



DEVELOPMENT OF AN OPTIMUM PRE-FOUNDED COLUMN SYSTEM FOR TOP-DOWN CONSTRUCTION

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Abstract. In this work, circular concrete-filled steel tubular (CCFT) columns, rather than the more popular H-shaped columns, were suggested as pre-founded columns for top-down construction. In addition, a novel shear connection system with headed stud shear connectors between the CCFT columns and the flat slabs was developed. It was noted that a CCFT column with a design compressive strength similar to that of an H-shaped column without consideration of length effects can be easily installed, even into a smaller borehole. Furthermore, compared to the H-shaped column, less steel is required for the CCFT column. It was shown that the amount of steel needed can be reduced by decreasing the wall thicknesses or diameters of the CCFT column depending on the exposed length of the column during excavation. The fillet-welded joint of the developed shear connector system was also tested before its in-situ application. The test results revealed that the joint possessed sufficient shear and deformation capacities. The CCFT column with the developed shear connection system was ultimately applied to an actual top-down construction process. The good constructability of the CCFT column system and reductions in construction costs and time were confirmed.

Keywords: pre-founded column, top-down construction, circular concrete-filled tube, shear connector.

1. Introduction

In congested urban areas, underground construction using such methods as traditional bottom-up construction is becoming increasingly difficult due to the presence of surrounding buildings and roads, as well as possible complaints from residents regarding noise, dust, and vibrations during construction. Meanwhile, top-down construction has the advantage of being able to protect surrounding buildings and underground infrastructures (Tatum *et al.* 1989; Song *et al.* 2009) and can thus be employed as an alternative method to traditional bottom-up construction. Top-down construction has actually been used in crowded Asian cities in Taiwan, Singapore, China (especially Hong Kong and Shanghai), Japan, and Korea (Moh, Chin 1994; Zhu *et al.* 2006; Yamamoto *et al.* 2009). The demand for top-down construction has also increased because of the ability to construct both super- and sub-structures simultaneously.

The main elements involved in the top-down construction method are perimeter retaining walls, pre-founded columns, and floors. Diaphragm walls along the perimeter of a construction site are generally constructed first as retaining walls. Pre-founded columns, whose lower ends are placed on the top of or embedded into piers, are then constructed through boreholes. Such columns serve to support a temporary construction load. After the pre-founded columns are installed, excavation proceeds

downward. Construction begins when excavation reaches the level required for the construction of the basement floor. In this process, flat slabs are preferred because they are useful in reducing the height of a floor and have a good resistance to punching shear failure due to capitals when the substructures are mainly used as parking spaces. Furthermore, flat slabs can be constructed on the ground, such as in slab-on-ground foundations, and the construction method has the advantages of speed and safety because the formwork is very simple and shoring is not required under the forms.

H-shaped columns with head stud shear connectors between flat slabs and H-shaped columns are generally used as pre-founded columns in Korea. The headed studs that are welded to the H-shaped columns at a workshop are very often damaged during excavation and are replaced with new ones. Even though the work required to replace damaged headed studs is not difficult, it is not easy to verify the conditions of all headed studs. Furthermore, even though a borehole is sufficiently large to obtain the bearing capacity required for a pier, the constructability inside the borehole decreases because of the section dimension comprised of an H-shaped column and studs. As a result, boreholes become larger, and the equipment required for the borehole excavation may change. Such changes can cause construction delays and an increase in the construction cost.

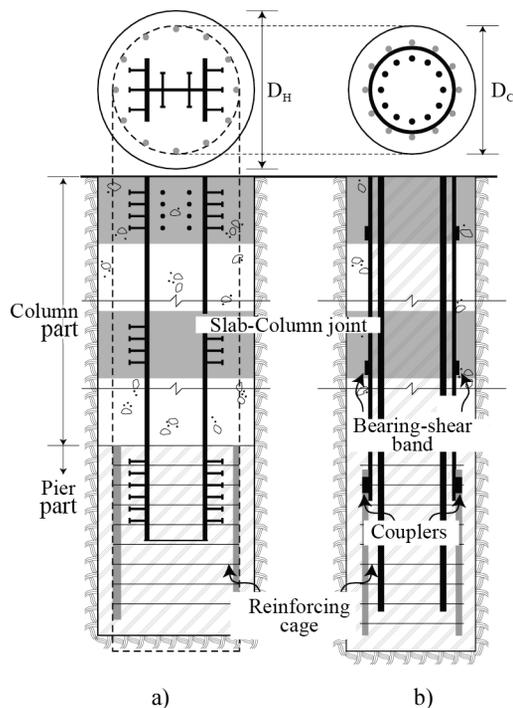
In this paper, circular concrete-filled steel tubular (CCFT) columns, rather than H-shaped columns, were suggested as pre-founded columns for top-down construction. A novel shear connection system with shear connectors of headed studs between flat slabs and CCFT columns was also developed. A CCFT column with the developed shear connection system was assessed economically by comparing it with conventional H-shaped columns with pre-welded studs. In addition, the capacity of the fillet-welded joint between the shear connection system and a CCFT column was tested before applying the CCFT column with the shear connection system to an actual top-down construction process. The feasibility of this pre-founded column system and its effects on construction costs and time were then evaluated based on the results of using the developed system in an actual top-down construction process.

