Research for Practical Applications of CFT and CEFT Columns with High-Strength Steel

고강도 강재를 적용한 충전형 및 피복충전형 합성기둥의 실용화를 위한 연구

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이 호 준
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Abstract

Research for Practical Applications of CFT and CEFT Columns with High-Strength Steel

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Concrete-filled steel tubular (CFT) and concrete-encased-and-filled steel tubular (CEFT) columns have been popular in building constructions worldwide, owing to the structural advantages that come from beneficial interactions between the two materials as well as the construction efficiency with rapid erection, framework, and formwork. In recent years, to meet more complicated and advanced requirements of modern constructions, high-strength steel and concrete are increasingly utilized in practice. This thesis deals with various experimental and analytical studies on the columns and beam-column connections for practical applications of the CFT and CEFT columns especially with high-strength steel.
In practice, CFT columns with the high-strength steel are often classified as noncompact or slender section due to its high yield strength. However, the relevant design codes and research are not readily available, limiting the application of high-strength steel. Experimental programs of concentric and eccentric compression loading include rectangular slender and circular noncompact section with the steel yield strength of 746 MPa and 565 MPa, respectively. The test strengths sufficiently exceeded the predictions of current design standards and were comparable to the nominal plastic capacity. In order to upgrade the structural performance, stiffened CFT specimens are included, and a theoretical design model for the stiffener rigidity is suggested.

Currently, pushing the limitations on material strengths is a key agenda in international code revisions for steel-concrete composite columns. In particular, improved design methods of the CFT are essential for the extension since it is one the most ideal structural system for the high-strength steel. The present study aims to develop analytical and design models for assessment of the axial-flexural capacity of CFT columns incorporating various material strengths and sectional slenderness. In the case of rectangular CFTs, the effects of design parameters are investigated based on existing experimental databases, and the effective peak stress of the steel tube is defined. The proposed model is applicable to the high-strength steel addressing the influence of concrete early crushing. In addition, simplified equations for axial-flexural design of the composite section are presented for practical use. In the case of circular CFTs, a strength degradation model of confined concrete is proposed considering the effect of strain gradient. The proposed model is validated by comparison with previous test results covering the high-strength steel.

Reinforced concrete is often added to CFT columns in the lower floors to
enhance the load-carrying capacity as well as the fire resistance. Since the structural performance of such CEFT columns is highly dependent on the integrity of the concrete encasement, the role of reinforcement details is critical. In the experimental study on CEFT columns, various encasement details are tested under eccentric compression with considerations of tube shape and yield strength. The early spalling of the thin concrete encasement was not serious in all specimens, and the ductility was significantly increased by the high-strength circular tube. Based on the experimental results, the applicability of current design codes in terms of strength and flexural stiffness is investigated.

In steel beam-CFT column joints, semi-rigid connection or simple welding without stiffening plates is preferable from the viewpoint of economy. In the case of CEFT columns, however, because the out-of-plane distortion of the high-strength steel tube may be detrimental to the concrete encasement, it is important to control the wall deformation. Thus, the experimental study on steel beam-CEFT column joints focuses on the flexural behavior of the connection. A set of monotonic tension tests on flange plate-to-column connections is conducted, in which the punching shear failure occurred. The peak loads are compared with predictions of yield line models while the local load-distortion relationships are simulated by 3D finite element analysis. Cyclic flexure tests are then carried out on exterior beam-column joints with square or circular tubes. Based on the test results, failure modes and flexural resistance of the connections are investigated.

Owing to the presence of concrete encasement, connecting concrete beams to the CEFT column is easier than to the CFT column. Practical concerns for the field application though include the tube penetration, splice, and anchorage of beam flexural rebars. In this regard, connection schemes such as through-beam type with
thin-wall opening and interrupted-beam type with coupler splice are suggested, considering the constructability and seismic performance, respectively. The associated structural behavior is examined through cyclic loading tests for interior and exterior beam-column joints.

