Behavior of Short Columns Constructed using Engineered Cementitious Composites under Seismic Loads

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ABSTRACT

The present research reports the application of engineered cementitious composites (ECC) as an alternative to conventional concrete to improve the brittle shear behavior of short columns. Experimental and finite element investigation was conducted by testing five reinforced engineered cementitious composite (RECC) concrete columns (half-scale specimens) and one control reinforced concrete (RC) specimen for different shear-span and transverse reinforcement ratios under cyclic lateral loads. RECC specimens with higher shear-span and transverse reinforcement ratios demonstrated a significant effect on the column shear behavior by improving ductility (>5), energy dissipation capacity (1.2 to 4.1 times RC specimen), gradual strength degradation (ultimate drift $>3.4\%$), and altering the failure mode. The self-confinement effect of ECC fibers maintained the integrity in the post-peak region and reserved the transmission of stress through fibers without noticeable degradation in strength. Finite element modelling of RECC specimens was carried out by adopting simplified constitutive material models. It was apprehended that the model simulated the global response (strength and stiffness) with an accuracy of about 95%, and captured the shear and flexure crack patterns reasonably well.

INTRODUCTION

To meet the functional requirements for ample space and large span, in most reinforced concrete (RC) frame buildings and high-rise building constructions, short columns have become unavoidable, specifically in the basement floors. Contrarily, short columns were also formed due to partial height walls, large openings in walls, and mezzanine floors (Basha and Kaushik 2019). When such buildings are exposed to moderate- to strong-earthquakes, short columns exhibit brittle shear failure, leading to significantly lower ductility, energy dissipation, and deformation capacities (Moretti and Tassios 2007; Zhou and Liu 2010). As soon as the columns observed sudden shear failure (Fig. 1), load redistribution causes continuous damage to the adjacent members

leading to the collapse of the structure. Various attempts in the past (Wakabayashi et al. 1980; Tomii et al. 1987; Ricles and Paboojian 1994; Liu et al. 2009; Shi et al. 2012; Mizuki et al. 2016) were made to address the brittle shear failure of columns by increasing the transverse reinforcement ratio, using tubed-short columns and steel-reinforced concrete composite columns. Wakabayashi et al. (1980) proposed bi-diagonal shear reinforcement in short columns to improve their hysteretic and energy dissipation characteristics. Tomii et al. (1987) first investigated tubed RC short columns in building structures to improve the shear strength and ductility. Shi et al. (2012) proposed high-strength short columns confined using high-strength stirrups to improve bearing capacity and deformation performance. Ricles and Paboojian (1994), Liu et al. (2009), and Mizuki et al. (2016) investigated steel-reinforced concrete columns to improve the shear capacity and ductility of RC short columns. The inherent low shear deformation ability of concrete followed by detachment and bond-slip of steel and concrete surfaces led to limited load carrying capacity and deformation characteristics.

Fig. 1. Damage of RC short columns in past earthquakes: (a) shear and axial compression failure of first storey columns, 1999 Kocaeli earthquake (EERI 2021); (b) and (c) shear failure in a split-level RC frame, 2008 Wenchuan earthquake (Lieping et al. 2013 WCEE); and (d) damage in captive columns, 1999 Athens earthquake (Pardalopoulos et al. 2013)

Although the aforementioned strengthening techniques augmented the strength and deformation characteristics of short columns, concrete brittle shear behavior was not improved significantly. In recent years, cement composites reinforced with steel or polymer fiber commonly known as high performance fiber-reinforced cementitious composites (HPFRCC) or engineered cementitious composites (ECC) (Li et al. 1995) or strain-hardening cementitious composites (SHCC) (Li and Wu 1992) or ultra-high toughness cementitious composites (UHTCC) (Xu et al. 2010) were introduced to improve the mechanical (tension, compression, flexural toughness) properties of standard conventional concrete. ECC is significantly different from the traditional steel fiber reinforced concrete or ultra-high-performance concrete (UHPC) due to its high strain-hardening response under axial loads. This enables formation of multiple microcracks by the bridging fibers under large deformation. ECC aims at overcoming the lack of tensile deformation capacity of conventional

concrete. As a result, the stress-strain characteristics of ECC are analogous to metallic material possessing tensile strain-hardening behavior. These influencing characteristics provide a better opportunity for the application of ECC material in structural engineering.

Nowadays, ECC is widely used due to its excellent mechanical properties in RC frame buildings such as beams (Xu et al. 2012), columns (Li et al. 2017), joints (Qudah and Maalej 2014; Razak and Said 2016), and slabs (Said et al. 2015; Zheng et al. 2018). Past studies (Aviram et al. 2014; Osorio et al. 2014; Panagiotou et al. 2015; Xu et al. 2017) on HPFRC bridge columns reported enhanced deformation behavior, energy dissipation capacity, and high-damage tolerance than the regular concrete columns. Most of the past research mainly focused on improving the dissipation and ductility of long column plastic hinge areas. On the other hand, the study of imparting shear resistance is of utmost importance in case of short columns. Parra-Montesinos and Wight (2000) studied the cyclic response of the ECC joint without any transverse reinforcement. It was reported that the ECC joint observed better shear deformation characteristics and suffered minor damage. Van Zijl (2007) reported that the ECC V-shaped notch plate exhibited pronounced ductility in diagonal tension. Recently, Hou et al. (2018, 2019) conducted a study to understand the seismic performance of steel plate ECC reinforced coupling beams. The experimental results reported that coupling beams with a small span-to-depth ratio observed ductile behavior with excellent hysteretic response and large energy-dissipating capacity. Shear cracks were distributed all over the span region with a tightly controlled crack width, displaying excellent crack control ability. Deng et al. (2018) also reported that ECC jacketed columns failed in ductile modes and exhibited better plastic deformation and energy dissipation capacity. In addition to the experimental investigations, a few analytical studies (Zohrevand and Mirmiran 2010; Gencturk et al. 2013; Roehm et al. 2015; Wu et al. 2018) were also conducted to propose concrete models considering the constitutive material properties of fiber reinforced concrete (FRC). Wu et al. (2018) proposed uniaxial cyclic constitutive material models in compression and tension, followed by a numerical modeling technique to simulate short column ECC members using OpenSees (Mazzoni et al. 2007). Recently, Pang et al. (2019) also developed 3D nonlinear finite element models to simulate different FRC (steel fibers reinforced concrete, polypropylene fibers reinforced concrete, and steel polypropylene hybrid fiber reinforced concrete) columns.