2. Pre-founded columns with shear connectors

2.1. Conventional H-shaped column system

An H-shaped column is the most popular type of pre-founded column in Korea because H-shaped members can be easily obtained. Headed studs are also used as shear connectors between flat slabs and H-shaped columns in almost all top-down construction processes. These studs are generally stud-welded to the surfaces of H-shaped columns in a workshop.

In the sequence of top-down construction, boreholes are backfilled once with gravel to prevent soil collapse that may result from extraction of the outcasing and the buckling of H-shaped columns after the piers have been constructed. The process is illustrated in Fig. 1(a).



(D_H is the borehole diameter for an H-shaped column and D_C is the borehole diameter for a CCFT column)

Fig. 1. Comparison of pre-founded columns based on column sections: (a) H-sections and (b) circular CFTs

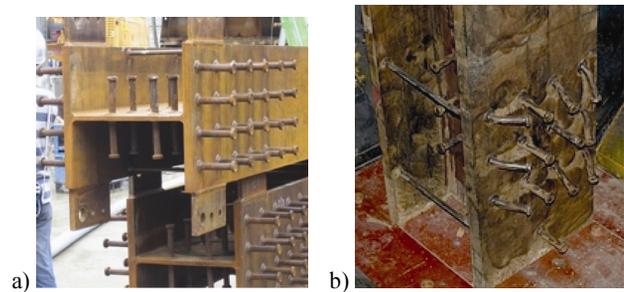


Fig. 2. Headed studs damaged by excavators during excavation: (a) before excavation and (b) after excavation

Excavation then proceeds downward. During excavation, excavators often encounter headed studs. As such, many headed studs are damaged when they are exposed after excavation (see Fig. 2). These damaged headed studs are removed, and new ones are welded to the H-shaped columns in the field before a slab is constructed. Even though the work required to replace damaged headed studs is not difficult, it is not easy to verify the conditions of all headed studs. Therefore, some damaged headed studs may remain and become embedded in the slab concrete. Such a scenario will affect the structural reliability of the building.

2.2. Circular CFTs instead of H-sections

The concrete-filled steel tubular (CFT) column is constructed by filling the hollow section of a steel tube with concrete. Besides reducing construction time due to using a steel tube as formwork, a CFT column has many advantages of high compressive stiffness, high axial load-carrying capacity, high ductility, and earthquake-resistance in result. It is because the steel tube confines the concrete core inside it and the concrete core supports the axial load and helps preventing local buckling of the steel tube. The concrete core also improves the fire resistance: the steel columns with plain concrete showed the fire resistance time to failure of one to two hours, as compared to about five minutes for the unfilled steel columns (Kodur 1999).

There are various section shapes of CFT columns: circular, rectangular, square, and multi-side like octagonal. CFT columns also can be classified to two types according to the form of concrete core: with solid and hollow core (Kuranovas, Kvedaras 2007). Among them, the CCFT column with solid concrete core is the focus of discussion in this paper, which is the more commonly used type in many structures. Analytical and experimental researches have indicated that CCFT columns provide better performance than CFT columns with other section shapes: the better confining effect (Susantha *et al.* 2001), more axial load-carrying capacity (Knowles, Park 1969; Tomii *et al.* 1977), more post-yield ductility (Schneider 1998), and greater bond-stress transfer (Roeder *et al.* 1999). Furthermore, the cross-section of the CCFT has no weak-axis in contrast to an H-shaped member that has both weak- and strong-axes.