**Keywords**: concrete-filled steel tube (CFT); concrete-encased-and-filled steel tube (CEFT); high-strength steel; noncompact section; slender section; stiffener; local buckling; concrete lateral confinement; axial-flexural strength; composite beam-column connection; concrete beam

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<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_c$</td>
<td>Area of concrete, mm$^2$</td>
</tr>
<tr>
<td>$A_{ce}$</td>
<td>Area of the concrete encasement, mm$^2$</td>
</tr>
<tr>
<td>$A_{cf}$</td>
<td>Area of the concrete infill, mm$^2$</td>
</tr>
<tr>
<td>$A_g$</td>
<td>Area of gross section, mm$^2$</td>
</tr>
<tr>
<td>$A_h$</td>
<td>Area of transverse reinforcement, mm$^2$</td>
</tr>
<tr>
<td>$A_r$</td>
<td>Area of the longitudinal reinforcement, mm$^2$</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$A_{r1}$</td>
<td>Area of the top longitudinal reinforcement, mm$^2$</td>
</tr>
<tr>
<td>$A_{r2}$</td>
<td>Area of the bottom longitudinal reinforcement, mm$^2$</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Area of steel, mm$^2$</td>
</tr>
<tr>
<td>$A_{ss}$</td>
<td>Area of vertical stiffener, mm$^2$</td>
</tr>
<tr>
<td>$a$</td>
<td>Depth of compressive stress block, mm</td>
</tr>
<tr>
<td>$B$</td>
<td>Outer width of the rectangular tube, mm</td>
</tr>
<tr>
<td>$b$</td>
<td>Inner width of the rectangular tube = $B-2t_w$, also for the cold-formed section if not specified, mm</td>
</tr>
<tr>
<td>$b_b$</td>
<td>Width of the concrete beam, mm</td>
</tr>
<tr>
<td>$b_c$</td>
<td>Width of the concrete-encased steel column, mm</td>
</tr>
<tr>
<td>$b_f$</td>
<td>Width of the beam flange, mm</td>
</tr>
<tr>
<td>$b_s$</td>
<td>Width of the vertical stiffener, mm</td>
</tr>
<tr>
<td>$c$</td>
<td>Depth of neutral axial in member section, mm</td>
</tr>
<tr>
<td>$D$</td>
<td>Outer diameter of the circular tube, mm</td>
</tr>
<tr>
<td>$d$</td>
<td>Depth of the steel or concrete beam, mm</td>
</tr>
<tr>
<td>$d_b$</td>
<td>Bar diameter, mm</td>
</tr>
<tr>
<td>$d_f$</td>
<td>Flange center-to-center depth of the steel beam, mm</td>
</tr>
<tr>
<td>$E_c$</td>
<td>Modulus of elasticity of concrete, MPa</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Modulus of elasticity of steel = 200,000 MPa</td>
</tr>
<tr>
<td>$EI_{eff}$</td>
<td>Effective stiffness of composite section, N·mm</td>
</tr>
<tr>
<td>$e$</td>
<td>Axial-load eccentricity, mm</td>
</tr>
<tr>
<td>$F_{cr}$</td>
<td>Theoretical local buckling stress = $9E_s(b/t)^2$, MPa</td>
</tr>
</tbody>
</table>
List of Symbols

\( F_u \) Tensile strength of steel, tube in general, MPa

\( F_{uc} \) Tensile strength of the circular tube in circumferential direction, MPa

\( F_{uf} \) Tensile strength of the flange plate, MPa

\( F_{ul} \) Tensile strength of the circular tube in longitudinal direction, MPa

\( F_{ur} \) Tensile strength of the longitudinal reinforcement in composite column, MPa

\( F_{us} \) Tensile strength of the vertical stiffener, MPa

\( F_{uw} \) Tensile strength of the web plate, MPa

\( F_y \) Yield strength of steel, tube in general, MPa

\( F_{yc} \) Yield strength of the circular tube in circumferential direction, MPa

\( F_{yf} \) Yield strength of the flange plate, MPa

\( F_{yl} \) Yield strength of the circular tube in longitudinal direction, MPa

\( F_{yr} \) Yield strength of the longitudinal reinforcement in composite column, MPa

\( F_{ys} \) Yield strength of the vertical stiffener, MPa

\( F_{yw} \) Yield strength of the web plate, MPa

\( f'_c \) Compressive strength of concrete, MPa.

