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Flexural Capacity of Concrete-Filled Tubular Columns Encased with Precast Concrete

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Abstract. In this paper, concrete-filled steel tubular columns encased with precast concrete (PC) were studied. Four eccentrically loaded columns and a concentrically loaded column were tested to investigate the load-carrying capacity. The test parameters were the eccentricity, column length, and reinforcement details of the PC cover. The test results indicate that despite the small thickness, the cover precast concrete contributed to the load-carrying capacity, though in some specimens the load carrying capacity quickly decreased after the peak strength, due to the spalling of the cover concrete. The peak strengths of the specimens were compared with the predictions by current design codes.

Keywords: concrete-encased-and-filled tubular column, axial loading, cover concrete, hollow PC

1. Introduction

The structural characteristics of concrete-encased-and-filled steel tubular columns (CEFT columns) are generally more similar to those of concrete-encased steel columns (CES columns) than to those of concrete-filled steel tubular columns (CFT columns). In practice, CEFT columns have been used to improve the load-carrying capacity and the fire resistance of CFT columns. For instance, in a top-down construction, underground CFT columns are additionally encased with reinforced concrete after erecting the tubes and filling the inner concrete. Also, CEFT columns can be used in a precast concrete (PC) method by prefabricating the hollow PC columns combined with the tubes in factories, and filling the inner concrete at fields. In the present research, CEFT columns were designed for the purpose of the latter. Hollow PC columns especially have the advantage in reducing the lifting weight of large columns

Some researchers [1]-[3] in Japan, where steel tubes with a various types of combination with concrete are widely used, have conducted studies on composite tubular members. However, the previous researches mostly focused on encased-type tubular members and combined flexural-and-axial behaviors of CEFT columns were not experimentally investigated. Furthermore, the hollowness ratios (hollow section-to-gross section area ratio) of the former column specimens were mostly smaller than 36%. However, in order to lighten the hollow PC columns enough to be applicable for a large section, the hollowness ratio should be greater than 50%.

In this research, eccentric axial load tests were performed to investigate the structural capacities of CEFT columns with relatively thin encasement thickness. The test parameters were not only the eccentricity and column length, but also the reinforcement details of cover PC such as the use of fiber reinforcement for high performance and welded wire mesh (WWM) for simple fabrication.

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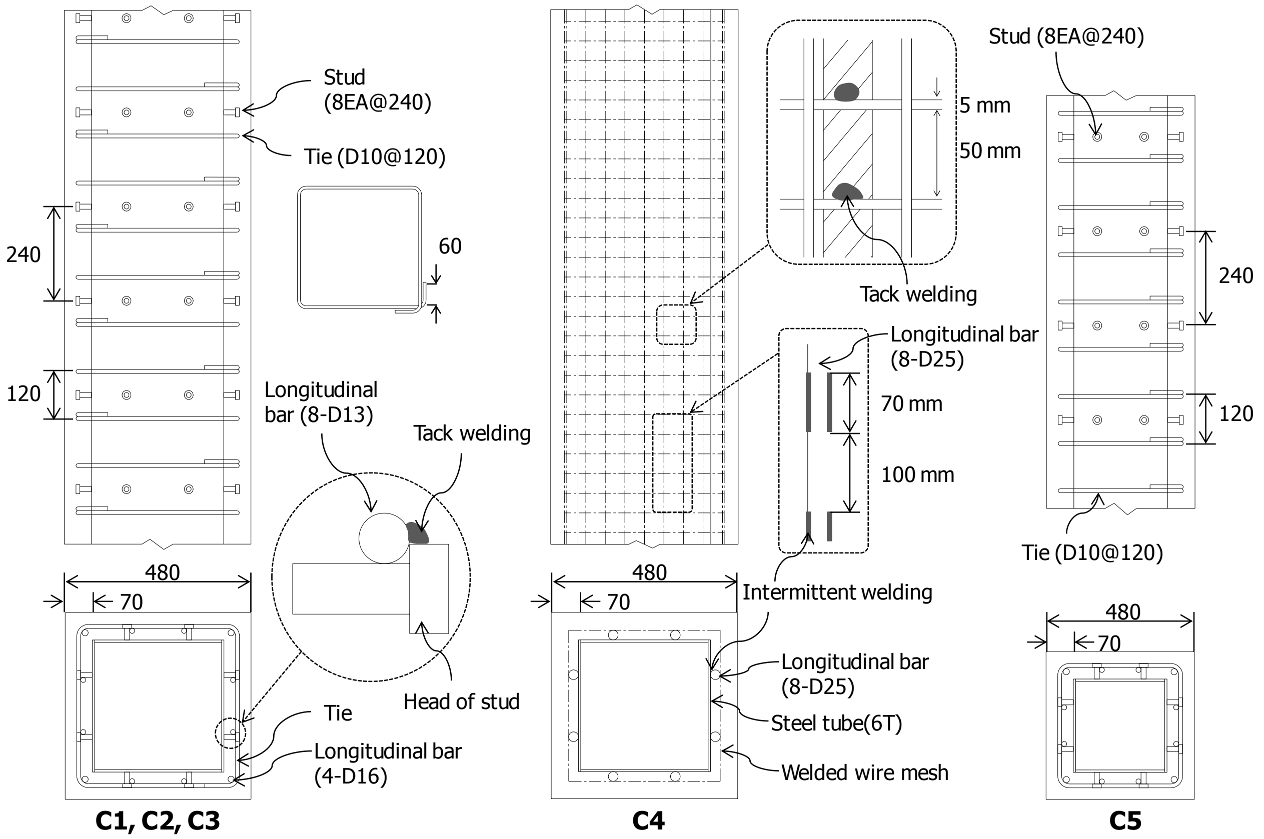

