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Publication Date

2017-08-15

DOI

10.1016/j.compstruct.2017.04.058

Peer reviewed

Seismic response of a rocking bridge column using a precast hybrid fiber-reinforced concrete (HyFRC) tube

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ARTICLE INFO

Article history:

Received 26 September 2016

Revised 13 January 2017

Accepted 23 April 2017

Available online 25 April 2017

Keywords:

Hybrid fiber-reinforced concrete
High-performance fiber-reinforced
cementitious composites
Reinforced concrete
Seismic
Bridge column
Accelerated bridge construction

ABSTRACT

With the expected replacement of deteriorated reinforced concrete bridges to occur in the next several decades, an opportunity exists to design more economic bridge columns in seismically active regions which are rapidly constructed while having enhanced crack resistance for extended service life. A precast hybrid fiber-reinforced concrete (HyFRC) tube, which contains a column's steel reinforcement, is proposed for accelerating construction and is to be filled with plain concrete at the construction site. An experimental column was subjected to static, uni-directional, cyclic loading and utilized a base-rocking design for further ductility enhancement. At the conclusion of the test, the column reached a peak drift ratio of 13.1% and showed minor damage, including elimination of cover spalling. The HyFRC tube column design contributed effective resistance against longitudinal reinforcing bar buckling, allowing the column to maintain 93% of its peak load capacity at 9.5% drift. Compared to a monolithic HyFRC column, the precast HyFRC tube column was constructed with less HyFRC volume while having similar seismic performance, making it a more sustainable alternative.

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1. Introduction

The deteriorating quality of bridges is a major durability and economic issue in the United States. Recent infrastructure studies revealed approximately 11% of 607,000 American bridges were classified as structurally deficient, requiring rehabilitation or complete replacement for potential reduction in designed load capacity [1]. Because bridge costs are directly related to the duration of onsite field operations, designs focused on accelerated bridge construction (ABC) to reduce project duration may be a worthwhile consideration [2,3].

As the conventional design of bridges is altered to accommodate ABC, evaluation of the causes of deterioration in bridges should be considered and improved upon in the design revision as well. Reinforced concrete (RC) bridges in seismic regions, highlighted here, are designed to deform in a ductile manner while preventing collapse of the structure [4]. In particular, structural column detailing currently allows for plastic hinge formation to occur at the column ends when large lateral displacements are induced. Damage of the column, in the form of cracking and con-

crete cover spalling, can occur during lower than design-level earthquake intensities [5]. Higher-intensity seismic events may cause longitudinal reinforcing bar (rebar) buckling and fracture, transverse rebar fracture, and concrete crushing. Repair and retrofit may be required to restore serviceability to such damaged bridge columns, which can be costly and not achieved immediately after an earthquake.

Prior to seismic loading, bridges may accumulate damage due to expansive deterioration processes such as rebar corrosion and alkali-silica reaction (ASR), which cause concrete cracking and spalling. Because damage caused by corrosion and ASR reduces a reinforced concrete member's load capacity [6–9], consideration of durability issues is paramount in new bridge designs, particularly those used in seismic regions.

To improve the damage-resisting properties of RC columns, a hybrid fiber-reinforced concrete (HyFRC) is utilized in this study. HyFRC is a type of high-performance fiber-reinforced cementitious composite (HPFRCC) that is designed by a multi-scale crack control approach [10,11]. This composite material was initially developed for bridge approach slabs with strong consideration of environmental loading caused by rebar corrosion as well as freeze-thaw cycling [12]. The concrete utilizes two to three fiber types, varying in material, shape, and length, to achieve progressive crack control from microcrack initiation to macrocrack propagation. Common fiber types include polyvinyl alcohol (PVA) microfibers and

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hooked-end steel macrofibers. HyFRC has recently been investigated specifically for bridge column use, including static seismic testing [13] and dynamic shake table testing [14]. The experimental tests of the columns revealed significant resistances towards spalling and longitudinal rebar buckling compared to conventionally designed columns. Fiber reinforcement has been shown to be effective in mitigating corrosion and ASR damage due to the fibers' enhanced crack resistance, which provides chemo-mechanical confinement of the expansive reaction products [15,16]. Particular to HyFRC composites, Blunt et al. [17] and Jen and Ostertag [18] investigated the corrosion resistance of flexurally loaded reinforced HyFRC beams and found a delay in corrosion initiation time and lower active corrosion rates compared to conventional reinforced concrete beams. Jen [19] found that HyFRC cast with aggregates susceptible to ASR provided effective constraint against the alkali-silica gel expansion and had improved retention of compressive strength and stiffness compared to plain concrete. Thus, use of the HyFRC composite is seen as a suitable replacement for plain concrete in bridge columns due to its high crack resistance when exposed to both seismic and environmental loading conditions.

