PRECAST CONCRETE-ENCASED CONCRETE-FILLED STEEL TUBE COLUMN-TO-COLUMN DRY CONNECTION

TECHNICAL FIELD

[0001] This application relates to the field of building and construction, and more particularly to concrete and steel building elements.

BACKGROUND

[0002] There is a need to improve the efficiency of constructing concrete building structures. Cast-in-place (CIP) structures involve pouring fresh concrete on-site and allowing sufficient for the concrete to fully cure. Inadequately cured concrete will not have the necessary strength and other properties to ensure that the building is safe for occupancy and use. The curing process therefore cannot be rushed. For buildings with multiple floors, it can be appreciated that structures on a higher floor cannot be fabricated before all load-bearing concrete structures on the lower floor are fully cured.

[0003] Various types of precast connections have been proposed for reinforced concrete (RC) columns, and they can be categorised as wet or dry connections, depending on the connections used for longitudinal steel reinforcement. One example of a wet connection is the grouted sleeve connection. The sleeves have to be grouted on-site. As wet connections require a curing period, following construction work may be delayed.

[0004] Some RC structures, such as conventional precast concrete-filled steel tube (CFST) structures, can be precast in the factory and interconnected on site. Examples of conventional connections include bolted flange connections, demountable connections, and in-situ grouting using high strength cementitious materials. The convention connections are not universally applicable to all types of RC structures. Other challenges remain with the use of precast

structures, for example, the considerable imperfection tolerance from structure to structure can affect the resultant strength and performance of the connected structure. Some RC structures are therefore not precast. For example, the conventional method of forming concrete-encased concrete-filled steel tube (CECFST) columns is wholly carried out in-situ.

SUMMARY

[0005] In one aspect, the present application discloses a column. The column includes: a steel tube, a first plurality of a lapped member, a second plurality of the lapped member, and a concrete body. The steel tube longitudinally extends between two open ends of the steel tube. The first plurality of the lapped member are partially disposed in an interior of the steel tube. The first plurality of the lapped member are coupled to an interior surface of the steel tube. The first plurality of the lapped member extend beyond one of the two open ends. The second plurality of the lapped member are partially disposed in the interior of the steel tube. The second plurality of the lapped member are coupled to the interior surface of the steel tube. The second plurality of the lapped member extend beyond another of the two open ends. The concrete body is cast about an exterior surface of the steel tube to form a precast concrete-encased steel tube. The steel tube is internally unfilled with concrete to provide a contiguous space in fluidic communication with the two open ends.

[0006] In another aspect, the present application discloses a method. The method includes aligning a top of first column and a bottom of a second column to define an interface therebetween, in which one or both of the first column and the second column are as described above, and in which the steel tube of the first column and the steel tube of the second column are aligned to define a contiguous space in fluidic communication across the interface.

[0007] In another aspect, the present appliction discloses a column-to-column connection system. The column-to-column connection system includes: a first plurality of a lapped

member, a second plurality of the lapped member, and a concrete core. In the column-to-column connection system, the first column and the second column in assembly includes: an outer reinforcement, the outer reinforcement including the rebar coupled to the column shoe.

[0008] In another aspect, the present application discloses a building structure including at least two columns in assembly using the column-to-column connection system as described above.

[0009] In yet another aspect, disclosed herein is a building structure including: a first precast column, a second precast column, a concrete core cast in situ, and a plurality of a lapped member. Each of the lapped member is fixed coupled to the steel tube of one of the first precast column and the second precast column. The plurality of the lapped member are embedded in the concrete core in an interleaved arrangement across the interface.

BRIEF DESCRIPTION OF THE DRAWINGS

[0010] Various embodiments of the present disclosure are described below with reference to the following drawings:

[0011] Figs. 1A-1G schematically illustrate precast column segments and a precast column-to-column dry connection system in accordance with some embodiments of the present disclosure;

[0012] Figs. 2A-2D schematically illustrate precast column segments and a precast column-to-column dry connection system in accordance with other embodiments of the present disclosure;

[0013] Fig. 3 shows images of a method of making PCDCs;

[0014] Fig. 4 illustrates diagrammatically some dimensions of pullout tests;

[0015] Figs. 5A-5F illustrate diagrammatically some dimensions of test specimens;

[0016] Fig. 6 is a schematic diagram of a set up for pullout tests;

[0017] Fig. 7 is a schematic diagram of a set up for connection tests;

[0018] Fig. 8 shows a measurement scheme of test specimens;

[0019] Fig. 9 is a plot showing bond stress-slip relationships of pullout tests;

[0020] Fig. 10 shows load–displacement relationships of CFST specimens;

[0021] Fig. 11 shows load–displacement relationship of CECFST specimens;

[0022] Fig. 12 shows failure modes of CFST connections;

[0023] Fig. 13 shows interfaces between reinforcing bars and inner concrete core;

[0024] Fig. 14 shows failure modes of CECFST connections;

[0025] Fig. 15 shows plots of load–strain relationships of CFST connections;

[0026] Fig. 16 shows plots of load–strain relationships of CECFST connections;

[0027] Fig. 17 shows charts of load–opening relationships of four connection specimens;

[0028] Fig. 18 illustrates stress and strain distribution of the RC connection at ultimate state;

[0029] Fig. 19 illustrates a load transfer mechanism of CFST connections;

[0030] Fig. 20 illustrates a failure mechanism of inner CFST connection of PCDC-A;

[0031] Fig. 21 illustrates a failure mechanism of inner CFST connection of PCDC-B;

[0032] Fig. 22 shows stress and strain distributions of lapped /anchored bars at elastic stage;

[0033] Fig. 23 shows stress and train distributions of lapped/anchored bars at plastic stage;

[0034] Fig. 24 shows the predication of load–opening relationships of CFST connections;

[0035] Fig. 25 is a chart showing a compatibility analysis of connection opening;

[0036] Fig. 26 is a chart on a parametric study of the ductility of PCDCs;

[0037] Fig. 27 is a flowchart of a design procedure of PCDCs;

[0038] Fig. 28 is a cross-section of a CECFST column;

[0039] Fig. 29 is a schematic cross-sectional view of some embodiments of the proposed connection at the interface;

[0040] Fig. 30 is a schematic cross-sectional view of other embodiments;

[0041] Fig. 31 shows an example of a precast CECFST column-to-column connection according to embodiments of the present disclosure, used as the basis for axial and eccentric compression analysis;

[0042] Fig. 32 shows diagrams of test specimens and their dimensions;

[0043] Fig. 33 shows the test setup and instrumentation used;

[0044] Fig. 34 shows plots of load-axial shortening relationships of test specimens;

[0045] Fig. 35 shows images of failure modes of the test specimens;

[0046] Fig. 36 shows a graphical representation of a failure mechanism of one example of the precast CECFST column-to-column connection;

[0047] Fig. 37 shows FE modelling of a precast CECFST column-to-column connection;

[0048] Fig. 38 shows stress-strain models used for steel components;

[0049] Fig. 39 shows a modelling of an RC connection;

[0050] Fig. 40 shows a modelling of a CFST connection;

[0051] Fig. 41 shows a modelling of the bond-slip behaviour of the lapped bars (lapped members);

[0052] Fig. 42 shows a codified bond-slip relationship;

[0053] Fig. 43 shows boundary conditions of the test specimen;

[0054] Fig. 44 shows prediction of load-lateral deflection relationships;

[0055] Fig. 45 shows a prediction of the failure mode;

[0056] Fig. 46 shows a typical N-M iteration curve of columns;

[0057] Fig. 47 shows a plastic stress distribution in a cross-section of a connection interface;

[0058] Fig. 48 shows a transformed cross-section of a precast connection;

[0059] Fig. 49 shows the neutral axis within the steel tube;

[0060] Fig. 50 shows the neutral axis outside the steel tube;

[0061] Fig. 51 shows the neutral axis coinciding with the centre line; and

[0062] Fig. 52 compares between model prediction and experimental and numerical results.

DETAILED DESCRIPTION

[0063] The following detailed description is made with reference to the accompanying drawings, showing details and embodiments of the present disclosure for the purposes of illustration. Features that are described in the context of an embodiment may correspondingly be applicable to the same or similar features in the other embodiments, even if not explicitly described in these other embodiments. Additions and/or combinations and/or alternatives as described for a feature in the context of an embodiment may correspondingly be applicable to the same or similar feature in the other embodiments.

[0064] In the context of various embodiments, the articles “a”, “an” and “the” as used with regard to a feature or element include a reference to one or more of the features or elements.

[0065] In the context of various embodiments, the term “about” or “approximately” as applied to a numeric value encompasses the exact value and a reasonable variance as generally understood in the relevant technical field, e.g., within 10% of the specified value.

[0066] As used herein, the term “and/or” includes any and all combinations of one or more of the associated listed items.

[0067] In one aspect, the present disclosure includes various embodiments of a column-to-column dry connection system for precast concrete columns. In another aspect, the present disclosure includes various embodiments of a precast concrete-encased concrete-filled steel tubular (CECFST) column (segment) configured to implement the proposed column-to-column dry connection system. In yet another aspect, various embodiments of the present disclosure includes buildings and other structures which are at least partially fabricated using one or more of the proposed column-to-column dry connection system and/or precast CECFST column.

Another aspect of the present disclosure relates to the related processes of making a precast segment.

[0068] To minimize self-weight of precast CECFST columns during transportation and hoisting, the outer RC 400 encasement and the embedded steel tube 200 can be fabricated in factories to reduce secondary on-site work. With the embedded steel tube 200 serving as permanent formwork, the inner concrete core 700 can be cast in-situ. In this paper, two innovative PCDCs 180, referred to as PCDC-A (Figs. 1A-1G) and PCDC-B (Figs. 2A-2D) will be described with reference to the appended figures. The PCDCs 180 consist of two independent parts: 1) The outer RC connection and 2) the inner CFST connection. Since tensile strength of concrete is negligible, only steel components such as longitudinal reinforcement and steel tubes were connected in the developed PCDCs. For ease of reference, a longitudinal axis or longitudinal direction 900 may be defined by the steel tube 200 or by the column/column segment 100 / 101 / 102.

[0069] According to various embodiments of the present disclosure, the column 100 (also referred to as a column segment or a segment) may be precast (e.g., off-site or in a factor) with a concrete body 400 cast about an exterior surface 224 of a steel tube 200. For example, the steel tube 200 may be described as longitudinally 900 extending between two open ends 210 of the steel tube 200. The precast concrete-encased steel tube 400/200 includes the steel tube 200 with concrete 400 cast only on the exterior of the steel tube 200. For example, precast concrete-encased steel tube 200 may be internally unfilled with concrete to provide a contiguous space in fluidic communication with the two open ends 210 of the steel tube 200. For PCDC-A, column shoes 600 were utilized to connect longitudinal reinforcement of outer RC encasement 400. Specifically, the longitudinal reinforcement of the upper column 100/102, referred to as upper reinforcement, were pre-welded to the column shoes 600. In some examples, a plurality of the column shoe 600 may be distributed external of the steel tube 200. Each column shoe

600 may be partially embedded in the concrete body 400 (cast about the exterior surface 224 of the steel tube 200). The column shoe 600 may be proximal to a bottom of the concrete body 400 (of an upper column). As illustrated in Figs. 1A-1G, the column shoe may include two caps 610,620 spaced apart along the longitudinal direction 900. The lower cap 610 provides a hole to receive a threaded end of a rebar 500 (lower reinforcement) such that the rebar 500 can be bolted to the lower cap 610. The upper cap 620 and the lower cap 610 are longitudinally spaced apart to provide an access space essentially free of grout / cement. The access space facilitates/enables fastening of the column shoe 600 with the respective/complementary rebar 500. The top ends of the longitudinal reinforcement 500 from the lower column 100/101, referred to as lower reinforcement, were threaded for a certain length. In some examples, the lower reinforcements 500 include a plurality of a rebar distributed external of the steel tube. Each rebar 500 may be partially embedded in the concrete body and partially extend from a top of the concrete body (of a lower column). Note that during the assembly process, threaded regions of the lower reinforcement 500 were designed to pass through the pre-drilled holes of the column shoes 600 and then to be tightened with nuts and washers by a torque wrench on-site. As for the inner CFST connection, the bond strength between reinforcing bars and concrete was designed to connect the upper and lower steel tubes.

[0070] In this regard, two groups of reinforcing bars (also referred to herein as "lapped members"), as shown in Figs. 1A and 1B, were pre-welded onto the inner surfaces of the upper and lower steel tube, respectively. The lapped bars were welded with banded steel cushions (Fig. 1A) to increase tolerance against fabrication imperfections. For example, each lapped member includes a lapped bar 310 that extends longitudinally 900. In some examples, the lapped member 300 may include a banded steel cushion 330 (also referred to as a "cushion" for the sake of brevity). The cushion 330 may be joined to the interior surface 222 of the steel tube 200 and extends transversely from the interior surface 222. The cushion 330 may be coupled

on opposing sides to the respective lapped bar 310 and to the interior surface 222 of the steel tube 200. That is, the lapped bar 310 may be spaced apart from the interior surface 222 of the steel tube 200.

[0071] Before assembly or installation on-site, the precast column 100/101/102 includes a plurality of the lapped member 300 (e.g., the lapped bars 310) partially disposed in the interior 220 of the steel tube 200 with a length of each of the lapped bar 310 extending beyond one of the open ends 210 of the steel tube 200. In some example, a first plurality of the lapped member 300/301 extends from one open end 210 of the steel tube 200 and a second plurality of the lapped member 300/302 extends from another (opposing) end 210 of the steel tube 100. Each lapped member 300 is welded or otherwise coupled to the interior surface 222 of the steel tube 200 and the lapped bars 310 form part of the precast column segment 100.

