Cyclic Response of Precast High-Performance Fiber-Reinforced Concrete Infill Panels

by E. C. Olsen and S. L. Billington

A seismic retrofit strategy using precast infill panels made of a ductile high-performance fiber-reinforced concrete (HPFRC) is investigated for use in critical-use, steel moment-frame buildings. The infill panel material is a self-consolidating concrete mixture that uses high-strength steel fibers. In the retrofit, the panels infill existing steel moment-frame bays, adding stiffness, strength, and energy dissipation capacity and use pretensioned, slip-critical bolted connections. Component-level single-panel tests with various reinforcement layouts and geometry are conducted and show promising cyclic behavior in terms of hysteretic stability, ductility, energy dissipation, and damage tolerance. A connection detail using threaded studs is verified for performance and repeated use. Recommendations for future phases of research, including large-scale testing, are given.

Keywords: critical-use; cyclic load; high-performance fiber-reinforced concrete; moment-frame; precast infill panel; seismic retrofit; self-consolidating.

INTRODUCTION

The vulnerability of critical-use facilities such as hospitals has received heightened attention for seismic retrofit. The 1994 Northridge earthquake in California caused 23 hospitals to suspend some or all of their services at the time in which they were needed most and resulted in more than $3 billion in hospital-related damages. California enacted legislation that made it mandatory for healthcare facilities to evaluate the seismic adequacy of their buildings by January 2001, upgrade these facilities to ensure life safety by 2008, and ensure full operation following earthquakes by 2030. The susceptibility of steel moment-frame buildings to have premature, brittle fracture at the welded beam-column connections under large deformations was also evident from the 1994 Northridge earthquake.

The research presented herein is a component-level evaluation of a seismic retrofit system intended for use in steel moment frames for critical facilities. Retrofit design challenges for critical-use facilities include the need for the building to remain operational during retrofit installation and be operational immediately following an earthquake when the facility is heavily needed. Furthermore, in many critical facilities such as hospitals, secondary systems and other operational constraints require retrofit designs that do not block entire bays with retrofit features. The retrofit technique investigated herein addresses each of these challenges with the goal of providing engineers with an additional method for mitigating seismic risk to vulnerable steel moment-frame buildings used as critical-use facilities.

The retrofit system under investigation is shown schematically in Fig. 1(a). The system uses precast infill panels made from a ductile high-performance fiber-reinforced concrete (HPFRC) to add stiffness, strength, and energy dissipation to deficient critical-use steel moment-frame buildings. This system extends previous research on ductile infill panels and panel-to-frame connection details. The research presented herein evaluates a different HPFRC material that contains coarse aggregate and has different properties, new reinforcement layouts, and a new connection detail. A different testing protocol and different instrumentation methods were used as well. It is noted that this retrofit system could be considered for new design as a replaceable energy-dissipating fuse, which would be a topic for future research.

The retrofit system is designed with three objectives:

1. Ease of installation to minimize disruption to current facility operations;
2. Protection of the steel frame and nonstructural systems in a seismic event; and
3. Simple, rapid repair after a seismic event to ensure minimal disruption to continued facility operations.

The panels are sized to be light and small enough to be brought into a building by elevator and maneuvered by hand or with a small lift. The panels are precast and use bolted connections to limit disruptions associated with concrete placement. Two panels are bolted together and to the existing frame beams to act as a single, fixed-end deep-beam flexural element. The panels are unlike traditional infill in that they are not connected to adjacent panels or to the frame columns. The connections are designed to be reusable if panels, which act sacrificially, are damaged and need to be replaced after an earthquake. Because the infill panel system is modular, partially filled bays can be used to tailor global building response and to incorporate facility use constraints. Moreover, recognizing that hospitals undergo frequent reorganization of floor space use, with some effort and reanalysis, these infills could be moved in the same way nonstructural partitions, which the panels might replace, could be moved.

High-performance fiber-reinforced cementitious composites

The high-performance concrete used in this research is part of a class of materials referred to as high-performance fiber-reinforced cementitious composites (HPFRCCs), which include mixtures that do not contain coarse aggregate. HPFRCC is so named because of having an ultimate tensile strength higher than its first cracking strength and the formation of multiple cracking during the inelastic deformation process. When a tensile crack forms in an HPFRCC material, the fibers bridge the crack and prevent its uncontrolled propagation. Stress is transferred between the...
fibers and matrix by the bridging fibers through interfacial shear and/or mechanical anchorage through fiber hooks, and eventually leads to the formation of another tensile crack in a new location at a higher load. As this process is repeated, multiple tensile cracks are formed until the maximum fiber-bridging stress is reached and the composite fails generally at a single crack (that is, crack localization). This multiple crack formation at increasingly higher load leads to a tensile-hardening behavior that classifies HPFRC materials and is analogous to strain hardening in steel. This hardening tensile response can lead to energy dissipation that is orders of magnitude higher than brittle cementitious composites and even traditional fiber-reinforced concretes. Due to the additional energy dissipation of HPFRCs over traditional cementitious materials, they have shown promising results in many seismic applications, which inherently require large inelastic deformation demands.5-9