The study presented herein investigates the feasibility of constructing and seismically evaluating a bridge column that synergistically adheres to the ABC and multi-scale crack control methodologies. A crucial requirement is to develop a strategy of utilizing HyFRC in a manner that professional construction engi-

neers could easily adopt into their current practices. Use of a precast HyFRC tube element is proposed as the primary design feature to achieve an expedited construction schedule while optimizing its placement in the column where its mechanical properties are most beneficial, improving the economy of the column. The precast tube, which contains the column's longitudinal and transverse reinforcing steel, is to be installed into a foundation and then filled with cast-in-place (CIP) plain concrete afterwards. This process aims to eliminate onsite rebar assembly for the column and provides a permanent formwork for the column.

The column was subjected to static, uni-directional, cyclic loading to evaluate its seismic performance. To further increase the damage-resistance of the experimental column, a base rocking design was utilized to avoid flexural cracking associated with plastic hinge formation at the column base. The seismic response of the column is compared to a similarly sized and tested rocking column fabricated with cast-in-place HyFRC.

2. Specimen design and construction

2.1. Design of column

The structural design of the column is presented in Fig. 1. The test specimen represents the lower half of a full-height bridge column deforming in double curvature, assuming a midheight inflec-

Fig. 1. Tube column design: (a) Structural details (units in mm); (b) Representative visualization of precast tube element. Select longitudinal rebar (T3, T10) are referenced for discussion.

Table 1
Summary of column design parameters.

Column	D (mm)	M/VD	Concrete	f'_c (MPa)	Cover thickness (mm)	ρ_l (%)	ρ_s (%)
Prototype column ^a	1830	4.2	Cast-in-place plain concrete	35 ^b	51	1.2	0.83
Tube column	406	4.1	Precast HyFRC	66	25	1.2	0.87
			Cast-in-place plain concrete	44	–		

^a Based on [20].

^b Typical nominal compressive strength.

tion point. The column's dimensions are based on a typical 15.2 m-tall column supporting a five-span box girder Californian bridge with a 1.8 m deck depth [20]. Based on the Caltrans seismic design criteria, key design parameters include the column diameter-superstructure depth ratio ($D/D_s = 0.7$ to 1.0) and longitudinal reinforcing ratio ($\rho_l = 1\%$ to 4%) [4]. The prototype column used for the tube column specimen was designed with diameter $D = 1.8$ m ($D/D_s = 1.0$), 44 No. 11 (35 mm diameter) longitudinal bars ($\rho_l = 1.2\%$), No. 8 (25 mm diameter) transverse hoops that are spaced 140 mm on center and result in a transverse reinforcing ratio of $\rho_s = 0.83\%$, and a concrete cover of 51 mm.

Table 1 provides a summary of pertinent design parameters. A geometric scaling factor of 4.5 was used to obtain the tube column's diameter ($D = 406$ mm). The column's shear span ratio, $M/VD = 4.1$, was nearly the same as the prototype's design value ($M/VD = 4.2$). The tube column used 12 No. 4 (13 mm diameter) steel bars (ASTM A706 Grade 420 [21]) for longitudinal reinforcement ($\rho_l = 1.2\%$). A continuous W4 (5.7 mm diameter) galvanized steel wire spiral (ASTM A82 [22]), spaced 32 mm on center, was used as transverse reinforcement ($\rho_s = 0.87\%$).

A precast HyFRC tube, which contains the column's steel reinforcement, was determined to be the most practical method for accelerating construction. The precast tube element expedites construction by eliminating the need for rebar assembly to be done onsite for the column as well as by providing permanent formwork for the interior plain concrete to be cast into. Additionally, casting of the HyFRC in a precasting facility will likely improve quality control, as cast-in-place construction of fiber-reinforced concrete is generally not common in the United States. A 16 gauge (1.6 mm thick) corrugated galvanized steel duct (ASTM A796 [23]), initially used as the inner permanent formwork of the precast tube, provided proper mechanical bonding between the precast HyFRC and plain concrete. The longitudinal rebar embedded in the tube extruded 457 mm from the tube's bottom surface to allow for later installation into the foundation and were capped with steel heads for anchorage. Because HyFRC is more expensive than plain concrete, the precast tube column design improved economy by containing approximately 43% less HyFRC by volume compared to a column cast entirely with HyFRC.