Fig. 1: Details of test specimens

2. Test Specimens

Fig. 1 shows the dimensions and the details of test specimens. The sectional dimension of eccentrically loaded specimens C1, C2, C3, C4 was 480 mm × 480 mm, and the dimension of the cross section for a centrically loaded specimen C5 was 380 mm × 380 mm considering the loading capacity of UTM. In every specimen, the thickness of tubes was 6 mm and the thickness of cover PC was 70 mm. The thickness of cover PC was decided as the minimum dimension for arrangement of longitudinal re-bars and cover concrete for transverse re-bars. The hollowness ratio was about 50% for the eccentrically loaded columns and 40% for the concentrically loaded column. The steel tubes were built up by welding and the dimensions were 340 mm × 340 mm in the eccentrically loaded specimens and 240 mm × 240 mm in the concentrically loaded specimen. The width-to-thickness ratio of tubes in C1~C4 was 54.7 which slightly exceeds the limitation for compact section defined by AISC 360-10.

In C2, C3, and C5, the original detail of cover PC using the conventional ties for the lateral confinement and the studs for the bond between tubes and cover PC was applied. In the original detail, four D16 longitudinal re-bars were placed at the corners of the cross section and D10 transverse re-bars were used with spacing of s = 120 mm (= a fourth of the cross section). In order to prevent cracking and early spalling of cover PC, eight D13 longitudinal re-bars were tack-welded with studs.

In C2, the eccentricity (eccentric distance-to-width of the cross section ratio) was 0.375 (e = 180 mm). The effective length (distance between a loading point and a reaction point) of C2 was L_e = 2,880 mm. To evaluate the slenderness effect, the effective length of C3 was designed to be L_e = 4,320 mm. The eccentricity of C3 was 0.125 (e = 60 mm). C5 was a concentrically loaded one. Although the dimension of the cross section was decreased compared with the eccentrically loaded specimens, the detail of cover PC is almost the same as C2 and C3. The effective length of C5 was L_e = 1,500mm.

In C1, the cover PC was reinforced with steel fiber to improve the resistance to cracks and the deformation capacity of thin cover PC. The volumetric ratio of steel fiber was limited to 0.8 % taking account of workability. Except for the use of steel fiber, C1 had no difference from the original detail. The effective length of C1 was L_e = 2,880 mm and the eccentricity was 0.125. The cover PC of C4 was reinforced with WWM (ϕ = 5 mm, □ = 50 mm × 50 mm) for the ease of production. For installation of WWM and restraining the local buckling of tubes, eight D25 longitudinal re-bars were welded intermittently with the tube. The eccentricity of C4 was 0.125.