In this paper, a CCFT column with a reinforcing cage for the pier connected to the lower part of the column by

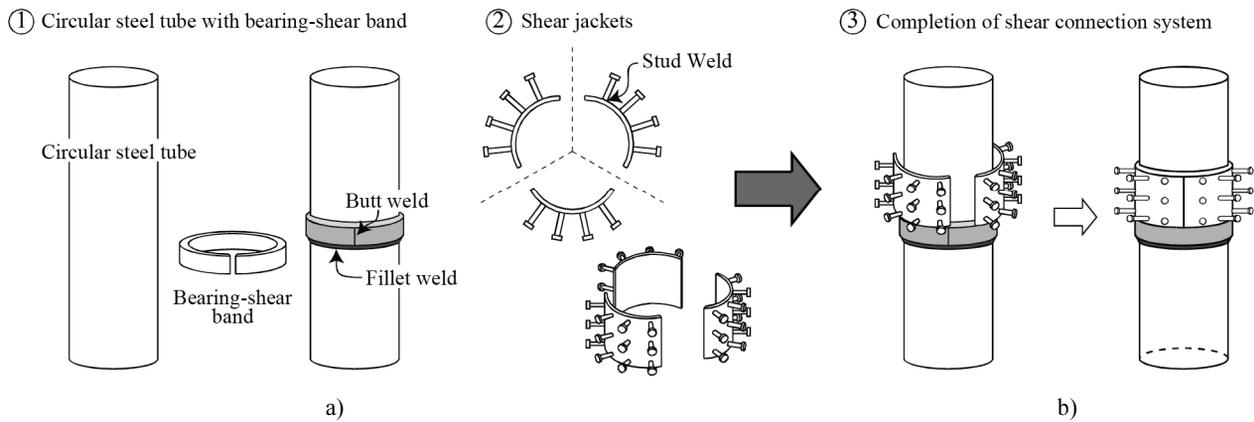


Fig. 3. Concept of the new shear connecting system: (a) application in a workshop and (b) in the field

several couplers is applied to a pre-founded column instead of an H-shaped column and it is illustrated in Fig. 1(b). More detailed comparisons of CCFT and H-shaped columns as pre-founded columns are discussed in Chapter 3.

2.3. Connections between flat slabs and CCFTs

The developed shear connection system between a flat slab and a CCFT column for top-down construction is illustrated in Fig. 3. The system consists of shear jackets with headed stud shear connectors for transferring the shear force across the concrete-steel interface and a bearing-shear band for supporting the shear jackets. The shear jacket is fabricated in a workshop by curving a plate into a column section shape and stud-welding the headed studs to it. The bearing-shear band is also a curved plate with a column section shape and is fillet-welded to the outer circumference of a circular steel tube.

In the field, the shear jackets are installed on the bearing-shear band in a manner similar to wrapping a column after excavation. As shown in Fig. 3, installation of the shear jackets is performed before the slab installations. Each column has more than two shear jackets, depending on the weight of the jacket itself. The shear jackets are lightly connected to one other using binding wires or steel screw-clamps so that fine hand adjustments are possible. This system can reduce top-down construction time because almost all necessary work is conducted in a workshop, and the system is expected to be easily installed in the field.

In the developed system, headed studs for transferring shear force are ultimately connected to a CCFT column by a fillet-welded joint between the bearing-shear band and a CCFT column. If the fillet weld is ruptured, the story shear at the interface between a flat slab and a CCFT column will not be transferred to the column. Therefore, the capacity of the fillet-welded joint was tested before in-situ application of the shear connection system.

3. Comparison of H-sections and circular CFTs

CCFT columns were compared with H-shaped columns as pre-found columns focusing on the constructability of how easily a steel column is plumbed inside a small and

deep borehole and the possibility of reducing the diameter of a borehole and the amount of material. A substructure with six basement floors constructed using the top-down construction method is considered as a case. It is assumed that a superstructure is constructed after the completion of the substructure. The height and column load of each basement floor of the case substructure are shown in Table 1 and the depth of a pier cap is 1.0 m.

Table 1. Case substructure

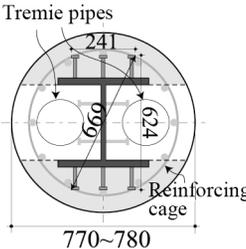
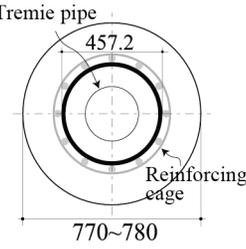
Basement story	Height (m)	Slab thickness (m)	Floor load per column (kN)	Factored axial load per column (kN)
1	5.10	0.4	2,548	2,548
2	3.77	0.3	833	3,381
3	3.05	0.3	833	4,214
4	3.05	0.3	833	5,047
5	3.05	0.3	833	5,880
6	6.40	0.35	931	6,811

3.1. Constructability inside boreholes

The H-shaped column as shown in Table 2 is chosen for the pre-founded column illustrated in Fig. 1(a) and is assumed to be inserted into an 800-mm-diameter borehole. The actual diameter of a borehole may be about 770–780 mm depending on drilling machines. The dimension of the CCFT column for the pre-founded column of Fig. 1(b) is decided to have the similar design compressive strength without consideration of a length effect to that of the H-shaped column.