The column utilized a base rocking design to avoid flexural crack localization caused by plastic hinge formation [13,24,25]. Flexural crack localization is caused by the higher mechanical bond that exists between HPFRCC and the reinforcing steel bar due to the HPFRCC's effectiveness in suppressing splitting cracks [26]. Base rocking was achieved by allowing the longitudinal reinforce-

ment to be unbonded in the lowest portion of the column over a length of 457 mm, with an additional 51 mm of unbonded length extending into the foundation. In a rocking column, the concrete in the unbonded region develops insignificant tensile strain. However, it remains susceptible to compression damage, including spalling of concrete cover [27–29]. Use of HyFRC in rocking columns has been shown to be effective in resisting compression damage at the rocking plane, improving the overall ductility of the column [13,14]. To unbond the rebar, rubber mastic tape was tightly wrapped around the bars to build thickness between the ribs such that anchorage would be negligible. A final layer of duct tape was used for additional slippage (Fig. 5a). Because the rebar crossing the rocking interface is covered, exposure to corrosion-inducing salts from the environment is mitigated while the column would be in service.

2.2. Materials and properties

A variant of the self-consolidating HyFRC design detailed by Jen et al. [30] was used for the column. Table 2 summarizes the mix proportions of the concrete used for the tube column. While the HyFRC was not required to maintain the self-consolidating properties of the baseline design, a slump flow diameter of 432 mm to 483 mm was achieved on the day of casting (Fig. 2). The concrete used 0.04 mm-diameter, 8 mm-long PVA fibers and 0.55 mm-diameter, 30 mm-long hooked-end steel fibers. Table 3 provides a summary of fiber properties. The PVA fibers and steel fibers were designed at 0.2% and 1.8% fiber volume fraction, based on total concrete volume, respectively. This total fiber volume fraction of 2% is greater than the total volume fraction of 1.5% typically associated with HyFRC, which has been suggested as an upper design limit for cast-in-place construction due to the tendency of fiber clustering and fresh concrete fluidity reduction caused by the inclusion of additional fibers [30]. The HyFRC for the tube column was able to exceed this critical fiber volume fraction as vibration equipment used during laboratory casting was able to mitigate the reduced workability caused by the fibers, mirroring the expected higher quality control and more varied casting techniques available in a concrete precasting facility as opposed to a field condition. A HyFRC with a greater fiber volume fraction may simultaneously be designed with a higher compressive strength while maintaining ductile behavior [31]. Because coarse aggregates, normally having a maximum size of 9.5 mm, were removed from the mix to better accommodate casting of the fresh HyFRC material through the tube's critical thickness of 64 mm, an increase in compressive

Table 2
Concrete mix proportions per 1 m³ of concrete.

Concrete type	Binder (kg)		Aggregates (kg)		Batched water (kg)	Chemical admixtures (wt.% binder)		Fibers (vol.%)	
	Cement	Fly ash	Coarse	Fine		SP ^a	VMA ^a	PVA	Steel
HyFRC	490	163	–	1306	213	1.09	1.59	0.2	1.8
Plain concrete	476	–	693	876	228	–	–	–	–

^a SP and VMA refer to superplasticizer and viscosity modifying admixture, respectively.

Fig. 2. Slump flow test result of fresh HyFRC.

strength was expected. The strength was further increased from

Table 3
Fiber properties.

Fiber type	Length (mm)	Diameter (mm)	Aspect ratio (length/diameter)	Strength (MPa)	Elastic modulus (GPa)
PVA	8	0.04	200	1600	43
Hooked-end steel	30	0.55	55	1100	200

the baseline design by reducing the water-binder ratio from 0.45 to 0.35.

Mechanical testing of concrete occurred six days after column testing. The HyFRC had a compressive strength (f_c) of 66 MPa after 73 total days of curing while the strain at peak stress (ϵ_0) was -0.0042 . Fig. 3 shows 4-point bending test results of beams with a square cross section of 152 mm by 152 mm and a length of 610 mm. The behavior of the beams satisfied the performance-based deflection-hardening criteria for HyFRC design as detailed in [10]. The plain concrete used for the interior of the column was designed with a nominal 28-day compressive strength of 35 MPa and had a water-binder ratio of 0.48. At an age of 46 days, it had a compressive strength of 44 MPa. Future design considerations may include the use of a more sustainable concrete with high replacement of cement by industrial waste products, such as fly ash, since early strength development of the interior concrete is not crucial in this precast tube column.

