SHAKE TABLE TESTING OF A FULL-SCALE FIVE-STORY BUILDING: SEISMIC PERFORMANCE OF PRECAST CONCRETE CLADDING PANELS

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ABSTRACT

A full-scale five-story building was tested on the unidirectional George E. Brown Jr. Network for Earthquake Engineering Simulation (NEES) Large Outdoor High-Performance Shake Table at the University of California, San Diego. The building was equipped with a wide range of nonstructural components including a functioning passenger elevator, stairs, partition walls, external cladding, piping, hospital equipment, fire sprinklers and a roof mounted cooling tower. The structure was seismically tested in two phases: (i) base isolated and (ii) fixed-base. The two upper levels of the building were fully enclosed with sixteen precast concrete cladding panels fastened to the structure according to modern precast construction practice in seismically active regions. Nine different types of connections, including one newly developed, were used to connect the panels to the building. Two different corner joint configurations were implemented. The cladding and its connections were closely monitored with accelerometers, displacement potentiometers, load cells and video cameras. This paper will focus on the design and behavior of a novel connection at the corners of the precast concrete cladding system. This connection utilizes a flat steel plate that acts as a ductile fuse allowing the use of smaller joints between panels.

Keywords: Precast concrete cladding panels, full scale experiments, push-pull connections, corner joints, seismic response

INTRODUCTION

Recent earthquakes have demonstrated that damage to nonstructural components and systems (NCSs) in buildings pose life safety hazards to the building occupants and lead to significant economic losses and repair downtime[1]. A particularly important and sensitive NCS is that which provides the exterior enclosure. Not only does this subsystem represent a significant portion of the cost of the building, varying between 9% and 18% of the total cost for different types of buildings[2], but exterior enclosures have also realized extensive damage in past earthquakes[3]. A type of façade commonly used worldwide is the precast concrete cladding (PCC) panel system. In this type of façade, concrete panels are fabricated at a precast facility and brought to the site just prior to installation. They are attached to the building with steel connections that must provide a load path to the structure transferring not only the weight of the panel, but also any lateral forces (wind/seismic) imposed on the panels. Connections that resist out-of-plane forces only are called *push-pull* or *tie-back* connections. The connection system, however, must allow relative motion of the panel and structure due to the horizontal building displacements – both in-plane and out-of-plane – during seismic motions. Two types of push-pull connections are used to do this, namely, sliding connections and flexing rod connections. Because of the different behavior of the panels in the in-plane and out-of-plane direction, corners where they meet are particularly critical. All PCC panel joints should be as small as possible for aesthetic reasons but in practice, large, unsightly joints are introduced at the corners to prevent panel collisions and connection overloading.

Historically, this façade type has performed well in earthquakes – including the 1994 Northridge event, which resulted in large story drifts in buildings with limited damage to precast cladding systems[4]. Nonetheless, instances of damage to these panel systems have been reported in several earthquakes. Recently for example, during the Christchurch earthquake in 2011 in New Zealand several panels failed due to inadequate detailing. Moreover, extensive cracking, corner crushing, residual displacement of the panels and rupture of the seal at panel interfaces were reported[3]. During the Chile earthquake in 2010, several PCC panels collapsed in the out-of-plane direction. In one case, the cause of collapse was local bending failure of the flanges of an embedded anchor channel, which allowed pullout of the sliding bolt[5]. Several pullout failures of tie-back connections were also observed following the L'Aquila earthquake in Italy in 2009 [6]. Despite these and other field evidence, as well as the knowledge of the importance of these systems, large scale testing of the PCC panels, particularly integrated within a building system, has been limited[7,8].

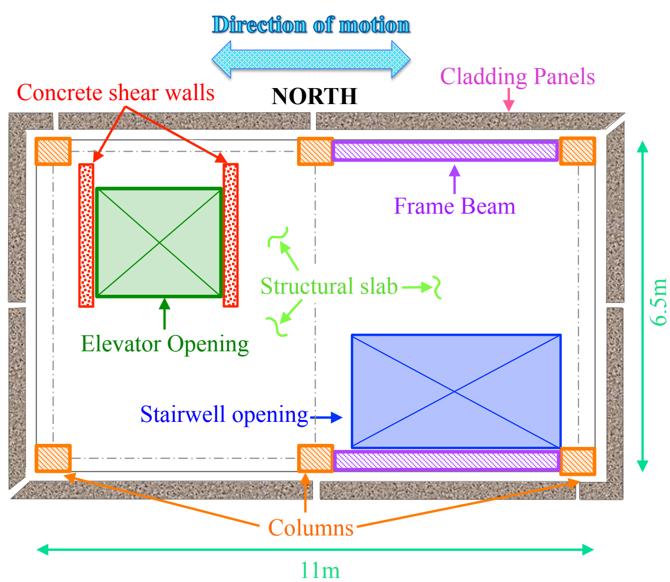
SCOPE OF THIS PAPER

In April and May 2012, a landmark test of a five-story building constructed at full-scale and completely furnished with nonstructural components and systems (NCSs) was conducted at the University of California, San Diego (UCSD) Network for Earthquake Engineering Simulation (NEES@UCSD, 2013) facility. The project, coined *Building Nonstructural Components and Systems (BNCS)* was realized by a unique collaboration between Academe, Industry and Government and hundreds of individuals with expertise in structural and nonstructural design, earthquake engineering, and construction and management practices[9]. The full-scale building-NCS system was seismically tested in a base isolated and fixed based configuration on the Large High-Performance Outdoor Shake Table (LPOSHT) at the UCSD-NEES facility. Wrapping the exterior of this building were two types of façades, namely light weight metal stud balloon-framing overlaid with a synthetic stucco finish (first three floors) and precast concrete cladding panels (PCC panels) (two upper floors).

This paper will focus on the PCC panels installed at the upper floors and namely describe the behavior of the panels at the corners of the building. This location was characterized by the presence of a newly developed type of push-pull connection with a ductile fuse that would allow for smaller and more aesthetically pleasant corner joints. In addition two different types of corner joints were tested.

DESCRIPTION OF THE EXPERIMENT

A poured in place reinforced concrete five story building was fully equipped with a wide range of NCSs, including a fully functional passenger elevator, stairs, mechanical and electrical services, ceiling and piping subsystems, as well as roof mounted equipment. The overall height of the specimen, including its foundation, was 22.8m, and its overall plan area occupied the full area of the shake table platen with a length of 11.5m and a width of 7m. The bare structure had an estimated weight of 3010 kN, excluding the foundation, which weighed 1870 kN. Including the NCSs, the building weighed approximately 4420 kN. The floor plan was characterized by the presence of two large openings (one for the stairs and one for the elevator) and two walls encasing the elevator shaft (Fig. 1). It is important to underline that the LHPOST at NEES@UCSD allows movement only in the East-West direction (longitudinal direction of the building). Two bays in the longitudinal (shaking) direction and one bay in the transverse direction provided the load bearing system. Lateral seismic resistance in the primary shaking direction was provided by a pair of identical one-bay special moment resisting frames in the Northeast and Southeast bays.



(a) (b)

Fig. 1 General views of the test building: (a) photograph of the North-West sides of the building and (b) plan view of a typical floor

While fixed to the shake table platen, the building was subjected to a suite of six earthquake motions of increasing intensity: the first two motions were serviceability level motions, the third and the fourth were the same long duration record amplified at 50% and 100% intensity, the fifth motion was meant to be a design earthquake (DE), while the last motion was a targeted MCE level motion (Table 1). In some cases the actual motion (AM) was used as the target, while in other cases the original motion was spectrally matched (SM) to the ASCE 7-10 design spectrum assuming a high seismic zone in Southern California (site class D).