From past literature, it was established that despite the development of various strengthening options, the brittle shear failure of short columns in RC frame buildings was not assessed clearly. Given the limitations, an

innovative emerging engineering cementitious strain hardening ECC composite using polyvinyl alcohol (PVA) fibers and fly ash was chosen instead of conventional concrete in this study. The applicability of ECC has not been explicitly comprehended in elevating the seismic performance of short columns under lateral loads. Further, the available numerical and constitutive material models do not reasonably simulate the shear behavior of ECC. Hence, a detailed experimental and analytical investigation was carried out on reinforced ECC short columns (RECC short columns named hereafter) to recognize the suitability of ECC and appraise the influence of shear-span and transverse reinforcement ratios. Five RECC and one RC half-scale short column specimens consisting of different shear-span and transverse reinforcement ratios were tested under cyclic lateral loads in the experimental campaign. Shear behavior of the RECC short columns in terms of hysteretic response, strength, stiffness, deformation capacity, and failure modes were evaluated. Later on, finite element modeling of the RECC short columns was carried out, and the developed models were verified with the experimental results.

EXPERIMENTAL PROGRAM

To evaluate the effectiveness of ECC concrete in delaying the shear failure of short columns, five RECC and one RC column specimens with different shear-span ratios ($\lambda = 1.5, 2.0$ and 2.5 , $\lambda = H/b$, where H represents the distance from the loading center to the bottom of the column and b is the width of the section parallel to the bending moment) and stirrup reinforcement ratios ($\rho_{sv} = 0.48\%, 0.61\%$ and 0.84%) were constructed and tested under displacement-controlled cyclic loading. As per national Chinese technical standard CMC (2011), columns were classified into long columns ($\lambda > 2.0$), short columns ($1.5 \leq \lambda \leq 2.0$) and ultra-short columns ($\lambda < 1.5$), and the stirrup reinforcement ratio of short columns should not be less than 0.60% . The model specimen represents a cantilever column, approximately half of the full-length column (Fig. 2). The specimens design followed the recommendations of national Chinese technical standard CMC (2011), which is similar to the guidelines followed in the various international design standards like ACI (2014) and Eurocode 2 (2004). Half-scale column specimens (300×300 mm) were constructed on a stiff RC base beam ($1400 \times 400 \times 500$ mm) to provide proper fixity under lateral loads (Fig. 3). The base beam was constructed using the conventional normal concrete (cubic compressive strength, C35), while the columns were constructed using ECC concrete.

Fig. 2. Model selection of test specimen.

Fig. 3. Reinforcement detailing of column specimens (All dimensions are in mm).

The ECC concrete section was extended into the base beam by about $600 \times 400 \times 100$ mm to ensure proper bonding and reduce joint failure. The column specimens were loaded at different heights ($H = 750$ mm, 600 mm, and 450 mm) to obtain various shear-span ratios (2.5, 2.0, and 1.5), as given in Table 1. Higher shear span ratio ($\lambda = 2.5$) in specimen RECC-3 was chosen to evaluate the variation in lateral load behavior for lower and higher bound shear span ratios. The longitudinal reinforcement ratio (ρ_s) was maintained same (2.68%) in all the specimens. In contrast, the transverse reinforcement consists of 8 mm diameter four-legged stirrups provided at a spacing of 80 mm, 110 mm, and 140 mm, which correspond to a transverse reinforcement ratio ($\rho_{sv} = A_{sv}/bs$, A_{sv} is the cross-sectional area of the stirrups, b is the width of the column and s is stirrup spacing) of 0.84%, 0.61%, and 0.48%, respectively. The longitudinal reinforcement was computed directly for the reduced column section. Scaling was applied only to column sections, and other parameters such as longitudinal reinforcement, shear reinforcement, and axial load ratio were calculated directly for the reduced column section. The longitudinal reinforcement was same in all the specimens as the major focus of the present study was to evaluate the influence of shear-span ratio and transverse reinforcement ratios on the lateral load performance of ECC

132 short columns.

133 **Table 1.** Details of Specimens

Specimen	H (mm)	λ	n_t	Longitudinal reinforcement			Stirrup			
				f_y (MPa)	ρ_s (%)	Size	f_{yv} (MPa)	D (mm)	s (mm)	ρ_{sv} (%)
RC	450	1.5	0.27	472.4	2.68	12-D16	596.6	8	80	0.84
RECC-1	450	1.5	0.27	472.4	2.68	12-D16	596.6	8	80	0.84
RECC-2	600	2.0	0.27	472.4	2.68	12-D16	596.6	8	80	0.84
RECC-3	750	2.5	0.27	472.4	2.68	12-D16	596.6	8	80	0.84
RECC-4	450	1.5	0.27	472.4	2.68	12-D16	596.6	8	110	0.61
RECC-5	450	1.5	0.27	472.4	2.68	12-D16	596.6	8	140	0.48

134 Note: H represents the distance from the loading center to the bottom of the column; n_t represents the axial compression
 135 ratio ($n_t = N/f_c A$); f_y represents the yield stress of longitudinal reinforcement; f_{yv} represents the yield stress of stirrup
 136 reinforcement; D represents the diameter of steel reinforcement; s represents the spacing of stirrup.

137

138 **Material Properties**

139 Material characterization of normal concrete (NC), ECC concrete, and steel reinforcement was carried out using
 140 the relevant technical standards (Table 2, Table 3, and Fig. 4). The ECC concrete matrix constituents consists of
 141 42.5 grade Portland cement (240 kg/m³), polyvinyl alcohol (PVA) fibers (2% of total volume), fine silica sand
 142 (550 kg/m³), superplasticizer (6 kg/m³), and mineral admixtures such as silica fume (160 kg/m³) and grade II
 143 fly ash (800kg/m³). PVA fibers of length 12 mm and diameter 0.038 mm with a tensile strength of 1600 MPa
 144 (density 1300 kg/m³) were used as reinforcement in the cementitious matrix. The water to binder ratio was taken
 145 as 0.25. More details about the material characterization of ECC concrete can be obtained from Hou et al. (2019).
 146 Uniaxial tension tests on dumbbell-shaped specimens for cement-based composites were carried out in
 147 accordance with JSCE (2008). All the specimens were moist cured for 28 days at room temperature (28°-29°C)
 148 under laboratory conditions before testing. Compression tests on 150 × 300 mm diameter cylinders (2018) were
 149 performed to evaluate the compressive stress-strain characteristics of NC and ECC concrete (Fig. 4).

