HIGH-PERFORMANCE FIBER-REINFORCED CEMENTITIOUS COMPOSITES FOR SEISMIC DESIGN: A REVIEW OF COLUMNS

W. Nguyen[1], Y. Shao[2], S.L. Billington[3], M.J. Bandelt[4], C.P. Ostertag[5]

[1] *Associate II, Wiss, Janney, Elstner Associates, Inc., wnguyen@wje.com*
[2] *PhD candidate, Stanford University, yishao@stanford.edu*
[3] *Professor, Stanford University, billington@stanford.edu*
[4] *Assistant Professor, New Jersey Institute of Technology, bandelt@njit.edu*
[5] *Professor, University of California, Berkeley, ostertag@ce.berkeley.edu*

Abstract

High-performance fiber-reinforced cementitious composites (HPFRCC) have emerged in recent decades as a promising class of materials for damage-resistant reinforced concrete structures, including bridge columns. Compared to ordinary concrete, HPFRCC carries tensile load after matrix cracking, has greater toughness under tension and compression, and promotes diffuse microcracking behavior which results in reduced maximum crack widths. Although the fundamental properties of HPFRCC materials have been widely studied, collective information on the large-scale seismic behavior of bridge columns incorporating these materials is less understood. This paper provides a brief review of the testing and response of HPFRCC bridge columns under earthquake loading. A variety of experimental columns constructed with HPFRCC are surveyed, including those incorporating precast elements, base rocking, posttensioning, and unconventional reinforcing materials. The discussed studies indicate that replacement of ordinary concrete with HPFRCC in columns can lead to greater damage tolerance and reduced residual deformations after an earthquake.

Keywords: HPFRCC; fiber-reinforced concrete; column; crack; plastic hinge

1. Introduction

With a growing number of deteriorated bridges requiring replacement in the coming decades, motivation exists to design and construct new bridge columns that are more durable and damage-resistant. Improving the serviceability of bridge columns located in seismically active regions is particularly important due to the structural, societal, and economic impacts of bridges in post-earthquake conditions. High-performance fiber-reinforced cementitious composites (HPFRCC) have been proposed and used in practice as an alternative to ordinary concrete due to HPFRCC's greater crack resistance, energy dissipation, and ductility [1]. Such material characteristics are desirable for bridge columns, which can undergo high deformations during a seismic event and are susceptible to concrete spalling and crushing, as well as rebar buckling and fracture.

This paper provides a brief review on the seismic response of experimental bridge columns utilizing HPFRCC, with a majority of the surveyed testing results published only within the past decade. Discussion on column designs incorporating HPFRCC, the influence of HPFRCC on damage states, and the influence of HPFRCC on residual drift performance are presented herein.

2. High-performance fiber-reinforced cementitious composites (HPFRCC)

Compared to conventional concrete, HPFRCCs have been characterized by multiple cracking behavior, smaller crack openings, and tensile pseudo-strain hardening after initial cracking [2]. The post-peak compression toughness of HPFRCC is also significantly greater due to fiber-bridging between cracks [3]. While different types of HPFRCCs have been researched for bridge columns, the following HPFRCCs are discussed in more detail due to their use in multiple bridge column research programs.

2.1 Engineered cementitious composites (ECC)

Engineered cementitious composites (ECC) are fiber-reinforced mortars that conform to a micromechanics-based design, resulting in high ductility with typical tensile strain capacities in the range of 1 to 5%, depending on the specimen type and test setup. ECCs that have been used in bridge column tests [4-9] are designed with polyvinyl alcohol (PVA) fibers at a volumetric fraction of 2%, based on total composite volume. Further information on the development and properties of ECC are reported by Li [10].

2.2 Hybrid fiber-reinforced concrete (HyFRC)

Hybrid fiber-reinforced concrete (HyFRC) is described by a fiber hybridization scheme that synergistically incorporates 2 or more fiber types, usually varied in fiber length, within a single concrete mixture. The fibers are understood to arrest and resist cracks of different lengths. In bridge column research, HyFRC based on the work of Blunt and Ostertag [11] has been specified by different researchers [12-16]. This HyFRC is typically designed with a total fiber volume fraction of 1.5% and consists of a hybridization of 8 mm-long PVA fibers and 30 mm-long to 60 mm-long steel fibers.

2.3 Ultra high-performance fiber-reinforced concrete (UHPFRC)

Ultra high-performance fiber-reinforced concrete (UHPFRC), also simply referred to as ultra high-performance concrete (UHPC), is a class of fiber-reinforced cementitious composites that achieve high compressive strengths, often in excess of 150 MPa [17]. Despite its nomenclature, UHPFRC can be designed as a mortar without coarse aggregate. The UHPFRCs used for bridge columns reviewed in this paper [9, 18-21] were designed with 2% to 3% steel fibers by volume when detailed information about the UHPFRC mix design was reported. Some of the reviewed UHPFRCs were also designed with a hybridization of straight steel fibers and hooked-end steel fibers within the same cementitious composite [20, 21].