Station-scale (Earthquake)	Name	Type	Notes
Canoga Park-100% (1994 Northridge)	FB-1:CNP100	SM	Serviceability level
LA City Terrace-100% (1994 Northridge)	FB-2:LAC100	SM	Serviceability level
ICA-50% (2007 Pisco-Peru)	FB-3:ICA50	AM	Long duration, multiple runs
ICA-100% (2007 Pisco-Peru)	FB-4:ICA100	AM	Long duration, multiple runs
Pump Station #9-67% (2002 Denali)	FB-5:DEN67	SM	~Design Earthquake (DE)
Pump Station #9-100% (2002 Denali)	FB-6:DEN100	SM	~Maximum Considered Earthquake (MCE)

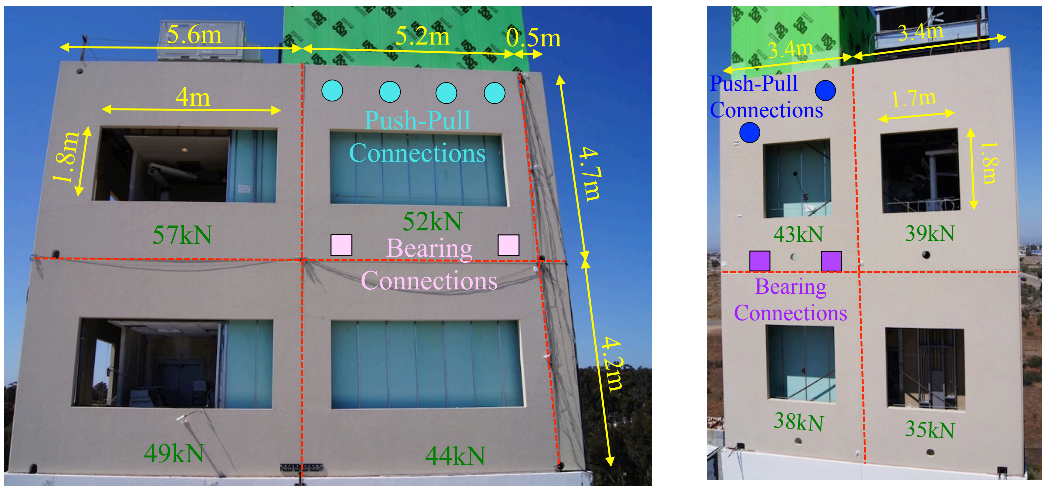
Table 1: Seismic test protocol imposed while the building was fixed to the shake table.

DESCRIPTION OF THE PRECAST CONCRETE CLADDING PANELS

Through support of the Charles Pankow Foundation and an industry advisory board within the Precast Concrete Institute (PCI), a team of researchers and precast concrete producers worked closely on the design, construction, installation, and instrumentation of the precast concrete panels tested within the BNCS building specimen.

OVERVIEW

Panels selected for this test program were punched window wall units, meaning they spanned from floor to floor with openings provided only for windows. Two panels per side of the building were installed at two floors, resulting in a total of 16 panels mounted on the test building. Eight panels translate predominantly in the in-plane direction (denoted as *IP* panels) and eight panels tilt predominantly in the out-of-plane direction (denoted *OP* panels). Connection of the panels to the building skeleton were facilitated by steel embeds installed in the slab, beams and columns. The panels were supported by two bearing connections at the bottom welded to embeds in the floor slab or beam and push-pull connections at the top. Each of the larger IP panels had four push-pull connections welded to embeds in slabs and beams. The IP panels on the western half of the building were connected at the top of the panel to the building via sliding type push-pull connections, while the IP panels on the eastern half were connected via flexing rod push-pull connections. Each OP panel had only two upper connections: the inner connection was either a sliding of a flexing rod connection and it was welded to a slab embed while the outer (corner) connection was a newly developed push-pull connection with a ductile fuse attached to a column embed. The IP panels had an average dimension of 5.4m x 4.4m and an average weight of 50 kN, while the OP panels were smaller, with an average dimension (not considering the return corner) of 3.4m x 4.4m and an average weight of 39 kN. All panels were 125 mm in thickness. Details of the panel geometry can be found in Fig. 2. Two different types of corner joints were tested, namely, miter joints and butt return joints. Miter joints were installed in the South-West and North-East corners, while butt joints were installed on the North-West and South-East corner. Nomenclature of the panels on the 4th floor and location of the different types of corner joint are shown in Fig. 3.



(a) (b)

Fig. 2 View of the panels showing the geometry and the typical location of the connections: (a) IP panels on the South side and (b) OP panels on the East side

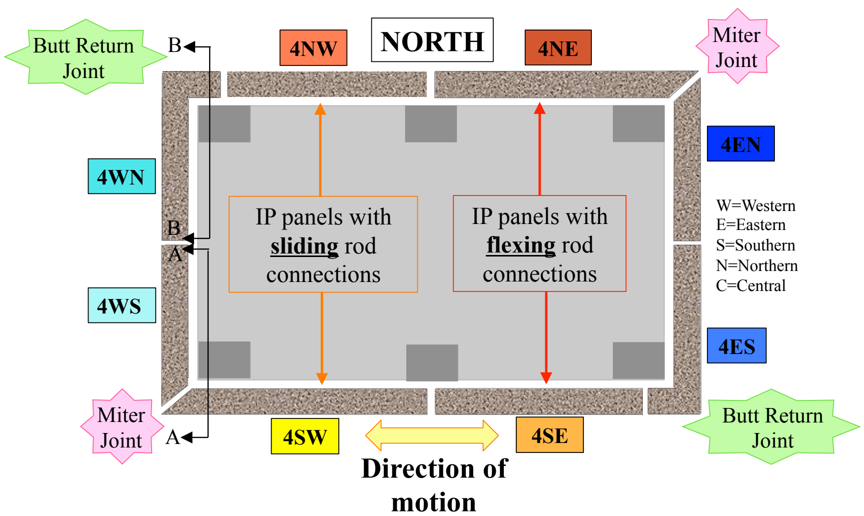


Fig. 3 Nomenclature used to identify the panels on the fourth floor.

PARAMETERS OF SPECIFIC INTEREST IN THE TEST PROGRAM

The corner connections of the PCC panels constitute a particularly critical location in the system because this is the point where the IP panels and OP panels meet. Ideally, the IP panels move rigidly with the lower slab while the upper portions of the OP panels are attached to and move with the upper slab (in fact these panels may flex and also tilt about their weak axis). The relative motion of the tops of the two panels is essentially the same as the relative motion of the upper and lower floors.

The current state of practice is to oversize the relevant panel joints to prevent panel collisions at the corners of the building during a large seismic event, but this may result in an unappealing reveal on the building exterior. For this reason, in this test program a new connection with a ductile fuse allowing for smaller corner joint was explored and installed at each corner connection. In this new corner system the floor-to-floor relative displacements are (ideally) absorbed as follows:

- *Elastic drifts* are absorbed by the closing of the vertical corner joint, with the joint sized sufficiently to avoid impact; and
- *Inelastic drifts* are larger than the vertical corner joint and therefore intended to result in impact of the joint. However, upon impact the ductile fuse is designed to prevent connection overload through the fuse mechanism, ensuring that the panels remain attached to the building after the event. In this work, the ductile fuse was in the form of a cantilevering bending plate that deformed during impact. At the same time, it was sized to avoid damage to the other parts of the panel/connection. A picture of this connection is shown in Fig. 4a. A schematic of the corner system during inelastic drift demands for the South-East (SE) corner is shown in Fig. 4b and c.

PANEL DESIGN

Panel design, construction, and installation were performed by a U.S. West-Coast precaster with expertise in precast concrete cladding systems. Structural design criteria and detailing conformed to requirements of ASCE 7-10 ACI 318-08, PCI MNL 120-04 and ANSI/AISC 360-05[10-13] for regions of high seismicity. Nonlinear time history analyses of the building were conducted and used to predict interstory drift ratios (IDRs) anticipated during design and maximum credible earthquake events[14] and thereby size the joints and design the slotted and flexing rod connections. Design forces were estimated using the building target S_{DS} and the linear force distribution estimate of ASCE 7 (equations 13.3-1).