150

151 **Table 2.** Material properties of normal and ECC concrete

Type of concrete	Compressive strength (MPa)	Average (MPa)	Peak compressive strain (%)	Average (%)	Elasticity modulus (×10 ⁴ MPa)	Average (×10 ⁴ MPa)	Tensile strength (MPa)
NC-1	28.6		0.13		2.38		
NC-2	25.8	28	0.13	0.15	3.89	3.34	2.5
NC-3	29.7		0.17		3.75		
ECC-1	27.7		0.34		1.01		
ECC-2	25.5	26	0.31	0.36	0.85	0.94	3.6
ECC-3	25.2		0.44		0.96		

Table 3. Tension properties of steel reinforcement

Diameter of reinforcement	Yield strength (MPa)	Average (MPa)	Ultimate strength (MPa)	Average (MPa)	Elongation (%)	Average (%)	Elasticity modulus ($\times 10^5$ MPa)	Average ($\times 10^5$ MPa)
D8-1	600.6		790.1		27.5		2.14	
D8-2	592.2	596.6	784.7	781.0	25.0	26.3	1.65	1.83
D8-3	597.1		768.1		26.3		1.69	
D16-1	468.7		624.3		23.8		1.72	
D16-2	470.4	472.4	627.8	625.1	22.5	22.5	1.66	1.70
D16-3	478.2		623.1		21.3		1.71

Fig. 4. Stress-strain characteristics of: (a) Normal and ECC concrete under compression; and (b) ECC concrete under tension.

The average cylinder compressive strength of ECC and NC was about 26 MPa and 28 MPa, respectively (Table 2). The specimens exhibited significant strain hardening (ultimate tensile strain is over 3%) under tensile loading, and the strength attenuation of ECC under compression is very gradual than that of the normal concrete (Table 2, Fig. 4b). The average tensile strength of ECC concrete was 3.6 MPa (ranging from 3.3 MPa to 4 MPa), and for normal concrete was about 2.5 MPa. The material characterization of both normal and ECC concrete (compressive strength and tensile strength) was carried on the day of testing of column specimens which was about 60 days after the construction. The tensile properties (elastic modulus, yield- and ultimate- strength) of the high-strength steel reinforcement from coupon tests are given in Table 3.

Test Setup and Loading Protocol

The specimens were tested under quasi-static displacement-controlled cyclic loading applied near the column top-level using a 1000 kN servo-controlled hydraulic actuator with a stroke length of ± 500 mm as shown in Fig. 5. The hydraulic actuator was firmly connected to the specimen using four stiff steel rods and two steel

plates. The specimens were loaded in two stages: elastic stage and plastic stage following the recommendations of JGJ/T101 (2015) and ISO 16670 (2003), till the models attained significant damage or failure. Only one cycle (0.125% and 0.25%) drift levels were applied in the first stage to capture the specimen's elastic behavior, as shown in Fig. 6 (enlarged sectional view). In the second stage, two cycles for each drift level with an increment of 0.5% drift were applied till the specimens were severely damaged or the capacity was reduced to 75% of its maximum. The vertical load was applied on the top of the column specimens using a hinged servo-hydraulic jack attached to a frictionless sliding cart. The sliding cart provided effortless mobility to the hydraulic jack to move smoothly with the specimen under lateral loads. A stub column section was provided at the top free end of the column to avoid local damage due to significant concentrated loads under lateral and horizontal directions. Additionally, the stub column section was beneficial in limiting the cracking due to the stress concentration at the free end of the column specimens and avoid interference with the main observation area. Due to laboratory constraints, shear behavior of short columns was evaluated by testing the vertical cantilever specimens (Fig. 5) under the combined action of axial and lateral forces, including both the experimental and actual force state conditions. Simultaneously, under fundamental forces, short columns deformation includes bending deformation and shear deformation (which make it difficult to separate the shear resistance). The axial load ratio ($n_t = N/Af_c$) was calculated as the ratio of axial load (N) to the cross-sectional area (A) times concrete compressive strength (f_c) and it was found to be approximately 27%. The axial load applied on the columns was based on the design load calculations of the selected building. The base beam of the specimens was prevented from sliding using hydraulic jack restraints. Specimens were instrumented with strain gauges, linear variable differential transducers (LVDTs), laser displacement transducer (Fig. 7a), and load cell in the actuator arm. The lateral displacement of the specimens near the mid-point of loading was monitored using an external laser displacement transducer, rather than using the displacement recorded in the actuator arm which may involve non-quantifiable uncertainties. Strain gauges were bonded to the longitudinal and transverse steel reinforcement at various locations to measure the steel strain (Figs. 7b, c). The response of the specimens under lateral loads was recorded continuously using the IMP data acquisition system.

Fig. 5. Experimental setup: (a) Details of testing rig (drawing is not to scale); and (b) Photograph of specimen.

Fig. 6. Schematic layout of the loading protocol.

Fig. 7. Details of instrumentation: (a) LVDT and laser transducer locations; and layout of strain gauges in (b) transverse; and (c) longitudinal reinforcements.

EXPERIMENTAL OBSERVATIONS

The experimental observations of the specimens were discussed based on low (1.5, RECC-1, -4, -5), intermediate (2, RECC-2), and higher (2.5, RECC-3) shear-span ratios considering the similarity of the initiation of cracks followed by their proliferation and final failure mechanisms. Fig. 8 shows the specimens before the

211 commencement of the test and their corresponding damaged state after the termination of the test. In the case of
 212 RC specimen, cracks were initiated as minor hairline flexural cracks along the edge of the columns at a drift
 213 level of 0.36% (-272 kN, +346 kN), followed by the diagonal tension shear cracks corresponding to a drift level
 214 of 0.67% (-346.7 kN, +417.7 kN). With increased monitored drift, multiple oblique tension shear cracks
 215 developed on either side of the main diagonal. From the analysis of the strain data, it was found that with an
 216 increase in drift, the longitudinal reinforcement at the bottom of the column observed yielding in all specimens
 217 (Fig. 9). The yielding of longitudinal reinforcement in the RC specimen was observed at -0.90% (-415.7 kN)
 218 drift, and the stirrups yielded at 1.52% (+504.7 kN) drift. In the subsequent drift levels, widening of diagonal
 219 shear cracks and flexural cracks was observed, followed by spalling of cover concrete, and the same continued
 220 till 3.00% drift level. The test was terminated at 3.50% (-170.6 kN, +354.1 kN) drift when the crushing of core
 221 concrete along the diagonal shear cracks and buckling of longitudinal reinforcement was observed.
 222

Fig. 8. (a) Undamaged state of specimens before test; (b) Final damage state of specimens after the test; (c) Schematic crack patterns at different drift levels.