3. Design considerations

Compared to conventionally designed, cast-in-place reinforced concrete bridge columns, the majority of experimental bridge columns surveyed in this paper include novel design features in addition to HPFRCC usage. Some of these design features are discussed next. For simplicity, the term "HPFRCC bridge column" is understood to describe a reinforced concrete column that has partial or full substitution of concrete with HPFRCC.

3.1 Precast and segmental construction

To incorporate HPFRCC material into a bridge column, precast elements are commonly specified by researchers (Fig. 1). A precast element avoids cast-in-place operations of fresh HPFRCC, which can have different workability characteristics than ordinary concrete and with which it can be difficult to ensure proper consolidation under field conditions due to congestion caused by fibers. Precast elements also conform to accelerated bridge construction (ABC) methodologies, as the precast elements can be more rapidly assembled onsite without the need to cure HPFRCC in the field, potentially reducing direct and indirect costs of bridge construction [22].

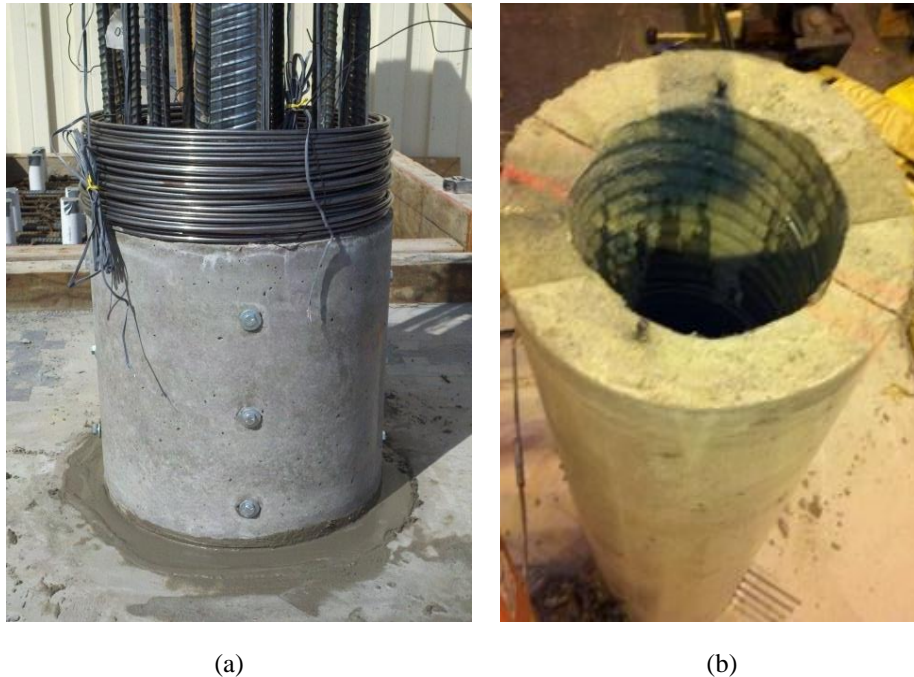(a)   (b)

Fig. 1 – HPFRCC precast elements: (a) Solid element at column base (Trono et al. [12]);
(b) Full-height tube (Nguyen et al. [14]).

Precast HPFRCC elements have been placed at the expected plastic hinge region of a column [7, 9, 12, 15, 16], fabricated as a full-height continuous precast element [14], and fabricated as one or more precast HPFRCC segments as part of a column assembled from multiple precast segments [6, 18-21, 23, 24]. Because HPFRCC is more expensive than conventional concrete and because damage-resisting properties of HPFRCC are not fully utilized in portions of the column that deform elastically, researchers have limited the placement of HPFRCC to the expected plastic hinge region, which is where the column is most susceptible to seismic damage. For a column design consisting of multiple precast segments, the segments are typically designed with openings to allow for unbonded posttensioning strands to pass through and provide a tensile load path for the segmented column. When fabricated as an HPFRCC precast element, the cross section of the element may be solid [6, 23, 24], hollow [18-21], or hollow with concrete infilled later [18, 21].