CORNER SYSTEM DESIGN

Vertical Joint Size

For a conventional design, the joints are sized to accommodate the maximum ID. During this test, the design drift at the fourth floor was 85mm. With the ductile fuse design method, the joint is sized for elastic drifts. Therefore, the design inelastic drift predicted was reduced by dividing the drift by the code deflection amplification factor C_d (=5.5 for special moment concrete frames), resulting in a minimum gap size of 13mm. Both types of corner joints were sized to allow this drift, with a joint size selected as 25mm for the butt-return joint and 19mm for the miter joint (ID allowed~27mm due to the inclination of the joint).

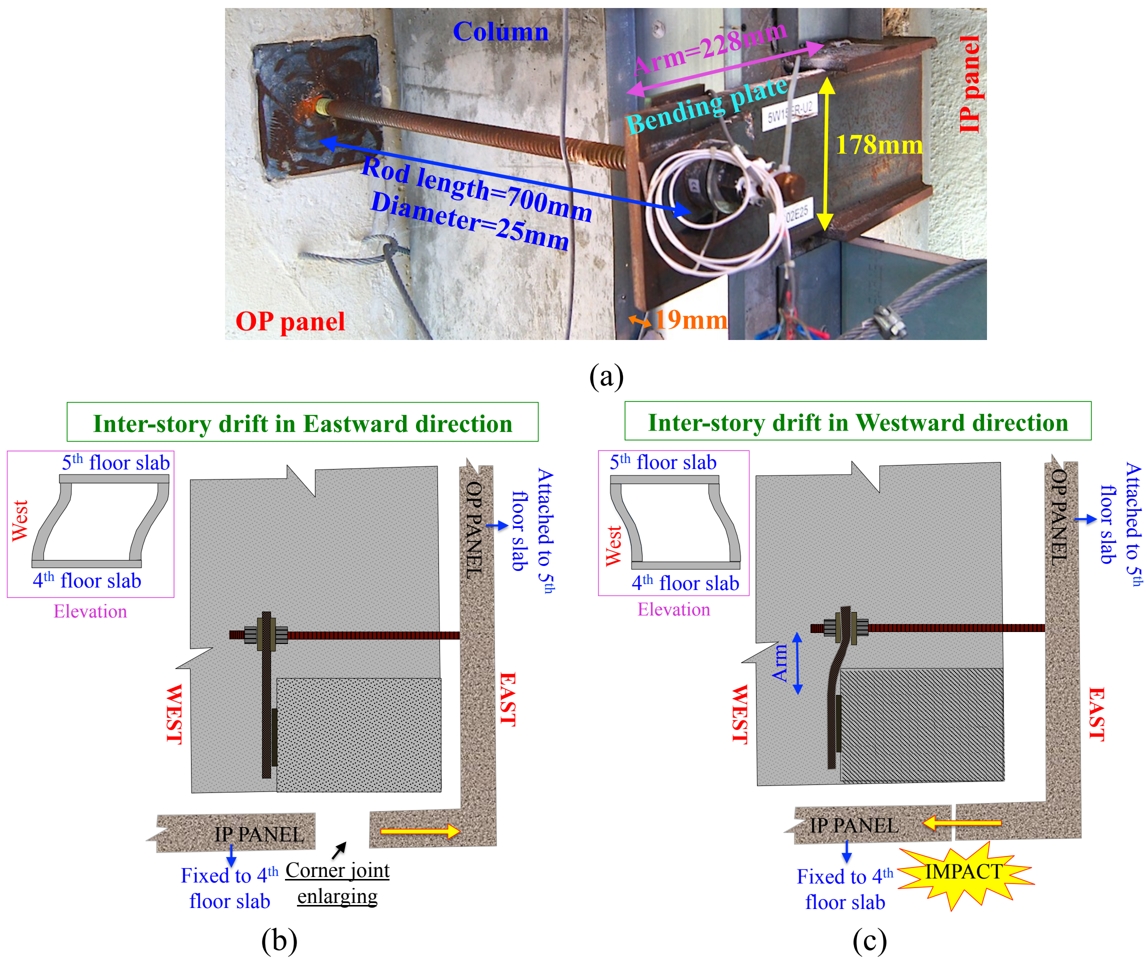


Fig. 4 Push-pull corner connection with ductile fuse on the OP panels (a) photograph, (b), (c) conceptual schematic showing the desire behavior of a corner connection in the 4ES panel during Eastward and Westward motion

Inertial Forces in the Connections

The design of the new corner connections with ductile fuse used the ASCE 7 equation for estimating forces to components placed at elevation within a building (Equation 13.3.1):

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

Where; S_{DS} = short period spectral acceleration (= target 1.59 for this building), z = height above grade of the connection, h = total height of building, a_p = component amplification factor, R_p = component response modification factor reduction factor, I_p = importance factor, and W_p = component weight. Consider an example at the fifth floor (z = 17.06m, h = 21.33m, a_p = 1.0 for bodies of connection and 1.25 for fasteners of the connection, R_p = 2.5 for bodies of connection and 1.0 for fasteners of the connection, I_p = 1.0, and W_p = 15kN), results in an F_p for the body and fasteners of the connection calculated as 9.9kN and 31kN, respectively.

Ductile Fuse Design

The maximum flexural capacity of the cantilevering plate, including strain hardening effects and expected material over strength values, may be estimated as:

$$M_{p\text{-}expected}=1.1\cdot R_y\cdot F_y\cdot Z \tag{2}$$

Where; R_y= Ratio of expected yield stress to the specified yield stress (e.g. AISC 341, Table A3.1 for various materials, or for A36 Hot-rolled shapes and bars, R_y = 1.5), F_y = yield strength (= 248 MPa) and Z = plastic section modulus = ($h*t^2/4$) = ($178*(19)^2/4$) = 16064mm^3 where h is the height of the plate and t in the thickness. The maximum value of the force in the rod is then obtained by dividing the moment by the arm (=228mm), for this example resulting in an F_{max} = 29kN. Ductility is provided by ensuring the expected plastic flexural capacity of the plate is less than the capacity of the welds, the rod in tension, and the concrete anchorages of the embeds in the column and the panel itself.

PANEL PROPERTIES

Panels were 127mm thick and reinforced with #4 bars A615 Grade 60 rebar spaced at 300mm on center. Additional #5 bars were added around window openings, and #3 horizontal bars at 150mm o/c were used in piers adjacent to windows. The specified compressive strength of concrete at 28 days was 34MPa with a unit weight of 2400 kg/m^3. Coil rods and bolts were ASTM A108 steel with yield and ultimate strengths of f_y = 410MPa and f_u = 550MPa, respectively.

INSTRUMENTATION

A total of 65 analog sensors monitored the behavior of the cladding panels. In relation to the corner system it was monitored:

- The relative displacement of the OP panel respect to the structural column at the location of the push-pull connection with ductile fuse (PPDF): the sensor (string potentiometer) measuring this displacement is shown in Fig. 5. This sensor was installed as close as possible to the connection so that its measurement provides only the part of displacement absorbed by the connection itself (namely due to the bending of the plate since the axial deformation of the rod is a much stiffer mechanism);
- The force in the connection rods: the load cell measuring the force in the rod is indicated in Fig. 5 and it was able to measure only tension in the rod.

- The in-plane displacement of the top of the IP panels respect to the top slab: this measurement is very important because it indicates the actual movement of the IP panel.