Different design codes predict the axial capacity of a CCFT column in different ways by reflecting the design philosophies and practices in the respective countries (Shanmugam, Lakshmi 2001; Kuranovas *et al.* 2009). Comparative researches on the accuracy of different design codes for predicting the axial capacity of a short CCFT column concluded that Eurocode 4 (2004) generally showed the best agreement with the test results, while AIC (2001) and AISC (2005) provided conservative results (Zhao *et al.* 2009). Eurocode 4 (2004) predicted

Table 2. Comparison between H-sections and circular CFTs as pre-founded columns

Pre-founded column	H-shaped column	Circular CFT column
Section		
Dimension (mm)	H – 414 x 405 x 18 x 25	$D = 457.2, t = 16$
F_y (MPa)	325	325
f'_c (MPa)	–	35
E_s (MPa)	205,000	205,000
E_c (MPa)	–	29,779
Verticality control	Difficult	Easy
Pumping concrete of pile	Difficult	Easy
Required diameter of tremie pipe	Not satisfied	Satisfied
Required depth of concrete cover	Not satisfied	Satisfied

*1) concrete for the pier is not considered

well the ultimate compressive strength of both short and long CCFT columns, too (Kuranovas *et al.* 2009). Here, AISC (2005) that has a tendency to conservatively predict the design compressive strength of a CCFT column is applied because a CCFT column is mainly compared with an H-shaped column in aspects of the constructability inside a borehole and the possibility of reducing the amount of steel. The conservative value will put a CCFT column in the worse condition related to the dimension in comparison with an H-shaped column. The design compressive strength of the H-shaped column is calculated with AISC (2005) as well.

The design compressive strength without a length effect is defined as:

$$\phi_c P_n, \quad (1)$$

where ϕ_c is the resistance factor (0.9 for H-section and 0.75 for CCFT) and P_n denotes the nominal compressive strength.

The nominal compressive strength, P_n , is as follows:

$$P_n = A_s F_y \quad \text{for H-section}; \quad (2a)$$

$$P_n = A_s F_y + C_2 A_c f'_c \quad \text{for CCFT}, \quad (2b)$$

where A_s and F_y are the area and the yield strength of the steel section, respectively, C_2 is the coefficient for the increased concrete strength due to confinement by a steel tube (0.95), and A_c and f'_c are the area and the strength of the concrete core, respectively. The dimension and cross-section of the CCFT column decided are presented in Table 2.

The maximum diagonal dimension of the H-section in Table 2, including shear studs, is 669 mm because headed studs are typically welded to an H-shaped co-

lumn in a workshop and then delivered to the field. The distance between the end of the stud and the inner borehole wall is less than 55.5 mm and it is less than the maximum allowable error for plumbing a steel column of 91 mm (the maximum error was calculated by $H/300$, where H = total column length). Therefore, control of the verticality of the H-shaped column within the borehole is difficult. However, in the case of the CCFT column, more than 156.4 mm, sufficient for plumbing, exists between the outside of the column and the inner borehole wall.

Concrete for a pier of the H-shaped column was poured using two small tremie pipes smaller than general tremie pipes of 250–300 mm in a diameter because of the section shape. In contrast, concrete for a CCFT column pier can be poured using one tremie pipe large enough to accommodate the maximum aggregate size of the pier concrete within the circular steel tube.

The required depth for the cover concrete of a pier is 80 mm. However, in the case of the H-shaped column, the maximum depth remaining inside the borehole is less than 52 mm after installing the reinforcing cage for the pier. In the case of the CCFT column, the depth between the outside of the reinforcing cage and the inner borehole wall is greater than 113 mm. The depth of over 113 mm is satisfied with the required minimum cover depth.

The borehole for the H-shaped column should be drilled so that it is larger than that for the CCFT column, even though the pier is large enough to support the column load during construction. As a result, a change in the drilling method may be necessary. Such a change may entail a shift from the Percussion Rotary Drilling method (PRD) to the Reverse Circulation Drilling method (RCD), which is namely a change from a smaller to a larger drilling diameter. This, in turn, may cause construction delays and an increase in construction costs.

3.2. Amount of material according to effective length

In top-down construction, excavation generally proceeds downward over more than one phase and the buckling restraint condition of a pre-founded column is changed according to the excavation phases. Until the final excavation phase, the lower end of the exposed pre-founded column is not restrained by the concrete but embedded into back-filled material such as gravel while the upper end is restrained by the concrete of a slab and the column of the upper floor. Therefore, the upper and lower ends can be idealized as being fixed and pinned at a virtual supporting point as illustrated in Fig. 4. The length from the exposed soil surface to the virtual supporting point, l_0 , depends on the section properties of pre-founded columns. The effective buckling lengths of the H-shaped and CCFT columns are thus different when the exposed lengths of them are identical.