Fig. 4 plots the stress–strain behavior of the reinforcing steel used for the column. The longitudinal reinforcement exhibited a yield stress of 480 MPa and an ultimate stress of 660 MPa. The spiral transverse reinforcement, which required straightening prior to testing, did not exhibit a yield plateau. The 0.01% proof stress of the spiral transverse reinforcement was 450 MPa at a strain of 0.0032 and the ultimate stress was found to be 640 MPa. Due to plastic deformation that occurred to allow direct tensile testing of the original spiral-shaped material, the yield strain of the spiral steel may have been overestimated.

Fig. 3. 4-point bending test results of HyFRC.

Fig. 4. Tensile test results of reinforcing steel.

2.3. Construction of test specimen

The tube column was constructed in the University of California, Berkeley, structural engineering laboratory in three primary stages:

1. After internal data instrumentation was applied onto the longitudinal rebar, the bars were wrapped for debonding and then assembled with the spiral reinforcement (Fig. 5a). The tube was cast with HyFRC in a single batch with a volumetric yield of 0.26 cubic meters, which was prepared in the laboratory mixer. The corrugated steel duct provided permanent formwork for the interior surface of the tube, while removable formwork was used for the exterior surface. The HyFRC was poured from the top elevation of the tube and flowed to its base with the aid of vibration equipment. After 14 days of curing, the outer formwork was stripped, revealing no significant defects or voids.
2. The precast tube was installed into the foundation block after 18 total days of curing. A crane was used to hoist and position

Fig. 5. Tube column during construction: (a) Steel reinforcement prior to casting of tube, showing duct tape used for rebar debonding; (b) Precast HyFRC tube aligned prior to casting of foundation; (c) Overhead view of specimen prior to casting of its hollow interior.

the tube such that the bottom surface of the tube would be coincident with the foundation's top finished surface (Fig. 5b). Concrete for the foundation was provided by a mixing truck.

- After the foundation cured for 9 days, the hollow tube was filled with plain concrete. Fig. 5c shows the specimen prior to the casting of its interior. The concrete was produced in the laboratory mixer in a single batch. At the time of column testing, the tube HyFRC, foundation concrete, and column interior concrete had an age of 67 days, 49 days, and 40 days, respectively.

3. Experimental setup and test program

The column was tested under static, uni-directional, cyclic loading. Two horizontal actuators, inclined at 45 degrees in the direction of displacement, were used to apply lateral load to the column (Fig. 6). To simulate gravity load, a constant vertical load ($P = 445 \text{ kN}$) was applied onto the column through a steel spreader beam attached to two vertical actuators. The value of vertical load

was selected by setting $P/f_c A_g = 0.1$, using $f_c = 35 \text{ MPa}$ as the nominal compressive strength, which is common in bridge column design. A_g is the gross cross-sectional area of the column. Four post-tensioned rods were used to prevent uplifting of the foundation from the structural testing floor.

The column was loaded cyclically in the east-west direction under a drift ratio-controlled protocol. The drift ratio θ is defined as the lateral displacement at the point of load application over the vertical distance from the same point to the column base. Table 4 summarizes the loading protocol of the column. Loading sequences up to the peak drift ratio of $\theta = 3.6\%$ were tested in three consecutive cycles prior to increase in amplitude. All sequences after the peak drift ratio of 3.6% consisted of one cycle.

Strain gauges were placed along the longitudinal rebar within its unbonded length and were also placed on the transverse spiral steel near the base of the column. Within the unbonded region, linear variable displacement transducers (LVDTs) were placed in series along the column elevation on the north, south, west, and east

Fig. 6. Column test setup: (a) Plan view details; (b) Global view.

Table 4
Loading protocol.

Sequence (3 cycles)	Peak drift ratio (%)	Sequence (1 cycle)	Peak drift ratio (%)
1	0.15	10	4.2
2	0.30	11	4.8
3	0.44	12	6.0
4	0.60	13	7.1
5	1.2	14	8.3
6	1.8	15	9.5
7	2.4	16	11.3
8	3.0	17	13.1
9	3.6		

faces. Wire potentiometers measured the translation of the column at the point of load application.