3.2 Rocking and unbonded reinforcement

Columns can rock at their bases by unbonding longitudinal reinforcement near joint interfaces (e.g., column-foundation interface). When properly designed, base rocking can avoid damage caused by plastic hinge formation during earthquake loading by reducing large curvatures where the hinge would occur and reducing residual displacements. Some researchers have detailed columns for base rocking by unbonding longitudinal rebar over a length equivalent to or greater than the gross diameter of the column [12-14]. Other researchers [9, 20, 25] did not design columns for base rocking, though did unbond longitudinal reinforcement over relatively shorter lengths. When unbonded, the longitudinal rebar can plastically deform over its unbonded length and avoid strain localizations that otherwise may occur during loading. The phenomenon of strain localization will be further discussed in Section 4.

3.3 Pre- or posttensioning

Pre- or posttensioning in columns is known to result in lower residual lateral displacements due to the restoring force provided by pre- or posttensioned strands. The strands are typically unbonded from concrete or HPFRCC along the column height. Use of these systems is pertinent in segmental construction as the strands provide tensile load continuity between segments.

3.4 Unconventional rebar materials

In addition to the use of HPFRCC materials in place of ordinary concrete, some researchers have investigated other metals as partial or complete substitution for low-carbon steel in longitudinal rebar. Panagiotou et al. [13] designed a bridge column with A316 stainless steel rebar, which is characterized by strain-hardening behavior after initial yielding, with no intermediate yield plateau regime, and has greater strain capacity at fracture. Similarly, Finnsson [15] specified 1.4362 duplex stainless steel rebar to determine the influence of the stainless steel's greater strain capacity.

Saiidi et al. [4] used a nickel-titanium alloy, commonly referred to as nitinol, as longitudinal reinforcement within the expected plastic hinge region of a column. Under stress, nitinol exhibits superelasticity, reaching high elastic strains even under repeated cyclic loading and resulting in lower residual deformations of the column after loading. The bars were smooth and were not fabricated with deformed ribs. Following this work, several bridge columns were also tested using nitinol longitudinal reinforcement [5, 7].

4. Tensile damage

4.1 Crack localization

Depending on the column design, flexural cracking of HPFRCC can be the first sign of observable damage during lateral loading. Although HPFRCC is known to result in finer, more distributed microcracking than concrete, continued lateral loading of an HPFRCC column can result in the widening of a few cracks and localized deformation where the widened cracks occur. At these cracks, longitudinal reinforcement may fracture at lower column deformations levels than expected due to strain localization.

Aviram et al. [25] tested two HPFRCC columns and a reference reinforced concrete column. The HPFRCC columns contained supplementary longitudinal dowel reinforcement at and near the column-footing interface in combination with continuous longitudinal reinforcement. The column designated as S1 was designed with partial unbonding of dowel reinforcement at and near the termination of the dowels within the column. The column designated as S2 had fully bonded dowel reinforcement, though a portion of continuous longitudinal reinforcement was unbonded at the elevation were the dowels terminated. After the columns reached a drift of 2.6%, widening of a few cracks could be observed for both HPFRCC columns, while the reference column did not exhibit such behavior. Sample S1 appeared to show localized widening at two or more flexural cracks and had all continuous longitudinal rebar fractured after reaching a peak drift of

10.7%. Sample S2 developed single-crack localization where the dowel reinforcement terminated within the column and some longitudinal rebar fractured after reaching 5.4% drift.