[0072] The fabrication of a column structure 100/950 from two or more column segments 101/102 may include aligning a top of a first precast column 101 with a bottom of a second precast column 102 to define an interface 150 therebetween. In alignment or in assembly, the steel tube 200 of the first precast column 101 and the steel tube 200 of the second precast column 102 are aligned to define a contiguous space 220 in fluidic communication across the interface 150.

[0073] After joining the upper and lower hollow-core precast CECFST columns 101/102 together by tightening the nuts (e.g., to couple respective complementary ones of a rebar 500 and a column shoe 600), the inner concrete core (contiguous column of concrete) 700 was cast-in-situ. The two steel tubes 200 are in fluidic communication such that fresh concrete may be poured in via the open (top) end 210 of the upper steel tube 102, and the fresh concrete will first fill up the interior space 220 of the lower steel tube 101. A contiguous column of concrete (also referred to as a concrete core 700) can thus be cast in situ to form a concrete-encased concrete-filled steel tubular (CECFST) column from two or more CEST column segments 101/102. To

make full use of the bond strength between the inner concrete core 700 and lapped bars 310, the arrangement of the lapped bars 310 should preferably comply with the following requirements: 1) for each lapped bar 310, only half of its length protruded out of the steel tube, with the other half welded onto the tube inner surface (e.g., interior surface 222); 2) the upper and the lower lapped bars 302/301 were evenly staggered alongside the tube edge upon assembly, and they "overlapped" with each other inside the assembled steel tube 200 (Fig. 1D); 3) the lapped bars 310 were sufficiently long to prevent pullout failure.

[0074] In assembly or on formation of the CECFST column 100 from connecting a first column 101 and a second column 102, the lapped members 300 of the first column 101 and the second column 102 may be interleaved around the edges of the steel tubes 200, with the lapped members 300 extending across the interface 150. In the CECFST column 100, the lapped members 300 will also be fully embedded in the concrete core 700. It can be appreciated that completed CECFST column, joined using the column-to-column dry connection 180 proposed herein, forms a building structure or a part of a building structure 950.

[0075] For PCDC-B, the outer RC connection is similar to that of PCDC-A, while the inner steel tubes are connected by a different configuration, as shown in Figs. 2A-2D. Instead of two separate groups of lapped bars, a group of anchored bars were evenly welded onto the inner surface of the upper steel tube with banded steel cushions, and they protruded downwards with a certain anchorage length. A group of shear studs were welded on the inner surface of the lower steel tube to improve the roughness of the interface between the inner concrete core and the lower steel tube. The anchored bars were welded on the steel tube and they lay side by side with the bottom welded shear studs (Fig. 2D).

[0076] For these two precast connections (PCDC-A and PCDC-B), neither on-site welding nor formwork was required for the assembly process. Moreover, the connections provided flush appearance, considerable tolerance against fabrication imperfections and excellent fire

resistance. A comparison on the PCDC-A and PCDC-B is summarised in Table 1, where the similarities and differences between these two connections are presented. Table 2 introduces the detailed arrangement of connection components. Compared with PCDC-A, the fabrication process of PCDC-B is more complex due to the welding of shear studs. However, shear studs not only enhance the interaction between the concrete and the steel tube but also reduce protrusions, facilitating transportation and assembly on-site. Fig. 3 presents an exemplary construction procedure of the PCDCs. The outer RC component and the embedded steel tube were first prefabricated in a precast yard and the inner concrete core was cast-in-situ after connection assembly.

[0077] Tests and Experimental Results

[0078] A total of ten specimens were tested in the present experimental programme, including four component tests and six connection tests.

[0079] Component tests

[0080] The component tests consisted of four pullout tests, namely two concentric pullout (CP) tests and two eccentric pullout (EC) tests. The detailed dimensions of the component specimens are shown in Fig. 4. The pullout tests were carried out to investigate the effect of thin concrete cover on bond stress-slip behaviour of reinforcing bars, since all the lapped/anchored bars of the inner CFST connection (Fig. 1D and Fig. 2D) were eccentrically placed along the edges of the steel tubes with a small concrete cover of 8 mm (Fig. 4 (b)). Note that since the component tests served as a basis for the connection tests, the steel tubes, reinforcing bars and the concrete utilised in pullout tests were same as those of corresponding connection tests.

[0081] Connection tests

[0082] For connection tests, two groups of specimens were designed to investigate tensile performance of the inner CFST connections and corresponding CECFST connections. Each

group has two column-to-column connections and one entire column as a reference. The basic information on these six test specimens is summarised in Table 3, where *A* and *B* indicate the configuration of PCDC-A and PCDC-B, respectively; *R* indicates the reference specimen; *L* is the length of the specimens; *D* and *t* are the outer diameter and thickness of the steel tube, respectively; *H* is the width of the CECFST specimens; d_s and d_{ts} are the diameter of unthreaded and threaded regions of longitudinal reinforcement, respectively; d_{cs} is the diameter of lapped/anchored bars; d_{sc} is the diameter of shear studs; l_{ts} is the length of effective threaded region of lower reinforcement. Note: the effective threaded region is defined as a combination of two separate threaded regions: 1) threaded region between upper and lower nuts and 2) threaded region below lower nuts, as shown in Fig. 1C and Fig. 2C.

No.	Specimen Label	L (mm)	$H \times H$ (mm)	$D \times t$ (mm)	d_s (mm)	d_{ts} (mm)	d_{cs} (mm)	d_{sc} (mm)	l_{ts} (mm)
1	CFST-R	800	---	114 x 4	---	---	---	---	---
2	CFST-A	800	---	114 x 4	---	---	13	---	---
3	CFST-B	800	---	114 x 4	---	---	13	13	---
4	CECFST-R	800	220 × 220	114 x 4	13	---	---	---	---
5	CECFST-A	800	220 × 220	114 x 4	13	10.2	13	---	25
6	CECFST-B	800	220 × 220	114 x 4	13	10.2	13	13	25

[0083] Figs. 5A-5F show the detailed dimensions of the test specimens. For the four connection specimens, six pieces of short reinforcing bars with an overall tensile strength of 505 kN were utilised to achieve nominal tensile strength of the reference specimen CFST-R ($N_{ny,cfst}$ = 508 kN), which is obtained by a strength model proposed by Han et al.. To prevent brittle pullout failure, lapped bars/anchored bars and shear studs where applicable, were designed as per applicable industrial standards (e.g., Eurocode 2 / EN 1992-1-1, Eurocode 2: design of concrete structures–part 1-1: general rules and rules for buildings, European Committee for Standardization (CEN), Brussels, 2004. 2004; and Eurocode 4. EN 1994-1-1, Eurocode 4: Design of Composite Steel and Concrete Structures – Part 1-1: General Rules and Rules for Buildings, European Committee for Standardization (CEN), Brussels, 2004. 2004.,

also referred to as EN-1992-1-1 and EN-1994-1-1, respectively). In EN 1992-1-1, the respective requirements on lap length and anchorage length of reinforcing bars are different. Particularly, the required lap length of the lapped bars (PCDC-A) should be larger than 12 d_s (156 mm), while the specified anchorage length of the anchored bars (PCDC-B) should be larger than 14 d_s (182 mm). In this experimental programme, to avoid pullout failure, the lap length of lapped bars and the anchorage length of anchored bars were uniformly set as 200 mm conservatively. According to EN-1994-1-1, the shear resistance of headed studs should be greater than the overall tensile strength of anchored bars (505 kN). For this reason, a group of twelve headed studs with a shear resistance of $N_{y,sc}$ = 509 kN were welded on the inner surface of the lower tube, as shown in Fig. 5 (c).

[0084] Material properties

[0085] Table 4 presents material properties of concrete and steel components. The compressive strength of concrete was measured from compression tests of standard concrete cylinders with a diameter of 100 mm and a height of 150 mm. To obtain material properties of the steel tubes and steel reinforcement, coupon tests conforming to the standard requirements of ASTM (ASTM A. E8/E8M-08. Standard test methods for tension testing of metallic materials. West Conshohocken, PA; 2016) were conducted in a 50 kN servo-hydraulic tensile testing machine. For shear studs, the information provided by the supplier was used. The lapped/anchored bars and longitudinal reinforcement used in this study were from the same batch, and all specimens were cast at one time.

Table 4.

Concrete				
		Inner concrete		Outer concrete
Strength (MPa)		102.5		42.7
Steel				
	Longitudinal reinforcement	Lateral reinforcement	Steel tube	Shear studs
Yield strength (MPa)	552	367	414	450
Elastic modulus (GPa)	208	207	279	---
Yield strain (%)	0.27	0.18	0.39	---

Ultimate strength (MPa)	634	446	556	500
Ultimate strain (%)	9.8	7.2	12.6	---

[0086] Test setup

[0087] For pullout tests, as shown in Fig. 6, the top O-shaped jig of the testing machine gripped the embedded reinforcing bars, while the CFST blocks were fixed on the bottom V-shaped jig via a commonly used clamping system. One linear variable differential transducer (LVDT) was installed at the bottom end of the embedded reinforcing bar to record the end slip. A displacement-control loading scheme with a rate of 2 mm/min was adopted.

[0088] Fig. 7 shows the setup for six connection tests, including three CFST specimens and three CECFST specimens. To apply external tensile force, one pair of dismountable "Tee"-shaped holders with pre-drilled holes were connected to the endplates of the specimens by eight 10.9 grade high strength bolts with 20 mm diameter. Subsequently, the vertical plates of the "Tee" holders were clamped by the V-shaped jigs of the testing machine. A computer-controlled actuator with a tensile capacity of 2000 kN was used, and tension was applied under displacement control at a rate of 0.5 mm/min. As shown in Fig. 8 (a), four linear variable displacement transducers (LVDTs), namely, L1, L2, L3, and L4, were installed to measure axial deformation. Moreover, another three LVDTs, L5, L6, and L7, were installed on the four connection specimens to measure potential openings since they would reflect connection stiffness and ductility. Fig. 8 (b) shows the arrangement of strain gauges. For all four connection specimens, SG-1 was affixed on connected steel tubes, SG-2 on the lapped/anchored bar, while SG-3 was bonded on the unthreaded region of lower H13 reinforcement.

[0089] Test results - Load versus deformation relationships

[0090] Component tests

[0091] Fig. 9 presents the measured load–slip relationships of the four pullout tests, including two eccentric tests (EP-1 and EP-2) and two concentric tests (CP-1 and CP-2). A small concrete cover would lead to a reduced ultimate bond strength and a brittle failure, which

should be prevented in structural design. In contrast, in this study, the measured ultimate bond stress of eccentric pullout tests with a concrete cover of 8 mm was comparable to corresponding concentric pullout tests with a large concrete cover of 45 mm. This demonstrated that the thin concrete cover of 8 mm did not undermine the bond–slip performance of reinforcing bars enclosed by CFST components. This is probably because the steel tube provided large confinement to the concrete cover and prevented it from splitting. Note that the above pullout test results supported the fundamental concept of the developed precast CECFST connections where lapped/anchored bars had a small concrete cover as shown in Fig. 5 (b) and (c). Specifically, the bond strength between the lapped/anchored bars and the inner concrete core would not be weakened despite a small concrete cover of 8 mm. Thus, it could be utilised to connect the steel tubes as expected.

[0092] Connection tests

[0093] Fig. 10 presents load–displacement relationships of the three CFST specimens. At the initial stage, the stiffness of connection specimens CFST-A and CFST-B was comparable to that of the reference specimen CFST-R. With the applied load exceeding the yield strength of the lapped/anchored bar group ($N_{y,csg}$ = 439 kN), load–displacement curves of the two connections began to soften with a tensile strength over 500 kN, larger than the nominal yield strength of corresponding CFST columns ($N_{ny,cfst}$ = 494 kN). This indicated that the connections developed for the inner CFST component of CECFST columns were effective and reliable in terms of tensile strength. The reference specimen CFST-R showed a larger tensile strength than specimens CFST-A and CFST-B. This is because the yield strength of steel tubes provided by the supplier was unexpectedly higher than 355 MPa, as shown in Table 4. According to the measured material properties (Table 4), tensile strength of CFST columns $N_{y,cfst}$ was around 596 kN, which exceeded $N_{ny,cfst}$ by 88 kN.

[0094] Fig. 11 presents load–displacement relationships of the three CECFST specimens. Connection specimens CECFST-A and CECFST-B showed a comparable initial stiffness to the

reference specimen CECFST-R. Moreover, the ultimate tensile strength of all the two CECFST connections exceeded 90 % nominal yield strength of the connected precast CECFST columns N_{ny} =796 kN, which was obtained by a conventional strength model. To ensure that precast connections could meet the code requirement, the tensile resistance of column-to-column connections should be larger than 50% nominal yield strength of the connected precast columns N_{ny}. In this regard, the two proposed PCDCs have sufficient tensile resistance and could achieve the code requirement. Note that the tensile strength of the PCDCs can be as high as the connected columns provided that sufficient number of lapped/anchored bars are incorporated to connect the inner steel tubes and that the diameter of the threaded region of lower reinforcement is larger than that of the longitudinal reinforcement of the connected columns.

[0095] Failure modes

[0096] Failure modes of precast connections provide researchers with critical insights into load transfer mechanisms of corresponding configurations. In this paper, failure modes of all the four connection specimens were recorded for subsequent connection analyses.