Research significance

The research presented herein builds on previous experimental and finite element simulation investigations, which showed promise for this retrofit system.2 Several shortcomings in the previous research were observed, including low ductility in the hysteretic response and a connection detail that could be difficult to implement in standard building configurations. Furthermore, damage in cumulative cycles was not investigated. The new set of panel tests conducted and presented herein uses a new self-consolidating HPFRC mixture design containing coarse aggregate (hence reducing cost) and having a higher stiffness and tensile strength. New panel geometries and reinforcing details for improved ductility and energy dissipation are investigated as well as the feasibility and reusability of a new, more easily implemented connection detail. Finally, information regarding both the flexural and shear deformation is presented.

EXPERIMENTAL PROGRAM

The experimental program consisted of a 2:1 aspect ratio, roughly 2/3-scale component-level single panels tested cyclically as a vertical cantilever. Panels were loaded laterally (Fig. 2) at the theoretical inflection point to take advantage of the symmetry of a proposed double height fixed-fixed infill panel unit.

Self-consolidating HPFRC

The infill panels were made from a self-consolidating HPFRC material.10 This material uses 1.5% by volume of 30 mm (1.2 in.) long high-strength (2300 MPa [333.5 ksi] in tension) hooked-steel fibers with an aspect ratio of 80. The mixture designs used in the panel experiments are shown in Table 1.

Panel design and fabrication

The panels were initially designed using monotonic nonlinear finite element analyses with a material model using assumed material properties to study optimum panel ductility.11 Reinforcement layout and panel geometry were the main variables of the study. Five HPFRC panel designs were chosen for experimental investigation, with a reinforced concrete control panel for comparison. Four of the HPFRC panel designs and experimental results are presented herein. The specimen schedule and naming convention are shown in Table 2, where WWR refers to welded-wire reinforcement and No. 3 and No. 2 bars refer to 9.5 and 6.4 mm (0.375 and 0.25 in.) diameter reinforcement, respectively. Construction drawings of the panels discussed herein are shown in Fig. 3(a) through (e).

The design objectives for each panel were as follows:

No. 3 P-Bar—Simple rectangular geometry and reinforcing layout; baseline, easy-to-construct panel.

No. 2 P-Bar—Impact of tailoring panel strength; investigate if delaying high reinforcement strains and increasing energy dissipation through multiple cracking is achievable with smooth (that is, low mechanical bond) reinforcement, recognizing that HPFRC has a strong bond to deformed reinforcement causing a quick change from yielding to fracture of steel.

Taper—Spread damage and increase ductility by tapering panel geometry to reduce capacity where internal moment decreases; achieve good strength and ductility with less material.

V-Bar—Spread damage and increase ductility using easy-to-construct rectangular geometry and tapering only the No. 3 reinforcing bar (therefore similar reinforcement as Taper panel).

Control—Best-case reinforced concrete panel using traditional concrete and the same geometry as No. 3 P-Bar; additional internal reinforcement is used, recognizing that traditional concrete has less shear capacity and no matrix tensile contribution to flexural capacity as is the case with HPFRC.

In traditional terms, the reinforcement ratio for the HPFRC panels ranged from 0.35 to 0.6% and the
reinforcement ratio for the concrete Control panel was 1%. It is noted, however, that the strength of the various reinforcements differ (Table 3).

The panels were cast flat in plywood forms and covered with plastic sheeting to cure for a minimum of 28 days, with the exception of the V-Bar-2 panel, which was tested at 21 days. Cast acrylic cylindrical inserts were used to form the holes for the connection through-bolts.

**Threaded stud connection detail**

In consultation with several practitioners, a connection detail was designed using a proven construction technique and threaded welded studs (schematic shown in Fig. 1(b)) with the following construction steps: (1) the existing concrete slab is cored to expose the top flange of the frame beam; (2) a threaded stud longer than the depth of the hole is welded within this hole to the frame beam; (3) the hole is filled with grout; (4) a steel U-channel (half an HSS section) is bolted to the exposed threads of the stud; (5) an infill panel is lowered into place; (6) the tolerance between the base of the panel and the U-channel is filled with nonshrink grout; and (7) after the grout has cured, the panel is secured with pretensioned bolts. The retrofit sequence is similar to a proven construction technique referred to as “retrofit-upgrading to composite construction.” As this is a proven technique, and for simplicity in the lab, in this research, the studs were welded to a section of wide-flange beam prior to casting the representative existing slab instead of casting the slab first and core-drilling.