Through use of novel longitudinal rebar materials, the effects of flexural crack localization and rebar strain localization can be less influential on column ductility. Panagiotou et al. [13] tested a HyFRC column designated as TS-2 that also showed crack localization, as shown in Fig. 2a. During a quasi-static load cycle that was defined by a peak drift of 3.6%, the maximum crack opening was reported to be 10 mm wide and the measured strain in the longitudinal rebar was on the order of 7%. Despite strain localization, rebar fracture did not occur until the column reached drifts greater than 9.5%, owing to the high strain capacity of stainless steel compared to low-carbon steel. Finnsson [15] tested two HyFRC columns that differed in the selection of longitudinal rebar material. Sample PreT-BS was designed with low-carbon steel, while sample PreT-SS was designed with stainless steel. Both columns showed single-crack localization. Fracture of low-carbon steel rebar first occurred at a drift of 4.0% while fracture of the stainless steel rebar occurred after a greater drift of 7.2%. The greater tensile strain capacity of stainless steel was partly limited due to bar buckling. Tazarv and Saiidi [7] tested a bridge column with a precast ECC element located at the column base. A single dominant crack in the ECC formed and had a measured width of 21 mm at 5% drift, though fracture of the nitinol longitudinal reinforcement did not occur until after the column reached a peak drift of 10%. The delay of rebar fracture to greater column drifts and the lack of bar buckling in the column was attributed to the superelasticity of nitinol, which reduces plastic deformation under cyclic loading compared to low-carbon steel.
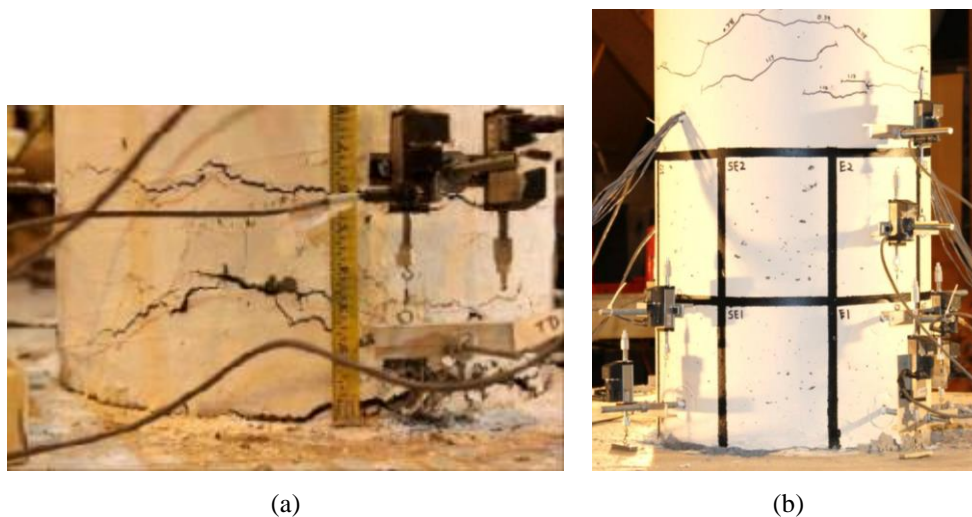

(a)   (b)

Fig. 2 – Observed cracking in HPFRCC bridge columns after reaching 3.6% drift:
(a) Crack localizations (Column TS-2, Panagiotou et al. [13]); (b) Lack of flexural cracking near the column base due to unbonded longitudinal reinforcement (Nguyen et al. [14]).

The phenomenon of flexural crack localization in laterally loaded HPFRCC bridge columns detailed with conventional longitudinal rebar is related to the high mechanical bond between HPFRCC and rebar. The bond between the materials resists plastic deformation of the rebar, shifting rebar plasticity to cracked sections of HPFRCC where bond is degraded. This shift leads to localized deformation of both the rebar and HPFRCC at a crack. Multiple localized cracks can open depending on characteristics such as HPFRCC type and reinforcement ratio. Further information on this behavior is found in the literature [26, 27].

4.2 Effect of unbonded rebar

Unbonding of reinforcement at and near a column-footing or column-cap beam interface has been observed to be effective in mitigating flexural crack localization and provides hysteretic energy dissipation through yielding and plastic deformation of rebar over the unbonded length. Where the longitudinal rebar is unbonded from HPFRCC and where a cold joint exists between the column and the footing, the HPFRCC is subjected to insignificant tensile stresses and does not form flexural cracks. Although HPFRCC is not

utilized for its tensile capacity under this condition, its high compression toughness can significantly contribute towards an overall damage-resistant bridge column. Discussion of the compression characteristics of HPFRCC is presented in Section 5.

Nguyen et al. [14] tested a HyFRC bridge column detailed for base rocking behavior with unbonded longitudinal reinforcement under quasi-static loading. The length of unbonded longitudinal rebar within the column was slightly greater than the column diameter. Flexural cracking was insignificant where the rebar was unbonded (Fig. 2b) and the rebar did not fracture until reaching a loading cycle with a peak drift of 11.3%. Trono et al. [12] designed a rocking, posttensioned HyFRC bridge column with unbonded reinforcement at and near the rocking plane, bonded longitudinal rebar that was discontinuous and armored with steel plates at the rocking plane, and unbonded posttensioning strands. The length of unbonded longitudinal rebar in the column was equivalent to the column diameter. During shake table testing, the column did not form dominant flexural cracks and the unbonded reinforcement contributed to hysteretic energy dissipation through yielding based on the measured flag-shaped hysteretic response. Longitudinal reinforcement did not fracture until after the column was subjected to a total of 11 ground motions and reached a peak drift of 8.8%.

Mohebbi et al. [9] dynamically tested a two-column bent assembly with one ECC column and one UHPFRC column. Unbonded longitudinal bars were detailed at or near column-cap beam (191 mm unbonded length) or column-footing (63 mm unbonded length) joints. Both specified unbonded lengths were less than the width of the square column (356 mm). Yielding was reported to occur where the longitudinal rebar was unbonded and flexural crack localization was not observed. The bent assembly reached peak drifts of respectively 5.7% and 9.6% prior to and during the ground motion that caused rebar fracture.