4. Experimental results and discussion

4.1. Damage mitigation

The tube column showed excellent overall damage resistance during testing. Fig. 7 reveals a global view of the southeast face of the column at the conclusion of select drift ratio cycles. Damage up to $\theta = 4.8\%$ was generally limited to hairline flexural cracks at elevations above the unbonded region and was negligible at the column base (Fig. 7a–b). Pens were used to trace all observed surface cracks during testing, though the marking widths were not indicative of the actual crack openings produced. After $\theta = 4.8\%$, no detectable crack propagation was observed for existing cracks away from the unbonded region of the column. However, cracking could be detected at the column base (Fig. 7c–d). Fig. 8a–c shows the local observed state of the southeastern face of the column at increasing loading cycles. A splitting crack was initially observed on the southeast face at $\theta = 6.0\%$ (Fig. 8b), being the first significant crack detected within the unbonded region. Additional loading cycles continued to increase the crack width, which was the location of the most severe damage to the HyFRC (Fig. 8c). On the northwest face of the column, damage consisted of very fine splitting cracks, and a dominant crack was not observed (Fig. 8d). Concrete cover spalling did not occur throughout the entirety of the test and visible lateral deformations of longitudinal rebar were also not detected at the conclusion of testing (after $\theta = 13.1\%$), as shown in Fig. 7d. A total of two longitudinal reinforcing bars was noted to have fractured, one each during the $\theta = 11.3\%$ and $\theta = 13.1\%$ cycles.

Terzic and Stojadinovic [32] tested a conventionally designed and constructed bridge column (specimen Base45) with the same diameter, same longitudinal reinforcing ratio and similar transverse reinforcing ratio as the tube column. The column was tested statically, though with bi-directional lateral displacement. After the column reached a drift ratio of 3.9%, severe spalling was observed and cracking penetrated the concrete core, reducing its volume. Seismic testing of Base45 ceased after this event. In statically loaded, circular RC bridge columns subjected to cyclic lateral displacement, longitudinal rebar buckling spanning several spiral spacings is expected in the load cycle immediately following the cycle in which complete cover spalling and exposure of reinforcement occurs [33]. Though the loading protocol for Base45 and the HyFRC tube column differed, the latter column was subjected to several additional cycles with peak drifts much higher than Base45's peak drift of 3.9% while having significantly less concrete damage.

4.2. Measured response

Fig. 9 presents the lateral load versus drift ratio response of the tube column up to $\theta = 6\%$ and for the complete test. Positive lateral displacements correspond to eastward movement. The column reached a peak lateral load resistance of 120 kN during the drift cycle of $\theta = 4.8\%$. The post-peak curve of the column showed only mild load resistance degradation up to $\theta = 9.5\%$, revealing retention of over 90% of peak load capacity. During the sequence of $\theta = 11.3\%$, the column maintained 83% of its peak resistance despite one of the 12 longitudinal bars fracturing. In conventional RC bridge columns, a general, though arbitrary, measure of column failure occurs when the lateral load resistance has reduced to 80% of its peak value [33]. The tube column did not meet this criterion until the final loading cycle, which had a maximum drift of $\theta = 13.1\%$. Throughout the test, the applied gravity load of $P = 445$ kN was resisted and did not significantly deviate.

The column base rotations, based on displacement transducer measurements located nearest the column–foundation interface, are plotted against the imposed drift ratio of the column in Fig. 10 for drift cycles up to $\theta = 7.1\%$. The response of a perfectly rigid column is shown in the plot as having equivalent values of drift ratio and base rotation. The measured data shows good agreement with the idealized behavior, suggesting the debonding of the reinforcing steel was effective in producing the intended rocking behavior.

Fig. 7. Observed global state of southeast (SE) face of tube column at neutral position after completion of peak drift ratio θ : (a) $\theta = 1.2\%$; (b) $\theta = 4.8\%$; (c) $\theta = 9.5\%$; (d) $\theta = 13.1\%$. The painted gridlines on the column face are completely within the column's unbonded region. Observed cracks are traced with pens during testing.

Fig. 8. Observed local state of tube column at neutral position after completion of peak drift ratio θ : (a) Southeast (SE) face, $\theta = 4.8\%$; (b) SE face, $\theta = 6\%$; (c) SE face, $\theta = 9.5\%$; (d) Northwest (NW) face, $\theta = 9.5\%$. Observed cracks are traced with pens during testing.

Fig. 9. Lateral load versus drift ratio response: (a) Up to $\theta = 6.0\%$; (b) Complete response.