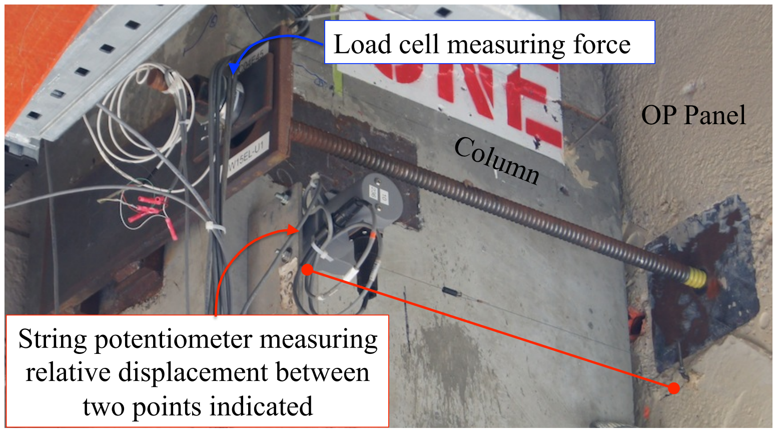
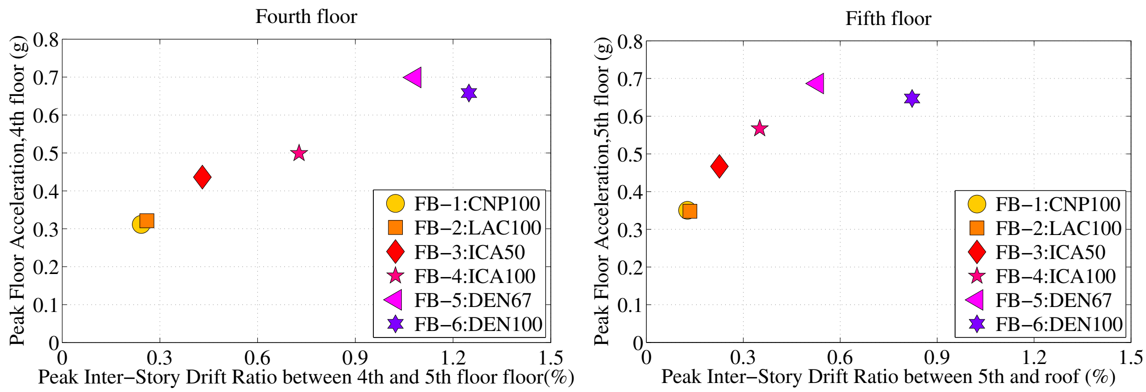


Fig. 5 Analog sensors monitoring the behavior of the behavior of a PPDF connection

Due to the large number of panels and connections compared to the number of sensors available, measurement locations were concentrated in the South-East corner of the building, as this was considered the most flexible. In addition to the analog sensors, four video cameras were installed to monitor the behavior of the panels during the FB testing: these recorded the behavior of the two types of corner joints, namely a PPDF connection and a flexing rod connection.

GLOBAL BUILDING RESPONSE

Fig. 6 shows the peak floor accelerations (PFA) and peak interstory drift ratios (PIDR) of the building at its fifth and fourth floor recorded during the FB motions. It is noted that accelerations were measured at every corner of each floor of the building, and displacements were obtained by double integration of these accelerations. The peaks in Fig. 6 are the average of the maximum values of each of the four corners. The maximum value of PFA was obtained during motion FB-5: DEN67, when the structure observes considerable plastic deformation. The largest PIDRs obtained on levels 4-5 and 5-roof were observed during the final motion FB-6: DEN100, with 1.2% and 0.8% attained on levels 4-5 and 5-roof, respectively. These values were lower anticipated, as soft story mechanism developed in the lower levels of the building, with very large PIDRs approaching 6% for the first two floors[14].



(a) (b)

Fig. 6: Peak interstory drift ratio versus peak floor acceleration achieved while the building was fixed at its base: (a) fourth floor, (b) fifth floor

BEHAVIOR OF THE CORNER SYSTEMS OF THE PANELS

This section will describe the behavior at the corners for the panels installed on the 4th floor during the motion sequence summarized in Table 1. Results will focus on the panels of the 4th floor because this was the floor where the larger IDs were obtained. The main differences in the four corners were:

a) The type of corner joint installed on the OP panel: both on the eastern and western side of the building one panel had a miter joint (MJ panel) while the other had a butt-return joint (BRJ panel).
b) The type of push-pull connection in the closest IP panel: the western OP panels were in contact with IP panels supported by sliding rod connection while the OP panels on the east side were close to panels with flexing rod connections (as can be seen Fig. 3).

Observed physical damage to the OP panels is described and subsequently select measured response is presented.

DEVELOPMENT OF PHYSICAL DAMAGE

Interior inspection of the cladding panels and connections was performed after each motion. This inspection allowed a close monitoring of the damage in the lower portion of the panels and a monitoring only of major damage in the upper portion of it (i.e. permanent deformation of a connection and large cracks). In contrast, exterior inspections were more complete and allowed a detailed check of the interior upper portion of the panels. Time constraints resulted in exterior inspections performed only after motions FB:3-ICA50, FB-4:ICA100 and FB-6:DEN100.

Damage in the OP panels on the fourth floor was concentrated in the panels on the western side of the building (4WS and 4WN) and consequently the description will focus mainly on these panels. A schematic of the damage progression in these panels is shown in Fig. 7a and b (the location of these drawings corresponds respectively to section A-A and B-B in Fig. 3).The following damage description is divided into description of damage close to the corner connection, damage in the other areas of the panel and permanent misalignments of the panels.

Damage to the Corner Connection Area

As can be seen in Fig. 7a, the damage to panel 4WS (MJ panel) was mainly characterized by a network of cracks forming in the panel close to the corner connection embed. Cracks of average width = 0.2mm spanned from the corner of the window up to the corner connection embed and upper edge of the panel and formed at an early stage (after FB-4:ICA100; a motion resulting in ~0.7% IDR at this level). New cracks at this same location formed during FB-5:DEN67 and FB-6:DEN100, when IDRs of respectively 1.1% and 1.2% were achieved. In the interior these cracks were almost perpendicular to the old ones and pretty wide (up to 0.5mm) while the exterior crack that formed was parallel to the previous cracks and thinner (0.1mm). The final crack pattern, shown in Fig.7c, seems to indicate that the upper corner of the panels was folding outward when pushed by the IP panel. During the last inspection this same type of cracks (but much thinner and less developed) was observed basically in every OP panel close to a miter joint while it was never observed in an OP panel close to a butt-return joint confirming the relevance of the type of joint in the determination of the damage pattern. The plate of the corner connection of panel 4WS was closely inspected after the final motions and it did not look visibly bent. If it did plastically deform, the amplitude was small, on the order of a few millimeters.

The damage to panel 4WN (BRJ panel) was characterized by the bending of the ductile plate of the corner connection. A permanent bend was observed after FB-5:DEN67 while a much more visible bending deformation was observed after FB-6:DEN100 (Fig.7d). No cracks in the panel close to the corner connection embed were observed. This panel was the only one where a visible bending of the plate was observed.

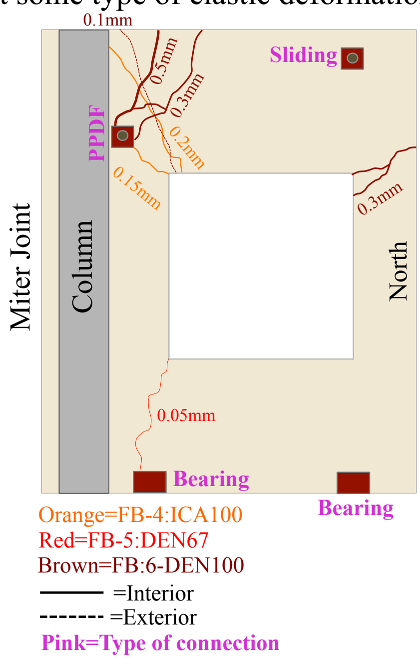
Since the only parameter considerably different between these two panels were the corner joints, it is possible to conclude that this different behavior is due to this. Namely it is possible to conclude that the presence of a BRJ gives stiffness to the corner thus reducing flexural distortion in the panel and forcing the ductile fuse in the connections to absorb the deformation. On the other side in the MJ panels the most flexible part of the corner system is not the ductile fuse but the corner of the panel itself and it that ends up getting damaged.