5. Compressive damage

5.1 Spall resistance

Due to the crack-bridging effect of fibers, HPFRCC columns have significantly greater cover spalling resistance than reinforced concrete columns. Spalling is understood to be the physical detachment of concrete from a column. When quasi-statically tested to large drifts in excess of 10%, some HPFRCC column designs have shown no observable spalling [4, 14]. Shake table tests in which HPFRCC columns were subjected to maximum drifts of up to 8% also showed insignificant spalling [12]. Although HPFRCC may not spall from a column, splitting cracks and bulging of the cover can occur under high compressive strains, as shown in Fig. 3. The presented damage is relatively minor compared to widespread spalling that is typical of reinforced concrete.

(a)   (b)

Fig. 3 – Compression damage in rocking HPFRCC bridge columns: (a) After 11 tested ground motions (Trono et al. [12]); (b) During loading cycle with a peak drift of 11.3% (Nguyen et al [14]).

Although HPFRCC covers have resulted in high spalling resistance for some column designs and testing, HPFRCC may be susceptible to spalling where crack localizations occur. Aviram et al. [25] tested two HPFRCC columns, with spalling prominent after subjecting the columns to 5.4% drift. The spalling was located near large flexural cracks that formed in prior loading cycles and longitudinal rebar buckling was observable at spalled regions. The ECC column tested by Tazarv and Saiidi [7] also showed spalling damage at and near a single flexural dominant crack, though rebar buckling was not reported.

Under shake table excitation, Motaref et al. [6] tested columns fabricated with four segments along the column height. The lower two segments positioned nearest the column-footing interface were fabricated with ECC. At the interface between ECC segments, some spalling was visible on both segments after the columns reached 5% drift. Additional spalling occurred with increased column drifts. Spalling was attributed to large compressive strains developed during opening and closing of the interface between the segments, as well as to the unconfined state of ECC cover near the joint.

UHPFRC materials are particularly resistant to spalling. Mohebbi et al. [9] dynamically tested a UHPFRC column and an ECC column that were connected to the same footing and cap beam. After subjecting the column-cap beam assembly to a total of 4 ground motions, with the fourth ground motion considered to be at the 100% design level, minor spalling was observed in the ECC column at the column-cap beam connection, while no spalling was noted for the UHPFRC column. UHPFRC used in segmental construction has also shown high spall resistance [18, 19, 21].

5.2 Longitudinal rebar buckling

The expected damage characteristics of a reinforced concrete column under earthquake loading are described initially by flexural cracking and cover spalling, leading to more severe damage states such as longitudinal rebar buckling. After cover spalling, rebar buckling deformations are resisted through tightly-spaced transverse reinforcing spirals or ties. Because HPFRCC cover has high spall resistance, some researchers have hypothesized that HPFRCC cover, even when cracked, can provide lateral constraints against rebar buckling and hence that transverse reinforcement requirements can be relaxed. HPFRCC has also been reported to increase shear strength compared to conventional concrete [10], countering the effect of reduced transverse reinforcement, which provides shear capacity within a column. Although transverse reinforcement detailing affects confinement behavior of reinforced concrete, limited information exists in the literature regarding the influence of transverse reinforcement on the confinement properties of reinforced HPFRCC.

Aviram et al. [25] designed two HPFRCC columns with a transverse reinforcement ratio of approximately 0.37% and a conventionally designed reinforced concrete column with a transverse reinforcement ratio of approximately 0.75%. The HPFRCC columns exhibited transverse spiral reinforcement fracture and longitudinal rebar buckling during the loading cycle with a peak drift of 5.4%. Spiral fracture and rebar buckling were not noted for the reinforced concrete column, though that column was tested to a lower maximum drift of 3.9%. Specimen TS-2 tested by Panagiotou et al. [13] was also designed with a transverse reinforcement ratio of 0.37% and had spiral fracture and longitudinal rebar buckling at a drift of 6%. In comparison, the reinforced concrete bridge column designated as column 415 and tested by Lehman et al. [28] was designed with a greater transverse reinforcement ratio of 0.70% and exhibited cover spalling and rebar buckling after reaching a drift of 5.2%. The limited results suggest HPFRCC columns with reduced transverse reinforcing can reach similar or greater drift ratios prior to rebar buckling than conventional reinforced concrete columns. However, because transverse reinforcement fracture preceded rebar buckling in the discussed HPFRCC columns, reduction in code-conforming transverse reinforcement may have other significant effects beyond those related to rebar buckling.