Fig. 9. Variation of strain in longitudinal reinforcement at different drift levels: (a) RC; (b) RECC-1; (c) RECC-2; (d) RECC-3; (e) RECC-4; and (f) RECC-5.

In the case of first category low shear-span ratio (RECC-1, -4, and -5) specimens, the initiation and propagation of cracks were found to be similar, with slight variations observed due to the various transverse reinforcement ratios. Minor hairline cracks were initiated in the depth region at a drift level of 0.67%, corresponding to a lateral load of approximately 320 kN. With further increase in drift level, minor hairlines were modified into multiple diagonal shear cracks and proliferated towards the opposite corners. It continued till a drift level of 1.53%, 2.0%, and 1.53%, in the case of RECC-1, -4, and -5 specimens, respectively. The

238 yielding of longitudinal reinforcement and stirrups in RECC-1 specimen was observed at a drift level of 1.18%
239 (443.6 kN) and 1.25% (431.0 kN), respectively. Whereas, in RECC-4 and RECC-5 specimens yielding of
240 longitudinal reinforcement (0.54% and 1.09%) and stirrups (0.70%) was observed at slightly smaller drift levels
241 (Fig. 9). At 2% drift, spalling of cover concrete and widening of shear cracks was affirmed, and it continued till
242 the termination of the test in RECC-4 and RECC-5 specimens. Widening of shear cracks was observed at a
243 slightly higher drift level (3%) in RECC-1, which is mainly due to the presence of higher transverse
244 reinforcement ratio ($\rho_{sv} = 0.84\%$) when compared to RECC-4 ($\rho_{sv} = 0.64\%$) and RECC-5 ($\rho_{sv} = 0.48\%$)
245 specimens. From the observed experimental observations of low shear-span ratio specimens, it was observed
246 that there was no fall-out of the core concrete in RECC-1 and RECC-4 specimens except in RECC-5 specimen
247 where the disintegration of core concrete was initiated. This clearly shows the beneficial influence of the ECC
248 and transverse reinforcement ratio in resisting crack propagation.

249 In intermediate (RECC-2) and higher (RECC-3) shear-span ratio specimens, formation of cracks and failure
250 mechanisms were found to be significantly similar with minor deviations. Minor hairline diagonal shear cracks
251 near the mid-height of the column were initiated at 0.67% (4.02 mm) drift in RECC-2, whereas, similar cracks
252 formed at 0.67% (5.03 mm) drift in RECC-3 specimen. With an increase in the monitored drift of 1% (6 mm,
253 324 kN), multiple diagonal cracks on either side of the main diagonal and flexural cracks along the column
254 edges formed in RECC-2 specimen. Very few cracks were observed in RECC-3 higher shear-span ratio
255 specimen until 2% (15 mm, 271 kN) drift. Formation of flexural-shear cracks and slight spalling of cover
256 concrete near the corners was ascertained at a significantly higher drift level of 2.5% (18.75 mm, 262 kN) than
257 the RECC-2 specimen (7.5 mm, 348 kN). ECC concrete and high transverse reinforcement ratio significantly
258 enhanced the specimens lateral load behavior in delaying the formation of cracks, crack development, and
259 strength degradation at higher drift levels. Widening of shear cracks was observed at a drift level of 3.5% (21
260 mm, 340 kN) and 4% (30 mm, 236 kN) in RECC-2 and RECC-3 specimens, respectively, and it continued till
261 the termination of the test. Flexural-shear cracks initiated along the edges were not transferred to the opposite
262 corners instead propagated along the column length in the case of RECC-3, which is absent in other specimens.
263 The yielding of longitudinal reinforcement was observed at a drift of 0.72% (4.32 mm, 288.4 kN) and 1.22%
264 (9.15 mm, 251.9 kN), respectively, in RECC-2 and RECC-3 specimens (Fig. 9). In comparison, the yielding of
265 transverse reinforcement was observed at a drift of 0.64% (3.84 mm, 262.5 kN) and 2.06% (15.5 mm, 256.2

kN), respectively. There is no fall-out or disintegration of the cover concrete, highlighting the benefits of the ECC concrete matrix in inhibiting the disintegration of core concrete even at higher drift levels. The tests were terminated at a drift level of 6% in RECC-2 and RECC-3 when the capacity of the specimens was reduced to about 75% of maximum.

The experimental observations ascertained that specimens RECC-2 and RECC-3 failed in the flexural-shear mode due to their larger shear-span ratio. It should be noted that many hairline cracks around the existing shear cracks appeared and developed, as shown in Fig. 8, which is seldom observed in conventional RC specimen. In RECC specimens, fracturing fibers sound was noted during the widening and propagation of cracks. Compared to the RC specimen, a slight outward bulge of ECC concrete was observed in the compressive region instead of crushing and spalling. The self-confinement effect of fibers in ECC retained the transmission of tensile stress before forming major cracks, thus limiting the crack width and showing high ultimate tensile strain (Li and Wu 1992; Li et al. 1995). As a result, the cracks of ECC specimens were dense and numerous. In addition, owing to large tensile deformation capacity of ECC, the compatibility between ECC substrate and steel reinforcement was maintained much better than conventional concrete, which may explain the different failure mechanisms of ECC and RC specimens.

INFLUENCE OF ECC, SHEAR-SPAN RATIO AND TRANSVERSE REINFORCEMENT RATIO

The assessment of ECC concrete, shear-span ratio, and transverse reinforcement ratio on the lateral load behavior of short columns was evaluated in terms of hysteretic and envelop (lateral load vs. drift) response, stiffness degradation, energy dissipation, deformation characteristics, and strength attenuation.