On the other hand, when excavation proceeds to the final planned level, the lower end is also restrained by the concrete of a pier and both ends can be idealized as being fixed. As a result, there are no differences in the effective buckling lengths of the H-shaped and CCFT columns when the exposed lengths of them are identical. Furthermore, the load that is exerted on the exposed part of a pre-founded column is greatest during this final excavation phase.

Therefore, the design compressive strengths with length effects for the H-shaped and CCFT columns in Table 2 during the final excavation phase were compared, which is shown in Fig. 5.

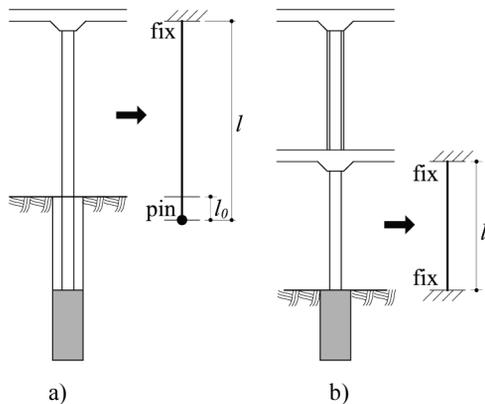


Fig. 4. Idealization of both ends of the exposed steel column in the excavation phases: (a) during excavation and (b) in the final excavation phase

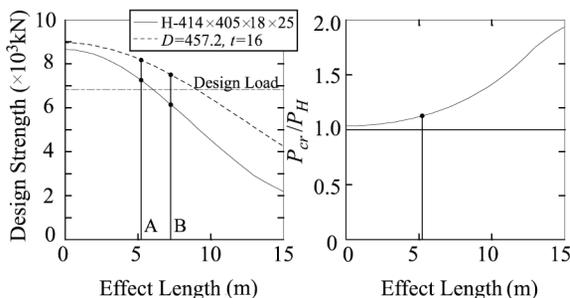


Fig. 5. Design compressive strengths: (a) comparison of section shapes and (b) the compressive capacity ratio, P_{cr} / P_H

According to AISC (2005), the each design compressive strength, P , is defined as:

$$P = \phi_c \left(0.658^{\frac{P_e}{P_n}} P_n \right) \text{ for } P_e \geq 0.44 P_n; \quad (3a)$$

$$P = \phi_c (0.877 P_e) \text{ for } P_e < 0.44 P_n, \quad (3b)$$

where P_e denotes the elastic buckling load by the Euler equation.

The elastic buckling load, P_e , is as follows:

$$P_e = \frac{\pi^2 (EI)}{(kl)^2}; \quad (4a)$$

$$EI = E_s I_s \quad \text{for H-section}; \quad (4b)$$

$$EI = E_s I_s + C_3 E_c I_c \quad \text{for CCFT}; \quad (4c)$$

$$C_3 = 0.6 + 2 \left(\frac{A_s}{A_s + A_c} \right), \quad (4d)$$

where k is the effective length factor (0.65), l is the length of the column for buckling, E_s and I_s are the elasticity modulus and moment of inertia of the steel section, respectively, and E_c and I_c are the elasticity modulus and moment of inertia of the concrete core, respectively.

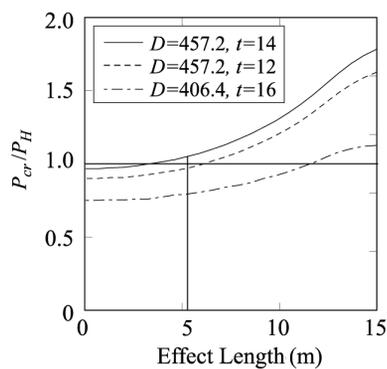
The design compressive strength for the CCFT column is getting higher than that for the H-shaped column as the effective length becomes longer while the design compressive strengths of them without consideration of a length effect is almost same. In the case of the substructure presented in Table 1, only the lowest single floor can be excavated at once if the H-shaped column is chosen for a pre-founded column as presented in Fig. 5 and Table 3. However, with the CCFT column, two floors of the fifth to sixth basement floors can be excavated during the final excavation phase. The number of excavation phases can thus be reduced depending on the excavation planning and the more work space under the constructed slab of the upper floor can be guaranteed. It may result in reducing construction time.