3. Test Results

3.1. Failure mode and load-displacement relationship

Fig. 2 and Fig. 3 show the failure modes and the load-displacement relationships of specimens respectively. The loading was conducted until an axial strength dropped to the 70% of the maximum load. The dropping of cover PC is not clearly described in Fig. 2 because the column part of specimens was lapped during the experiments. The failed regions were distinguished from the observation of local buckling in tubes through the breakup of specimens.

In all specimens, failure occurred with the delamination of cover PC which was followed by the local buckling of longitudinal bars and tubes. C1 reinforced with steel fiber showed the increased load-carrying capacity even after the compressive cracking. At the damaged region, a plenty of fine cracks was generated through the test. The spalling of cover PC was also restrictive and the load resistance gradually decreased after the maximum load P_u = 6,844 kN.

In all specimens except for C1, the maximum load coincided with the compressive cracking, and the brittle failure of cover PC immediately followed afterwards. The specimens experienced a quick drop of strength after the peak load and went through a gradual decrease caused by the local buckling of re-bars and tubes. A fall of load resistance after attaining the peak load (P_u = 4,195 kN) was not severe in C2, for the contribution of cover PC to axial load was smaller than in the other specimens with low eccentricity.

Compared to C2, C3 showed a larger decrease in load-carrying capacity immediately after the peak load (P_u = 6,883 kN) because the contribution of cover PC to the strength was significant by its low eccentricity (e = 0.125). C4 having the same eccentricity with C3 also went through a sudden drop of axial load after its peak strength (P_u = 7,674 kN). Because a thin layer or densely-welded wire mesh was installed, the delamination of cover PC occurred in the wide range of the compressive face. After the test, welded wire mesh turned out to be failed by weld fracture, and the local buckling of the tube was found.