Strain gauge measurements of rebar T10 are presented in Fig. 11a. The location of the bar in the column cross section is identified in Fig. 1a. The plot shows measurements taken at different elevations ($h = 102$ mm and 203 mm) within the unbonded length of the rebar, showing good equivalency between the two sets approximately up to the completion of eastward movement during $\theta = 4.8\%$. The similar strain values at different elevations on the bar suggest a uniform spread of plasticity in that domain, owing to the debonding of the rebar from the HyFRC matrix. During the load

cycle of $\theta = 4.8\%$, disparities in the minima of the measurements in rebar T10 become apparent, indicating the initiation of localized strain deformation related to bar buckling. This nonuniform behavior was followed by the formation of a visible splitting crack formed near the location of rebar T10 in the next loading sequence ($\theta = 6\%$) (Fig. 8b), which coincided with softening of the lateral load resistance of the column (as shown in Fig. 9). During the same cycle, the measured compressive strain of the west column face reached approximately $\varepsilon = -0.0040$ (Fig. 11b), nearly matching

Fig. 10. Base rotation versus drift ratio response up to $\theta = 7.1\%$, with a reference line representing perfectly rigid column deformation.

the compressive strain at peak load of the HyFRC ($\varepsilon_0 = -0.0042$). The results suggest the onset of lateral load softening is governed in part by macrocrack formation in the HyFRC cover and by initial rebar buckling behavior.

Fig. 12 shows the strain profile of the column's spiral transverse reinforcement at select drift ratio cycles. Strain gauges located on the western and eastern faces of the column are considered for the plot. The strain development in the spiral is minimal up to the drift ratio of $\theta = 7.1\%$ and the steel remains elastic up to $\theta = 9.5\%$ despite the peak lateral load of the column occurring several load cycles prior ($\theta = 4.8\%$). When a conventional RC column is in a compressive stress state, volumetric dilation of the concrete core against the transverse reinforcing steel causes a stress response in the transverse reinforcing steel, which in turn applies lateral confining stresses on the concrete core by the steel. These confining stresses improve the overall strength and ductility of the reinforced concrete [34], and the concrete core is defined as the volume of concrete enclosed by the transverse reinforcement. In the tube column, the corrugated steel duct confines and restricts volumetric dilation of the plain concrete interior, but does not confine the HyFRC. Insufficient composite material dilation perpendicular to the spiral steel's tangential direction resulted in delayed stress development in the steel, suggesting the HyFRC does not benefit from confinement by the transverse steel when the spiral steel itself is not stressed. Despite minimal strain development in the spiral steel, the high retention of residual peak lateral load, being 97% of its peak resisted load at $\theta = 7.1\%$, indicates satisfactory overall post-peak toughness with unconfined HyFRC. The result is consistent with the work performed by Trono [35], who reported that the compressive post-peak softening curves of spirally-confined reinforced HyFRC, up to transverse reinforcing ratios of $\rho_s = 1.0\%$, had the same ductile response as reinforced HyFRC lacking transverse reinforcement, suggesting unconfined HyFRC composites can exhibit similar ductility as confined HyFRC composites.

Under high compressive strains, longitudinal reinforcement is susceptible to buckling after the concrete cover has cracked and spalled from a bridge column, resulting in load capacity degradation. Lateral displacements of the longitudinal bars are typically restricted by the presence of transverse reinforcing steel. However, the aforementioned lack of measured strain in the spiral reinforcement of the tube column up to $\theta = 7.1\%$ indicates insignificant lateral deformation of the longitudinal bars against the spiral

Fig. 11. Strain measurements plotted as a function of testing time: (a) From strain gauges placed on longitudinal rebar T10; (b) From displacement transducers placed on west column face. The abscissa is labeled as the time when a sequence of drift ratio cycles has completed. Strain gauges are located at an elevation h above the column base. Displacement transducers have a gauge length located between indicated elevations h of column.

reinforcement, which would otherwise produce a hoop stress in the spiral steel. It is suggested that lateral deformations of the longitudinal bars are restrained continuously along their lengths by the HyFRC, which showed no observable spalling damage due to its high crack resistance, rather than being restrained at discrete locations where the bar is in contact with the spiral steel. Fig. 13 shows the effectiveness of the longitudinal rebar buckling restraint for rebar T3, which was located near the extreme western column face. Manual excavation of the HyFRC cover revealed the bar only had mild buckling deformation after the conclusion of the test. Future iterations of the tube column design may consider lowering the transverse reinforcing ratio due to the delayed contribution of the steel spiral towards concrete core confinement and longitudinal bar buckling as described.