Hysteretic and Envelop of Lateral Load-Drift Response

The hysteretic response of RC and RECC specimens is shown in Fig. 10, and the experimental results of the tested specimens are summarized in Table 4. The RECC short column hysteretic behavior was proportional in shape both in pre- and post- peak regions compared to the RC short column reference specimen. The ECC concrete did not enhance the lateral load-carrying capacity of the RECC specimens than the RC specimen, but the specimens exhibited stable hysteretic response except in the case of RECC-5, which may be due to lower transverse reinforcement ratio ($\rho_{sv} = 0.48\%$). The lateral load resistance of RC specimen (515 kN) was slightly higher (8%) compared to RECC-1 (477 kN), which may be due to the difference in compressive strength of normal (28 MPa) and ECC (26 MPa) concrete (7.7%). Moreover, the degradation in the capacity of RC specimen

was very sudden compared to RECC specimens. The brittle nature of the conventional concrete and ECC concrete ductile behavior was pragmatic from the hysteretic response comparison.

Fig. 10. Hysteretic response of specimens: (a) RC; (b) RECC-1; (c) RECC-2; (d) RECC-3; (e) RECC-4; and (f) RECC-5.

Table 4. Experimental results of the specimens

Specimen	K/K_{RC}	ED/ED_{RC}	V_y (kN)	δ_y (%)	V_m (kN)	δ_m (%)	$V_{0.8m}$ (kN)	δ_u (%)	$v_{un}/\sqrt{f'_c}$	μ	V_f/V_m	V_{GB}/V_m	Failure mode
RC	1.0	1.0	349	0.5	515	1.7	412	3.0	1.08	6.02	0.97	1.03	Shear-Flexure
RECC-1	0.7	1.2	299	0.6	477	1.9	381	3.4	1.04	5.59	1.05	1.17	Shear-Flexure
RECC-2	0.6	2.1	247	0.6	369	2.0	295	5.1	0.80	8.50	1.02	1.44	Flexure-Shear
RECC-3	0.5	4.1	189	0.7	271	2.0	217	5.2	0.59	7.95	1.11	1.89	Flexure-Shear
RECC-4	0.7	0.7	315	0.7	471	2.0	377	3.1	1.03	4.69	1.06	0.97	Shear-Flexure
RECC-5	0.7	0.5	308	0.7	421	1.5	337	2.4	0.92	3.69	1.19	0.95	Shear-Flexure

Note: K and K_{RC} represent the initial stiffness of the RECC and RC specimens; ED and ED_{RC} represent the cumulative energy dissipation of RECC and RC specimen; V_y represents yield force corresponding to yield drift (δ_y); V_m represents the maximum lateral load, and δ_m is the corresponding drift; $V_{0.8m}$ represents the load corresponding to 80% maximum and δ_u is the corresponding drift; v_{un} represents the nominal shear stress calculated as the ratio of maximum lateral load to the cross-sectional area of the column; μ represents the ductility drift calculated as δ_u/δ_y ; V_f represents the shear capacity due to the flexural resistance; V_{GB} represents the predicted shear strengths using CMC (2011).

The pinching phenomenon was more pronounced in RC specimen, whereas it decreased with an increase in the shear-span ratio of RECC specimens. RECC-3 and RECC-2 specimens observed a lesser amount of pinching compared to RC, RECC-1, RECC-4, and RECC-5 specimens (Fig. 10). This may be due to the crack inhibition and significantly less damage owing to the beneficial influence of ECC as described previously and gradual lateral load resistance reduction in the post-peak regime when compared to other specimens. In case of similar shear-span ratio specimens, RECC-1 observed a lesser amount of pinching and, consequently, higher

316 deformation capacity and energy-dissipation due to a slightly higher transverse reinforcement ratio than RECC-
 317 4 RECC-5. For the same shear-span ratio, the transverse reinforcement ratio greatly influenced the hysteretic
 318 behavior of test specimens, and the increase in stirrup ratio improved the behavior of short columns.

319 Fig. 11a shows the comparison of envelop lateral load-drift response of specimens. It was observed that
 320 specimens using ECC concrete showed higher deformation ability, and an increase in the transverse
 321 reinforcement ratio increased the load-carrying capacity of the specimens (Fig. 11a). The shear resistance of
 322 both RECC-1 and RECC-4 specimens was similar, which emphasized that a higher transverse reinforcement
 323 ratio is not noteworthy in enhancing the capacity of the specimen; instead, the capacity was mainly dominated
 324 by the behavior of concrete under compression (Fig. 11a). On the other hand, the shear-span ratio significantly
 325 influenced the strength and deformation performance of specimens. The capacity of specimens decreased with
 326 increase in shear-span ratio. However, the ultimate drift level corresponding to 80% of peak load was
 327 substantially higher.

Fig. 11. Comparison of lateral load behavior of specimens: (a) envelop lateral load-drift response; (b) stiffness degradation; (c) cumulative energy dissipation; and (d) equivalent viscous damping.

334 The degradation in strength in RECC specimens was more gradual than RC specimen (Fig.11a). The self-
335 confinement effect of fibers in ECC maintained the integrity in the post-peak region, preserves the transmission
336 of stress through the fibers without noticeable degradation in strength. Xu et al. (2017) from their study on ECC
337 short and slender columns, reported that ECC concrete enhanced the lateral strength of the columns by about
338 1.3 times in short columns ($\lambda = 2$) with higher axial load ratios when compared to the corresponding RC
339 specimen. On the other hand, the increase in strength was insignificant in slender columns ($\lambda = 4$) even with
340 increased transverse reinforcement ratios. Similar observations were reported for 85% of post-peak capacity for
341 both short and slender column specimens.