Damage to Other Areas of the Panel

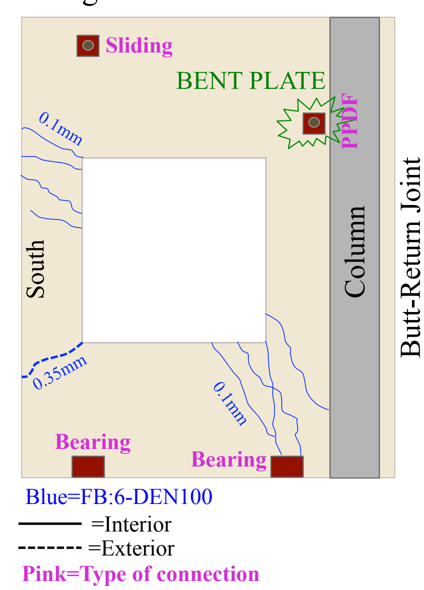
During the last two motions several cracks formed in the sections of the panels far from the corner connection as observed in Fig. 7a and b. The crack pattern suggests that impact with the IP panels created a folding of the upper-outer section of the panel while the bottom of the panel and its inner side remained basically fixed. The cracks in the MJ panel (4ES) were very similar, in terms of thickness and length, to the ones typically observed in many other OP panels confirming that they were created by the typical deformation of these panels. In contrast, the cracks in the BRJ panel (4WN) were many more than what was typically observed.

Permanent Misalignment

The upper portions of both the southern and northern panels on the west side observed a permanent westward misalignment with respect to the bottom of the panels on the 5th floor and consequently severe damage to caulking was also observed. The southern MJ panels had a permanent westward misalignment of ~15mm (Fig. 7e). Since the plate of the connection did not visibly bend, the misalignment was likely created by some plastic deformation of the panel itself, as confirmed by the thick cracks on the interior of the panel. The upper portion of the BRJ panel ended up with a permanent displacement of 40mm (Fig. 7e). Fifteen of the total 40mm can be attributed to the final plastic deformation of the plate in the connection while the remaining 25 were likely due to plastic deformation in the panel. No permanent misalignment was observed at these same locations on the east side of the building, however, the damage to the caulking confirmed that some type of elastic deformation occurred during the motions.



(a)



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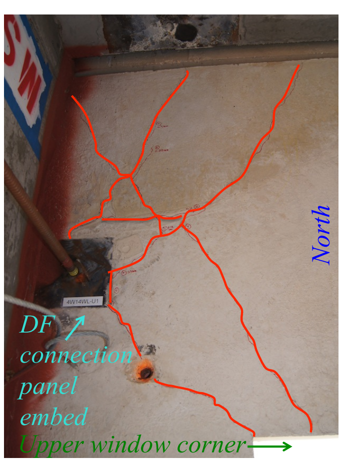

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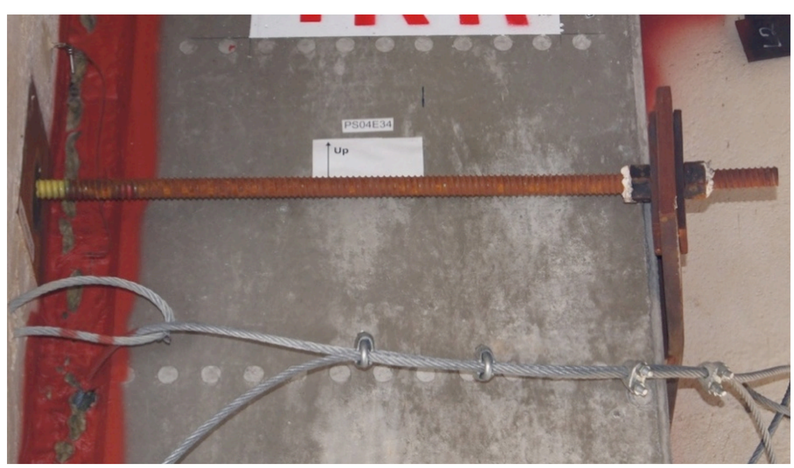

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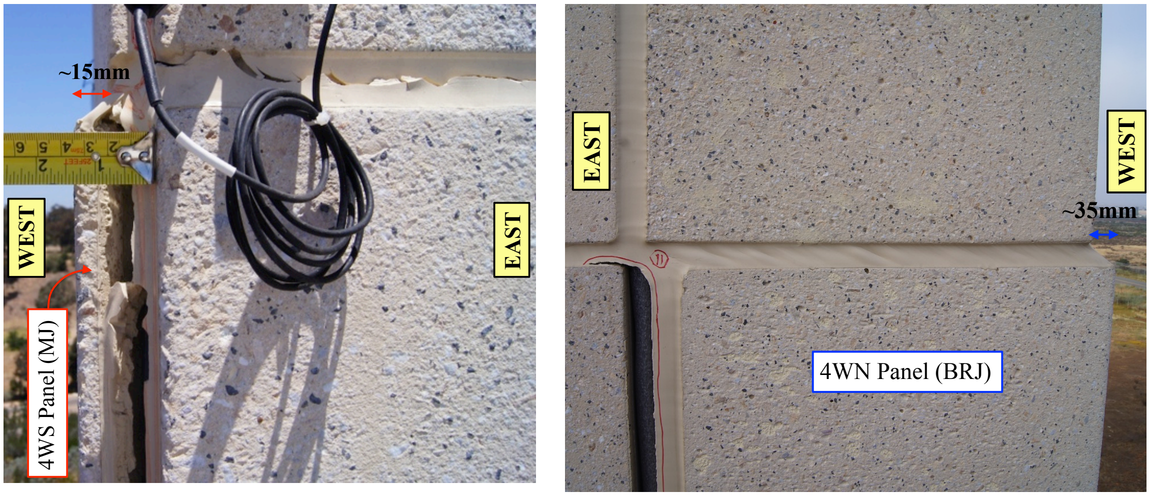


(e) (f)

Fig. 7 Observed damage to corner connection areas of the panels (a) crack pattern in the 4WS panel (section A-A in Fig. 3), (b) crack pattern in the 4WN panel (section B-B in Fig. 3), (c) cracks net close to the embed of the corner connection is panel 4WS after FB-6:DEN100, (d) bending of the panel in the corner connection of panel 4WN after FB-6:DEN100, (e) permanent displacement of panel 4WS, (f) permanent displacement of panel 4WN. (a) and (b) represent schematics of the elevation of these panels. (parts a-d on prior page).

MEASURED FORCES AND DISPLACEMENTS

This section will focus of the behavior on the instrumented corners: in the corner connections of the eastern panels both forces and displacements were measured, while on the western side of the building the northern panel (BRJ) was instrumented with a displacement potentiometer while the southern one was not instrumented . The following results will be presented:

- A general description of the displacement-force behavior as recorded in the eastern panels and its relationship to the building behavior (acceleration and drifts);
- Comparison of the displacements recorded in the BRJ panels in the east and west side of the building; and
- Comparison of the displacement and force behavior on the MJ and BRJ panels on the eastern side of the building.