The residual lateral load capacity of the tube column was relatively high compared to conventional RC bridge columns. The specimen designated as column 415 and tested by Lehman et al. [33] had a diameter of $D = 610$ mm, longitudinal reinforcing ratio of $\rho_l = 1.5\%$ and transverse reinforcing ratio of $\rho_t = 0.70\%$. The column was also tested under static, uni-directional cyclic lateral displacement. At the initial cycle with peak drift ratio of 5.2%, final cover

Fig. 12. Strain profile of spiral steel transverse reinforcement. Strain measurements consider gauges located on western and eastern faces of the column, whose magnitude is taken as the greater of the strain measured during maximum lateral displacement in the westward or eastward direction.

Fig. 13. Observed final state of rebar T3 after manual excavation of HyFRC cover.

spalling occurred and longitudinal rebar buckling was observed. A 20% reduction in peak lateral load capacity was measured at the third consecutive cycle with this same peak drift ratio. Use of HyFRC is thus advantageous for its spalling resistance and restraint against rebar buckling, which resulted in higher retention of peak lateral load capacity at higher drift ratios compared to column 415.

4.3. Comparison to a cast-in-place HyFRC rocking column

The performance of the tube column is compared to a column of similar design and loading protocol, but was cast entirely with a self-consolidating HyFRC in monolithic cast-in-place construction [13,36]. The CIP HyFRC bridge column had the same diameter ($D = 406$ mm), same longitudinal reinforcing ratio ($\rho_l = 1.2\%$) and similar height ($H = 1708$ mm) as the tube column. Additionally, the CIP column utilized a similar rocking mechanism as the HyFRC tube column by debonding of longitudinal reinforcement from the HyFRC within the expected plastic hinge region, though the length of debonding within the CIP column was 406 mm, which was 51 mm less compared to the precast HyFRC tube column. The

transverse reinforcement of the tube column ($\rho_s = 0.87\%$) differed from that of the CIP column ($\rho_s = 0.37\%$). Both columns were subjected to similar seismic testing protocols, using static, uni-directional, cyclic loading at the same target peak drift values, though sequences at $\theta = 3.0\%$ and $\theta = 13.1\%$ were excluded from the testing program of the CIP column. The same magnitude of constant gravity load ($P = 445$ kN) was also used.

The HyFRC of the tube column had a greater fiber volume fraction ($V_f = 2\%$ compared to $V_f = 1.5\%$) and greater compressive strength ($f_c = 66$ MPa compared to $f_c = 35$ MPa) than the self-consolidating HyFRC of the CIP column. As previously described in Section 2.2, the HyFRC used for the tube column was designed with a greater total fiber volume fraction than typical HyFRC, despite a reduction in workability, due to the expected higher quality control available in a precasting facility. The design strength of the HyFRC is allowed to increase due to the greater fiber volume fraction, as the composite maintained satisfactory deflection-hardening behavior. Despite differences in the HyFRC material properties between the two columns, comparison of the tube column and the CIP column is appropriate when considering how the method of column construction dictates the type of HyFRC that should be used. A self-consolidating HyFRC is not required for fabricating precast elements, though would be beneficial for casting bridge columns at a construction site.

Use of a precast HyFRC tube appears to be an acceptable alternative construction method in lieu of conventional cast-in-place designs. Fig. 14 shows the complete lateral load versus drift ratio response of the tube column and CIP column in superposition. The responses of the two columns are similar up to approximately $\theta = 4.8\%$. The post-peak toughness of the precast HyFRC tube column is noticeably greater than that of the CIP HyFRC column, which was still ductile. At $\theta = 7.1\%$, the tube column and CIP column maintained at least 97% and 80% of peak load resistance, respectively. Further loading to $\theta = 9.5\%$ revealed poorer peak load capacity retention for the CIP column (61%) than the tube column (93%).

The cover thickness of the columns appeared to influence the softening response of the columns. Because the precast tube element was designed such that the longitudinal and transverse reinforcing steel was located approximately at the tube's midthickness, the HyFRC cover in the tube column (25 mm) was greater than that of the CIP column (13 mm). During seismic testing of the CIP col-

Fig. 14. Lateral load versus drift ratio response of HyFRC tube column and cast-in-place HyFRC column [13], with envelope curves explicitly plotted.

umn, initial buckling of a longitudinal rebar was observed as early as $\theta = 2.4\%$. The unexpected behavior was attributed to a construction flaw of locally thin HyFRC cover (8 mm) where the buckling deformation occurred, highlighting the effect of cover thickness on buckling resistance. Four of the 12 total longitudinal reinforcing bars in the CIP column were visually observed to have some buckling deformation after reaching $\theta = 7.1\%$. After completing the same drift ratio cycle, the tube column showed no visible rebar buckling behavior and the spiral reinforcement exhibited minimal strain development, indicating the HyFRC cover dominated the resistance against rebar lateral deformation, as discussed in Section 4.2. The ineffective response of the spiral steel also suggests a negligible effect of a higher transverse reinforcing ratio present in the tube column compared to the CIP column up to $\theta = 7.1\%$.