\( f'_{cc} \) Compressive strength of confined concrete, MPa

\( f'_{cce} \) Compressive strength of confined concrete with eccentricity, MPa

\( f'_{ccn} \) Compressive strength of confined concrete with axial-load
List of Symbols

- ratio, MPa
- $f_{ce}'$: Compressive strength of the concrete encasement, MPa
- $f_{cf}'$: Compressive strength of the concrete infill, MPa
- $f_y$: Yield strength of reinforcement, MPa
- $f_{yh}$: Yield strength of hoop and crosstie reinforcement, MPa
- $H$: Outer depth of the rectangular tube, mm
- $h$: Inner depth of the rectangular tube, mm
- $I_c$: Moment of inertia of the concrete section about the elastic neutral axis of the composite section, mm$^4$
- $I_r$: Moment of inertia of reinforcing bars about the elastic neutral axis of the composite section, mm$^4$
- $I_s$: Moment of inertia of steel shape about the elastic neutral axis of the composite section, mm$^4$
- $h_c$: Depth of the concrete-encased steel column, mm
- $L_b$: Length of the beam in beam-column joint, mm
- $L_{b0}$: Length between the beam tip and column face, mm
- $L_c$: Length of the column in beam-column joint, defined as the net column length in compression test, mm
- $L_e$: Effective length of the column in compression test, mm
- $M_b$: Joint moment cause by the beam shear force, N·mm
- $M_{bcr}$: Moment at the critical section cause by the beam shear force, N·mm
- $M_c$: Joint moment cause by the column shear force, N·mm
- $M_{bp}$: Plastic moment capacity of the steel beam, N·mm
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_{by}$</td>
<td>Yield moment capacity of the steel or concrete beam, N·mm</td>
</tr>
<tr>
<td>$M_{b_{\text{test}}}$</td>
<td>Maximum joint moment that occurred in the beam-column joint testing, N·mm</td>
</tr>
<tr>
<td>$M_p$</td>
<td>Plastic moment capacity of the composite section, N·mm</td>
</tr>
<tr>
<td>$n$</td>
<td>Axial load ratio</td>
</tr>
<tr>
<td>$P$</td>
<td>Compression force, N</td>
</tr>
<tr>
<td>$P_p$</td>
<td>Plastic strength of composite section, N</td>
</tr>
<tr>
<td>$P_{cr}$</td>
<td>Critical buckling load of column, N</td>
</tr>
<tr>
<td>$P_{no}$</td>
<td>Nominal strength of composite section, $P_p$ in case of compact section, N</td>
</tr>
<tr>
<td>$P_{\text{test}}$</td>
<td>Maximum load in compression test, N</td>
</tr>
<tr>
<td>$s$</td>
<td>Spacing of transverse reinforcement, mm</td>
</tr>
<tr>
<td>$T$</td>
<td>Tensile force on the flange-tube connection, N</td>
</tr>
<tr>
<td>$T_{PS}$</td>
<td>Tensile strength based on the CIDECT punching shear model, N</td>
</tr>
<tr>
<td>$T_{\text{test}}$</td>
<td>Maximum load in tension test, N</td>
</tr>
<tr>
<td>$T_u$</td>
<td>Ultimate tensile strength of the connection, N</td>
</tr>
<tr>
<td>$T_{YL}$</td>
<td>Tensile strength based on the yield line model, N</td>
</tr>
<tr>
<td>$t$</td>
<td>Thickness of the tube plate, mm</td>
</tr>
<tr>
<td>$t_e$</td>
<td>Thickness of the concrete encasement, mm</td>
</tr>
<tr>
<td>$t_f$</td>
<td>Thickness of the flange plate, mm</td>
</tr>
<tr>
<td>$t_r$</td>
<td>Total thickness of the flange plate and weld metal, mm</td>
</tr>
<tr>
<td>$t_s$</td>
<td>Thickness of the vertical stiffness, mm</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>---------</td>
<td>------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$t_w$</td>
<td>Thickness of the web plate, mm</td>
</tr>
<tr>
<td>$V_b$</td>
<td>Beam shear force, N</td>
</tr>
<tr>
<td>$V_{by}$</td>
<td>Beam shear force corresponding to the beam yield moment, N</td>
</tr>
<tr>
<td>$V_{bp}$</td>
<td>Beam shear force corresponding to the beam plastic moment, N</td>
</tr>
<tr>
<td>$V_{b\text{rest}}$</td>