To facilitate replaceability, the grout in the connection was installed in two layers. The bottom layer of grout remained intact after the top layer was chiseled after a test and was reused six times. Bolt pretensioning was performed after the second layer of grout was allowed to cure for a minimum of 36 hours. A calibrated torque wrench was used to torque the 19 mm (3/4 in.) diameter ASTM A490 bolts to 178 kN (40 kips) each. Tension in the bolts was monitored using direct-tension-indicating washers.

**Single-panel test setup and procedure**

A schematic of the test setup is shown in Fig. 2. No axial loads were applied to any of the panels. In the full system, a vertically slotted-hole center connection will be used (validated in a different phase of this research), which prevents axial load buildup from live load or lateral loads. Out-of-plane bracing was provided for all panel tests except for the No. 3 P-Bar panel. The cyclic loading protocol was adapted from the SAC-recommended protocol and previous experimental tests on precast reinforced concrete infill walls, with consideration for the expected drift capacity of the specimens being tested. The drift levels for the cyclic loading (with the number of cycles in parentheses following) were: 0.1% (3), 0.25% (3), 0.375% (3), 0.5% (2), 0.75% (2), 1% (2), 1.5% (2), 2% (2), and 3% (1) followed by a monotonic push to failure. The panel tests were displacement-controlled, pushing and pulling to each drift level, which was defined as the displacement at the top bolts divided by the distance from this point to the point of fixity below—914 mm (36 in.).

The panels were instrumented with nine linear variable displacement transducers (LVDTs) to measure drift profile, out-of-plane displacement, and shear distortion (Fig. 2). Six strain gauges were installed on the primary reinforcement. In

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**Table 1—Mixture proportions for HPFRC and concrete used**

<table>
<thead>
<tr>
<th>Mixture</th>
<th>H₂O</th>
<th>Cement</th>
<th>Fly ash</th>
<th>Sand</th>
<th>CA</th>
<th>High-range water-reducing admixture</th>
<th>Viscous agent</th>
<th>Steel fibers</th>
</tr>
</thead>
<tbody>
<tr>
<td>By weight of cement</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HPFRC</td>
<td>0.59</td>
<td>1</td>
<td>0.5</td>
<td>1.7</td>
<td>1</td>
<td>0.0096</td>
<td>0.0095</td>
<td>0.245</td>
</tr>
<tr>
<td>Concrete</td>
<td>0.48</td>
<td>1</td>
<td>—</td>
<td>1.55</td>
<td>1.45</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>451</td>
<td>760</td>
<td>381</td>
<td>1293</td>
<td>760</td>
<td>7.3</td>
<td>7.2</td>
<td>186</td>
</tr>
<tr>
<td>HPFRC</td>
<td>416</td>
<td>867</td>
<td>—</td>
<td>1343</td>
<td>1257</td>
<td>—</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>

1Type III portland cement.
2Class C fly ash.
3Flint silica sand ASTM 50-70.
4Coarse aggregate (13 mm [0.5 in.]) stone.
5Viscosity-modifying agent.
6High-strength hooked fibers.
Notes: 1 yd³ = 0.765 m³; 1 lb = 0.454 kg.
addition, 32 optical markers were installed in a 127 x 127 mm (5 in. x 5 in.) grid to track three-dimensional (3-D) panel movement with an optical motion measurement system.

Measured properties of panel materials

The mechanical properties of the concrete, the self-consolidating HPFRC, and the steel reinforcement were obtained experimentally. For the concrete and HPFRC mixtures, 75 x 150 mm (3 x 6 in.) cylinders were cast for each material and tested in compression. The average compressive strength was 36.5 and 51.7 MPa (5300 and 7500 psi) for the concrete (three cylinders) and the HPFRC (four batches, three to six cylinders per batch), respectively. Additionally, for the HPFRC mixtures, dogbone specimens with a 175 mm (7 in.) gauge length and 25 x 50 mm (1 x 2 in.) cross section were cast and tested in uniaxial tension, and 150 x 150 x 500 mm (6 x 5 x 20 in.) beams were cast and tested using third-point bending, following ASTM C1609/C1609M-05.16 Figure 4 shows representative curves of the dogbone and ASTM beam tests. Tension tests on samples of the steel reinforcement were also performed and are summarized in Table 3.