342 As seen in Table 4, the values of V_f/V_m and V_{GB}/V_m for short columns (RC, RECC-1, RECC-4, RECC-5) are
343 pretty close to each other, which exhibited a shear-bending coupling state in these specimens. On the other hand,
344 in RECC-2 and RECC-3 specimens, the values of V_{GB}/V_m are significantly greater than that of V_f/V_m clearly
345 indicating the flexural resistance of the specimen was reached before the shear capacity. It was also observed
346 that for specimens with the same shear-span ratio and lower transverse reinforcement ratio (RECC-1, RECC-4,
347 RECC-5), the shear capacity (V_m) was reached before the shear demand as the flexural resistance (V_f) was
348 attained (Table 4). Because of this reason, shear failure was found to be the most dominant mode of failure in
349 the case of specimens with a relatively small shear span ratio ($\lambda = 1.5$) and lower transverse reinforcement ratio.
350 This was observed previously in the experimental observations where the specimens observed diagonal tension
351 shear cracks before the flexural cracks were initiated along the column length. On the other hand, specimens
352 with a shear-span ratio higher than 1.5 (RECC-2, RECC-3) observed flexural cracks before the shear cracks
353 initiation. Based on the limited experimental investigation conducted in the present study, it was found that the
354 shear capacity was higher than the shear demand due to the flexural resistance of the columns and flexure failure
355 was the dominant mode in flexure-shear failure. The shear strength decreased with an increase in the shear-span
356 ratio, as observed in the present study. It was observed that the realistic shear capacity estimated following the
357 Chinese design standard was larger than the experimental values for the present study. At the same time, in the
358 case of Chinese standard GB 50010 (2011), the contribution of concrete to shear strength is calculated using the
359 tensile strength of the concrete rather than the compressive strength. The tensile strength of ECC concrete was
360 approximately twice that of conventional concrete (Table 2), which led to a higher contribution of concrete in

imparting shear resistance. The limited analysis affirmed that due consideration needs to be given to the contribution shear-span ratio in calculating the shear capacity of columns. ACI 318 (2014) recommends the maximum shear to be resisted by the transverse reinforcement to $0.66\sqrt{f_c}bd$, where f_c is the compressive strength of concrete cylinder, b and d are the width and depth of the column section. The contribution of shear capacity provided by the transverse reinforcement was overestimated without considering its maximum upper limit.

Stiffness Degradation

The initial lateral stiffness of RECC specimens decreased with an increase in shear-span ratios, and it was in the range of 0.5 to 0.7 times that of the RC specimen. This may be due to the higher lateral load resisted by the RC specimen for the same initial drift level, and the elasticity modulus of ECC concrete (0.94×10^4 MPa) was lesser (3 times) than that of normal concrete (3.34×10^4 MPa), as given in Table 2. Among all the specimens, the RECC-3 specimen exhibited lower initial stiffness due to its higher shear-span ratio. For the case of various transverse reinforcement ratio specimens (RECC-1, -4, -5), the initial stiffness was approximately 0.7 times that of the RC specimen, highlighting the insignificant influence of the transverse reinforcement ratio in stiffness contribution. Previously, Xu et al. (2017) reported that secant stiffness decreased with an increase in shear-span ratio and decreased transverse reinforcement ratios. The secant stiffness was about 0.7-0.9 times and 1.3 times that of the corresponding RC specimen in slender columns and short columns, respectively. Fig. 11b shows the variation of secant stiffness of the specimens at different drift levels. The degradation of lateral stiffness in RC specimen was significant in the initial drift levels compared to RECC specimens. In RECC specimens, the lateral stiffness reduced to 80% of its maximum at almost the same drift level (2.54% to 2.91%), whereas, in RC specimen, it was observed at a slightly lower drift level (2.17%).

Energy Dissipation and Equivalent Viscous Damping

Figs. 11(c, d) shows the variation of cumulative energy dissipated and equivalent viscous damping for different drift levels. The equivalent viscous damping (β_{eq}) was estimated as per the recommendations of ATC 40 (1999) and Basha and Kaushik (2020) as given in Eqs. (1) and (2).

$$K_{eq} = \frac{V^+ - V^-}{\Delta^+ - \Delta^-} \quad (1)$$

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$$\beta_{eq} = \frac{2}{\pi} \frac{E_c}{K_{eq} (\Delta^+ - \Delta^-)^2} \quad (2)$$

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where K_{eq} is the effective secant stiffness, Δ^+ and Δ^- are the displacements corresponding to maximum forces V^+ and V^- in pull and push directions, respectively, and E_c represents the energy dissipated per displacement cycle level. The total energy dissipated by the RECC specimens was increased with an increase in the shear-span ratio, and it was in the range of 1.2 to 4.1 times that of the RC specimen. Conversely, the initial stiffness decreased with an increase in the shear-span ratio. RECC-3 specimen observed higher energy dissipation followed by RECC-2. The total energy dissipated in RECC-3 and RECC-2 specimens was increased by about 152% and 52% compared to RECC-1, respectively. RECC-3 observed higher energy dissipation when compared to other specimens which may be due to the high shear-span ratio, minor damage sustained, and lower pinching observed under lateral loads. The increase in the equivalent viscous damping ratio (Fig. 11d) was found to be gradual in the initial drift levels ($< 1.5\%$) till different dissipating mechanisms were stimulated. RECC specimens observed similar behavior in which RECC-3 showed the highest equivalent viscous damping ratio followed by RECC-2.

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413 **Deformation Analysis**

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In the present study, yield drift (δ_y) and the corresponding yield force (V_y) were calculated using the farthest point method developed by Feng et al. (2015). Ultimate drift (δ_u) was defined as the drift at which the specimen

capacity reduced to 80% of its max ($V_{0.8u}$). Generally, columns are designed to exhibit better ductility characteristics to resist strong-ground-motions. The calculation of the ductility ratio for all the specimens is given in Table 4. The ductility ratio in RECC-1, RECC-2, and RECC-3 was higher than the recommended ductility ratios ($\mu > 5$) following FEMA guidelines (1999). On the other hand, the ductility factors in RECC-4 (4.7) and RECC-5 (3.7) specimens were smaller than other specimens, which may be due to slightly larger yield drift and smaller ultimate drift. The calculated ductility ratios ascertained that with an increase in shear-span ratio and transverse reinforcement ratio, the deformation capacity of short column specimens (RECC-2, RECC-3) increased. It was also comprehended that for smaller transverse reinforcement ratios (RECC-5), the deformation characteristics were not explicitly enhanced in short columns even though high ductile concrete was used. Further, it was noted that a slight variation of yield or ultimate drift led to a relatively significant difference in ductility calculation. At the same time, the available unified method for the estimation of yield drift was not well established. Past research (2002) also emphasized that deformability was more consistent and more straightforward than the ductility ratio to understand the inelastic properties of structures.

