designed to slide but instead deform in a bending mode can achieve large rotations at low story drift amplitudes, making fracture of the rod suspect. Several modifications are needed to improve the performance of sliding connections. The simplest strategy is to eliminate or significantly minimize the rod's free length. In these tests, snug sliding connections consistently demonstrated good performance. Alternatively, the compression plate washer can be removed, thus minimizing the potential for the development of clamping forces at the connection. Sliding connections absent a compression plate washer responded in a reliable sliding mode for all clear lengths $L_c$ and diameters $d$ tested; however, they also exhibited combined sliding and plastic bending deformations at high $L_c/d$ values. For this reason, it is recommended that $L_c/d$ be limited to 5.3. Removing the compression plate clearly creates a tension connection only, and compression loads will have to be resisted by some other mechanism or connection.

- **Corner system.** During the system-level test, a new corner connection characterized by a ductile fuse mechanism was designed and tested. In this system, corner joints were sized to match the predicted elastic story drift of the structure, and as a result, corner panels were intended to impact during inelastic drifts. Forces that would otherwise be generated by this collision were not transferred to the remaining portions of the connection due to the presence of a capacity-limited cantilever ductile bending plate. This new corner system worked well in the case of butt-return joints because the return stiffens the panel and allows the transmission of the impact forces to the connection without cracking the panels itself. During large story drifts, moderate damage of the corner areas, including tearing of the caulking and cracking of the panels, was observed.

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## Notation

$a_p$ = component amplification factor

$d$ = diameter

$D_{pI}$ = seismic relative displacement

$F_p$ = seismic force

$h$ = average roof height of the structure with respect to the base

$I_p$ = importance factor

$L_c$ = clear length

$L_f$ = free length

$PD$ = peak story drift

$r^2$ = coefficient of determination

$R_p$ = response modification factor

$S_{DS}$ = spectral acceleration at short periods

$W_p$ = weight

$z$ = height in the structure of the point of attachment of the component

$\Delta$ = constant displacement amplitude applied in the component tests on flexing connections

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## Abstract

Architectural precast concrete cladding is a nonstructural system sensitive to both seismic floor accelerations and story drifts. Architectural precast concrete panels must be designed to resist forces in the out-of-plane direction of motion while accommodating in-plane story drifts. This requirement presents specific challenges to engineers, namely in terms of the design and detailing of connections intended to allow in-plane drifts and corners of the system. Presently, these important issues are addressed rather broadly in design codes, leaving the details to the discretion and experience of the designer. With the goal of providing practical guidance to designers, system and component tests on representative architectural precast concrete cladding and tieback connections were performed. This paper summarizes key results obtained from these experiments and applies these findings to developing guidelines for the drift-compatible design of tieback connections and corner systems.

## Keywords

Architectural precast concrete, cladding, connection, corner joint, drift, facade, fixed bearing, seismic floor acceleration, story drift, tieback.

## Review policy

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