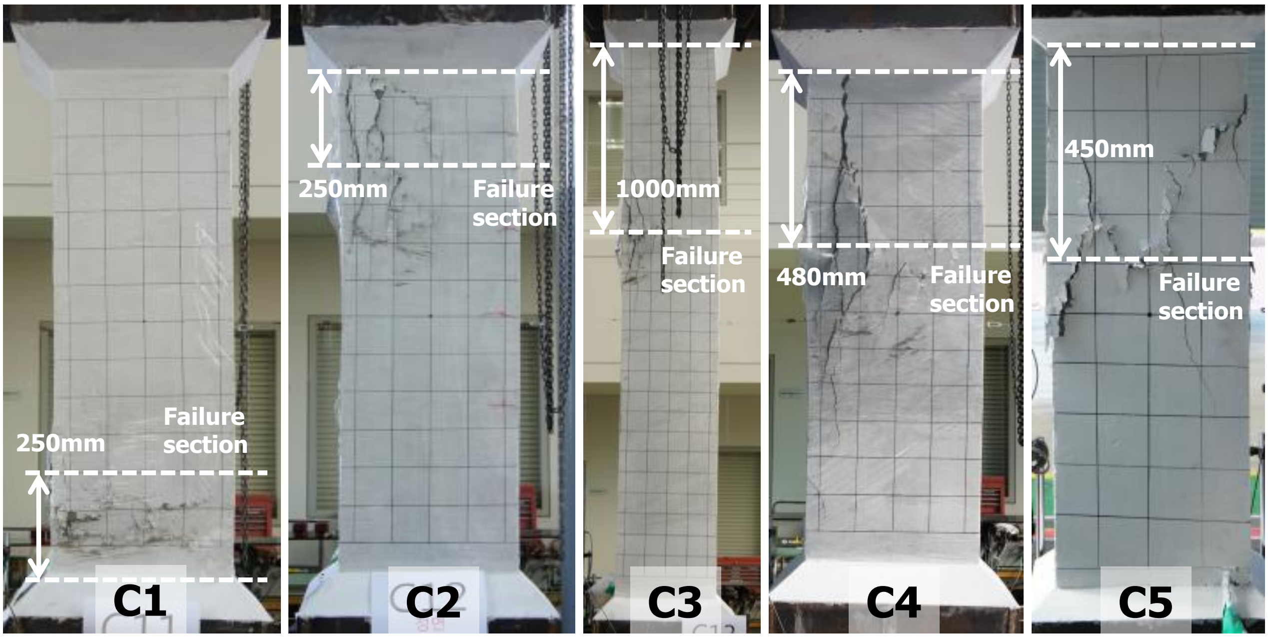

Fig. 2: Damaged specimens at the end of testing

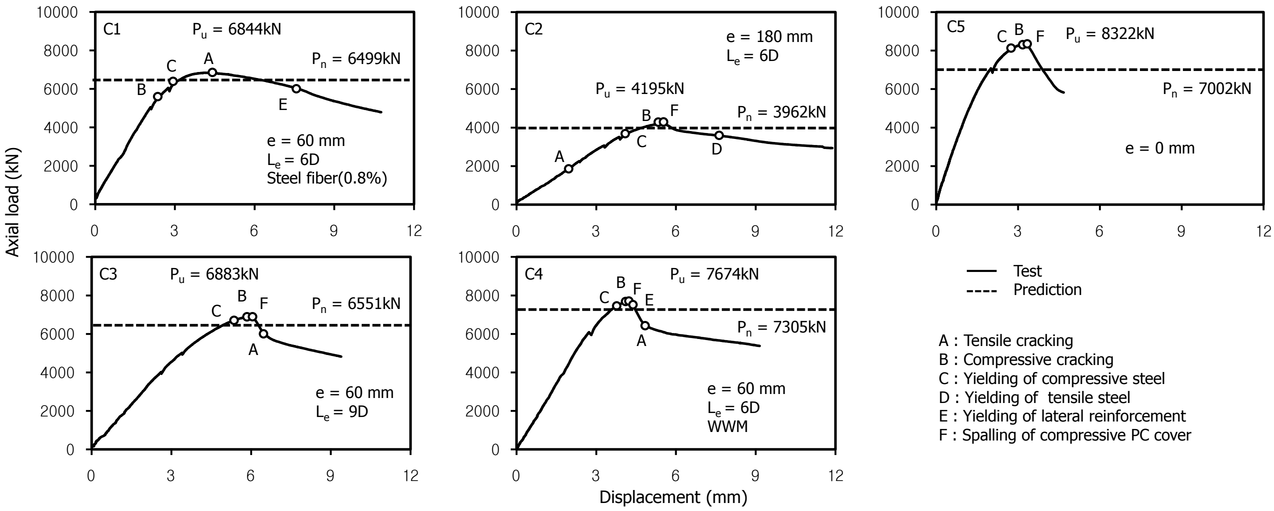

Fig. 3: Load-axial displacement relationships of test specimens

In C5, a rapid fall of axial load also happened immediately after the maximum load (P_u = 8,322 kN). The failure of C5 was brittle because the section-area ratio of cover PC was higher than the eccentrically loaded specimens, and all of the cover PC contributed to axial load. The local buckling of re-bars and a tube was not obvious from the experiment because the width-to-thickness ratio of the tube (b/t = 38.0) was smaller than in C1~C4, which resulted in the unlikelihood of local buckling, and the test ended with the brittle failure for which the tube did not even experienced a large axial deformation.

3.2. Flexural-and-Axial Capacity

In order to examine the applicability of current design codes to CEFT columns, the peak strengths from the tests were compared with the axial load-moment relationships (P - M relationships) calculated from design codes: ACI 318-11 [4], ANSI/AISC 360-10 [5], Eurocode 4 [6], and AIJ [7]. First of all, the nominal compressive strength of the composite section can be calculated as follows.

$$P_n = 0.85 f_{ck} A_c + F_y A_s + F_{yr} A_r \tag{1}$$

where P_n = nominal compressive strength of composite section; f_{ck} = strength of concrete; F_y = yield strength of steel; F_{yr} = yield strength of re-bar; A_c = area of concrete section; A_s = area of steel section; and A_r = area of re-bar section. Any reduction factors or material factors were ignored to directly compare different design codes. In calculations, the properties of concrete, steel, and re-bar acquired by material tests were used.