Because HPFRCC cover may not spall during experimental testing, visual observations of longitudinal rebar buckling can be difficult to achieve. Destructive removal of HPFRCC cover is also not commonly performed to visually inspect embedded rebar after structural testing. However, some researchers have reported the state of buckled reinforcement in HPFRCC columns. Kawashima et al. [29] tested a polypropylene fiber-reinforced cement composite (PFRC) bridge column with a cover that cracked but did not spall throughout the entirety of testing. The test concluded with a maximum applied drift of 4.4%. A steel

fiber-reinforced concrete (SFRC) and a reference reinforced concrete column were also tested to maximum applied drifts of 4.4% and 4.2%, respectively. During testing, a hammer was used to audibly detect the separation of cover from the column core and buckling deformations of longitudinal rebar were later verified by manually removing cover after testing. All columns in the testing program developed buckled longitudinal rebar. Compared to the PFRC and SFRC columns, the average buckling lengths and average lateral displacements in the reference column, which had significantly more cover spalling, were 18% to 40% and 16% to 42% greater, respectively. The results indicate that longitudinal rebar can buckle in the presence of a cracked HPFRCC cover, though the overall buckling deformations are reduced. Nguyen et al. [14] determined that the onset of longitudinal rebar buckling and observations of initial HPFRCC cover cracking occurred within the same loading cycle during quasi-static lateral loading. With continued loading, the cracked HPFRCC cover in addition to transverse reinforcement resisted lateral deformation of longitudinal rebar, with only limited buckling deformations incurred despite testing the column to a peak applied drift of 13.1% (Fig. 4).
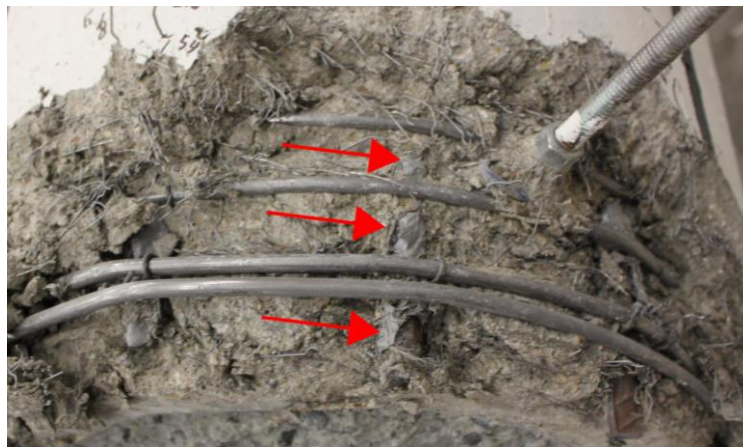

Fig. 4 – Limited longitudinal rebar buckling after test conclusion and manual HPFRCC cover removal. The column reached a peak drift of 13.1% (Nguyen et al. [14]).

6. Residual drift

Limitation of residual lateral drift is an important parameter for determining the serviceability of reinforced concrete columns after seismic events. As a measure of performance, some researchers have referenced the maximum allowable residual drift of 1% based on the seismic design specifications published by the Japan Road Association (JRA) [30]. Due to the influence of pre- or posttensioning on residual drift, column designs without and with systems are discussed separately in this section of the paper.

6.1 Column designs without pre- or posttensioning

A limited number of residual drift results from tested HPFRCC columns is available for evaluation from the literature. Kawashima et al. [31] subjected an HPFRCC column to 6 total ground motions based on the 1995 Kobe earthquake. A reinforced concrete column used for comparison in the study was tested under the same ground motions except the sixth and final ground motion, resulting in 5 total test excitations. The reference column was cast with conventional concrete and had a 2000 mm diameter circular cross section, as opposed to an 1800 mm square cross section for the HPFRCC column. After the fourth and fifth ground motions, the reference column had reported residual drifts of 0.3% and 1.8%, respectively. In comparison, after the fifth ground motion, the HPFRCC column had a lower residual drift of 0.49%, highlighting the difference in residual drift between the columns. The HPFRCC column continued to exhibit low residual drifts after a sixth ground motion test, after which the residual drift measured at 0.13%. The results indicate use of HPFRCC resulted in lower residual drifts, though the difference between residual drifts was more apparent after a greater accumulation of tested ground motions.