[0097] Fig. 12 presents the failure mode of specimens CFST-A and CFST-B. It was observed that both of them failed by fracture of respective lapped/anchored bars. Fracture of lapped/anchored bars implied that the lap/anchorage length was sufficient to prevent potential pullout failure for both specimens and that the arrangement of shear studs for the specimens CFST-B was appropriate. This indicated that it is feasible to determine the length of lapped/anchored bars and the number of shear studs for the developed CFST connections as per codified provisions of EN 1992-1-1 and EN 1994-1-1. Furthermore, to study the interaction between lapped/anchored bars and the inner concrete core, respective interfaces of specimens CFST-A and CFST-B were photographed by extracting the fractured reinforcing bars via an electrical saw, as shown in Fig. 13. It showed that for specimens CFST-A and CFST-B, concrete ribs at the interface remained intact even after the fracture of lapped/anchored bars. This

indicated that only limited slip occurred between the lapped/anchored bars and the inner concrete core. In other words, the connection detailing was sufficiently ductile to prevent brittle pullout failure.

[0098] Fig. 14 presents the failure mode of the two CECFST connections. As expected, specimens CECFST-A and CECFST-B failed by fracture of inner lapped/anchored bars and outer threaded longitudinal reinforcement (H13). It should be pointed out that during the tests, the outer lower H13 reinforcement fractured earlier than the inner reinforcing bars, implying that the ultimate deformability of the outer RC connection was weaker than that of the inner CFST connection.

[0099] Strain analysis

[00100] This section presents a strain analysis on the tested connection specimens. In Fig. 15, the longitudinal strain of steel tubes (SG-1) increased linearly as the tensile force increased. In contrast, the strain of lapped bars (SG-2) increased significantly after the applied load exceeded the nominal yield strength of the lapped bar group ($N_{ny,csg}$). It indicated that the deformation of CFST connections was mainly concentrated on the lapped bars, consistent with the observed failure modes (Fig. 12). Note that the strain gauge affixed on the anchored bar of specimen CFST-B (SG-2 (CFST-B)) was damaged when the load exceeded 350 kN.

[00101] For CECFST connections, as shown in Fig. 16 (a), with an increase in the applied load, the longitudinal strain of connected tubes (SG-1) increased linearly, whereas that of lapped/anchored bars (SG-2) showed large strain increments when the load approached the yield strength of CECFST connections (N_{yc}=612 kN). This is consistent with the findings obtained from corresponding CFST connections. Additionally, Fig. 16 (b) shows that the strain of unthreaded regions of H13 reinforcement (SG-3) increased linearly as the tensile force increased, but did not exceed yield strain (ε_{ys}=0.27%) although outer RC connections showed severe deformations and even fractured (Fig. 14). The reason was that under axial tension,

failure of outer RC connections was governed by threaded regions of lower H13 reinforcement due to its reduced cross-sectional area.

[00102] Connection opening and ductility

[00103] Fig. 17 presents load versus connection opening relationships of the four connection specimens under axial tension. Due to a minor eccentricity during testing, some differences existed between the data recorded by the LVDTs (L5, L6, and L7). In this regard, an average load–opening curve labelled as "L-Ave" was drawn for each specimen. As shown in Fig. 17 (a)–(b), with an increase of tensile force, the opening of specimens CFST-A and CFST-B increased linearly at the initial stage until the load reached around 450 kN, almost equal to the yield strength of the lapped/anchored bar group $N_{y,csg}$. Afterwards, the opening of these two specimens increased significantly with a minor increase of tensile force, indicating that the deformation lapped/anchored bars governed the connection opening. As for CECFST connections (Fig. 17 (c)–(d)), the load–opening curves demonstrated a similar trend as those of corresponding CFST specimens. However, the opening of CECFST connections under the ultimate state was slightly lower than that of corresponding CFST connections.

[00104] To evaluate ductility of the proposed precast column-to-column connections, a ductility index (*DI*) is defined as the ratio of O_u over O_y, where O_y and O_u are connection openings at the yield and ultimate loads, respectively. The measured value of O_y and O_u of the four connection specimens were summarised in Table 5. Moreover, the ductility index (DI) of each connection specimen was calculated accordingly. It was observed that the ductility of all the four connections was larger than or equal to 4.0, which is a minimum requirement for grouted precast column-to-column connections in low-to-moderate earthquake-prone regions. It indicated that the developed connections could be classified as ductile connections.

Table 5. Ductility of the tested connections

Specimens	O_y (mm)	O_u (mm)	DI_e (O_u/O_y)

CFST-A	0.6	3.7	6.2
CFST-B	0.6	3.6	6.0
CECFST-A	0.8	3.2	4.0
CECFST-B	0.7	3.0	4.3

[00105] Connection analysis and component-based models

[00106] For the design of precast connections, ductility and strength are of paramount importance, particularly in earthquake-prone regions. Therefore, besides the experimental investigation, a series of theoretical analyses were conducted to explore the key factors that affect ductility and strength of the proposed PCDCs. Prior to connection analyses, load transfer mechanisms of the developed PCDCs were first proposed, followed by the development of component-based models for ductility assessment and strength prediction.

[00107] Load transfer mechanisms

[00108] Given that the PCDCs consist of two independent parts, namely the outer RC connection and the inner CFST connection, load transfer mechanisms of respective RC and CFST connections are presented as follows.

[00109] Load transfer mechanism of outer RC connection

[00110] For the outer RC connection, since the longitudinal reinforcement was designed to be directly connected by steel column shoes with nuts, the load transfer mechanism was straightforward. Specifically, under axial tension, the tensile force incurred by the upper longitudinal reinforcement was transferred to the column shoe due to the butt weld. Then, the force was transferred to the lower reinforcement through the nuts and washers (Fig. 1 (c) and Fig. 2(c)).

[00111] Fig. 18 shows the stress and strain distribution of the outer RC connection at the ultimate state, where Ns is the tensile force experienced by longitudinal reinforcement; “UR” indicates upper H13 reinforcement; “SH” indicates the column shoe; “ETR” and “BR” indicate the effective threaded region and unthreaded region of lower H13 reinforcement, respectively. Under axial tension, due to a reduction of cross-section area, the effective threaded regions of the H13 reinforcement would yield first and attain ultimate strain at ultimate load, while the

other parts would remain elastic. It indicated that severe strain concentration occurred at the effective threaded region of the lower H13 reinforcement when the external load exceeded the yield strength of the outer RC connection, consistent with the failure mode reported in Fig. 14.

[00112] Load transfer mechanisms of inner CFST connections

[00113] Fig. 19 presents load transfer mechanisms of the inner CFST connections, where $N_{c,cfst}$ is the tensile force experienced by the CFST connections. For the PCDC-A, tensile force from the upper tube was first transferred to the upper lapped bars through the butt weld. Afterwards, the protruding part of upper lapped bars interacted with adjacent lower lapped bars via inclined concrete struts, as shown in Fig. 19 (a). Since the lap length was sufficiently long, the tensile force from the upper steel tube was effectively transferred to the lower steel tube through the bond between the lapped bars and the inner concrete core. For the PCDC-B, tensile force from the upper steel tube was first transferred to the welded anchored bars. Subsequently, as shown in Fig. 19 (b), through inclined concrete struts formed between the anchored bars and welded shear studs, the tensile force was transferred to the lower steel tube. For both PCDC-A and PCDC-B, as the connection region opened with an increase of tensile force, the inner concrete core fractured at the edge of the steel tube. Subsequently, as shown in Fig. 20 (a) and Fig. 21 (a), where N_{cs} is the tensile force experienced by lapped/anchored bar, the inclined concrete struts close to the tube edges were pulled out in a conical shape due to a lack of lateral restraint, resulting in an exposed zone for the respective lapped/anchored bar. Fig. 20 (b) and Fig. 21 (b) present close-up views of the failed connections, which showed that the inner concrete core fractured, and all the four connection specimens failed in conical mode. This indicated that the proposed load transfer mechanisms took place.

[00114] Component-based models

[00115] Based on the load transfer mechanisms, component-based models were proposed to calculate the opening of the outer RC and the inner CFST connections under yield and ultimate

states. The proposed models will be used to predict the ductility and tensile strength of the PCDCs.

[00116] According to Fig. 18, the opening of the outer RC connection varied with the length of the effective threaded region. Therefore, Eqs. (1) and (2) can be used to calculate the opening of the outer RC connection at yield and ultimate states, respectively.

[00117]

$$O_{\mathrm{yc,rc}} = l_{\mathrm{ts}} \times \varepsilon_{\mathrm{ys}} \tag{1}$$

$$O_{\mathrm{uc,rc}} = l_{\mathrm{ts}} \times \varepsilon_{\mathrm{us}} \tag{2}$$

[00118] where $O_{\mathrm{yc,rc}}$ and $O_{\mathrm{uc,rc}}$ are openings of the outer RC connection at yield and ultimate states, respectively; $\varepsilon_{\mathrm{ys}}$ and $\varepsilon_{\mathrm{us}}$ are yield and ultimate strains of longitudinal reinforcement.

[00119] As for the inner CFST connection, its opening consists of two parts: 1) elongation of lapped/ anchored bars and 2) slip of lapped/anchored bars. Based on the load transfer mechanisms presented in Section 4.1, the protruding lapped/anchored bar close to the tube edge would be exposed (Fig. 21) from the inner concrete core due to conical failure of inclined concrete struts. In this regard, under the elastic stage ($N_{\mathrm{c,cfst}} \leq N_{\mathrm{y,csg}}$), as shown in Fig. 22 (a), the lapped/anchored bar can be classified into two zones ("*α"* and *"γ"*). The stress of the exposed region ("*α"* zone) f_{cs} is uniform, whereas the stress of the other part ("*γ"* zone) varies linearly from f_{cs} to zero. Under the plastic stage ($N_{\mathrm{c,cfst}} > N_{\mathrm{y,csg}}$), as shown in Fig. 23 (a), the lapped/anchored bar can be classified into three zones ("*α"*, *"β"* and *"γ"*). The stress of the exposed region ("*α"* zone) f_{cs} is uniform, whereas the stress of the other region varies linearly from f_{cs} to f_{ycs} ("*β"* zone*)*, and then from f_{ycs} to zero ("*γ"* zone), where f_{ycs} is yield strength of lapped/anchored bars. Strain distributions of the lapped/anchored bar at elastic and plastic stages are presented in Fig. 22 (c) and Fig. 23 (c), where $\varepsilon_{\mathrm{cs}}$ is the strain of exposed region ("*α"* zone), and $\varepsilon_{\mathrm{ycs}}$ is yield strength of the lapped/anchored bars.

[00120] Based on the above analysis, the length of "*α*" zone l_{α} varies with the arrangement of connection components, i.e., reinforcing bars for the PCDC-A and shear studs for the PCDC-

B. Assuming that the angle of the inclined concrete struts θ is 45º , the length of "α" zone of the PCDC-A $l_{\alpha,a}$ is equal to the distance between lapped bars D_a (Fig. 19 (a)). Likewise, the length of "α" zone of the PCDC-B $l_{\alpha,b}$ is equal to the maximum distance between the anchored bar and adjacent shear studs D_b (Fig. 19 (b)). For both PCDC-A and PCDC-B, the length of "β" zone l_β (Fig. 23 (a)) can be calculated by Eqs. (3) and (4), where τ is the average bond stress. Finally, the length of the "γ" zone l_γ can be calculated by Eq. (5). l_β is equal to zero when the connection is at the elastic stage.

$$\tau = \frac{f_{cs} \times \pi \times \frac{d_{cs}^2}{4}}{\pi \times d_{cs} \times (l_s - l_\alpha)} \tag{3}$$

$$l_\beta = \frac{(f_{cs} - f_{ycs}) \times \pi \times \frac{d_{cs}^2}{4}}{\pi \times d_{cs} \times \tau} \tag{4}$$

$$l_\gamma = l_s - l_\alpha - l_\beta \tag{5}$$

[00121] where l_s is the lap/anchorage length of lapped/anchored bars.

[00122] Based on the above analysis, the elongation of the lapped/anchored bars (E_{cs}) can be calculated by Eq. (6).

$$E_{cs} = \begin{cases} \varepsilon_{ycs} \times l_\alpha + \frac{\varepsilon_{ycs}}{2} \times l_{y,\gamma}, \ N_{c,cfst} \le N_{y,csg} \\ \varepsilon_{cs} \times l_\alpha + \frac{\varepsilon_{cs} + \varepsilon_{ycs}}{2} \times l_\beta + \frac{\varepsilon_{ycs}}{2} \times l_\gamma, \ N_{c,cfst} > N_{y,csg} \end{cases} \tag{6}$$

[00123] In terms of the slip of lapped/anchored bars S_{cs}, with the bond stress τ obtained by Eq.3, it can be calculated by bond stress- slip models (Eq. 7) provided in CEB-FIB codes.

$$\tau = \tau_{max} (S_{cs} / S_1)^{\mu} \tag{7}$$

[00124] where τ and S_{cs} are the bond stress and corresponding slip of lapped/anchored bars, respectively; τ_{max} is the maximum bond stress depending on compressive strength of the inner concrete core f_{ic}, $\tau_{max} = 2.5\sqrt{f_{ic}}$; S_1 and μ are two parameters and should be 1 mm and 1, respectively.

[00125] After obtaining the elongation (E_{cs}) and the slip of lapped/anchored bars (S_{cs}), the opening of the inner CFST connection $O_{c,cfst}$ can be calculated by Eqs. (8).

$$O_{c,cfst} = E_{cs} + S_{cs} \tag{8}$$

[00126] Note that the tensile force experienced by the CFST connection $N_{c,cfst}$ that corresponds to the $O_{c,cfst}$ can be calculated by Eq. (9)

$$N_{c,cfst} = n_{cs} \frac{\pi d_{cs}^2}{4} f_{cs} \tag{9}$$

[00127] where n_{cs} is the number of lapped/anchored bars.