**Table 2—Single-panel specimen designs and capacities**

<table>
<thead>
<tr>
<th>Panel name</th>
<th>Material</th>
<th>Shape</th>
<th>Reinforcement</th>
<th>Predicted peak load, kN (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 3 P-Bar</td>
<td>HPFRC</td>
<td>Rectangle</td>
<td>No. 3 perimeter bar + WWR</td>
<td>48 (10.8)</td>
</tr>
<tr>
<td>No. 2 P-Bar</td>
<td>HPFRC</td>
<td>Rectangle</td>
<td>No. 2 perimeter bar + WWR</td>
<td>43 (9.7)</td>
</tr>
<tr>
<td>V-Bar*</td>
<td>HPFRC</td>
<td>Rectangle</td>
<td>No. 3 tapered bar + WWR</td>
<td>47 (10.6)</td>
</tr>
<tr>
<td>Taper</td>
<td>HPFRC</td>
<td>Taper</td>
<td>No. 3 tapered bar + WWR</td>
<td>47 (10.6)</td>
</tr>
<tr>
<td>Control</td>
<td>Concrete</td>
<td>Rectangle</td>
<td>No. 3 perimeter bar + WWR + interior No. 3 U-bar to midheight</td>
<td>38 flexure (8.5) 30+ shear† (6.7)</td>
</tr>
</tbody>
</table>

*V-Bar panel test was repeated due to slippage-induced connection failure of first specimen. V-Bar tests are referred to in this paper as V-Bar-1 (slip) and V-Bar-2 (no-slip).

†Assumes only concrete contribution to shear resistance, although recognized that welded-wire reinforcement will resist some shear.

**Table 3—Reinforcement tension test results**

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>$f_y$, MPa (ksi)</th>
<th>$f_u$, MPa (ksi)</th>
<th>$\varepsilon_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 3 deformed bar</td>
<td>469 (68)</td>
<td>634 (92)</td>
<td>0.1</td>
</tr>
<tr>
<td>No. 2 round bar</td>
<td>662 (96)</td>
<td>745 (108)</td>
<td>0.02</td>
</tr>
<tr>
<td>3 x 3 W1.2 x W1.2</td>
<td>696 (101)</td>
<td>828 (120)</td>
<td>0.03</td>
</tr>
</tbody>
</table>

**Predicted peak strength of panels using measured material properties**

Theoretical predictions of the flexural strength of the panels were calculated using the measured material properties for the steel, HPFRC, and concrete and are given in Table 2. The reinforced concrete Control panel was analyzed for ultimate flexural strength per ACI 318-05.17 The ultimate shear strength for the concrete panel assumed only the concrete contribution $V_c$ as given by ACI 318-05.17 This estimate is expected to underestimate the panel shear strength as it ignores the welded-wire reinforcement (WWR). The wire reinforcement, however, was made up of two overlapping 152 mm (6 in.) grids to create a 76 mm (3 in.) spacing of the wires (due to lack of availability of a 76 mm [3 in.] grid); therefore, full development of the WWR is not expected.

The HPFRC panel capacities were calculated based on their theoretical flexural strength, as only flexural failures were expected based on previously observed panel response with a different HPFRC material.2 For the HPFRC flexural strength calculations, three aspects of the analysis differed from traditional reinforced concrete ultimate strength calculations: (1) the HPFRC panels are considered to reach peak strength when a dominant crack in the HPFRC forms, which corresponds to softening of the HPFRC in tension (for example, roughly 0.5% strain in Fig. 5(a)); (2) it is assumed that before a dominant crack forms, the HPFRC can carry tension and forms a “tension block” in the sectional stresses; and (3) when the HPFRC forms the dominant crack, it is assumed that in compression the material is still elastic. The
peak strength of the HPFRC panel is evaluated assuming plane strain across a section at the base of the panel (just above the confined connection zone) and considering an extreme tensile strain in the panel of 0.005 (when the HPFRC is expected to form a single dominant crack based on uniaxial tension tests [Fig. 4(a)]). The neutral axis is assumed and iterations conducted until equilibrium in the cross section is reached with a linear-elastic (triangular) compression block. The moment capacity is determined and the estimate of the peak lateral load to reach this moment capacity is summarized in Table 2 for comparison later with experimental results. As expected, the capacity of the Taper and V-Bar panels is slightly lower than the No. 3 P-Bar panel due to the reduced depth to the No. 3 perimeter bar reinforcement, which has already begun to taper at the critical section slightly above the confined connection zone.

**PANEL EXPERIMENT RESULTS**

**Single-panel test results**

*Hysteretic behavior*—Plots of the applied load versus panel drift are shown for all of the specimens in Fig. 5(a) to (f). Cracking patterns (discussed in the following sections) are shown in Fig. 6. Table 4 shows a summary of the peak load, the maximum drift reached with at least 80% of peak load maintained, the panel initial stiffness from the 0.1% drift cycle, the post-cracking stiffness from the 0.375% drift cycle, and the residual strength, defined as the panel load at the maximum drift level prior to completion of testing.

The general trend of the panel hysteresis begins with a linear region followed by a slightly nonlinear positive slope caused by the multiple cracking and pseudo-strain-hardening of the HPFRC material and eventual yielding and strain hardening of the reinforcing steel. After the peak strain capacity of the HPFRC material is reached, crack strain localization begins—that is, the crack opening continues at one localized crack—and the hysteretic behavior begins to soften. Depending on the extent of the crack localization, this softening occurs either with a negative slope within the cycle (for example, the Control panel shown in Fig. 5(f)) or with cyclically degrading, slightly positive slopes.