<td>Peak beam shear force in the test of beam-column joint, N</td>
</tr>
<tr>
<td>$V_c$</td>
<td>Column shear force, N</td>
</tr>
<tr>
<td>$V_{cnb}$</td>
<td>Column shear force corresponding to the beam flexural failure, N</td>
</tr>
<tr>
<td>$V_{cnj}$</td>
<td>Column shear force corresponding to the joint shear failure, N</td>
</tr>
<tr>
<td>$V_{c\text{test}}$</td>
<td>Peak column shear force in the test of beam-column joint, N</td>
</tr>
<tr>
<td>$V_{nc}$</td>
<td>Shear strength of the joint concrete, N</td>
</tr>
<tr>
<td>$V_{ns}$</td>
<td>Shear strength of the joint panel web, N</td>
</tr>
<tr>
<td>$V_j$</td>
<td>Joint shear force, N</td>
</tr>
<tr>
<td>$w$</td>
<td>Width of the subpanel, mm</td>
</tr>
<tr>
<td>$X$</td>
<td>Vertical length between out-of-plane bending yield lines, mm</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Strength reduction factor for the high-strength concrete</td>
</tr>
<tr>
<td>$\beta_1 F_y$</td>
<td>Local buckling strength accounting for the post-buckling reserve, MPa</td>
</tr>
<tr>
<td>$\beta_2$</td>
<td>Strength reduction factor accounting for the early crushing of concrete infill</td>
</tr>
<tr>
<td>$\Delta$</td>
<td>Displacement, lateral deflection in the eccentric compression test, mm</td>
</tr>
<tr>
<td>$\Delta_y$</td>
<td>Yield displacement, mm</td>
</tr>
</tbody>
</table>
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<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta_u$</td>
<td>Ultimate displacement, mm</td>
</tr>
<tr>
<td>$\delta$</td>
<td>Local tensile deformation of the flange-tube connection, mm</td>
</tr>
<tr>
<td>$\delta_e$</td>
<td>Axial-load contribution of the concrete encasement</td>
</tr>
<tr>
<td>$\delta_s$</td>
<td>Axial-load contribution of the steel section</td>
</tr>
<tr>
<td>$\varepsilon$</td>
<td>Axial strain, mm/mm</td>
</tr>
<tr>
<td>$\varepsilon_c$</td>
<td>Compressive strain of concrete, mm/mm</td>
</tr>
<tr>
<td>$\varepsilon_{co}$</td>
<td>Concrete compressive strain corresponding to $f_{c'}$, mm/mm</td>
</tr>
<tr>
<td>$\varepsilon_{cr}$</td>
<td>Theoretical local buckling strain = $F_{cr}/E_s$, mm/mm</td>
</tr>
<tr>
<td>$\varepsilon_{cu}$</td>
<td>Ultimate compressive strain of unconfined concrete, mm/mm</td>
</tr>
<tr>
<td>$\varepsilon_{ini}$</td>
<td>Initial local buckling strain, mm/mm</td>
</tr>
<tr>
<td>$\varepsilon_y$</td>
<td>Yield strain of steel = $F_y/E_s$, mm/mm</td>
</tr>
<tr>
<td>$\eta$</td>
<td>Stress reduction factor accounting for the depth of neutral axis</td>
</tr>
<tr>
<td>$\lambda_B$</td>
<td>Normalized sectional slenderness with respect to outer width = $B/t\sqrt{(F_y/E_s)}$, mm/mm</td>
</tr>
<tr>
<td>$\lambda_{coeff}$</td>
<td>Normalized sectional slenderness or slenderness coefficient, $(b/t)\sqrt{(F_y/E_s)}$ in case of the rectangular tube, mm/mm</td>
</tr>
<tr>
<td>$\lambda_c$</td>
<td>Column slenderness ratio</td>
</tr>
<tr>
<td>$\lambda_p$</td>
<td>With-to-thickness ratio limit dividing the compact and noncompact section (AISC 2016), mm/mm</td>
</tr>
<tr>
<td>$\lambda_r$</td>
<td>Width-to-thickness ratio limit dividing the noncompact and slender section (AISC 2016), mm/mm</td>
</tr>
<tr>
<td>$\mu_\Delta$</td>
<td>Displacement ductility</td>
</tr>
<tr>
<td>$\mu_\phi$</td>
<td>Curvature ductility</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$\rho_h$</td>
<td>Area ratio of transverse reinforcement</td>
</tr>
<tr>
<td>$\rho_t$</td>
<td>Volumetric ratio of the steel tube</td>
</tr>
<tr>
<td>$\sigma_c$</td>
<td>Compressive stress of concrete, MPa</td>
</tr>
<tr>
<td>$\phi_y$</td>
<td>Yield curvature, 1/mm</td>
</tr>
<tr>
<td>$\phi_u$</td>
<td>Ultimate curvature, 1/mm</td>
</tr>
</tbody>
</table>
Chapter 1. Introduction