[00128] According to the proposed component-based model, the opening of CFST connections $O_{c,cfst}$ at each load level can be calculated by knowing the corresponding strain/stress value of the lapped/anchored bars. Therefore, to validate the model, the load–opening relationships of connection specimens CFST-A and CFST-B were predicted based on the measured stress-strain data of the lapped/anchored bars, as shown in Fig. 24, assuming that the load–opening curve is linear during the initial elastic stage. It showed that the predictions were in good agreement with the corresponding test results. Besides, the predicted yield and ultimate openings of the CFST connections match well with the corresponding test results as summarised in Table 6, where $O_{pc,cfst}$ and $O_{ec,cfst}$ are the predicted and experimental openings of the CFST connections. This indicated that it is feasible to predict the opening of inner CFST connections via the proposed component-based model.

Table 6. Calculations of connection openings

Specimens	Status	l_α (mm)	l_β (mm)	l_γ (mm)	S (mm)	$O_{pc,cfst}$ (mm)	$O_{ec,cfst}$ (mm)	$O_{pc,cfst}/O_{ec,cfst}$
CFST-A	Yield	23	0	177	0.40	0.70	0.6	1.17
	Ultimate	23	23	154	0.46	4.07	3.7	1.10
CFST-B	Yield	18	0	182	0.39	0.68	0.6	1.13
	Ultimate	18	24	158	0.45	3.61	3.6	1.01

[00129] Prediction of ductility and tensile strength

[00130] Ductility quantification

[00131] With the component-based models proposed in Section 4.2, the opening of the outer RC and the inner CFST connections of the PCDCs can be determined as per the connection detailing, respectively. Furthermore, to determine the opening of the overall PCDCs, a simplified compatibility analysis on these two components was conducted, as shown in Fig. 25, where $O_{yc,cfst}$ and $O_{uc,cfst}$ are the yield and ultimate openings of the CFST connections, respectively; N_{yc}, $N_{yc,rc}$, $N_{yc,cfst}$ are the yield strength of the PCDCs, the outer RC connection and the inner CFST connection, respectively. According to the load-opening relationship of PCDCs (Fig. 25), for conservatism, the yield opening of PCDCs O_{yc} can be assumed as the larger of $O_{yc,rc}$ and $O_{yc,cfst}$, while the ultimate opening of PCDCs O_{uc} can be assumed as the smaller of $O_{uc,rc}$ and $O_{uc,cfst}$. As a result, the ductility of the PCDCs can be calculated by Eq. (10).

[00132] Table 7 summarises the predicted (DI_{pc}) and experimental (DI_{ec}) ductility index of specimens CEFST-A and CECFST-B. It revealed that the proposed model can predict well the ductility of the developed PCDCs.

$$DI_c = \frac{O_{uc}}{O_{yc}} = \frac{Min\ (O_{uc,rc}, O_{uc,cfst})}{Max\ (O_{yc,rc}, O_{yc,cfst})} \tag{10}$$

Table 7. Predictions on ductility index (DI) of tested PCDCs

Specimens	$O_{yc,rc}$ (mm)	$O_{uc,rc}$ (mm)	$O_{yc,cfst}$ (mm)	$O_{uc,cfst}$ (mm)	DI_{pc}	DI_{ec}	DI_{pc}/ DI_{ec}
CECFST-A	0.08	2.45	0.70	4.07	3.50	4.0	0.88
CECFST-B	0.08	2.45	0.68	3.61	3.60	4.3	0.84

[00133] Based on the proposed quantification approach, a parametric study was conducted to explore the influence of two main factors (D_a /D_b and l_{ts}) on the ductility of the PCDCs, as shown in Fig. 26. It showed that with a simultaneous increase of D_a /D_b, and l_{ts}, the ductility of the PCDCs increases significantly. This indicates that the ductility of the PCDCs developed in this study could be improved by increasing the length of effective threaded region of lower reinforcement (l_{ts}) and the distance between connection components, i.e., lapped/anchored bars and shear studs (D_a /D_b).

[00134] Tensile strength

[00135] As shown in Fig. 25, the tensile strength of the PCDCs also relies on deformation compatibility between the outer RC and the inner CFST connections. Specifically, with a compatible zone, the tensile strength of the PCDCs (N_c) would attain the superposition of yield strengths of respective RC ($N_{yc,rc}$) and CFST ($N_{yc,cfst}$) connections. Otherwise, lower tensile resistance will be obtained. In this regard, to maximise the tensile resistance of the PCDCs, a relatively large compatible zone should be ensured ($DI > 1$). In this way, the tensile strength of the PCDCs can be calculated by Eqs. (11)–(13).

$$N_c = N_{yc,rc} + N_{yc,cfst} \tag{11}$$

$$N_{yc,rc} = n_{ts} \times \frac{f_{ys} \pi d_{ts}^2}{4} \tag{12}$$

$$N_{yc,cfst} = n_{cs} \times \frac{f_{ycs} \pi d_{cs}^2}{4} \tag{13}$$

where n_{ts} is the number of lower reinforcement, and f_{ys} is yield strength of longitudinal reinforcement.

[00136] According to the measured dimensions and material properties, the predicated tensile strength of specimens CECFST-A and CECFST-B was 612 kN, which was lower than the measured ultimate strength of around 700 kN, as shown in [0021]. This indicated that Eqs. (11)–(13) can be used to conservatively predict the tensile strength of the developed PCDCs.

[00137] Design method and worked example

[00138] Design method

[00139] As shown by a flowchart in Fig. 27, the design of the PCDCs developed in this paper can be generally divided into five steps: 1) Obtain geometric and material properties of the target precast CECFST columns, and determine the material properties of the connection components; 2) Determine the detailing of the outer RC connection; 3) Determine the detailing of the inner CFST connection; 4) Calculate the ductility of the designed connections by Eqs. (1)–(10); 5) Calculate the tensile resistance of the designed connections by Eqs. (11)–(13). To further clarify the design procedure, a worked example is presented in the following section.

[00140] Worked example

[00141] In general, CECFST columns are used in high-rise buildings. Therefore, as shown in Fig. 28, a large cross-section with a width of 1000 mm is chosen for illustration purpose. Geometric and material properties of the specimen are given as follows: D = 508 mm, t = 12 mm, f_y = 355 MPa, f_{ys} = f_{ycs} =550 MPa, ε_{ys}= 0.27 %, ε_{us} = 9.8 %, f_{ic} = 102.5 MPa. In the following design procedure, subscribes "*a*" and "*b*" represent PCDC-A and PCDC-B, respectively.

[00142] *Step 1*: The material properties of lapped/anchored bars are similar to that of the longitudinal reinforcement, and shear studs with a yield strength of 500 MPa is used.

[00143] *Step 2*: According to Fig. 28, twenty column shoes with sufficient tensile resistance are used to connect respective longitudinal reinforcement. Moreover, to ensure a high ductility for the overall connection, the length of effective threaded regions of lower reinforcement is set as 50 mm in accordance with Fig. 26.

[00144] *Step 3*: The tensile resistance of the inner CFST $N_{y,cfst}$ is 6638.5 kN. To fully connect the inner CFST components, the diameter of lapped/anchored bars is tentatively set as 32 mm, and thus the number of lapped/anchored bars n_{cs} is calculated by:

[00145] $$n_{cs} = \frac{6638.5}{3.14 \times 16^2 \times 550} = 15.1$$

[00146] For conservatism, n_{cs} is set as 16 for both PCDC-A and PCDC-B. Furthermore, the number of shear studs for the PCDC-B is calculated by:

$$n_{sc} = \frac{6683.5}{0.7 \times 3.14 \times (25/2)^2 \times 500} = 38.7$$

Likewise, n_{sc} is set as 40 conservatively.

[00147] In according to EN 1992-1-1, to prevent brittle pullout failure, the lap length of lapped bars (PCDC-A) should be larger than 12 d_{cs} = 384 mm, and the anchorage length of anchored bars (PCDC-B) should be larger than 14 d_{cs} = 448 mm. Conservatively, the lap length of lapped bars is set as 400 mm, and the anchorage length of anchored bars is set as 450 mm in this illustration.

[00148] *Step 4*: After obtaining the required number of lapped/anchored bars and shear studs, they are drawn to scale, as shown in [0039] and Fig. 30, where $D_a = 45$ mm, $D_b = 30$ mm. Note that 10 mm thick cushions are incorporated to provide tolerance for fabrication imperfections.

[00149] Based on the proposed component-based models, the ductility of the designed connections is calculated by obtaining the opening of the outer RC and the inner CFST connections under yield and ultimate states, respectively. For the outer RC connection, as per Eqs. (1) and (2):

$$O_{yc,rc} = 50 \times 0.27\ \% = 0.135 \text{ mm}$$
$$O_{uc,rc} = 50 \times 9.8\ \% = 4.9 \text{ mm}$$

[00150] For the inner CFST connection, prior to calculating the opening, the respective length of α, β, and γ zones (Fig. 22 and Fig. 23) of lapped/anchored bars is determined. $l_{\alpha,a} = D_a = 45$ mm, $l_{\alpha,b} = D_b = 30$ mm. To calculate the length of β zone l_β, the average bond stress of the lapped/anchored bars under the ultimate state is obtained by Eq. (3):

$$\tau_{u,a} = \frac{630 \times 3.14 \times 32^2 / 4}{3.14 \times 32 \times (400 - 45)} = 14.2 \text{ MPa} \quad \tau_{u,b} = \frac{630 \times 3.14 \times 32^2 / 4}{3.14 \times 32 \times (450 - 30)} = 12.0 \text{ MPa}$$

,

Afterwards, the length of β and γ zones can be calculated as per Eqs. (4–5):

for the PCDC-A:

$$l_{\beta a} = \frac{(630 - 550) \times 3.14 \times \frac{32^2}{4}}{3.14 \times 32 \times 14.2} = 45 \text{ mm}, l_{y,\gamma a} = 400 - 45 = 355 \text{ mm},$$

[00151] $l_{u,\gamma a} = 400 - 45 - 45 = 310$ mm

for the PCDC-B:

$$l_{\beta b} = \frac{(630 - 550) \times 3.14 \times \frac{32^2}{4}}{3.14 \times 32 \times 12} = 53 \text{ mm}, l_{y,\gamma b} = 450 - 30 = 420 \text{ mm},$$

[00152] $l_{u,\gamma b} = 450 - 30 - 53 = 367$ mm

[00153] Subsequently, the slip of lapped/anchored bars under yield and ultimate states is determined from Eq. (7), with the average bond stress under the yield state calculated by Eq. (3) as follows:

$$\tau_{y,a} = \frac{550 \times 3.14 \times 32^2 / 4}{3.14 \times 32 \times (400 - 45)} = 12.4 \text{ MPa} \quad \tau_{y,b} = \frac{530 \times 3.14 \times 32^2 / 4}{3.14 \times 32 \times (450 - 30)} = 10.5 \text{ MPa}$$

$$S_{y,a} = 12.4/(2.5\times\sqrt{102.5}) = 0.5 \text{ mm}, \quad S_{y,b} = 10.5/(2.5\times\sqrt{102.5}) = 0.4 \text{ mm}$$
$$S_{u,a} = 14.2/(2.5\times\sqrt{102.5}) = 0.6 \text{ mm}, \quad S_{u,b} = 12.0/(2.5\times\sqrt{102.5}) = 0.5 \text{ mm}$$

With the above information, the opening of the connection under yield and ultimate states can be calculated by Eqs. (6-8) accordingly:

$$O_{yc,cfsta} = 0.0027\times 45 + \frac{0.0027}{2}\times 355 + 0.5 = 1.1 \text{ mm}$$
$$O_{uc,cfsta} = 0.098\times 45 + \frac{0.0027+0.098}{2}\times 45 + \frac{0.0027}{2}\times 310 + 0.6 = 7.7 \text{ mm}$$
$$O_{yc,cfstb} = 0.0027\times 30 + \frac{0.0027}{2}\times 420 + 0.5 = 1.1 \text{ mm}$$
$$O_{uc,cfstb} = 0.098\times 30 + \frac{0.0027+0.098}{2}\times 53 + \frac{0.0027}{2}\times 367 + 0.5 = 6.6 \text{ mm}$$

Finally, the ductility of the designed connections can be calculated by Eq. (9):

$$DI_{ca} = \frac{Min(4.9, 7.7)}{Max(0.1, 1.1)} = 4.5, \text{and } DI_{cb} = \frac{Min(4.9, 6.6)}{Max(0.1, 1.1)} = 4.5$$

[00154] Since the ductility of the designed connections is greater than 4.0, they can be used in low-medium seismic regions.

[00155] *Step 5*: Given that the ductility of the designed connections is larger than 4.0, there should be a sufficient compatible zone (Fig. 25) for the outer RC connection and inner CFST connection. Hence, the tensile resistance of the designed connections can be calculated by Eqs. (11)–(13), with the diameter of the threaded regions d_{ts} assumed as 29 mm.

$$N_{yc,rc} = 20\times 550\times 3.14\times\frac{29^2}{4} = 7262.0 \text{ kN}$$
$$N_{yc,cfst} = 16\times 550\times 3.14\times\frac{32^2}{4} = 7073.7 \text{ kN}$$
$$N_c = 7262.0 + 7073.7 = 14335.7 \text{ kN}$$

[00156] According to previous research, the tensile resistance of the connected CECFST columns is:

$$N_{cecfst} = 20\times 550\times 3.14\times\frac{32^2}{4} + 6638.5 = 15480.7 \text{ kN}$$

[00157] Finally, the tensile resistance of the designed connection is compared with that of the connected columns.