Based on the load-drift ratio responses and observed damage of both the tube column and the CIP column, contribution of a deflection-hardening, spall-resistant HyFRC material to the overall toughness of a rocking bridge column is most effective when restraining longitudinal rebar buckling. The design of the tube column optimizes the use of HyFRC for this task, limiting the placement of a higher-strength HyFRC primarily around the longitudinal bars in the form of a precast tube element with an increased HyFRC cover. This optimized use of HyFRC in a precast tube, while using a plain concrete interior, may be considered a more economic and sustainable alternative due to the reduction of required HyFRC volume, while still maintaining satisfactory seismic performance. It should be noted that the CIP HyFRC column design remains a viable option for bridge column construction due to its toughness enhancement compared to conventional RC columns, which is further detailed in [13].

5. Summary and conclusions

This paper presents the seismic response of a precast hybrid fiber-reinforced (HyFRC) tube column detailed for accelerated bridge construction. The precast HyFRC tube, which contains all of the column's longitudinal and transverse reinforcement, expedites construction by eliminating onsite steel assembly for the column, eliminating temporary column formwork, and avoiding onsite casting of fiber-reinforced concrete. Compared to a monolithic HyFRC column, the tube reduced HyFRC material volume consumption by 43% by specifically placing it where its damage-resisting benefits would be optimized, namely at the column base, where concrete compressive damage and rebar buckling are most likely to occur, and directly surrounding the column's steel reinforcement, for enhancing the column's resistance against steel rebar corrosion while in service. A base rocking design was used to eliminate flexural crack localization where the column's plastic hinge would typically occur, and the column was tested under static, uni-directional, cyclic loading conditions up to $\theta = 13.1\%$. The conclusions are as follows:

1. The HyFRC tube column exhibited greater toughness during seismic loading compared to a monolithic cast-in-place (CIP) HyFRC column that had similar geometry, longitudinal steel reinforcing ratio, and rocking mechanism [13]. The lateral load-drift ratio responses of the two columns were similar up to $\theta = 4.8\%$. Continued testing at larger peak drift ratios revealed that the HyFRC tube column had a higher residual load capacity than the CIP HyFRC column – at $\theta = 9.5\%$, the HyFRC tube column and CIP HyFRC column maintained 93% and 61% of their peak load capacity, respectively.
2. Compared to the conventionally designed reinforced plain concrete column Base45, which had similar geometry, longitudinal steel reinforcing ratio, and transverse steel reinforcing ratio

[32], the HyFRC tube column exhibited superior damage resistance. The Base45 specimen had extensive spalling of cover concrete at its base after a peak drift of 3.9% had been reached. The HyFRC tube column did not show visible surface cracking at its base until $\theta = 6\%$, which was in the form of minor splitting cracks. At the end of the test, the HyFRC tube column showed no spalling or visible rebar buckling.

3. The presence of an internal steel corrugated duct, which was necessary for fabrication of the precast tube element, confined and restricted the volumetric dilation of the enclosed plain concrete, resulting in a minimal strain response of the spiral transverse reinforcement up to $\theta = 7.1\%$. HyFRC in the column does not receive lateral confining stresses from the transverse reinforcement to at least that drift cycle.
4. Use of HyFRC improved the overall toughness of the column by resisting longitudinal rebar buckling. The lack of strain development in the transverse spiral steel at drift ratios up to $\theta = 7.1\%$ further suggests lateral deformations of the rebar were restrained continuously along the HyFRC, rather than at discrete points where the longitudinal bars were in contact with the spiral reinforcement.
5. Rocking behavior eliminated flexural cracking where debonding of the longitudinal rebar from the HyFRC was defined. Strain in rebar T10 was nearly equivalent over a 102 mm unbonded length up to $\theta = 4.8\%$, suggesting distributed plasticity.

Acknowledgments

The authors gratefully acknowledge the support provided by the Pacific Earthquake Engineering Research (PEER) Center for this study. The authors would also like to thank L. Stepanov for his assistance during specimen casting and J. Carr and P. Wong for their assistance during structural testing.

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