The average ultimate drift ratio ranged from 3.1% (RECC-4) to 5.2% (RECC-3) and was found to be higher than RC specimen (3%) except in the case of RECC-5 (2.4%). Thus, the experimental results affirmed that an increase in shear-span ratio and transverse reinforcement ratio enhanced the short columns deformation capacity as observed in ductility ratios, with the trend being consistent for the considered ratios. To allow for proper comparison of shear stresses across different specimens, normalized shear stress ($v_{un}/\sqrt{f'_c}$) was estimated following ACI (2014) and is shown in Table 4. The nominal shear stress of short columns ($\lambda < 2.0$) was exceeding the limit specified (0.83 MPa) in ACI 318 (2014) and the specimens observed acceptable ductility ranging from 3.69-5.59. The cost of proposed ECC columns was found to be higher due to the ECC matrix's constituents (polyvinyl alcohol (PVA) fibers, fine silica sand, mineral admixtures and superplasticizer) compared to conventional concrete. Considering the improvement in the seismic performance (strength degradation, post-peak deformation characteristics), the proposed RECC concrete columns can be a potential alternative, especially in ground stories of buildings that had a nominal influence on overall construction cost.

Strength Attenuation

The decrease in the capacity with an increase in the number of cycles for the same drift level (i.e., strength attenuation), which reflects the development of damage and sustained strength retention of the structures was

evaluated in the present study. Fig. 12 shows the variation of average strength retention capacity (F_n/F_1 , F_n , and F_1 represent the capacity of n^{th} and first-cycle) of specimens at different drift levels. In the present study, specimens were subjected to two cycles for each drift level, and three load values (F_1 , F_2 , and F_3) were taken for the same drift level in which the load F_3 was provided by the next successive drift cycle.

Fig. 12. Variation of strength retention curves with respect to: (a) materials; (b) shear-span ratio; and (c) transverse reinforcement ratio.

The strength retention capacity of RECC specimens (Fig. 12a) was significantly better than the RC specimen, especially at higher drift levels. The strength attenuation rate of RECC-1 was slightly higher at larger drift levels. It was also observed that as the shear-span ratio increased, the strength decay rate became very gradual in RECC specimens (Fig. 12b). It was clearly observed that the ratio of capacity (F_2/F_1) in RECC-2 and RECC-3 specimens was constant (0.97-0.93) for different drift levels. The ratio F_3/F_1 was decreased to 0.92 up to 1.5% drift and remained almost the same at higher different drift levels, emphasizing the higher strength retention capacity of specimens. The significant difference in strength retention capacities was mainly due to the lesser damage observed in the high toughness concrete than conventional concrete. From 12(c), it was observed that till 2% drift, the transverse reinforcement ratio has an inconsequential effect on the strength attenuation, and the ratio F_2/F_1 and F_3/F_1 was more than 0.90. After 2% drift, the strength of the specimen with a low transverse reinforcement ratio deteriorated rapidly, ranging from 0.84-0.73, which is probably due to the sudden increase of the width of the prominent cracks and their swift propagation in the subsequent loading stages.

LATERAL LOAD BEHAVIOR AND DAMAGE ASSESSMENT USING FINITE ELEMENT MODEL

In the present study, finite element modeling of the RECC specimens was carried out to evaluate the lateral load capacity and damage patterns using advanced finite element modeling tool ATENA (Červenka and Červenka 2015). The realistic behavior of concrete structures, explicitly yielding of reinforcement, cracking of concrete and post-peak behavior (crushing of concrete) can be simulated in detail using ATENA. Steel reinforcement was modeled discretely using truss elements assuming a uniaxial state of stress following elastic-perfectly plastic bilinear law (Fig. 13a). The uniaxial stress-strain relationship for the reinforcement is given in Eqs. (3) and (4).

$$\sigma_s = E_s \varepsilon_s \quad \text{for } \varepsilon_s \leq \varepsilon_y \quad (3)$$

$$\sigma_s = f_y \quad \text{for } \varepsilon_y < \varepsilon_s \leq \varepsilon_{s,h} \quad (4)$$

where σ_s , ε_s , represent the tensile stress and strain, respectively, and ε_y , $\varepsilon_{s,h}$ represent the yield and ultimate strains in the reinforcement, respectively. The engineered cementitious composite (ECC) concrete was modeled using 3D Nonlinear Cementitious 2 User elements and concrete compressive stress-strain model, as shown in Fig. 13b. The bilinear compression model proposed by Maalej and Li (1994) was used before the peak load (shown in solid lines) and is given in Eqs. (5)-(8). The dotted curve in the ascending branch represents the actual compressive stress-strain curve of concrete. The yield stress σ_{cc} is taken as 0.66 times of σ_{cp} (peak stress). A simplified bilinear descending branch in the post-peak region was defined based on the test results. The knee stress σ_{cl} was taken as 0.5 times of σ_{cp} . The residual stress σ_{cu} was taken as 0.2 times the peak load.

$$\sigma_c = \frac{\sigma_{cc}}{\varepsilon_{cc}} \sigma \quad \text{for } 0 < \varepsilon \leq \varepsilon_{cc} \quad (5)$$

$$\sigma_c = \frac{\sigma_{cp} - \sigma_{cc}}{\varepsilon_{cp} - \varepsilon_{cc}} (\sigma - \sigma_{cc}) + \sigma_{cc} \quad \text{for } \varepsilon_{cc} < \varepsilon \leq \varepsilon_{cp} \quad (6)$$

$$\sigma_c = \frac{\sigma_{cl} - \sigma_{cp}}{\varepsilon_{cl} - \varepsilon_{cp}} (\sigma - \sigma_{cp}) + \sigma_{cp} \quad \text{for } \varepsilon_{cp} < \varepsilon \leq \varepsilon_{cl} \quad (7)$$

$$\sigma_c = \frac{\sigma_{cu} - \sigma_{cl}}{\varepsilon_{cu} - \varepsilon_{cl}} (\sigma - \sigma_{cl}) + \sigma_{cl} \quad \text{for } \varepsilon_{cl} < \varepsilon \leq \varepsilon_{cu} \quad (8)$$

The tensile stress hardening in the multi-crack development stage of ECC was simplified and assumed as a straight-line segment [BC, Fig. 13c]. The average value of initial and ultimate tensile stress in concrete due to cracking was taken as f_t . The uniaxial tension stress-strain relationship is given in Eqs. (9) to (11).