[00158] $$\frac{N_{\mathrm{c}}}{N_{\mathrm{cecfst}}}=\frac{14335.7}{15480.7}=0.93$$

[00159] This comparison shows that the designed connections have sufficient tensile resistance, since the strength of the connections is over 50 % that of the connected columns.

[00160] Two innovative precast CECFST column-to-column connections featuring rapid construction and reduced demand for on-site labour were developed in this paper. Subsequently, four component tests and six connection tests were conducted to investigate tensile performance of the developed precast connections. Furthermore, based on the test results, a series of theoretical analyses were performed. According to the above experimental and analytical studies, the following conclusions can be drawn:

[00161] 1) The developed PCDCs may be analyzed as two independent parts: i) the outer RC connection and ii) the inner CFST connection. For the outer RC connection, column shoes were utilised to connect longitudinal reinforcement of outer RC components. For the inner CFST connection, the bond strength between lapped/anchored bars and inner concrete core was incorporated to connect the steel tubes.

[00162] 2) Without on-site welding, connection grouting, formwork, or propping system during construction, the developed PCDCs significantly reduced the demand for labour on-site and expedited the construction process, thus demonstrating a huge application potential.

[00163] 3) The developed PCDCs demonstrated a large tensile strength, which exceeded corresponding code requirements. Besides, all the test connections showed a ductile failure mode of reinforcing bar fracture and a high ductility index (*DI*) no less than 4.0. This satisfied the minimum ductility requirement for precast column-to-column connections in low-moderate seismic regions.

[00164] 4) Load transfer mechanisms of the PCDCs were proposed, which was followed by the development of component-based models to predict connection ductility and tensile strength. The proposed component-based models can be used to predict the connection ductility with good accuracy. Moreover, based on the proposed models, the ductility of the PCDCs can be improved by varying the arrangement of connection components.

[00165] 5) Direct superposition method can be used to predict tensile strength of the PCDCs conservatively, provided that the effective threaded region of lower reinforcement is sufficiently long to guarantee deformation compatibility between the outer RC and inner CFST connections.

[00166] 6) Based on the experimental and analytical studies, a general design method of the developed PCDCs under axial tension is presented with a flowchart, followed by a worked example.

[00167] 7) Despite excellent tensile performance, shear behaviour of the developed connections may differ from that of the corresponding entire columns. Therefore, besides the tensile performance, the behaviour of the developed connections under shear force will be studied in the future.

[00168] The following describes further experimental and numerical investigations on the precast CECFST column-to-column connections when subjected to axial and eccentric compression.

[00169] An experimental programme including five connection tests was conducted to investigate compression performance of the proposed precast connection. The test results show that compression resistance of the precast connection could achieve that of CECFST columns, indicating that it is feasible to connect CECFST columns by the proposed precast connection to enhance construction productivity. Furthermore, an extensive finite element (FE) modelling programme was carried out to generate more numerical data by conducting parametric studies.

The experimental and numerical results show that connection configuration plays a pivotal role in load-carrying capacity of the precast connection. Based on EN 1994-1-1, one simplified strength prediction model was proposed for the proposed precast CECFST column-to-column connection. Compared with experimental and numerical results, the predicted strength was conservative and consistent, indicating that the proposed approach can be used in the design of the precast connection.

[00170] Fig. 31 shows an exemplary configuration of the proposed connection, which includes two parts: 1) an outer RC connection and 2) an inner CFST connection. To form prototypes of the precast CECFST columns, the RC encasement in conjunction with the steel tubes was prefabricated in an offsite precast factory, and the inner concrete core was designed to be cast in-situ after connection assembly. This method has three main benefits: i) Transportation fees of precast segments from precast factories to construction sites are very sensitive to self-weight of each segment. The proposed method of casting the inner concrete core on-site could lower self-weight of the segments, and thus reduce the overall project cost. ii) Since crane fees are also charged based on self-weight of the precast segments, the deduction of segment self-weight would significantly decrease the crane fees. iii) Leaving the inner concrete cast on-site could make it possible to utilise the proposed novel CFST connection to connect the inner steel tubes of the precast CECFST columns without on-site welding. As for the RC connection, the upper and lower longitudinal reinforcement were connected through column shoes (Fig. 31 (b)) by tightening the nuts with a torque wrench, while the interfaces between the upper and the lower ends of outer concrete were carefully roughened to ensure that compression force carried by the upper concrete could be fully transferred to the lower counterpart via surface contact. In this series of experiments, threading operation reduced effective diameter of the lower reinforcement (Table 11), which resulted in a lower connection resistance compared to the column strength. In actual engineering practices, to enhance

connection efficiency, this issue could be addressed by using specially designed reinforcement where the effective diameter of threaded ends is no less than that of unthreaded regions.

[00171] Note that since the threaded longitudinal reinforcement might fracture under a large eccentricity, the column shoes were grouted with low-strength mortar to prevent fractured reinforcement from hurting people. The upper longitudinal reinforcement were directly welded onto the respective column shoes by fillet welding (Fig. 31 (b)), which might reduce ductility of the reinforcement under seismic loads. In terms of the CFST connection, the embedded steel tubes were connected via the bond between the lapped bars (Fig. 31 (c)) welded on respective steel tubes and the inner concrete. The upper and the lower lapped bars were provided with small steel cushions (Fig. 31 (b)) to increase connection tolerance for on-site assembly. After column assembly, the respective lapped bars were staggered evenly along the circumference of tube edges. Subsequently, the inner concrete core was cast on-site. The tensile force from the upper tube was first transferred to the upper lapped bars through the butt welding. The protruding part of upper lapped bars interacted with adjacent lower lapped bars via the inner concrete core. The lap length is sufficiently long for the tensile force from the upper steel tube to be effectively transferred to the lower steel tube through the bond between the lapped bars and the inner concrete core.

[00172] The proposed precast connection can significantly reduce overall construction costs by eliminating rent fees of formwork and propping systems, and by reducing on-site labour. It is expected that the construction time can be shortened as no formwork or propping system will be required.

[00173] Test Specimens

[00174] To validate the effectiveness of the developed connection under compression, an experimental programme carried out a series of compression tests on a total of five connection specimens, including four precast connections and one reference cast-in-place (CIP)

connection, as shown in Table 11. Herein, “PC” denotes precast connections, “U” denotes unconnected outer RC, “CIP” denotes cast-in-place connection, and the numbers following the alphabets denote load eccentricities, L is the specimen length, B is the width of specimens, t and D respectively denotes the thickness and diameter of the tube , ϕ_s and ϕ_{ts} are diameters of unthreaded and threaded regions of longitudinal reinforcement, respectively; ϕ_{cs} represents the diameter of lapped bars; e_0 denotes nominal design eccentricity*; e* is the measured eccentricity using a laser beam. In the present experimental programme, since the testing objective is the proposed precast connection, in the design of test specimens, the connection section was designed to be weaker than other column sections. In this way, the specimens could fail at the connection section rather than the column section. As such, the measured peak load and failure mode of the test specimens would directly reflect structural behaviour, i.e., load-carrying capacity and failure mechanism, of the precast connection. However, in the design of real structures, to ensure structural safety, “Strong connection-weak column” concept should be implemented.

[00175] To provide insights into compression performance of the proposed precast connection, specimens PC-0, PC-44 and PC-110 were tested under various compression load eccentricities, designed from 0 to 114 mm. According to various embodimetns of the proposed connection configuration (Fig. 31), the outer RC connection plays an important role in compression resistance of the connection. In this regard, specimen PC-U-110, where the outer reinforcements were not connected, was tested to quantify the contribution of the outer RC connection to overall cross-sectional resistance of the developed precast connection. In addition to these four examples of the proposed precast connections, a conventional CIP CECFST column-to-column connection was included as the reference specimen CIP-110. Owing to the limitation of the specimen size, butt-welding was incorporated for the continuity of both the

longitudinal reinforcement and the steel tubes. The CIP specimen was intended as a benchmark for evaluating structural behaviour of the proposed precast connection.

[00176] Fig. 32 presents detailed descriptions of the test specimens. For the precast connection specimens, lap length of the upper and lower lapped bars (of ϕ_s = 13 mm) was uniformly set as 200 mm, larger than the required lap length of $12\phi_s$, to prevent brittle pullout failure. Dimensions and material properties of the CIP specimens were the same as those of the four precast connections.

Table 11 **Details of test specimens**

No	Specimen Label	L (mm)	B (mm)	$D \times t$ (mm)	ϕ_s (mm)	ϕ_{ts} (mm)	ϕ_{cs} (mm)	e_0 (mm)	e (mm)	N_{Exp} (kN)	N_{FE} (kN)	N_{FE}/N_{EXP}
1	PC-0	800	220	114 x 4	13	10.2	13	0	0	2388.7	2501.2	1.05
2	PC-44	800	220	114 x 4	13	10.2	13	44	48	1279.6	1236.8	0.97
3	PC-110	800	220	114 x 4	13	10.2	13	110	112	570.4	542.3	0.95
4	PC-U-110	800	220	114 x 4	13	10.2	13	110	113	470.1	448.3	0.95
5	CIP-110	800	220	114 x 4	13	---	---	110	107	487.2	---	---
	Mean											0.98
	COV											0.04

[00177] Materials Properties

[00178] Inner high-strength concrete (60 MPa) and outer normal concrete (32 MPa) were used for the CECFST columns. Compressive strength of the two types of concrete was obtained by standard compression tests on respective concrete cylinders with dimensions of 100 mm x 150 mm (Table 12). Note that the cylinder tests were conducted on the same day as the connection tests. Tensile coupon tests conforming to ASTM were conducted in a servo-hydraulic machine with a loading rate of 0.05 mm/min to determine material properties of longitudinal reinforcement, lateral reinforcement and steel tubes. Yield and ultimate strength, yield, ultimate and fracture strain, as well as elastic modulus of respective materials are summarized in Table 12.

Table 12. Material Properties

Concrete			
	Inner		Outer
Compressive strength (MPa)	60		32
Steel			
	Longitudinal reinforcement	Lateral reinforcement	Steel tube
Yield strength (MPa)	552	367	414
Elastic modulus (GPa)	208	207	279
Yield strain (%)	0.27	0.18	0.39
Ultimate strength (MPa)	634	446	556
Ultimate strain (%)	9.8	7.2	12.6
Fracture strain (%)	18	14	20

[00179] Connection Tests

[00180] The specimens with a uniaxial eccentricity were tested under a pin-ended boundary condition, while the specimen under axial compression was tested with a fixed boundary condition. Fig. 33 presents the details of the test setup adopted in this study. Note that the effective length of the test specimens L_e under pin-ended boundary condition was 950 mm, which is the distance between two rotation centres (knife edges). To prevent premature end failure, one pair of 5 mm thick demountable steel clamps were fabricated to strengthen top and bottom ends of the test specimens (Fig. 33). All the five connection tests were performed under a 500-ton hydraulic compression machine at a loading rate of 0.3 mm/min under displacement control. To reflect cross-sectional behaviour of the proposed connection, slenderness ratio $\overline{\lambda}$ of the test specimens was calculated by the following equation:

$$\overline{\lambda} = \sqrt{N_a / N_{cr}} \quad \text{(Eq. 14)}$$

where N_a and N_{cr} are the nominal yield strength and the elastic critical load, as calculated by Eq. (15) and Eq. (26), respectively.

$$N_a = 0.85 f_c^{ic} A_c^{ic} + f_c^{oc} A_c^{oc} + f_y A_a + f_{ys} A_s \quad \text{(Eq. 15)}$$

$$N_{cr} = \pi^2 \left(EI\right)_{eff} / L_e^2 \quad \text{(Eq.16)}$$

where f_c^{ic} and f_c^{oc} respectively denote compressive strengths of the inner and outer concrete; f_y and f_{ys} respectively denote the yield strength of steel tubes and longitudinal reinforcement; A_c^{ic}

, A_c^{oc} , A_a and A_s are the respective areas of the corresponding materials; $(EI)_{eff}$ is effective flexural stiffness.

$$(EI)_{eff} = E_a I_a + E_s I_s + 0.6E_{cm}^{ic} I_c^{ic} + 0.6E_{cm}^{oc} I_c^{oc} \quad \text{(Eq. 17)}$$

where E_a and E_s respectively denote the elastic modulus of steel tubes and longitudinal reinforcement; E_{cm}^{ic} and E_{cm}^{oc} respectively denote the secant modulus of the inner and outer concrete; I_a, I_s, I_c, and I_c^{oc} are corresponding moments of inertia.

[00181] From the reported material and geometric properties, the relative slenderness ratio of all the five test specimens is equal to 0.156, which indicates that the specimens were classified as short columns as per EN 1994-1-1. Therefore, it is feasible to investigate cross-sectional resistance of the connections through the designed test specimens and test set-up. As shown in Fig. 32, a series of strain gauges were affixed on the lapped bars and longitudinal reinforcement to record strain development at critical positions with increased applied loads. Besides, as shown in Fig. 33, multiple linear variable differential transformers (LVDTs) were installed to measure global deformation of the specimens: L_1 and L_2 for axial shortening; L_3 for horizontal displacement; L_4 for connection opening.