In the case the pre-founded columns of the H-shaped and CCFT columns are excavated by the plan A of Table 3, the design compressive strength for the CCFT column is 1.13 times higher than that for the H-shaped column even though the amount of steel per a unit length for the CCFT column is 75% of the H-shaped column. It is because the concrete that is placed into the circular steel tube prior to excavation, contrary to the H-shaped column, contributes the design value in the case of the CCFT column. On the other hand, by changing the CCFT column into that with a smaller wall thickness or diameter, the amount of steel can be more reduced because the CCFT column has a 13% greater design compressive strength. Changing the wall thickness rather than the diameter in the cases presented in Fig. 6 seems to be more effective way to reduce the amount of steel by keeping the identical gross area with the CCFT column. The CCFT column with a wall thickness of 14 mm has a higher design compressive strength than the H-shaped column when the effective length is 5.23 m. In the case column when

Table 3. Excavation plans for the case substructure

Excavation plan	A	B
Final excavation part	<ul style="list-style-type: none"> Sixth basement floor Pile cap Additional excavation of 1 m for the construction of a pile cap 	<ul style="list-style-type: none"> Fifth and Sixth basement floors Pile cap Additional excavation of 1 m for the construction of a pile cap
Exposed length of a column (m)	8.05	11.15
Effective length of a column (m)	5.23	7.25
Design load per column (kN)	6,811	
Design compressive strength (kN)	H-section	7,238
	CCFT	8,167
		7,512

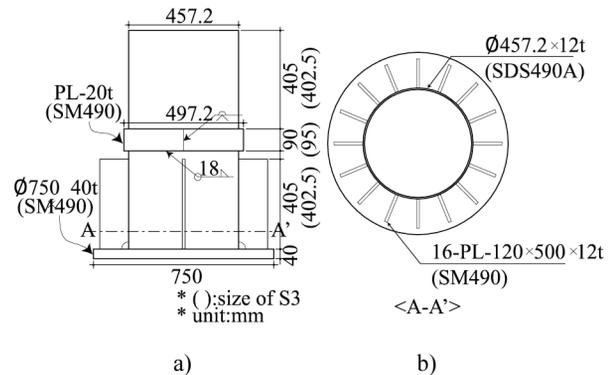
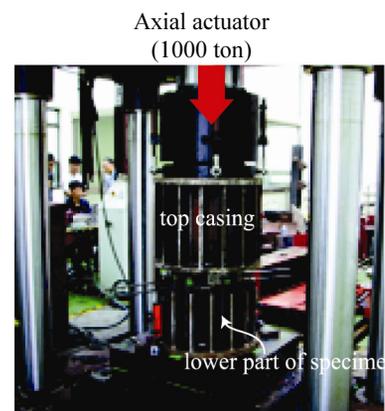
the effective length is 5.23 m. In the case of choosing the CCFT column with a wall thickness of 14 mm, the amount of steel is thus expected to be reduced by 14% more per a unit length.

**Fig. 6.** Influence on the compressive capacity ratio of thickness and diameter

4. Test for the shear connection system

4.1. Specimen and set-up

The specimen used in this study is a full-scale circular steel tube with a bearing-shear band that corresponds to a column in the first basement floor of the residential building introduced in Chapter 5 and shown in Fig. 7. The purpose of this test was to survey the shear capacity of the fillet-welded joint between the bearing-shear band and a column subjected to an axial load. The inside of the specimen was not filled with concrete. Instead, the lower part of the specimen was reinforced with 16 stiffeners (thickness = 12 mm, $F_y = 325$ MPa), as shown in Fig. 7. The outer diameter of the circular steel tube was 457.2 mm (thickness = 12 mm, $F_y = 325$ MPa), and the bearing-shear band was comprised of a 90- or 95-mm plate (thickness = 20 mm, $F_y = 325$ MPa) (see Fig. 7) that was curved like a circular steel tube. After the bearing-shear band was set around a circular steel tube, sides of the end were groove-welded, and the bottom was fillet-welded to the outer circumference of a steel tube (E70 electrode, leg size = 18 mm). A total of three specimens (S1, S2, and S3) that were identical except for their bearing-shear band height were fabricated and tested.

**Fig. 7.** Details of the specimen: (a) side view, (b) cross section and (c) photo of the specimen**Fig. 8.** Test set-up

As shown in Fig. 8, an axial load was applied directly to the bearing-shear bands of the specimens through a top casing using a hydraulic actuator installed on the tops of the specimens. The top casing corresponds

to the shear jackets in the shear connection system and was reinforced with 16 stiffeners.

The load and the total vertical displacement were respectively measured with a load cell and a linear potentiometer installed in the actuator.