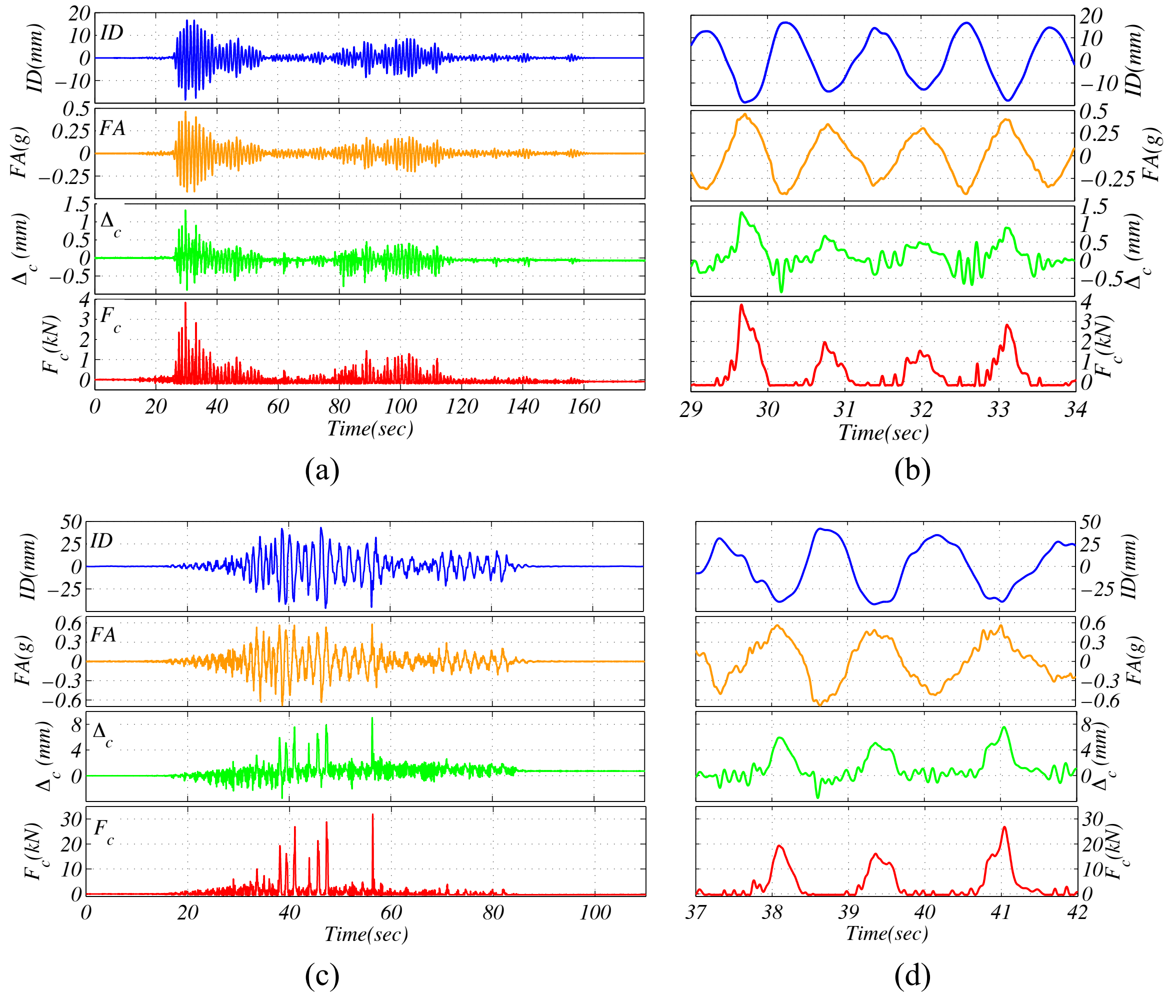


Fig. 8 Time history of FA, ID, Δ_c and F_c in panel 4ES (a),(b)recorded during FB:3-ICA50 (entire time history and zoomed view of strong motion period), (c),(d)recorded during FB:5-DEN67 (entire time history and zoomed view of strong motion period).

Time Histories of the Corner Connection Response Compared with the Building Response

Time histories recorded during FB-3:ICA50 and FB-5:DEN67 in the corner connections in panel 4ES are shown in Fig. 8. Plots show:

- Inter-story drifts (ID) on the SE corner between the fourth and the fifth floors. Positive indicates that the building is displacing eastward;
- Floors accelerations (FA) in the SE corner of the fifth floor slab. Positive indicates that the building is accelerating eastward;
- Displacement of the panel with respect to the column (Δ_c) as measured by the string potentiometer. Positive implies that the panel is moving away from the structure; and
- Force in the rod (F_c). Sensors were able to measure only tension in the rod, which is indicated as a positive force.

From the time histories recorded during FB:3-ICA50 (Fig. 8a and b) nearly perfect synchronization between the structure and panels is observed. The difference in behavior during the eastward and westward motion can be clearly seen: when the ID is in the eastward direction (positive) the corner joint is enlarging and the rod does not take extra tensile force while when the ID is directed westward (negative) peak tensile forces and positive displacements are recorded. The impact phenomenon (as explained in Fig. 4) can be observed for FB-5:DEN67 (Fig. 8c and d): during this test both time histories of F_c and Δ_c exhibit large spikes in the positive direction created when the IP and OP panels push against each other. The peak force actually measured at this corner during the design motion was 32kN, which is three times the design force F_p used for the body of the connection. Moreover, it is 10% larger than the capacity of the ductile fuse (= 29kN). These forces were not short duration/high frequency impacts, but rather their energy was concentrated in the frequency range below 5Hz. These same spikes were recorded also during FB-6:DEN100.

Displacements in the BRJ Panels: East Versus West Side

Damage to the panels on the west and east sides of the building varied, despite the consistent inter-story drifts. To understand this phenomenon, displacements are examined for the NW and SE corner (both having BRJ), namely:

- Interstory-drift recorded at that corner (ID_{4NW} and ID_{4SE});
- Displacement of the corner connections in the OP panels (Δ_{c4WN} and Δ_{c4ES}); and
- Displacement of the IP panel in that corner (Δ_{c4NW} and Δ_{c4SE}).

These three displacements for the NW corner of the 4th floor (where the plate plastically bent) recorded during FB-5:DEN67 are presented in Fig. 9a and b. Fig. 9b shows clearly that no displacement in the connection is recorded before the joint gap closes: the first movement of the connection is recorded at second 33 when the ID gets equal to 25mm (exactly the gap size for the BRJ). Up to this point the displacement of the IP panels and the ID are very similar. The first permanent bending of the corner connection in the OP panel occurs at second 38 when ID reaches 50mm: at this point the corner connection absorbed 20mm of total displacement of which 10mm were elastic and 10mm were plastic. The pushing action of the OP and IP panels against each other seems to affect also the movement of the IP panel, in fact the movement in the eastward direction in the IP panel (the direction of pushing) becomes larger than the ID. This might have been cause by a plastic deformation of the rods in the sliding rod connections of the IP panel. This IP panel ended up with an eastward residual displacement of 4mm.

The same plots for the SE corner are shown in Fig. 9c and d. In similar fashion, no deformation of the corner connection is recorded until the gap in the joint completely closes (ID = -25mm). It can be seen that, even if the ID were similar, the behavior in this corner is completely different: in this case when the OP panel impacts the IP panel (when the building is moving westward) the effect is a deformation of the IP panel. In fact, when the OP panel pushes against the IP panel the displacement absorbed by the IP panel connection is less than the ID meaning that the panel was not moving rigidly with the bottom slab but rather its upper portion was deforming in the direction of motion of the top slab. In this case Δ_{c4ES} remained smaller than 9mm and was mainly elastic.