The unloading response is initially elastic with very little permanent deformation at the cycles under ±0.25% drift. At higher drifts, pinching of the load-versus-drift response occurs during unloading and reloading as the cracks gradually close. During the closure of these fiber-bridged cracks, the panel has very low stiffness. The panel stiffness increases as the cracks fully close and begin to bear in compression.

As shown in Table 4, the peak load in the No. 3 P-Bar, V-Bar-2, and Taper panels was between 46 and 50 kN (10.3 and 11.2 kips) with the failure plane occurring near the base of the...
rather than flexure. Had the concrete been of equivalent
discussed in the following. The No. 2 P-Bar panel was weaker
in the connection region, leading to a connection failure that is
increase in flexural and shear capacity (the panel would still have been expected to fail in shear. An
capacity is attributed to both the concrete strength relative to
that of the HPFRC and the susceptibility of this panel
tapering the reinforcing bar within the rectangular panel,
reinforcement, either by tapering the panel shape or by
to fail in shear. Had the concrete been of equivalent
connection slippage. The Panel had similar initial and
shear in the panel. The Control panel failed in shear. Sketches of the cracking
approximately 11 kN (2.5 kips) of shear resistance prior to
concrete been of equivalent strength to the HPFRC—that is, 51.7 MPa (7500 psi)—the
reinforcement, either by tapering the panel shape or by
tapering the reinforcing bar within the rectangular panel,
connection slippage. The Taper panel had similar initial and
post-cracking stiffness for all of the panels showed more variation. The V-Bar-1 panel
less stiff than the V-Bar-2 panel because it experienced
connection slippage. The Taper panel had similar initial and
post-cracking stiffness as the Control panel despite its
tapered cross section. The similar stiffness is attributed to an
connection slippage. The Taper panel was designed to help distribute the damage
in flexural and shear capacity (the Vc component) of
maximum drift at which 80% of the peak load was
was maintained was 1% for most panels. The exceptions were the
No. 2 P-Bar panel, which was less ductile, and the Control panel, which maintained 80% of its negative direction peak
load up to 1.5% drift after abrupt shear failure had occurred in the positive direction at 1% drift. The panel initial stiffness
was similar for all of the tests. The post-cracking stiffness for all of the panels showed more variation. The V-Bar-1 panel
was less stiff than the V-Bar-2 panel because it experienced
damaged region through an abrupt shear failure. The Taper panel had similar initial and
post-cracking stiffness as the Control panel despite its
tapered cross section. The similar stiffness is attributed to an
increased stiffness provided by the HPFRC, which contains
an extra 1.5% by volume of steel for the fibers relative to
traditional concrete. Panel residual strength was typically
between 40 and 50% of peak strength at the end of each test, except for the Control panel, which showed a low residual
strength of 16% due to the abrupt shear failure.

The general trend for the cyclic strength degradation—that is, the difference in strength between the first and second
time the drift level was reached during testing—was similar
for all of the panels. As expected, cyclic degradation was
negligible for all panels in their elastic range. By the 1.0% drift, cyclic strength degradation was typically 15 to 20% in
all of the panels. The panels that were able to maintain higher residual strengths after softening (crack localization) showed
less cyclic degradation as well as their post-localization drift
levels (for example, Taper, V-Bar-2, and—to some extent—the No. 3 P-Bar, although this test was stopped after one
cycle at 2% drift).

Experimental versus predicted strength—The peak loads measured in the HPFRC panels with No. 3 reinforcement that did not experience connection slippage
were within 4% of those predicted by the theoretical
calculations. The average strength of the No. 2 P-Bar panel was 18% lower than predicted. The larger difference
between the actual and predicted capacities for the No. 2 P-
Bar panel is attributed to the actual tensile block being
smaller than assumed for the No. 2 P-Bar panel, which used a
smooth rod as opposed to the deformed No. 3 bar that was
present in the rest of the HPFRC panels. The assumed tensile
block for the HPFRC was taken as the full tension zone from
the extreme fiber to the neutral axis of the cross section. With
defomed bars, the size of the multiple cracks in HPFRC will
be better controlled than with a smooth rod. Therefore,
whereas the assumption of the full tension zone carrying
shear in the HPFRC seems to work well for the panels with
defomed reinforcement, it leads to an overprediction in
strength for panels with only smooth reinforcement. If the
lower 70% of the tensile zone is assumed to carry tension in
the HPFRC, the strength prediction for the No. 2 P-Bar panel would be approximately 35 kN (7.9 kips), similar to
what was experimentally measured. It is noted that reasonable
variations—5 to 10%—in the No. 2 reinforcement strength
could not account for the observed 20% difference in panel
strength. Although the method used herein predicted the
strength of these HPFRC panels well, further research is
recommended to validate the use of an HPFRC tension block in
predicting reinforced HPFRC component strengths.