4.2. Results

The load-total vertical displacement curves for the tested specimens are illustrated in Fig. 9. The maximum design load and the design shear strength for the fillet weld according to AISC (2001) are also plotted in Fig. 9. The design shear strength is the value obtained by multiplying the design shear strength of the longitudinally-loaded fillet weld by 1.5 when considering the load angle transverse from the weld longitudinal axis. For S3 and S1, loading was stopped by the force exerted during testing because of the hydraulic actuator capacity. Even though the maximum strength of S2 (8,207 kN) was the smallest measured, it was over three times greater than the maximum design load and two-fold stronger than the design shear strength of the fillet weld. Furthermore, the vertical deformation of the fillet welds under the maximum design load was less than 1 mm. This value is smaller than that attained for fillet welds used by Deng *et al.* (2003),

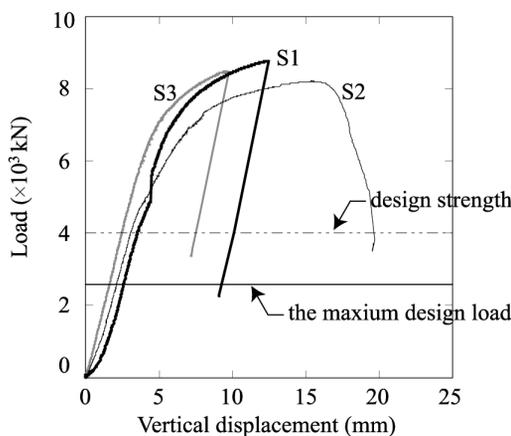


Fig. 9. Load-vertical displacement relationships

where a 3 mm fracture deformation (fracture strain of 0.169 x a weld leg size) was observed (see Fig. 9). Furthermore, no ruptures or significant damage to the fillet welds of the three specimens were observed during testing. Therefore, the fillet welds were considered to have sufficient shear and deformation capacities.

5. Application to actual top-down construction

A CCFT column with the developed shear connection system outlined in Section 2.3 was applied to the top-down construction of a residential building located in downtown Seoul. The building consisted of a 30-story residential tower and a three-story podium with seven basement floors for parking and commercial facilities. Fifty-eight pre-founded H-shaped columns were planned for the substructure. Among these, 19 columns that were located in the podium section were changed into CCFT columns using the shear connection system (the CCFT column system). For the case of the CCFT columns, 12 reinforcements with diameters of 32 mm were placed inside circular steel tubes.

Table 4 shows the details of the original H-shaped column and changed CCFT column.

The final excavation was planned in order to excavate the lowest single floor of 6.4 m, a pier cap of 1.0 m, and additional excavation of 1.0 m. The effective length was thus 5.23 m when an effective length factor of 0.65 was applied. The design compressive strengths for an effective length of 5.23 m, shown in Table 4, were calculated based on the Korean design provision of the days on which the building was planned. The current Korean design provision related to the design compressive strength was revised in 2009 and it is similar to AISC (2005) represented in the Eqs (1)–(4) except that the increased concrete strength due to confinement by a steel tube, which is defined as:

$$0.85(1+1.8(tF_y)/D[f_c C]^\dagger), \tag{5}$$

where *t* and *D* are the wall thickness and outer diameter of the steel tube, respectively, and the coefficient of 0.85 indicates the factor 0.85 in the stress block of concrete.

Table 4. Comparison of the H-shaped column and circular CFT column used in the seventh basement floor

	H-section	CCFT
Average design column load (kN)		7,644
Effective buckling length (mm)		5,233
Pre-founded column	H-shaped column	Circular CFT column
Dimension (mm)	H-450 × 450 × 20 × 35	Φ 457.2 × 12 (12 – D32)
Borehole (diameter, mm)	RCD (1500 mm)	PRD (800 mm)
F_y (MPa)	325	325
f_c (MPa)	–	35
F_{ry} (MPa)*1)	–	400
E_s (MPa)	205,000	205,000
E_c (MPa)	–	29,779
Design compressive strength (kN)	9,520	10,836
Amount per 1 m	Steel (tons)	0.31
	Concrete (m ³)	–