1.1 Brief Introduction of Composite Tubular Columns

1.1.1 Various Applications in Practice

Concrete-filled steel tubular (CFT) and concrete-encased-and-filled steel tubular (CEFT) columns have been popular in building constructions worldwide, owing to the structural advantages that come from beneficial interactions between the two materials as well as the construction efficiency with rapid erection, framework, and formwork.

Fig. 1-1 shows typical applications of composite tubular columns. In CEFT columns [Fig. 1-1(a)], local buckling of the CFT is further constrained by the concrete encasement, and the fire resistance is superior. Also, the composite column is easily connected with reinforced concrete beams as well as steel beams (Han et al. 2009; Liao et al. 2014). Double-skin concrete-filled steel tubes are featured by the configuration that maximizes flexural stiffness while saving materials and is suitable for large cross-sections [Fig. 1-1(b)]. Fig. 1-1(c) shows CFT sections strengthened with reinforcing bars (Arai et al. 2003; Shirai et al. 2014). This composite system enhances the economy of ordinary CFTs by reducing the tube thickness (O’Shea and Bridge 2000) and ensures ductile behavior of the concrete infill, which may be susceptible to brittle failure in the real-scale structure due to the plainness (Xiamuxi et al. 2011). The steel tube only needs to be thick enough to withstand the construction load before concrete casting. The CFTs with stiffeners, as illustrated in
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Fig. 1-1(d), exhibit the improved strength and ductility by delaying the local buckling of steel plates (Tao et al. 2005). Such columns are expected to be used for thin-walled steel tubes of mega columns or high-strength steels (Han et al. 2014).

Fig. 1-1. Various sectional configurations of composite tubular columns
1.1.2 Relevant Design Codes

This section briefly introduces current design standards for composite columns and beam-column joints with respect to structural steel tubes. More detailed information and related studies on specific structural systems will be introduced in Chapter 2.