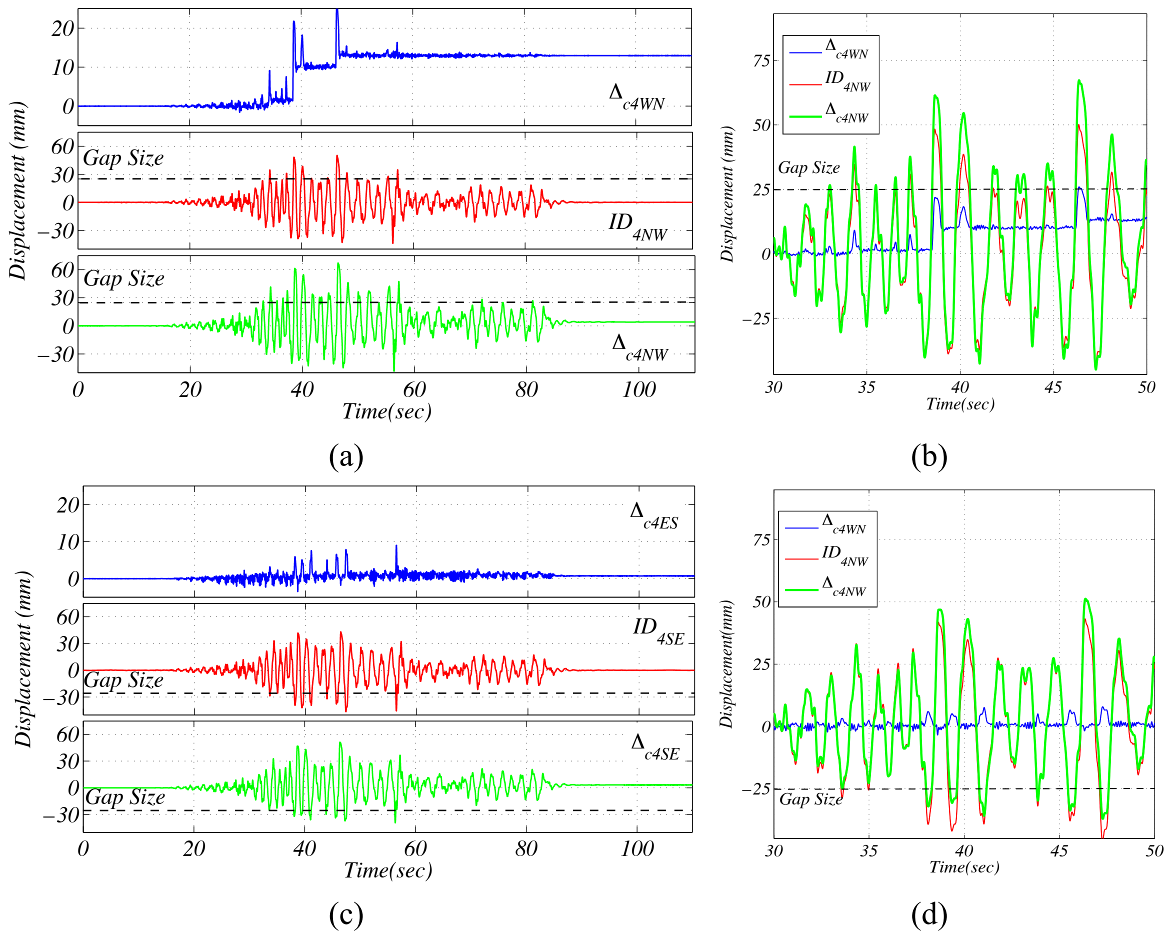


Fig. 9 Displacements recorded at the corners during FB-5:DEN67: (a),(b) close to NW corner at the 4th floor (entire time history, zoomed view, (c),(d) close to SE corner at the 4th floor (entire time history, zoomed view).

The different damage level and recorded behavior of the OP panels in the west and east side of the building can be explained considering the type of IP panels installed on each side: panels with sliding rod connections were installed on the west side while flexing rod connections were installed on the east side. Impacts between OP panels at the corner occurred on both sides but only panels on the west side got damaged. A hypothesis for this behavior is as follows:

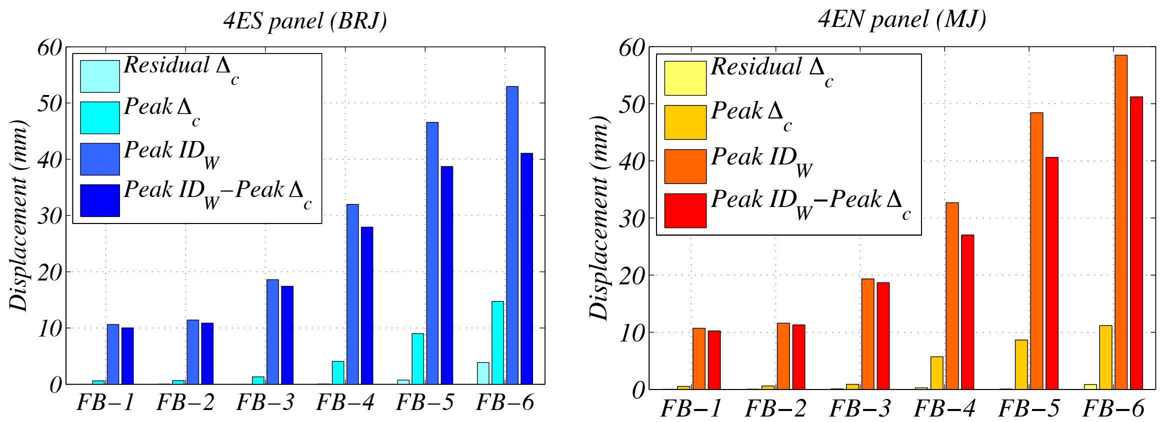
- West side, impact between OP panels and IP panels with sliding rod connection: the OP panels absorbed the deformation and got damaged. This means that the deformation mechanism in the IP panels was stiffer than the one in the OP panel. Either the activation of the sliding mechanism in the IP panel was very stiff or the mechanism was not working properly. In fact, extensive damage to the sliding connections of the fourth floor was reported[15] showing that the sliding rods behaved similar to very short flexing rods and ratcheted, thus explaining the incremental change in stiffness

East side, impact between OP panel and IP panels with flexing rod connection: the IP panels ended up absorbing part of the deformation while the OP panel did not get damaged, demonstrating that in this case the IP panels deformation mechanism was less stiff than that of the OP panels.

Maximum Displacements: MJ Versus BRJ panels

This section compares the displacement behavior of the corner connections in panels 4ES and 4EN. For each of the two panels examined the value of the displacement at the connection (Δ_c) and westward ID (ID_W) at the corresponding corner of the building were measured. These two values allowed calculation of the portion of drift not absorbed by a displacement in the connection, i.e. $ID_W - \Delta_c$. This drift is important because it represents the displacement absorbed by other deforming mechanisms, such as through the kinematics of the joint (vertical joint closure) and deformation of the OP and IP panels. A value of ID_W-Δ_c smaller than 20-25mm indicates that likely no impact occurred while a value much larger indicates that contact between the two panels occurred. Another important parameter is the residual Δ_c at the end of testing as this indicates the final deformation and alludes to the service state following a seismic event.

The residual Δ_c and the peak values of Δ_c, ID_W and ID_W-Δ_c recorded during each motion for the two corners under consideration are shown in Fig. 10. The peak values of ID and Δ_c confirm that during the first three motions no impact occurred as peak Δ_c were of the order or 1mm. A small impact probably occurred during FB-4:ICA100, in fact the ID reached more than 30mm and Δ_c of 4-5mm was recorded in both connections. During the design motion (FB-5:DEN67) the ID recorded was roughly twice the gap size (~50mm) confirming the inevitability of impacts. The peak connection displacement Δ_c reached 9mm, however, for both connections it was mostly elastic displacement. During the last motion the total ID in the southern side of the building was 53mm. For the BRJ southern panel it can be assumed that 25 mm (=47%) were absorbed by the closing of the corner joint, 11mm (~20%) were absorbed by the elastic deformation of the plate, 4mm (7%) by the plastic deformation of the plate and 13mm (26%) by other mechanisms. In the MJ panel (4EN) in total 58mm of displacement were absorbed as follows: ~27mm (46%) by the closing of the gap, 10mm (17%) by elastic deformation of the plate and 1mm (2%) by a plastic deformation of the plate, leaving 20mm (35%) to be absorbed by other mechanisms. This plots also confirms that the maximum elastic deformation of the connection was ~10mm.