[00182] EXPERIMENTAL RESULTS AND DISCUSSION

[00183] Load versus displacement behaviour

[00184] This section presents load–axial shortening relationships of the tested connections under uniaxial compression. As shown in Fig. 34 (a), initial stiffness and ultimate loads of the precast CECFST connections decreased with an increase in eccentricity. On the other hand, the failure of connections with a larger eccentricity was more ductile than those with a smaller eccentricity. This is because, with an increase in load eccentricity, the precast connection turned from brittle compression-controlled to ductile tension-controlled failure mode. Note that compression-controlled failure arising from concrete crushing is generally more brittle than tension-controlled failure mode, where the yielding of longitudinal reinforcement happens

before the concrete starts crushing. Recorded strain data was utilized to reflect failure mechanisms of the tested connections as shown in Fig. 34 (a), where red dots represent the yielding of tension reinforcement and pink dots represent the yielding of compression reinforcement. For the axially loaded specimen PC-0, longitudinal reinforcement yielded before reaching the ultimate load, indicating that the connection failed by pure compression. For specimen PC-44, longitudinal reinforcement at the compression side yielded before the ultimate load, whereas reinforcement at the tension side yielded during the post-peak stage. This shows that the connection experienced a typical compression failure. On the other hand, for specimen PC-110, longitudinal reinforcement at the tension side yielded in advance, whereas their counterparts in compression yielded much later, indicating that the connection experienced a tension failure. In actual engineering practice, the connections designed in this experimental program would be used to connect CEFST columns with 10 mm diameter longitudinal reinforcement as the effective diameter of threaded reinforcement was only 10.2 mm (Table 2). In this regard, to evaluate structural resistance of the proposed precast connection, cross-sectional resistance of the corresponding CECFST columns was compared to the test results as shown in Fig. 34 (a), where peak loads of all the three precast connection specimens reached respective cross-sectional resistance of corresponding CECFST columns. This indicated that resistance of the proposed precast connection could achieve that of the CECFST column, showing a huge potential to be utilised as an alternative to conventional CIP connections. Note that compression resistance of the proposed connection may vary depending on the arrangement of connection components, i.e., the number and diameter of the lapped bars (or lapped members) and the number of column shoes.

[00185] Fig. 34 (b) shows the load–axial shortening relationships of specimens PC-110, PC-U-110 and CIP-110. Given that effective diameter of the threaded reinforcement was reduced (Table 11), the precast connection PC-110 was expected to be weaker than the corresponding

reference specimen CIP-110 where longitudinal reinforcement and steel tubes were directly welded on site. However, from Fig. 34 (b), precast connection PC-110 showed a larger ultimate load than that of specimen CIP-110. After testing, the failure pattern of specimen CIP-110 was checked and it was found that premature welding failure (Fig. 35) occurred at the longitudinal reinforcement because of craftsmanship issues. In that case, specimen CIP-110 did not reach its real capacity, thus showing a lower peak load than specimen PC-110. In terms of specimen PC-U-110, where longitudinal reinforcement of the outer RC components were unconnected, its ultimate strength was significantly lower than that of the fully connected specimen PC-110. This implied that the outer RC connection contributed significantly to compression resistance of the overall precast connection. Hence, to ensure high compression resistance, longitudinal reinforcement of the outer RC encasements is preferably fully connected by an appropriate column shoe system. Besides, since all these three specimens in Fig. 34 (b) were loaded under a similar eccentricity, initial stiffness of the specimens was quite close. However, as the load increased, specimens PC-U-110 and CIP-110 showed a more brittle failure than specimen PC-110, which indicated that the integrity of these two connection specimens was lower than that of specimen PC-110. For specimen PC-U-110, since the outer RC part of the connection was not connected, its integrity should be lower than specimen PC-110. As for specimen CIP-110, its connection integrity was supposed to be higher than that of specimen PC-110. However, during the testing, welding of the longitudinal reinforcement fractured unexpectedly, resulting in a relatively brittle failure.

[00186] Failure mode and mechanism

[00187] This section presents the failure mode of the tested connections. For axially loaded specimen PC-0, as shown in Fig. 35 (a), the damage was mainly concentrated on the connection region. The outer concrete crushed significantly, and the surrounding stirrups failed due to a lack of bond strength. Longitudinal reinforcement buckled since the failed stirrups could not

provide adequate horizontal restraint for vertical reinforcement in compression. Failure modes of the precast connections loaded eccentrically (PC-44, PC-110, PC-U-110) were quite similar. Outer concrete crushed at the compression side of the connection region, while a large opening was observed at the tension side of the connection interface and no concrete cracks were found. The observed connection opening reflected that tensile deformation of the assembled CECFST column concentrated at the connection interface due to discontinuity of outer concrete and reduced cross-sectional area of longitudinal reinforcement (HT13). In terms of reference specimen CIP-110, as shown in Fig. 35 (c), a wide concrete crack with some minor tensile cracks was found at the tension side of the connection region, and a large area of crushed concrete was observed at the compression side. Furthermore, it was found that the welding of longitudinal reinforcement at the tension side fractured due to craftmanship issues, as shown in a close-up view. This corresponded to a sudden drop in the load-axial shortening curve of specimen CIP-110 at ultimate load, as shown in Fig. 34 (b).

[00188] From Fig. 35, failure of the precast column-to-column connection is concentrated at the connection interface. This is because both the steel tube and the outer concrete are discontinuous at the connection interface, making it the most critical cross-section along the jointed column. Therefore, strength of the connection could be represented by the resistance of the connection interface. Fig. 36 shows a schematic failure mechanism of the precast column-to-column connection under eccentric compression. The failure occurred at the connection interface, with concrete at the compression side being crushed and the connection opened up at the tension side.

[00189] Numerical modelling

[00190] Challenges and strategies

[00191] In the present study, numerical modelling is used to construct finite element models for the proposed precast connection with various connection configurations and loading

eccentricities, which were not included in the experimental programme. The simulation of precast connections is normally beset with severe convergence problems due to complex connection components. This poses a huge challenge to structural engineers studying precast connections. Specifically, for simulation of the proposed CECFST connection (Fig. 31), where lapped bars, threaded reinforcement, nuts and column shoes are utilised to achieve vertical continuity of columns, multiple aspects, including modelling of respective connection components, interactions among these components and contacts at connection interface are relevant considerations. In this study, a comprehensive finite element modelling was performed through Abaqus 6.14/Standard. Fig. 37 shows a schematic view of the FE model for the proposed CECFST column-to-column connection.

[00192] Strategies and assumptions adopted in the development of the FE model are presented, followed by verification of the FE models with the test data. Based on verified FE models, a series of parametric studies were carried out over various connection detailing and a large range of load eccentricities, respectively. The generated numerical data were used to examine the effectiveness of an analytical strength prediction approach for the precast connection subjected to combined compression and bending.

[00193] Element type and mesh size

Eight-node brick element with reduced integration C3D8R was adopted to simulate the inner concrete, outer concrete, steel tube, column shoe, subplate and clamp, while the 2-D beam element B31 was used to simulate the lapped bars, stirrups, and longitudinal reinforcement. Note that since low-strength mortar was used to grout the column shoes, it was not considered in the FE modelling. To balance computational efforts with accuracy of predictions, a mesh sensitivity study was conducted. It was found that for the simulation of the proposed precast connection, a maximum size of $B/10$ was suitable for 3D solid elements, e.g., the outer concrete,

steel tube, inner concrete, column shoe and subplate, while a maximum size of $L/40$ was appropriate for 2-D beam elements.

[00194] Material modelling

[00195] In the FE modelling, the steel tube, column shoe, subplate and steel clamp were assumed as an elastic-perfectly plastic material (Fig. 38 (a)). A modified elastic-perfectly plastic model considering the fracture of reinforcing bars as shown in Fig. 38 (b), was adopted for the lapped bars, longitudinal reinforcement and stirrups, to trace post-peak behaviour of the connections. InFig. 38, ε_y and ε_f are the yield and fracture strain of the lapped bars, respectively, and f_y is the yield strength of the lapped bars. Detailed information about these variables is provided in Table 12.

[00196] Concrete damage plasticity (CDP) model was used to describe material properties of the outer and inner concrete elements. Elastic modulus of the concrete was taken as $E_{cm}= 22 (f_c/10)^{0.3}$ based on EN-1992-1-1, and the Poisson's ratio was set as 0.2. Considering that the steel tube and transverse stirrups could provide inner and outer concrete with confinement effect, the concrete across the composite section was categorised into three parts: 1) inner confined concrete; 2) outer confined concrete; 3) outermost unconfined concrete cover. The dimension of the outer confined concrete was defined as B-c-d_{sw}, where B is the width of the specimen, c is the thickness of concrete cover, and d_{sw} represents the diameter of stirrups, as shown in Fig. 37 (d). Compression behaviour of the outermost unconfined concrete, the outer confined concrete, and the inner confined concrete were simulated by various constitutive models, and Tension behaviour of all the concrete was modelled with the tensile strength taken as $0.28 f_c$, using conventional models known to a person skilled in the art.

[00197] Interaction and boundary condition

[00198] General surface-to-surface contact was adopted to define mechanical interaction between the inner and outer concrete and the steel tube, as well as the interaction between outer

concrete and external steel clamps. “Hard contact” allowing separation under tension and preventing penetration under compression was defined in the normal direction, while “Mohr-Coulomb” model with a friction coefficient of 0.25 was defined in the tangential direction. This technique was also applied to define the connection interface consisting of two parts: 1) the horizontal interface between the upper and the lower steel tubes; 2) the interface between the subplate of the upper column and the concrete surface of the lower column. Note that for the interface between the upper and the lower steel tubes, a friction coefficient of 0.2 was adopted according to EN 1993-1-8. A command known as “Embedded region” in ABAQUS was employed to represent the interactions between steel reinforcement cages and outer concrete. “Tie” constraint was used to define the interaction among the outer concrete, the column shoes and the subplate to achieve better convergence. In this manner, the upper longitudinal reinforcement H13 were naturally tied with the column shoes, as shown in Fig. 39. On the other hand, the upper end of HT 13 was tied to the corresponding column shoe via multiple point coupling (MPC) techniques to simulate corresponding bolting system. This strategy avoids modelling of multiple contact pairs between the nuts, washers, threaded reinforcing bars and column shoes, significantly improving the convergence of the FE model.

[00199] As for modelling of the inner CFST connection, as shown in Fig. 40, the midpoints of the upper and lower lapped bars were directly tied to respective steel tubes through “Kinematic coupling” technique to simulate butt welding between the lapped bars and steel tubes. Besides, as shown in Fig. 41, bond-slip behaviour between the lapped bars and the concrete core was modelled through a series of nonlinear springs known as “translators”. Each spring element was defined by two nodes and a generalized force-slip relationship in the axial direction, while relative shear displacements between the connected nodes in the normal direction were neglected. The force-slip relationship of each spring element was established based on the bond-slip relationship provided in the FIB code, as shown in (Eq. , where F_s is the

equivalent axial force (in the direction of lapped bars) of each spring corresponding to a certain slip S; τ represents the bond stress corresponding to the slip S; d_c denotes the diameter of lapped bars; l_s denotes the distance between adjacent spring elements (Fig. 41 (b)).

$$F_s = \pi d_c l_s \tau \quad \text{(Eq. 18)}$$

Fig. 42 presents the codified bond stress-slip relationship for the lapped bars, where $\tau_{max} = 2.5\sqrt{f_{oc}}$; $\tau_f = 0.4\tau_{max}$; S_1, S_2 and S_3 are the slip of lapped bars, and they were recommended as 1, 2 and 5 mm, respectively.

[00200] As shown in Fig. 43, to apply boundary conditions, for the FE model, the top and bottom surfaces of the assembled column were coupled with respective loading points RP 1 and RP 2, with the distance between them equal to the effective length of specimen L_e. For the eccentric compression tests on specimens PC-44, PC-110, PC-U-110 and CIP-110, the lower loading point RP 2 was fully restrained but rotation about the y-axis (r_y) was released, while for the upper loading point (RP 1), all degrees of freedom were restrained except for vertical movement (U_z) and rotation about the y-axis (r_y). For the pure axial compression test PC-0, rotations of RP 1 and RP 2 about the y-axis (r_y) were restrained as well.

[00201] Validation of FE models

[00202] To verify the FE model, ultimate failure loads of the five tested connections were compared with the corresponding numerical results, as shown in Table 11. Clearly, the FE models could well predict load-carrying capacity of the CECFST connections, with the mean ratio of N_{FE}/N_{Exp} and the corresponding coefficient of variation (COV) equal to 0.98 and 0.04, respectively.

[00203] Besides, as shown in Fig. 44, load-lateral deflection relationships of two typical connection specimens (PC-44 and PC-110) from FEM were compared with corresponding test results to examine the effectiveness of the FE models. The comparison indicated that the developed FE models could well predict the initial stiffness and post-peak behaviour of the

precast connections under uniaxial compression. Failure modes obtained from the FE modelling were also compared with corresponding test results, as shown in Fig. 45, which indicated that the FE models could also predict failure mode of the connections. To sum up, the FE models developed in this study are reliable and thus can be subsequently used for parametric studies.

[00204] Parametric study

[00205] Based on verified FE models, a series of parametric studies were performed to identify the effect of connection components on compression performance of the proposed connection. Parameters adopted in this parametric study and corresponding numerical results are summarized in Table 12, where f_{ycs}, f_{ys}, d_{cs}, and d_s are the yield strength and diameter of lapped bars (subscript "cs") and threaded longitudinal reinforcement (subscript "s"), respectively.

[00206] From numerical results, for connections with a small eccentricity (48 mm), an increase of f_{ycs}, f_{ys}, d_{cs}, or d_s did not contribute to a significant increase for ultimate loads. On the other hand, for connections with a large eccentricity (e_0/B=0.5), the ultimate loads increased significantly (>20%) with an increase of f_{ycs}, f_{ys}, d_{cs}, or d_s. This is because connections with a small eccentricity failed by concrete crushing rather than reinforcement yielding. Improvement of reinforcing bars would not lead to a considerable enhancement of ultimate capacity. However, since connections with a large eccentricity failed by yielding of reinforcing bars, a small increase of f_{ycs}, f_{ys}, d_{cs}, or d_s could result in a relatively large increase of ultimate load. In brief, load-carrying capacity of the precast CECFST connection proposed by the authors could be improved by increasing the diameter or yield strength of longitudinal reinforcement and lapped bars. This supported the hypothesis made in that using larger diameter threaded bars for the precast connection could improve the connection compression performance.