1.1.2.1 Composite Tubular Columns

Steel-concrete composite columns are practically divided into concrete-encased steel (CES) columns with I-shaped steel section and CFT columns with square or circular steel tubes (AISC 2016; CEN 2004). The ANSI/AISC 360-16 (AISC 2016) is characterized by defining three section categories for CFTs, i.e., compact, noncompact, and slender sections. Eurocode 4 (CEN 2004) conservatively limits the width-to-thickness ratio to prevent early local buckling, allowing the rigid-plastic design of the composite section. On the other hand, more specific reinforcing details are provided for CES columns along with provisions for partially-encased I-sections. Design standard for composite structures of Japan (AIJ 2014a) is comprised of the one for steel reinforced concrete (SRC, identical to CES) structures (AIJ Standard for Structural Calculation of Steel Reinforced Concrete Structures, AIJ 2014b) and the one for CFT structures (Recommendations for Design and Construction of Concrete Filled Steel Tubular Structures, AIJ 2008). CEFTs are treated as SRC structures. In the recommendations for CFT structures (AIJ 2008), material models for the analytical approach are also provided. Although current standards provide a basic framework of design for ordinary CFT columns with the
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compact section, they are not readily applicable to thin-walled steel tubes of various cross-sectional configurations.

1.1.2.2 Composite Beam-Column Joints

Depending on the types of beams and columns, various connections details are available. Consequently, it is often difficult to standardize them in design codes. ANSI/AISC 341-16 (AISC 2016) deals with beam-column joints using reinforced concrete (RC), CES, and CFT columns, introducing typical connection details and relevant research. In particular, the provisions are featured by classification of moment-frame systems (i.e., ordinary, intermediate, and special) with specified seismic performance criteria regarding to story drift ratio. In Eurocode 4 (CEN 2004), the joint is categorized into either pinned, rigid, or semi-rigid (Eurocode 3, CEN 2005), and the component-based approach is adopted to analytically predict stiffness, strength, and rotation capacity. However, component models are standardized mainly for steel and partially-encased steel columns. In Japan, two major guidelines (Design and Construction of Mixed Structures Composed of Reinforced Concrete Columns and Steel Beams, AIJ 2001; AIJ 2008) are recommended for beam-column joints using RC and CFT columns, respectively (AIJ 2014a). CIDECT also publishes the design guide for tubular column connections (No. 9, Kurobane et al. 2004). The design specifications for composite beam-column joints generally cover steel or composite beams, not mentioning the connectivity with concrete beams. In addition, even in the case of the connections with steel beams, the validity of existing design methods is not assured for thin-walled steel tubes.
1.2 Research Backgrounds and Motivations

1.2.1 Applicability of High-Strength Materials

Recently, in a variety of special construction projects such as high-rise buildings and large-span structures, the use of high-strength steel and/or concrete often arises as a solution to realize the light-weight construction and to create greater architectural space. In the field of composite constructions, early applications of the high-strength materials have been found in CFT columns (Fig. 1-2). Since the CFT optimizes the interaction between steel and concrete, it is considered one of the most suitable structural systems for the new materials. Based on the results of numerous academic efforts and several field applications, some of the recently-revised design codes (AIK 2016; AS 2017) have narrowly allowed the use of high-strength materials (Table 1-1). Although ANSI/AISC 360 (AISC 2016) and Eurocode 4 (CEN 2004), which are the two major axes of composite structural standards, are still conservative on the extension, it is certain from the relevant studies, which are now carefully underway (Hajjar 2017; Varma and Lai 2017), that the issue is one of the main agenda items in the next revision.

In the AISC standards, CFT members are classified into compact, noncompact, and slender sections depending on cross-sectional aspect ratios of the wall, and different design equations are suggested for each type. In the case of high-strength steel, one of the reasons for the limited application is that there is not enough assurance of the structural behavior in the wide range of sectional slenderness. Even in the case of mild steels, the strength predictions for the slender section are significantly conservative when compared with the compact section. On the other
hand, if the yield strength of the steel increases with the same width-to-thickness ratio, the slenderness category of the section may be degraded (Fig. 1-3). Such situations are expected to occur frequently in practice since the thickness of the steel plates needs to be decreased for the economy. Therefore, in order to expand the applicability of high-strength steels in the current design standards, it is essential to clarify the behavioral characteristics accounting for various sectional slenderness as well as material grades, and rational analysis and design models need to be developed.