The effectiveness of HPFRCC towards lowered residual drift at greater loading severities is similar to the results of Saiidi et al. [4], who tested three columns under quasi-static lateral testing. One column was a reinforced concrete column that was fabricated with conventional concrete and low-carbon steel longitudinal rebar, the second column was fabricated with conventional concrete and nitinol longitudinal rebar at the base of the column, and the final column was fabricated with ECC and nitinol longitudinal rebar at the base of the column. For loading cycles with peak drifts up to 5%, use of ECC resulted in similar residual drifts. However, continued loading at larger imposed lateral displacements revealed lower residual drifts for the ECC column. For instance, after testing to 7% applied drift, the ECC column had 0.51% residual drift compared to 1.37% residual drift for the reinforced concrete column with the same nitinol reinforcing. The reference reinforced concrete column designed with steel reinforcement had the greatest residual drift (4.44%) after 7% applied drift, highlighting drift reduction due to the substitution of steel with nitinol, which exhibits superelasticity and can achieve greater elastic strain recovery than steel. Reductions in residual drifts using a combination of ECC and nitinol rebar in the plastic hinge region of a column, compared to a conventional reinforced concrete column, are also highlighted by Tazarv and Saiidi [7]. In the researchers' testing program, an ECC column exhibited noticeably lower residual drift compared to a reference reinforced concrete column after imposed peak drifts of 3% and greater. For instance, after 3% peak applied drift, the reference column exhibited a residual drift greater than 1%, while the ECC column did not reach this criterion until a peak applied drift of 10% was reached.

6.2 Column designs with pre- or posttensioning

Because pre- and posttensioning are effective in promoting recentering of a column after an earthquake, these methods have been incorporated into HPFRCC bridge column designs by some researchers. Haraldsson et al. [16] dynamically tested a pretensioned column with a precast, octagon-shaped HyFRC infilled shell element at the plastic hinge. A circular column constructed with cast-in-place conventional concrete was also referenced in the study to compare against the results of the HyFRC column. The reference column was not pretensioned. Partial test results from the study are presented in Table 1, showing the test run number, peak drift, and residual drift of the HyFRC column and reference column. After the fifth tested ground motion, the residual drifts of both columns were less than 1%. However, the difference in residual drifts between the columns became significantly greater after the sixth and seventh ground motions, highlighting the reduction of residual drifts through the use of HyFRC and pretensioning.

Table 1 – Partial shake table test results from Haraldsson et al. [16]: Measured peak and residual drifts.

Test number	Pretensioned HyFRC column		Reference column	
	Peak drift (%)	Residual drift (%)	Peak drift (%)	Residual drift (%)
5	5.3	0.22	5.7	0.83
6	5.5	0.23	6.0	1.57
7	9.2	1.25	10.7	6.70

Table 2 – Partial shake table test results from Trono et al. [12]: Measured peak and residual drifts.

Test number	Posttensioned HyFRC column		Reference column	
	Peak drift (%)	Residual drift (%)	Peak drift (%)	Residual drift (%)
5	6.2	0.1	5.8	0.9
6	5.0	0.2	6.1	1.6
7	8.0	0.4	10.8	6.8

Trono et al. [12] tested a HyFRC column detailed for controlled rocking at the column base. The column was designed with unbonded posttensioning strands, continuous longitudinal reinforcement that was unbonded from HyFRC at and near the column base, and longitudinal reinforcement that was discontinuous at the rocking plane and armored with headed plates. A conventional reinforced column of the same diameter and without posttensioning was used as a reference column for the experiment. Partial test results from the study are presented in Table 2. The HyFRC column showed significantly greater self-centering capability compared to the reference column. After 6 tested ground motions, the HyFRC column had a residual drift of 0.2% while the reference column had a residual drift of 1.6%. Although not shown in the table, the HyFRC column was tested to a total of 11 ground motions, reaching a peak drift of 8.8% and a final residual drift of 0.9%. The states of the HyFRC and reference columns after testing are shown in Fig. 5.

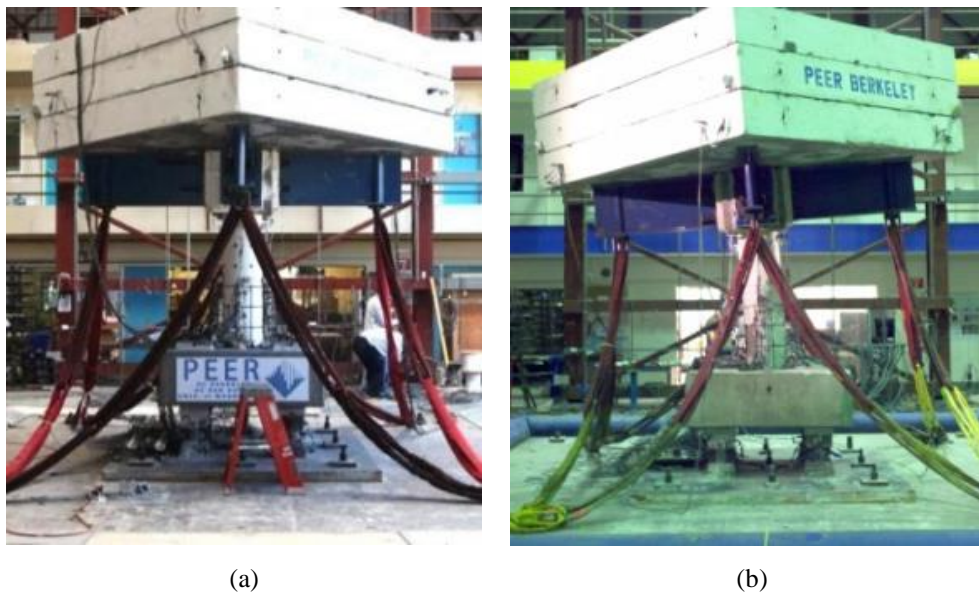(a)   (b)