(a) (b)

Fig. 10 Residual Δ_c, peak Δ_c peak ID_W and peak ID_W-Δ_c recorded in all the FB motions in the (a) 4ES panel and (b) 4EN panel

Force-Displacement Hysteretic Behavior: MJ Versus BRJ panels

The force-displacement behavior recorded in the last three motions for the corner connection in panel 4ES (butt-return joint) and 4EN (miter joint) is shown in Fig. 11. During motion FB-4:ICA100 (Fig. 11a), the connection on the BRJ panel shows an almost perfect linear behavior while that of the MJ panel presents a bi-linear behavior with a change in stiffness at a displacement of 1mm and force of 3kN. Fig. 11c shows the displacement versus force for FB-5:DEN67. During this motion large impact forces were recorded in both connections, and these are also present in the hysetersis of the connection. In this case both connections showed a nonlinear behavior: the connection of the south side (BRJ) presents a stiffening trend especially when displacements are larger than 6mm, while the connection in the northern panel (MJ) still tends to soften for displacements larger than 5mm. This softening behavior of the panel-connection with MJ is attributed to the formation of cracks in the corner of the panel, while the stiffening behavior in the panel with BRJ is more likely attributed to the nonlinear behavior of the steel connection (the connection deformed 9mm with 0.7mm of residual displacement). During the final, MCE-target motion FB-6:DEN100 even larger impacts were recorded and the behavior of the two connections more significantly diverged. During this motion, the connection of the BRJ panel plate developed plastic bending (as confirmed by a 4mm residual displacement) and several broad hysteresis loops confirm the dissipation of energy through continued plastic rotation of the plate. The behavior shows a stiffening trend also in this case. The connection in the northern MJ panel on the other hand observed smaller displacements and also did not exhibit broad hysteresis but rather a nearly linear force-displacement absent observed softening observed. This is likely due to the fact that sources of inelastic behavior in this panel-connection were limited to crack development in the panel, and this had occurred, and stabilized, in prior motions. The smaller connection displacement can be attributed to the fact that the plastic displacement of the panel was above the connection level.

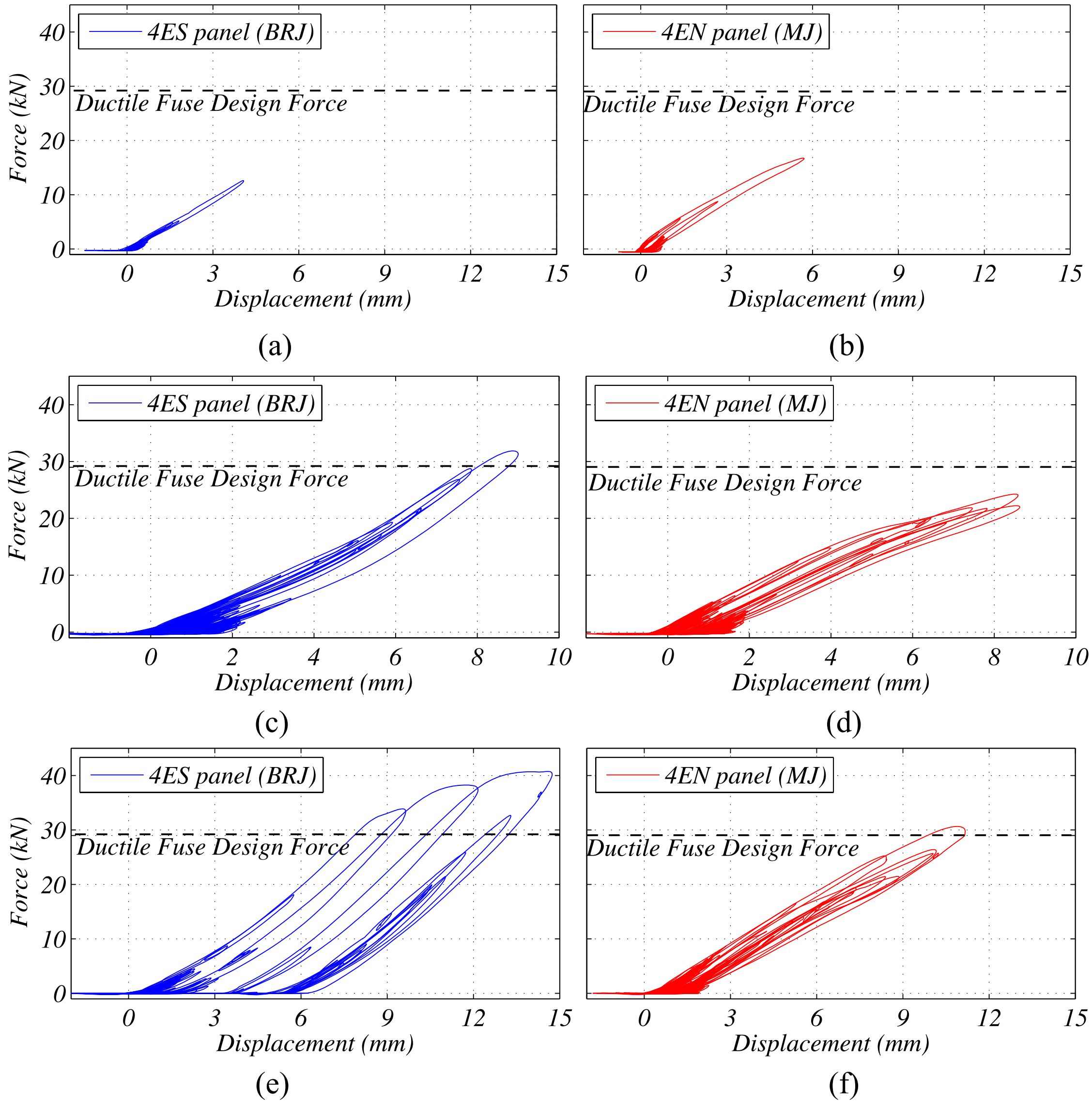


Fig. 11 Force-displacement response recorded in the 4ES and 4EN panels during (a),(b) FB-4:ICA100, (c),(d) FB-5:DEN67 and (e),(f) FB-6:DEN100

CONCLUSIONS

Sixteen precast concrete cladding panels were installed on the two upper floors of a full-scale five-story building and seismically testing at the NEES@UCSD shake table. In this paper, the behavior of a new ductile connection design was explored, with particular interest in observing its influence on the damage and recorded response to panels and connections installed at the corners of the test building.

- Damage to out-of-plane (OP) panels was more pronounced on the west side of the building. This was (possibly) due to the different types of IP panels installed at corners, namely a flexing connection installed on the east side absorbed a portion of the deformation created between OP and IP panels thus avoiding damage to the OP panels, while the stiffer and severely damaged sliding connections on the IP panels on the west side were not able to absorb the deformation caused by the impact that ended up damaging the OP panels;

- Butt return joint (BRJ) panels on the west side of the building activated the newly designed ductile mechanism, resulting in visible plastic deformation of the plate due to corner panel impact, while in the panels with miter joints (MJ) the plate did not bend and much of the deformation was absorbed by the panel bending and cracking. Damage to the panel in the case of the BRJ was probably avoided likely due to the additional stiffness of the panel created by the return;
- Time histories of the force and displacement recorded in the corner connections showed clearly that impact between the OP and IP panels occurred during the last three motions. Forces created by this contact were 10% larger than the design component forces and the strength of the ductile fuse;
- Displacements recorded in the connections verify that no inelastic displacements in the connection are generated before the gap in the corner joint closes.
- The force-displacement response measured in the ductile fuse connection observes a softening in the MJ panels, probably caused by the formation of cracks. In contrast, the panel with the BRJ showed a stiffening behavior, probably caused by stiffening in the steel of the connection. Maximum forces and displacement recorded confirmed the presence of nonlinearities during testing.

The intended goal of the ductile connection was to develop plastic deformation upon impact of corner precast concrete panels, while protecting (limiting the force transmitted) to other elements of the connection. In this fashion, joint sizes can be reduced and hence more aesthetically appealing. Results from these full-scale system tests successfully confirm that these objectives were achieved.

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