Table 12. Parameters adopted for parametric studies

No.	e (mm)	f_{ycs} (MPa)	f_{ys} (MPa)	d_{cs} (mm)	d_{ts} (mm)	N_{FE} (kN)	N_{FEC} (kN)	N_{FE}/N_{FEC}
1	48	700	552	13	10.2	1243.8	1236.8	1.01
2		552	552	16	10.2	1241.2		1.00

3		700	552	16	10.2	1241.2		1.00
4		552	552	13	13	1247.7		1.01
5		552	700	13	10.2	1286.6		1.04
6		552	700	13	13	1304.1		1.05
7		700	700	16	13	1305.2		1.06
8	112	700	552	13	10.2	577.1	542.5	1.06
9		552	552	16	10.2	585.3		1.08
10		700	552	16	10.2	589.4		1.09
11		552	552	13	13	615.3		1.13
12		552	700	13	10.2	617.9		1.14
13		552	700	13	13	660.5		1.21
14		700	700	16	13	663.7		1.22

Note: N_{FEC} is the numerical peak load of tested precast connections under respective eccentricities.

[00207] Simplified strength prediction model

[00208] The laboratory tests in Table 11 and 3D FE modelling in Table 13 provide valuable insights into compression behaviour of the precast connection. In this section, a simplified strength prediction model is proposed for the CECFST connection by providing the structural engineer with a cross-sectional *N*-*M* interaction curve. As shown in Fig. 46, a complete *N*-*M* interaction curve of the precast connection consists of four key points, namely, Point A (Pure compression failure), Point B (Pure bending failure), Point C (Balanced failure) and Point D (Compression failure). The *N*-*M* curve can be constructed with the following assumptions: (i) all the materials conform to the plane section assumption; (ii) the upper and lower steel tubes do not come into contact under compression; (iii) tension and compression forces experienced by the steel tube are transferred by the lapped bars; (iv) tensile strength of concrete is neglected.

[00209] From the failure modes (Fig. 35, Fig. 36), the most critical plane of the precast connection should be the connection interface, as shown in Fig. 47, where longitudinal reinforcing bars of 13 mm are threaded with 10.2 mm, and the lapped bars (H13) are staggered evenly along the circumference of the connected steel tubes (shown in dash lines in Fig. 47). To construct the cross-sectional *N*-*M* interaction curve, plastic stress distribution method from EN 1994-1-1 is adopted, where all steel reinforcement is assumed to be fully yielded, while

concrete compression stress is idealised as a rectangular stress block by providing respective concrete strengths f_{ic} and f_{oc} with a reduction factor 0.85, as shown in Fig. 47. In this way, the four key points of the curve (Fig. 46) could be obtained by determining the position of neutral axis for each point, respectively.

[00210] In the plastic stress distribution (PSD) method, distributed lapped bars would significantly complicate the derivation of the *N-M* interaction curve, particularly at point C. Hence, in this study, they are transformed into an "equivalent steel tube", as shown in Fig. 48 to simplify the strength prediction approach. To ensure that the equivalent steel tube can simulate the lapped bars, equilibrium conditions of forces and moments should be satisfied, viz, i) equivalent axial resistance and (ii) equivalent bending resistance, should be satisfied, respectively, as shown in (Eq. 19) and (Eq. 20).

$$n_{ls} A_{ls} f_{yls} = 2\pi r_{et} t_{et} f_y \quad \text{(Eq. 19)}$$

$$\sum A_{ls} e_i f_{ys} = 4 r_{et}^2 t_{et} f_y \quad \text{(Eq. 20)}$$

[00211] where n_{ls} and A_{ls} are the number and cross-sectional area of the lapped bars; r_{et} and t_{et} are the mean radius and thickness of the equivalent tube; e_i denotes the distance between the centre of each individual lapped bar and the centerline (C.A) of the cross-section.

[00212] Transferring the lapped bars into an equivalent steel tube, the four key points can be calculated by finding the position of neutral axis (N.A), respectively.

[00213] 1) For Point A:

[00214] Under pure axial compression in Fig. 19, it is assumed that all elements of the cross-section will yield. Thus, axial resistance can be calculated by (Eq. 21).

$$N_A = 0.85 A_{oc} f_{oc} + A_{ic} f_{ic} + f_y A_{et} + f_{ys} A_s \quad \text{(Eq. 21)}$$

[00215] where A_{oc}, A_{ic}, A_{et}, and A_s are respective areas of outer concrete, inner concrete, equivalent steel tube and longitudinal reinforcement; f_{oc}, f_{ic}, f_y, and f_{ys} are the corresponding yield strengths.

[00216] 2) For Point D:

[00217] To determine pure bending strength of the precast connection M_D, as shown in Fig. 46, two general cases are considered as per the location of neutral axis: a) N.A is within the steel tube (Fig. 49) ; b) N.A is outside the steel tube (Fig. 50). Note that since the thickness of the steel tube is relatively small compared with the overall dimensions of the cross-section, the neutral axis is assumed either within or outside the steel tube.

[00218] a) Assuming that the neutral axis is located within the steel tube, the position of neutral axis can be determined by (Eq. 22).

$$0.85A_{\mathrm{oc},c}f_{oc}+0.85A_{ic,c}f_{ic}+A_{s,c}f_{ys}+A_{et,c}f_{y}=A_{s,t}f_{ys}+A_{et,t}f_{y} \quad \text{(Eq. 22)}$$

$$A_{\mathrm{oc},c}=B\times h_0-r_m^2(\pi/2-\varphi-\sin 2\varphi) \quad \text{(Eq. 23)}$$

$$A_{\mathrm{ic,c}}=r_m^2(\pi/2-\varphi-\sin 2\varphi) \quad \text{(Eq. 24)}$$

$$A_{\mathrm{et,c}}=r_m t_{\mathrm{et}}(\pi-2\varphi) \quad \text{(Eq. 25)}$$

$$A_{\mathrm{et,t}}=r_{\mathrm{m}} t_{\mathrm{et}}(\pi+2\varphi) \quad \text{(Eq. 26)}$$

[00219] where $A_{\mathrm{oc,c}}$ and $A_{\mathrm{ic,c}}$ are areas of the compressed outer concrete and inner concrete, respectively; $A_{\mathrm{s,c}}$ and $A_{\mathrm{et,c}}$ are areas of the longitudinal compression reinforcement and steel tube, respectively; $A_{\mathrm{s,t}}$ and $A_{\mathrm{et,t}}$ are areas of the corresponding longitudinal tension reinforcement and steel tube; h_0 is the height of neutral axis; $\sin\varphi=(0.5B-h_0)/r_{\mathrm{m}}$.

[00220] With the neutral axis determined, pure bending resistance of the connection can be calculated by (Eq. 27)–(Eq. 30).

$$M_{\mathrm{u}}=A_{\mathrm{s,t}}f_{\mathrm{ys}}(0.5B-a)+A_{\mathrm{s,c}}f_{\mathrm{ys}}(0.5B-a)+4t_{\mathrm{et}}r_{\mathrm{m}}^2\cos\varphi f_{\mathrm{y}}+\frac{2}{3}r_m^3\cos^3\varphi\times 0.85f_{\mathrm{ic}}$$
$$+0.85f_{\mathrm{oc}}[(0.5B^2h_0-0.5Bh_0^2)-\frac{2}{3}r_m^3\cos^3\varphi-2t_{\mathrm{et}}r_{\mathrm{m}}^2\cos\varphi] \quad \text{(Eq. 27)}$$

[00221] b) Assuming that the neutral axis is located outside the steel tube, the location of neutral axis (N.A) can be determined from (Eq. 28)–(Eq. 30).

$$A_{\mathrm{oc,c}} f_{oc} + A_{\mathrm{s,c}} f_{\mathrm{ys}} = A_{\mathrm{s,t}} f_{\mathrm{ys}} + A_{\mathrm{et},t} f_{\mathrm{y}} \quad \text{(Eq. 28)}$$

$$A_{\mathrm{oc,c}} = Bh_0 \quad \text{(Eq. 29)}$$

$$A_{\mathrm{et,t}} = 2\pi r_{\mathrm{m}} t_{\mathrm{et}} \quad \text{(Eq. 30)}$$

[00222] With the neutral axis determined, pure bending resistance M_{B} can be calculated by (Eq. 31).

$$M_{\mathrm{u}} = f_{\mathrm{s}}(A_{\mathrm{s,t}} + A_{\mathrm{s,c}})(B/2 - a - h_0/2) + Bh_0(B - h_0)/2 \quad \text{(Eq. 31)}$$

[00223] 3) For Point C:

[00224] At the balanced failure point, where the bending moment reaches upon its maximum value, the neutral axis (N.A) should coincide with the centroid of the cross-section, as shown in Fig. 51. Therefore, M_{C} can be calculated by (Eq. 22)–(Eq. 26) with φ=0. Furthermore, the corresponding axial strength N_{B} can be calculated by (Eq. 32).

$$N_B = 0.85 f_{\mathrm{oc}}(Bh_0 - \pi r_{\mathrm{m}}^2)/2 + 0.85 f_{\mathrm{ic}} \pi r_{\mathrm{m}}^2 \quad \text{(Eq. 32)}$$

[00225] For Point B:

[00226] As shown in Fig. 46, moment of Point B (M_{B}) is the same as that of Point D (M_{D}), so that the neutral axis (N.A) of Point B should be symmetrical with that of Point D. Therefore, M_{B}=M_{D} and N_{D}=2N_{C}.

[00227] After obtaining these four key points, the *N*-*M* curve could be plotted as piece-wise linear, as shown in Fig. 52, which compares the predicted *N*-*M* interaction curve of the CECFST connections and the experimental and numerical data. It shows that the proposed strength prediction model can be used to predict the cross-sectional resistance of the precast connection conservatively.

[00228] The foregoing described na experimental, numerical and analytical study conducted to investigate structural behaviour of embodiments of a precast composite column-to-column connection proposed herein. The experimental programme consists of five connection

specimens, which were tested to provide valuable insights into the ultimate load-carrying capacity and failure mode of the proposed connection. The proposed precast connection exhibited excellent structural resistance, thus demonstrating significant potential to be used in actual engineering practice as an alternative to conventional cast-in-place connections.

[00229] The experimental programme was supplemented by an extensive numerical modelling programme, where FE models were developed and verified against the experimental results to generate further numerical data by parametric studies. It was found that the load-carrying capacity of the precast CECFST connection could be improved by increasing the yield strength and/or the diameter of threaded longitudinal reinforcement or the lapped bars. Besides, the outer RC connection plays a more critical role in the load-carrying capacity of the proposed CECFST connection, in comparison with the inner CFST connection.

[00230] Based on the plastic stress distribution (PSD) method, a simplified analytical strength prediction model was proposed for the developed precast CECFST connection through the typical *N*-*M* interaction curve of the precast connection under uniaxial compression. In this model, the inner lapped bars were replaced by an equivalent circular steel tube to simplify the analytical derivation. The validated FE models were then employed to conduct a parametric study on the precast CECFST connection subjected to a wide range of load eccentricities, with eccentricity ratio *e*/*B* ranging from 0 to 0.7. It was found that the predictions obtained from the strength model are in good agreement with the numerical and experimental results and are on the conservative side. This showed that the proposed model could be used to predict ultimate load-carrying capacity of the precast CECFST connection developed by the authors.

[00231] A column 100 (also interchangeably referred to herein as a precast column segment, PCS) includes: a steel tube 200, a first plurality of a lapped member 300/301, a second plurality of the lapped member 300/302, and a concrete body 400. The steel tube 200 longitudinally extends (along a longitudinal orientation or a longitudinal axis 900) between two open ends 210

of the steel tube 200. The first plurality of the lapped member 300/301 are partially disposed in an interior 220 of the steel tube 200. The first plurality of the lapped member 300/301 are coupled to an interior surface 222 of the steel tube 200. The first plurality of the lapped member 300/301 extend beyond one of the two open ends 210 (of the steel tube). The second plurality of the lapped member 300/302 are partially disposed in the interior 220 of the steel tube 200. The second plurality of the lapped member 300/302 are coupled to the interior surface 222 of the steel tube 200. The second plurality of the lapped member 300/302 extend beyond another of the two open ends 210 (of the steel tube). The concrete body 400 is cast about an exterior surface 224 of the steel tube 200 to form a precast concrete-encased steel tube. The steel tube 200 is internally unfilled with concrete to provide a contiguous space in fluidic communication with the two open ends 210 (of the steel tube).

[00232] The first plurality of the lapped member 300/301 include a lapped bar 310 extending longitudinally. The second plurality of the lapped member 300/302 include a lapped bar 310 extending longitudinally. The lapped bar 310 (of either the first plurality or the second plurality) is spaced apart from the interior surface 222 of the steel tube 200.

[00233] Each of the first plurality and the second plurality of the lapped member 300/301/302 includes a cushion 330 extending transversely (relative to the longitudinal orientation or the longitudinal axis) from the interior surface 222. The cushion 330 is coupled on opposing sides of the cushion to the lapped bar 310 and to the interior surface 222. The cushion 330 is coupled on one side to one of the lapped bars 310 and on an opposing side of the cushion to the interior surface 222.

[00234] The column 100 further includes a third plurality of a rebar 500 distributed external of the steel tube 200. Each of the third plurality of the rebar 500 is partially embedded in the concrete body 400 and partially extends from a top of the concrete body 400.