Fig. 5 – Observed state of columns after ground motion excitations, (Trono et al. [12]):
(a) Posttensioned, rocking HPFRCC column after 11 total ground motions (0.9% residual drift);
(b) Reinforced concrete column after 7 total ground motions (6.8% residual drift).

For posttensioned columns constructed with precast segments along the height of the column, HPFRCC may be less influential on reducing residual drift, as described next. The segmental columns discussed in this section contain discontinuous longitudinal reinforcement and achieve a tensile load path through posttensioning strands or tendons. Billington and Yoon [23] quasi-statically tested posttensioned segmental columns with an HPFRCC segment located at the plane of column fixity. Columns consisting of concrete segments were also tested for reference. For applied drift levels of 4% or less, all tested columns showed residual drifts less than 0.2%, regardless of HPFRCC inclusion. After an imposed drift of 8%, the residual drifts for all column samples were similar and approximately 1%. The similarity in residual drift was attributed to localized cracking developed in all sample types and posttensioned tendons nearly reaching yield. On a shake table, Motaref et al. [6] tested a precast segmental column using ECC in the lower segments. A reference segmental column precast with concrete in all segments was also tested. Both columns were subjected to 7 total ground motions, with the ECC column showing greater residual drifts than the reference column after the fourth and subsequent ground motions. The residual displacement of the reference column was less than 0.5% for all test excitations, while the ECC column exceed 0.5% after the sixth excitation and exceeded 1% after the seventh and final excitation. Based on the results, ECC did not significantly reduce residual drifts after shake table testing. Yang and Okumus [19] quasi-statically tested precast, segmental columns with posttensioning strands. The segment located at the base of column nearest the column-footing interface was fabricated with either reinforced concrete, reinforced UHPFRC, or only UHPFRC. The researchers reported that the influence of UHPFRC on the self-centering of the column was insignificant to moderate, depending on whether shear slip was explicitly accounted for during testing.

7. Conclusions

A variety of bridge column designs have been tested by different research groups to evaluate high-performance fiber-reinforced cementitious composites (HPFRCC) under earthquake loading, with most of the experimental results published within the past decade. Column designs reported in the literature have varied in HPFRCC type, reinforcement detailing, use of pre- or posttensioning, and construction methods. Although this paper only provides a brief review of HPFRCC bridge columns, some collective remarks can be made about the damage behavior of these columns and their benefits when compared to conventionally designed and constructed reinforced concrete bridge columns:

1. Despite the multiple cracking behavior and greater strain capacity of HPFRCC compared to concrete, HPFRCC bridge columns are susceptible to flexural crack localization and rebar strain localization. HPFRCC bridge columns detailed with partially unbonded longitudinal reinforcement generally do not exhibit flexural crack localization and have demonstrated distributed rebar plasticity along unbonded lengths. Novel rebar materials with greater strain capacities are also effective in delaying rebar fracture to greater drifts.

2. HPFRCC has significantly greater spall resistance than conventional concrete. For HPFRCCs that have particularly high spall resistance, compression damage at high levels of lateral displacement is typically characterized by splitting cracks at the column base. Although HPFRCC cover may not spall or physically detach from the column core, cover cracking can lead to the onset of longitudinal rebar buckling. However, even when the cover is cracked, HPFRCC columns can exhibit reduced buckling deformations of rebar compared to reinforced concrete columns.

3. Replacement of conventional concrete with HPFRCC can result in lower residual lateral drifts after seismic loading, though the influence of HPFRCC on residual drifts has been reported to be marginal when maximum applied drifts during quasi-static loading are less than 3%. Residual drifts in HPFRCC columns can be reduced further with superelastic nitinol rebar detailing, pretensioning, or posttensioning. HPFRCC generally does not improve self-centering capability of precast, segmental columns designed with discontinuous longitudinal reinforcement and posttensioning strands.

8. Acknowledgements

9. References

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