*1) F_{ry} = the yield strength of the reinforcement

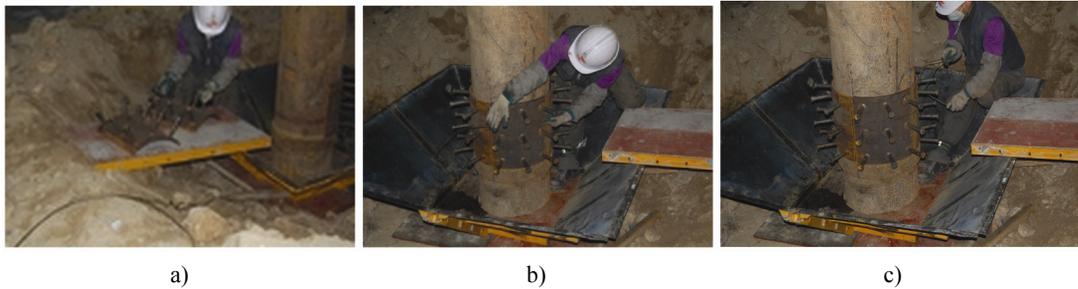


Fig. 10. Installation procedure for the shear connection system: (a), (b) placing of shear jackets on the bearing shear band, and (c) shear jacket connections

In the case of the CCFT column of Table 2, the concrete strength is considered to be increased by 135% according to the Eq. (5), which is 1.43-fold higher than the 95 % increased concrete strength in the Eq. (2b).

On the other hand, the former design provision incorporated a resistance factor of 0.85 for both the H-shaped and CCFT columns, and used properties modified according to the areas of the steel sections of the CCFT columns, which is similar to AISC (2001). The increased concrete strength by the confining effect was also estimated by the Eq. (5) but the coefficient of 0.85 was 0.6.

The design compressive strengths for the H-shaped and CCFT columns were 1.25-fold and 1.42-fold higher, respectively, than the design load. Changing the H-shaped column into the CCFT column with reinforcements results in a 14% greater design compressive strength. Here, the second-order moment due to the eccentricity that results from construction errors such as the controlling error of pre-founded column verticality was also considered.

Use of the CCFT column system resulted in a 57% reduction in the amount of steel compared to that used in the original H-shaped column. Furthermore, the diameter of the borehole was decreased and the drilling method was changed from RCD to PRD. The CCFT column system also led to reductions in both the amount of bored soil from the borehole and the amount of concrete needed for the pier.

The headed studs welded to many of the H-shaped columns located in the tower region were damaged. Such damage, which was revealed after excavation, led to additional work in removing the damaged studs and welding new headed studs to the H-shaped columns in the field. In contrast, the head studs for the CCFT columns were not damaged because they were installed after the excavation was complete. As such, no extra work was necessary. The installation procedure for the shear connection system with four shear jackets is shown in Fig. 10, and the installed system is shown in Fig. 11.

The shear jackets were ultimately fixed to one other via tag-welding. However, connecting the shear jackets lightly by hand in order to allow for fine adjustments is considered to be more desirable than welding the shear jackets to one other. The shear connection system is actually embedded into and restrained by the concrete after pouring the concrete for the slabs. Fixing the shear jackets to one other through processes such as welding is then unnecessary. As a result, the ease with which the shear connection system could actually be installed in the field was

confirmed. In addition, no further interruptions in the subsequent work were encountered, as shown Fig. 12.



Fig. 11. Shear connection system after installation



Fig. 12. Slab-column connections after slab bar placement

6. Conclusions

A CCFT column system for top-down construction has been developed and applied to actual top-down construction. The system consists of a CCFT member pre-founded column and a new shear connection system for a flat slab and a CCFT column. The CCFT system was first compared to the conventional H-shaped column system. The fillet-welded joint between the bearing-shear band and the CCFT column was then tested, and the CCFT system was applied to an actual top-down construction process. As a result, the following conclusions can be drawn:

- Compared to a conventional H-shaped column, a CCFT column with a similar design compressive strength (without consideration of length effects) could be easily erected, even within a smaller borehole. The amount of steel needed for the CCFT column was also lower than the amount needed for the conventional H-shaped column;

- With an increase in the effective length for buckling, the design compressive strength of the CCFT column increased more rapidly than that of the H-shaped column. Furthermore, the CCFT column was changed to CCFT members with smaller thicknesses or diameters when considering the exposed length of the column during excavation. As a result, the amount of steel needed was reduced;
- The developed shear connection system consists of shear jackets with headed stud shear connectors and a bearing-shear band to support the shear jackets. The fillet-welded joint between the bearing-shear band and the column was tested before applying the system to an actual top-down construction process. Test results showed that the fillet-welded joint had a sufficient shear and deformation capacity;
- The application of the CCFT column system, rather than the H-shaped column system, to an actual top-down construction process resulted in a reduction in the amount of steel used. In addition, no decline in the load carrying capacity was observed, and the borehole diameter was decreased. The ease with which the shear connection system could be installed was confirmed, and subsequent work through in-situ installations was not disturbed. Therefore, the CCFT column system is considered to have good constructability and is effective in reducing construction costs and time.

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