[00235] The column 100 further includes a fourth plurality of a column shoe 600 distributed external of the steel tube 200. The column shoe 600 is partially embedded in the concrete body 400 proximal to a bottom of the concrete body. The column shoe 600 includes: a lower cap 610 and an upper cap 620. The lower cap 610 and the upper cap 620 are longitudinally spaced apart from one another.

[00236] Disclosed herein is a method 910. The method 910 includes aligning a top of first column and a bottom of a second column to define an interface 150 therebetween, in which the steel tube 200 of the first column 101 and the steel tube 200 of the second column 102 are aligned to define a contiguous space in fluidic communication across the interface 150. In the method 910, each of the first column 101 and the second column 102 includes: a steel tube 200, a first plurality of a lapped member 300/301 partially disposed in an interior 220 of the steel tube 200, the steel tube 200 longitudinally extending between two ends 210 of the steel tube; a second plurality of the lapped member 300/302 partially disposed in the interior 220 of the steel tube 200, and a concrete body 400 cast about an exterior surface 224 of the steel tube to form a precast concrete-encased steel tube. In the method 910, the first plurality of the lapped member 300/301 are coupled to an interior surface 222 of the steel tube 200 and extend beyond one of the two ends 210 of the steel tube; and the second plurality of the lapped member 300/302 are coupled to the interior surface 222 of the steel tube 200 and extend beyond another of the two ends 210.

[00237] The method 910 includes casting in situ a concrete core 700, the concrete core 700 being a contiguous column of concrete extending from the interior 220 of steel tube 200 of the first column 101 across the interface 150 to the interior 220 of the steel tube 200 of the second column 102.

[00238] The method 910 includes: casting a concrete core 700 in the interior of at least the steel tube 200 of the first column 101 to form a concrete-encased concrete-filled steel tubular column.

[00239] The method 910 may include: interleaving the first plurality of the lapped member 300/301 of the first column 101 and the second plurality of the lapped member 300/302 of the second column 102; and after the interleaving, pouring fresh concrete into the contiguous space to in situ cast a concrete core 700, the concrete core 700 being a contiguously formed column extending from the interior 220 of the steel tube 200 of the first column 101 to the interior 220 of the steel tube 200 of the second column 102.

[00240] In the method 910, each of the first column 100/101 and the second column 100/102 may be a precast column, and the concrete core 700 is cast in situ.

[00241] The method 910 may include coupling respective complementary ones of a rebar 500 and a column shoe 600, in which the rebar 500 is partially embedded in the concrete body 400 of the first column 100/101, and in which the column shoe 600 is partially embedded in the concrete body 400 of the second column 100/102.

[00242] Disclosed herein is a column-to-column connection system 180. The column-to-column connection system 180 includes: a first plurality of a lapped member 300/301, a second plurality of the lapped member 300/302, and a concrete core 700. The first plurality of a lapped member 300/301 are fixedly coupled to an interior surface 222 of a first steel tube 200/201. The first steel tube200/201 is disposed in a first concrete body 400/401 cast about an exterior surface 224 of the first steel tube 200/201. The second plurality of the lapped member 300/302 are fixedly coupled to an interior surface 222 of a second steel tube 200/202. The second steel tube 200/202 is disposed in a second concrete body 400/402 cast about an exterior surface 224 of the second steel tube200/202. The concrete core 700 is a contiguous column of concrete cast in an interior of the first steel tube 200/201 and extending to at least partially fill an interior of the

second steel tube 200/202. The first plurality of the lapped member 300/301 and the second plurality of the lapped member 300/302 are embedded in the concrete core 700. Each of the lapped member 300 is disposed partially in an interior of the first steel tube 200/201 and partially in an interior of second steel tube 200/202.

[00243] In the column-to-column connection system 180, the first column 100/101 and the second column 100/102 in assembly includes: an outer reinforcement, the outer reinforcement including the rebar coupled to the column shoe, the rebar and the column shoe being respective mating ones of the rebar 500 of the first column 100/101 and the column shoe 600 of the second column 100/102. In the column-to-column connection system 180, the first column 1001/101 and the second column 100/102 in assembly includes: the first plurality of the lapped member 300/301 partially extending into the second steel tube 200/202. In the column-to-column connection system 180, the first column 100/101 and the second column 100/102 in assembly includes: the second plurality of the lapped member 300/302 partially extending into the first steel tube 200/201.

[00244] In the column-to-column connection system 180, each one of the lapped member 300 is spaced apart from the respective interior surface 222 of the first steel tube 200/201 and the second steel tube 200/202.

[00245] In the column-to-column connection system 180, each one of the first plurality of the lapped member 300/301 and the interior surface 222 of the first steel tube 200/201 are fixedly coupled to opposing sides of a cushion 330, in which each one of the second plurality of the lapped member 300/302 and the interior surface 222 of the second steel tube 200/202 are fixedly coupled to opposing sides of another cushion 330.

[00246] The column-to-column connection system 180 further includes a third plurality of a rebar 500 in respective engagement with a fourth plurality of a column shoe 600. The third plurality of a rebar 500 is distributed external of the first steel tube 200/201. Each of the third

plurality of the rebar 500 is partially embedded in the first concrete body 400/401 and partially extending from the top of the first concrete body 400/401. A fourth plurality of a column shoe 600 is distributed external of the second steel tube 200/202 and partially embedded in the second concrete body 400/402 proximal to a bottom of the second concrete body 400/402.

[00247] In the column-to-column connection system 180, the column shoe 600 includes: a shoe frame 630; a lower cap 610, and an upper cap 620. The lower cap 610 is coupled to the shoe frame 630 and the upper cap 620 is coupled to the shoe frame 630. The lower cap 610 and the upper cap 630 are longitudinally spaced apart from one another.

[00248] Disclosed is a building structure 950 including at least two columns 100 in assembly using the column-to-column connection system 180.

[00249] Disclosed herein is a building structure 950 including: a first precast column 100/101, a second precast column 100/102, a concrete core 700 cast in situ, and a plurality of a lapped member 300. Each of the first precast column 100/101 and the second precast column 100/102 includes: a steel tube 200 and a concrete body 400 cast external of the steel tube 200. The steel tube 200 has opposing ends 210 defining a longitudinal axis 900. The concrete body 400 defines a top and a bottom, the top and the bottom being longitudinally spaced apart from one another. The concrete core 700 is cast in situ in the steel tube 200 of the first precast column 100/101 and the steel tube 200 of the second precast column 100/102. The first precast column 100/101 and the second precast column 100/102 define an interface 150 therebetween. The concrete core 700 longitudinally extends contiguously across the interface 150. Each of the lapped member 300 is fixed coupled to the steel tube 200 of one of the first precast column 101/101 and the second precast column 100/102. The plurality of the lapped member 300 are embedded in the concrete core 700 in an interleaved arrangement across the interface 150.

[00250] All examples described herein, whether of apparatus, methods, materials, or products, are presented for the purpose of illustration and to aid understanding, and are not

intended to be limiting or exhaustive. Various changes and modifications may be made by one of ordinary skill in the art without departing from the scope of the invention as claimed.

CLAIMS

1. A column, comprising:

   a steel tube, the steel tube longitudinally extending between two open ends of the steel tube;

   a first plurality of a lapped member partially disposed in an interior of the steel tube, the first plurality of the lapped member being coupled to an interior surface of the steel tube and extending beyond one of the two open ends;

   a second plurality of the lapped member partially disposed in the interior of the steel tube, the second plurality of the lapped member being coupled to the interior surface of the steel tube and extending beyond another of the two open ends; and

   a concrete body cast about an exterior surface of the steel tube to form a precast concrete-encased steel tube, wherein the steel tube is internally unfilled with concrete to provide a contiguous space in fluidic communication with the two open ends.

2. The column as recited in claim 1, wherein each of the first plurality and the second plurality of the lapped member comprises a lapped bar extending longitudinally, and wherein the lapped bar is spaced apart from the interior surface of the steel tube.

3. The column as recited in claim 2, wherein each of the first plurality and the second plurality of the lapped member comprises a cushion extending transversely from the interior surface, and wherein the cushion is coupled on opposing sides to the lapped bar and to the interior surface.

4. The column as recited in any one of claims 1 to 3, further comprising:

a third plurality of a rebar distributed external of the steel tube, each of the third plurality of the rebar being partially embedded in the concrete body and partially extending from a top of the concrete body.

5. The column as recited in any one of claims 1 to 4, further comprising:

a fourth plurality of a column shoe distributed external of the steel tube and partially embedded in the concrete body proximal to a bottom of the concrete body, the column shoe including: a lower cap; and an upper cap, the lower cap and the upper cap being longitudinally spaced apart from one another.

6. A method, comprising:

aligning a top of first column and a bottom of a second column to define an interface therebetween, each of the first column and the second column comprises:

a steel tube, the steel tube longitudinally extending between two ends of the steel tube;

a first plurality of a lapped member partially disposed in an interior of the steel tube, the first plurality of the lapped member being coupled to an interior surface of the steel tube and extending beyond one of the two ends of the steel tube;

a second plurality of the lapped member partially disposed in the interior of the steel tube, the second plurality of the lapped member being coupled to the interior surface of the steel tube and extending beyond another of the two ends; and

a concrete body cast about an exterior surface of the steel tube to form a precast concrete-encased steel tube,

wherein the steel tube of the first column and the steel tube of the second column are aligned to define a contiguous space in fluidic communication across the interface.

7. The method as recited in claim 6, further comprising:

casting in situ a concrete core, the concrete core being a contiguous column of concrete extending from the interior of steel tube of the first column across the interface to the interior of the steel tube of the second column.

8. The method as recited in claim 6, further comprising:

casting a concrete core in the interior of at least the steel tube of the first column to form a concrete-encased concrete-filled steel tubular column.

9. The method as recited in claim 6, comprising:

interleaving the first plurality of the lapped member of the first column and the second plurality of the lapped member of the second column; and

after the interleaving, pouring fresh concrete into the contiguous space to in situ cast a concrete core, the concrete core being a contiguously formed column extending from the interior of the steel tube of the first column to the interior of the steel tube of the second column.

10. The method as recited in any one of claims 6 to 9, wherein each of the first column and the second column is a precast column, and wherein the concrete core is cast in situ.

11. The method as recited in any one of claims 6 to 10, comprising:

coupling respective complementary ones of a rebar and a column shoe, wherein the rebar is partially embedded in the concrete body of the first column, and wherein the column shoe is partially embedded in the concrete body of the second column.

12. A column-to-column connection system, comprising:

a first plurality of a lapped member fixedly coupled to an interior surface of a first steel tube, the first steel tube being disposed in a first concrete body cast about an exterior surface of the first steel tube;

a second plurality of the lapped member fixedly coupled to an interior surface of a second steel tube, the second steel tube being disposed in a second concrete body cast about an exterior surface of the second steel tube; and

a concrete core, the concrete core being a contiguous column of concrete cast in an interior of the first steel tube and extending to at least partially fill an interior of the second steel tube, wherein the first plurality of the lapped member and the second plurality of the lapped member are embedded in the concrete core, and wherein each of the lapped member is disposed partially in an interior of the first steel tube and partially in an interior of second steel tube.

13. The column-to-column connection system as recited in claim 12, wherein the first column and the second column in assembly comprises:

an outer reinforcement, the outer reinforcement including the rebar coupled to the column shoe, the rebar and the column shoe being respective mating ones of the rebar of the first column and the column shoe of the second column; and

the first plurality of the lapped member partially extending into the second steel tube; and

the second plurality of the lapped member partially extending into the first steel tube.

14. The column-to-column connection system as recited in claim 12 or claim 13, wherein each one of the lapped member is spaced apart from the respective interior surface of the first steel tube and the second steel tube.

15. The column-to-column connection system as recited in any one of claims 12 to 14, wherein each one of the first plurality of the lapped member and the interior surface of the first steel tube are fixedly coupled to opposing sides of a cushion, and wherein each one of the second plurality of the lapped member and the interior surface of the second steel tube are fixedly coupled to opposing sides of another cushion.

16. The column-to-column connection system as recited in any one of claims 12 to 15, further comprising a third plurality of a rebar in respective engagement with a fourth plurality of a column shoe, wherein the third plurality of a rebar is distributed external of the first steel tube, each of the third plurality of the rebar being partially embedded in the first concrete body and partially extending from the top of the first concrete body, and wherein a fourth plurality of a column shoe is distributed external of the second steel tube and partially embedded in the second concrete body proximal to a bottom of the second concrete body.

17. The column-to-column connection system as recited in claim 16, wherein the column shoe comprises: a shoe frame; a lower cap, the lower cap being coupled to the shoe frame; and an upper cap, the upper cap being coupled to the shoe frame, wherein the lower cap and the upper cap are longitudinally spaced apart from one another.

18. A building structure comprising at least two columns in assembly using the column-to-column connection system as recited in any one of claims 12 to 17.

19. A building structure comprising:

a first precast column and a second precast column, each of the first precast column and the second precast column including:

a steel tube with opposing ends defining a longitudinal axis; and

a concrete body cast external of the steel tube, the concrete body defining a top and a bottom, the top and the bottom being longitudinally spaced apart from one another;

a concrete core cast in situ in the steel tube of the first precast column and the steel tube of the second precast column, the first precast column and the second precast column defining an interface therebetween, the concrete core longitudinally extending contiguously across the interface; and

a plurality of a lapped member, each of the lapped member being fixed coupled to the steel tube of one of the first precast column and the second precast column,

wherein the plurality of the lapped member are embedded in the concrete core in an interleaved arrangement across the interface.