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$$\sigma_t = \frac{f_t}{\varepsilon_{c,t}} \varepsilon \quad \text{for } \varepsilon \leq \varepsilon_{c,t} \quad (9)$$

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$$\sigma_t = f_t \quad \text{for } \varepsilon_{c,t} < \varepsilon \leq \varepsilon_{u,t} \quad (10)$$

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$$\sigma_t = f_t - \frac{f_t}{\varepsilon_{l,t} - \varepsilon_{c,t}} (\varepsilon - \varepsilon_{u,t}) \quad \text{for } \varepsilon_{u,t} < \varepsilon \leq \varepsilon_{l,t} \quad (11)$$

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Both the base beam and column were modeled using the hexahedral elements, and detailed sensitivity analysis for various configurations of mesh sizes (25 mm × 25 mm, 50 mm × 50 mm and 100 mm × 100 mm) was carried out. It was found that larger mesh sizes (100 mm × 100 mm) observed significant deviation from the experimental results. On the other hand, smaller mesh size (25 mm × 25 mm) observed better correlation but required great computational effort, which is about three times necessary for 50 mm × 50 mm mesh size. Therefore, an intermediate mesh size of 50 mm × 50 mm was chosen for further simulation as the calculated results were in good agreement with the experimental results. The base beam of the specimen was fixed firmly to the ground and the loading steel plate was completely bonded to the specimen (Fig. 14), to replicate a realistic loading setup (as shown in Fig. 5). Constant vertical loads were applied and the lateral loading was applied under displacement control mode using the Newton-Raphson iteration method. Monotonic incremental displacement controlled nonlinear static analysis was carried out to evaluate the analytical response of RC/RECC specimens. The developed analytical model was verified by comparing the envelop lateral load-drift response and damage crack patterns followed by their ultimate failure modes as discussed in the following sections.

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Fig. 13. Constitutive relationships: (a) Uniaxial tensile stress-strain model of reinforcement; (b) compression and (c) tension stress-strain relationship of ECC concrete.

Fig. 14. Finite element model of the short column specimen.

VERIFICATION OF FINITE ELEMENT MODEL

Fig. 15 shows the comparison of estimated and experimental lateral load response of the short columns. The results showed that the finite element model response was matching well with the experimental results. The lateral load capacity in the RECC model was found to be same as that of the experiments (variation < 5%). The estimated global response (initial stiffness and lateral load capacity) using the finite element model was in good agreement with the experimental results.

Fig. 15. Comparison of experimental and analytical load-drift capacity response: (a) RC; (b) RECC-1; (c) RECC-2; (d) RECC-3; (e) RECC-4; and (f) RECC-5.

528 Slight variation in analytical drift values ($< 25\%$) corresponding to peak capacity may be attributed to modelling
 529 and experimental constraints, which may not be considered a major deviation from the obtained global response.
 530 Gradual degradation of the lateral load response of the specimens in the post-peak region was ideally traced
 531 with that of the experimental response (Fig. 15). Further, the damage patterns and failure mechanisms were
 532 compared to the predicted failure modes of the analytical model. Fig. 16 shows the variation of principal stress
 533 contours and crack distribution at the final drift level of each test specimen in the loading push direction.

534 The formation of shear cracks in the initial stages and their propagation towards the loading end matched
 535 reasonably well with the experimental results highlighting the accuracy of the analytical modeling. The
 536 formation of flexural-shear and flexure cracks on the front- and side- face in RECC-2 and RECC-3 specimens
 537 was captured ideally as observed in experiments (Fig. 16) in the push load direction. Furthermore, the crushing
 538 of the concrete near the corners in the high compression zones in RC and RECC specimens was also clearly
 539 simulated. From the comparison of global response (lateral load-drift) and predicted failure mechanisms, it was
 540 ascertained that the finite element model was found to be capable of predicting the lateral load behavior of RC
 541 and RECC columns. This provides a reasonable basis for further parameter analysis and the development of the
 542 detailed shear capacity model.

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Fig. 16. Comparison of experimental and finite element model damage patterns for: (a) RC; (b) RECC-1; (c) RECC-2; (d) RECC-3; (e) RECC-4; and (f) RECC-5.

CONCLUSIONS

The suitability of ECC concrete as an alternative to conventional concrete to improve the lateral load behavior of short columns was assessed by conducting an experimental and finite element investigation for different shear-span and transverse reinforcement ratio configurations. The experimental study affirmed that even though the contribution of ECC concrete to the load-carrying capacity of specimens was marginal, RECC specimens exhibited a more stable hysteretic response than RC specimen. Further, high toughness and strain-hardening ECC concrete demonstrated significantly better deformation and strength degradation characteristics in the post-peak region when compared to the conventional concrete specimen. The following conclusions were drawn from the limited research study.

- The lateral load capacity of the RECC specimens decreased by about 30%-50% for higher shear-span ratios (2-2.5) but provided better deformation and energy dissipation characteristics. On the other hand, increased lateral load capacity for higher transverse reinforcement ratios (0.84%) was not recognizable. Therefore, a balance must be exercised to limit the maximum transverse reinforcement ratio considering both strength

and ductility requirements, specifically in short columns.

- Plastic behavior of RECC specimens improved with an increase in both shear-span and transverse reinforcement ratios. The ultimate drift was in the range of 2.2%-4.7% and was more than 5% in the case of higher shear-span ratio specimens highlighting better shear ductility behavior of RECC specimens.
- The decay rate of strength in RECC specimens was lower even at higher drift levels. The reduction rate was significantly gradual for different shear-span and transverse reinforcement ratio configurations.
- The diagonal tensile shear cracks were very short, thin, and dense compared to RC specimen as the ECC fibers provided better confinement, leading to sizeable compressive strain capacity without spalling. RECC specimens either observed ductile-shear failure (low shear-span and transverse reinforcement ratio) or flexure-shear failure (higher shear-span and transverse reinforcement ratio) as compared to the brittle shear failure mechanism in RC specimen.
- Finite element model not only predicted the strength ($< 5\%$) and stiffness but also captured the damage crack patterns and failure mechanisms satisfactorily. The analytical study provided a way for further parametric analysis and for the establishment of a detailed shear capacity model.
- The limited experimental and analytical investigation ascertained that engineered cementitious composite concrete enhanced the ductile shear performance of the short columns and afforded a better alternative solution to conventional concrete to counteract the adverse shear failure of short column specimens.

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