For the Control panel, a shear failure was observed at a
lateral load of 41.4 kN (9.3 kips). The predicted strength
from concrete alone considering a nominal concrete shear
strength of 2 times the square root of the compressive
strength gave a prediction of 30 kN (6.7 kips). It can
therefore be assumed that the WWR present, which if fully
developed would be expected to contribute an additional 31 kN
(6.9 kips) of additional shear strength, is only able to contribute
approximately 11 kN (2.5 kips) of shear resistance prior to
shear failure in the panel.

Cracking, yielding, and modes of failure—All of the
HPFRC panels failed in a flexural mode, whereas the Control panel failed in shear. Sketches of the cracking
propagation for each specimen are shown in Fig. 6. For the
HPFRC panels, the dark crack indicates the location of
flexural crack localization, which became the failure plane.
Reinforcing bar fracture occurred at this location in all
panels except the No. 3 P-Bar panel, which was stopped at
2% drift due to out-of-plane movement, and the V-Bar-1
panel, which failed in the connection region due to slippage.
The failure plane for the HPFRC panels occurred toward the
base of the panel. The Control panel of reinforced concrete
failed in shear with large X-shaped cracks indicative of a
cyclic-shear failure.

The Taper panel was designed to help distribute the damage
through the height of the panel as compared to the No. 3 P-Bar
panel, for example, which had a constant panel capacity along its height. More distributed damage in the Taper panel was observed both in the cracking pattern (Fig. 6) and in comparing the measured steel strains. The longitudinal steel reinforcement reached strains of 0.009 and 0.042 at the middle gauge (Gauge L2 shown in Fig. 2(c)) in the No. 3 P-Bar and Taper panels, respectively. Higher up, (Gauge L3 shown in Fig. 2(c)), the steel in the No. 3 P-Bar panel reached a strain of 0.0026 compared with 0.0044 in the Taper panel.

Energy dissipation—Figure 7 compares the hysteretic energy dissipation of the first load-displacement cycle for each specimen at different drift levels. The energy dissipation from the panels is a result of the multiple cracking and tensile-carrying capacity of the HPFRC material, as well as the yielding of the main reinforcing steel. The panels that exhibited the greatest energy dissipation corresponded to those that showed the greatest cracking distribution and reinforcement strain gauge strain readings: the No. 3 P-Bar, Taper, and V-Bar-2 panels. The V-Bar-1 panel also showed relatively high energy dissipation until the connection slippage began at 1% drift, forming a major crack in the connection region and preventing further multiple cracking in the HPFRC. The Control and No. 2 P-Bar panels showed the least energy dissipation. The Control panel, being traditional reinforced concrete, was not expected to dissipate as much energy as the reinforced HPFRC panels and again failed in shear at 1% drift. The No. 2 P-Bar panel showed the least crack distribution during the test (Fig. 6), which correlates to the low calculated energy dissipation. Furthermore, the reinforcement was high strength with low ductility (Table 3) and hence was not expected to dissipate significant energy.

Figure 8(a) further compares the energy dissipation of each panel at different drift levels relative to an equivalent reinforced concrete panel assumed to have an elastoplastic response—that is, a parallelogram of initial stiffness and peak expected strength from a similarly reinforced concrete panel. An example of this is the Taper panel shown in Fig. 8(b). This comparison is similar in concept to the relative energy dissipation ratio used to evaluate precast moment frames for seismic regions (for example, ACI ITG 1.1-0118) with the exception of the peak value for the parallelogram being one for a reinforced concrete panel rather than an HPFRC panel. The energy dissipation of the experimental panel was calculated as the area of the hysteretic loop (load-displacement response) of the first cycle at a given drift level. The equivalent reinforced concrete panel’s energy dissipation was calculated as the area of an elastoplastic response where the initial stiffness was taken as that measured in the reinforced concrete Control panel. The ultimate strength was calculated assuming the same reinforcement details as the panel to which it is being
higher drifts, indicating a predominantly flexural response, which correlates with the flexural failure observed. All of the HPFRC panels had similar panel deformation profiles with the exception of V-Bar-1, which showed more rigid body rotation due to the connection slippage.11

Shear deformation was measured with LVDTs. The shear strain was approximated using a simplified measure of the average shear strain \( \gamma \) using two diagonal length change measurements (LV7 and LV8 in Fig. 2(d)), as well as initial horizontal and vertical distance measurements between the attachment points of LV7 and LV8. Figure 10 compares the shear stress-strain response of the No. 3 P-Bar panel and Control panel tests. At the peak load in each direction, which corresponded to 0.75% drift, the shear strain in the No. 3 P-Bar panel was between 0.03 and 0.3%, accounting for a maximum of 40% of the panel deformation. All of the HPFRC panel tests had maximum shear strains between 25 and 40% of panel deformation, with the exception of the V-Bar-1 panel, which experienced less shear strain due to slippage in the connection. The remaining deformation in the HPFRC panels was flexural, as very little rigid-body deformation from the connection slip was measured. The Control panel shear strain at peak load was between 0.3 and 0.5% and accounted for a maximum of 50% of the panel total deformation when shear failure occurred in the positive loading direction.