According to ACI 318-11, the flexural-and-axial capacity of composite sections is calculated assuming the compressive strain at an extreme fiber of concrete as ε_{cu} = 0.003 and the linear strain distribution over the cross section, so called a strain compatibility method. Method 1 in AISC 360-10 simplifies the P - M relationship of composite sections with two linear lines assuming the equivalent steel section and the plastic stress distribution. Eurocode 4 primarily recommends the section analysis on the basis of plastic stress distribution together with a simplified method described with four performance points. AIJ standard suggests a specific method to estimate the strength of CEFT members. A composite section is divided into three different materials and the P - M relationship of components are superimposed on one another. In this superposition, the filled concrete is assumed to resist only to axial load and the contribution to bending is not considered.

Fig. 4 shows the P - M relationships at the center of specimens. The specimens with low eccentricity e = 0.125 agreed with all standards and the specimen with high eccentricity e = 0.375 also satisfied the standards except for Eurocode 4. As shown in Fig. 4, ACI 318-11 most accurately predicted the strength of the specimens. Eurocode 4 overestimated the P - M relationship at the region of high eccentricity and AISC 360-10 highly underestimated the capacity of specimens in all regions. AIJ standard was similar to ACI 318-11 while slightly underestimating the specimens with low eccentricity.

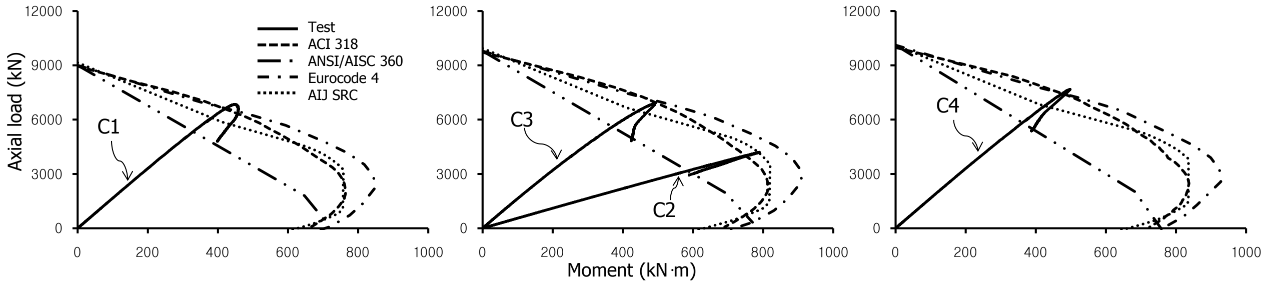

Fig. 4: Axial load-moment relationships of test specimens

4. Conclusions

In this study, eccentric and concentric axial load tests were performed to examine the flexural-and-axial capacity of CEFT columns. Summaries of the major outcomes in the present study can be described as follows.

- In C2, C3, and C5 applied with the original detail of cover PC, the load resistances sharply dropped immediately after the occurrence of compressive cracking which were accompanied by the spalling of cover PC and the local bucking of re-bars and tubes. The amounts of decrease in strengths were higher in the specimens with low eccentricity than in those with high eccentricity.
- In C1 with fiber-reinforced cover PC, a drop of load resistance was gradual and the specimen showed the best ductility in all specimens. The C4 revealed the similar behavior with specimens using the original detail of cover PC.
- ACI 318-11 using the strain-compatibility method which assumes the extreme concrete strain ε_{cu} = 0.003, safely and reasonably predicted the flexural-and-axial strengths of specimens. Method 1 in AISC360-10 largely underestimated the capacities of specimens, and Eurocode 4 which applies the plastic stress distribution slightly underestimated the test results.

5. Acknowledgements

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