**Connection performance**—Overall, the base connection performed well during the tests. The V-Bar-1 panel was the first panel tested and the only one to experience significant slippage. The slippage was attributed to poor grout consolidation in the connection region, leading to difficulties in transferring the pretensioning force into the panel base, as well as a pretensioning force that was too low at 111 kN (25 kips) per bolt. The pretensioning force was increased to 178 kN (40 kips) and a more fluid grout was used to aid in consolidation, resulting in a connection that performed well in the remaining six tests that were conducted.11 The two-lift connection grouting approach allowed seven panels to be installed and tested in the same setup, indicating the replaceable nature of the retrofit scheme. The interface between the grout and the connection channel was shown to be the weak interface, as the grout was typically still intact around the

![Figure 8](image_url)

**Fig. 8**—(a) Hysteretic energy dissipation of first full cycle relative to elastoplastic response of equivalent reinforced concrete panel; and (b) Taper panel response versus equivalent elastoplastic reinforced concrete panel. (Note: 1 kN = 0.225 kips.)

Panel deformation—The deformation of the panels was monitored during testing in terms of lateral deformation, shear deformation, and panel slip. Figure 9 shows the drift profile for the Taper panel. The panel’s displaced shape is linear at low drift levels with increasing panel curvature at
slip of less than 1.5 mm (0.06 in.). The slip in the V-Bar-1 panel was confirmed by the reinforcing steel strain gauge readings. In the V-Bar-1 panel, Gauge L1 reached a peak strain of 0.003, whereas Gauge L1 in the V-Bar-2 panel reached a strain of 0.0172.\textsuperscript{11}

Whereas the test setup prevented large out-of-plane displacements, small amounts (for example, 10 mm [0.39 in.]) were possible. Out-of-plane displacement was measured with both an LVDT and the optical motion measurement system. An out-of-plane movement at peak panel load of less than 5 mm (0.02 in.) was observed for all of the tests.

### PANEL COMPARISONS

#### Effect of reinforcement details

The strength of the No. 2 P-Bar panel was lower than the No. 3 P-Bar panel due to a smaller provided reinforcing resistance. Because the No. 2 P-Bar panel was smooth, it was of interest to observe if allowing slippage could increase drift capacity and delay reinforcing bar fracture, given that the HPFRC material bonds so tightly to deformed reinforcement. However, the drift capacity at peak load, energy dissipation at higher drift levels, amount of multiple cracking spread throughout the panel, and residual strength measured as a percentage of peak were all less in the No. 2 P-Bar panel than in the No. 3 P-Bar panel. These decreases in performance were attributed to the measured strain capacity of the No. 2 bar being much less than the No. 3 bar (refer to Table 3) as well as the low reinforcement ratio, which led to the rapid yielding of the reinforcement when cracking began, thus localizing the strain and further limiting the ability of the smooth bar to facilitate cracking distribution.

A comparison of the effect of the reinforcing details of the No. 3 P-Bar and V-Bar-2 panels shows that tapering the panel capacity with reinforcement alone can spread panel damage, increase energy dissipation, and increase drift capacity. The load-drift response of the two panels was remarkably similar; the V-Bar-2 panel exhibited slightly higher energy dissipation at larger drifts. The main benefit of the V-Bar-2 panel is its peak drift capacity. The No. 3 P-Bar panel load noticeably dropped off by approximately 16%, whereas the V-Bar panel dropped off only slightly (approximately 6%) at 1.0% drift. The V-Bar panel had slightly less capacity, which was expected as the depth of the tension reinforcement tapered down. Similar and extensive reinforcement yielding was observed from the strain gauge readings for both panels.\textsuperscript{11}

#### Effect of panel shape

Tapering both the reinforcement and the panel itself (Taper panel) was found to improve panel performance over the rectangular geometry with perimeter reinforcement. For a given drift, more curvature and reinforcing bar strain is expected and was observed in the Taper panel. Comparing the Taper panel to the No. 3 P-Bar panel, for example, the Taper panel showed the same strength, more energy dissipation, crack distribution higher up the panel, higher drift capacity at peak load, and more gradual softening behavior. The Taper panel maintained the same or slightly higher residual strength relative to its peak strength compared to all of the rectangular panels. Whereas all of the panels exhibited significant reinforcement yielding, measured strains were higher in the higher strain gauges in the Taper panel than in all of the rectangular panels. The Taper panel perimeter reinforcing bar fractured at a lower drift level (2.5%) than the V-Bar-2 panel (3.75%) with a similar reinforcement layout. The earlier fracture in the Taper panel is attributed to the Taper panel’s lower flexural stiffness.

#### Effect of panel material

The Control panel was approximately 25% weaker than the HPFRC panels with No. 3 perimeter bars. As discussed previously, this lower strength is partly attributed to the lower strength of the concrete relative to the HPFRC and partly due to the inability of a concrete panel of this geometry, reinforcement, and loading to resist shear as well as one with HPFRC, which inherently has a higher shear capacity due to the composite’s tensile load-carrying capacity. The Control panel failed in shear at midheight at +1% drift and just after –2% drift. The No. 3 P-Bar panel had a higher peak strength until approximately 1% drift, beyond which the strength reduced to just under that of the Control panel. The higher strength of the No. 3 P-Bar panel is attributed to the contribution of the reinforced HPFRC material, which diminishes once a crack localizes and the panel behaves more like a reinforced concrete panel in flexure—that is, HPFRC carrying compression and reinforcement carrying tension. In addition to increasing the strength of the panel, the reinforced HPFRC also helped avoid a brittle shear failure and exhibited a gradual rather than abrupt reduction in capacity at higher drifts. Whereas the Control panel showed severe spalling, the No. 3 P-Bar panel with HPFRC experienced only minor flaking of the material in compression.

### CONCLUSIONS

Single-panel component experiments for a proposed retrofit technique were conducted. The goal of the experiments was to observe the hysteretic response and failure modes of infill panels using a self-consolidating HPFRC material, as well as to verify a proposed connection detail. Different panel reinforcement details, panel geometry, and panel cement-based materials were used to achieve varying objectives of high ductility, optimal material usage, and ease of construction.
The results from the single-panel cyclic tests reported herein show that variation in strength, hysteretic behavior, energy dissipation, and failure mechanism can be achieved through reinforcement detailing, panel material, and geometry. The effect of the reinforced HPFRC material compared to the reinforced concrete Control panel was primarily to transform the failure mode from a brittle shear failure to a more gradual flexural failure. At drift levels beyond 2%, an average increase of 50% in energy dissipation was also achieved with the HPFRC panels relative to the reinforced concrete Control panel. The multiple cracking phenomenon of the HPFRC panels correlated to the amount of energy dissipation seen in the tests. The panels that exhibited the largest distribution of cracking showed the most energy dissipation. At peak load, the deformation of the HPFRC panels was between 60 and 75% flexural deformation and between 25 and 40% shear deformation with negligible panel base slip contribution.

The first panel tested (V-Bar-1) exhibited connection slippage during testing, which led to premature panel failure initiated in the connection region. In this test, slippage was attributed to poor grout consolidation and low bolt pretensioning. In subsequent tests, a more flowable grout was used, along with a higher bolt pretensioning force. With these adjustments, the connection region behaved very well and could be reused up to six times, thus verifying the potential reusability of this proposed connection detail. Out-of-plane deformations were determined to be small enough to not significantly affect panel performance.

Using a low reinforcement ratio (No. 2 (6.4 mm [0.25 in.]) P-Bar panel) with smooth reinforcement led to early crack localization at the base and prevented significant distribution of cracking and thus energy dissipation. Whereas the use of smooth reinforcement was anticipated to assist in delaying yielding and fracture (and thus distribute cracking), the reinforcement used in this panel was too small and had too low a strain capacity to adequately assess crack distribution.

Tapering of the reinforcement or panel itself was shown to improve panel performance in terms of drift capacity and energy dissipation. The tapering attempted to lower the capacity of the panel along its height as the demand was reduced and led to a better distribution of damage throughout the panel, delayed crack localization at the panel base, and helped use more of the high-performance material.

Hand predictions of reinforced HPFRC panel capacity assuming a full tensile block—that is, from the extreme tensile fiber to the neutral axis—of HPFRC contributing to the capacity was found to predict panel capacity well in the case of HPFRC reinforced with deformed reinforcement. With smooth reinforcement, a smaller tensile block leads to a better estimate of the capacity due to less multiple cracking and earlier crack localization (that is, opening of a dominant crack). A detailed study of the strength of reinforced HPFRC is needed to provide simple, transparent design and analysis guidelines.

This retrofit scheme has recently been further investigated through double-height panel tests to validate all connection details and out-of-plane behavior of a full-height panel system. In addition, the response of a two-story single-bay frame infilled with HPFRC panels has recently been evaluated using hybrid simulation. All of these tests are in-plane loading experiments, and further research is needed on system response to 3-D loading to understand the impact of out-of-plane movement on system performance.

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