![Diagram showing steel yield strength and concrete strength for various buildings](image)

Fig. 1-2. Field applications of high-strength materials in CFT columns
Fig. 1-3. Relationships between steel grades and width-to-thickness ratio limits (AISC 2016)

Table 1-1. Material strength limits in current design codes

<table>
<thead>
<tr>
<th>Design codes</th>
<th>Steel yield strength, $F_y$ (MPa)</th>
<th>Concrete compressive strength, $f_c'$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>AIJ (2008)</td>
<td>235 ~ 440</td>
<td>18 ~ 90</td>
</tr>
<tr>
<td>ANSI/AISC 360 (2016)</td>
<td>~ 525</td>
<td>21 ~ 69</td>
</tr>
<tr>
<td>KBC (2016)</td>
<td>~ 650</td>
<td>21 ~ 70</td>
</tr>
<tr>
<td>AS/NZS 5100.6 (2017)</td>
<td>230 ~ 690</td>
<td>25 ~ 100</td>
</tr>
</tbody>
</table>
1.2.2 Hollow Precast Concrete Construction

The use of precast concrete (PC) structures have been steadily growing in the construction market because of the advantages of excellent concrete quality and savings in construction time and cost. In recent years, the demand has extended to diverse large commercial and industrial buildings such as wholesale stores, warehouse buildings, high-tech plants, and power plants, which have high stories and long spans to accommodate equipment and mechanical tubes. These facilities usually require a high level of deflection and vibration control along with a large gravity load, which inevitably leads to the enlargement of PC columns. However, the use of conventional PC methods may not be advantageous because of difficulties in the transportation and lifting of such large-weight columns.

Alternatively, hollow PC columns, with reduced weight, can be used for the construction of such large columns. After erecting the hollow PC column on the construction site, concrete can be filled into the hollow core (Fig. 1-4). In manufacturing, however, due to the requirement of cross-ties, it is difficult to install and remove the inner form that is required to form the hollow section. Considering the difficulty of the use of an inner form, a thin-walled steel tube can be used for the hollow PC column as the permanent mold. In the concrete-encased-and-filled tubular column (CEFT column), cross-ties are not necessary, and the thin steel tube can be used as a structural element, resisting member forces. Furthermore, the concrete encasement can develop additional strength and stiffness, providing local-buckling restraint and fire resistance to the steel tube (Xu and Liu 2013).

In order to fully utilize the economy of the hollow PC method, the proposed
CEFT column needs to be detailed with thinner concrete shell and tube wall than used in CEFT columns with conventional configurations. In such conditions, the concrete encasement may be vulnerable to early delamination under axial compression, and the connection between the tube wall and steel beams may be susceptible to out-of-plane deformation under beam flexure. Thus, appropriate reinforcing details for integrity and robustness of the CEFT column are necessary. In addition, owing to the presence of concrete encasement to the steel tube, mixing with either RC or PC beams is a feasible option, and the development of relevant connection details is required.

Fig. 1-4. Concept of proposed hollow precast concrete construction

Thin-walled steel tube
Longitudinal rebars
Steel bracket
PC encasement
CIP concrete infill
1.3 Research Scopes and Objectives

In this thesis, CFT and CEFT columns are studied within the framework of the practical aspects mentioned in section 1.2. Research for CFT columns is focused on the application of high-strength steels and the associated design issues in rectangular and circular sections. In the study of CEFT columns, effects of the high-strength steel tube as well as reinforcing details of thin concrete encasement are investigated. For beam-column joints of the composite tubular columns, connectivity with steel and concrete beams is considered. Specific research scopes and objectives are summarized as follows.

1.3.1 CFT Columns

In practice, CFT columns with the high-strength steel are often classified as noncompact or slender section due to its high yield strength. However, the relevant design codes and research are not readily available, limiting the application of high-strength steel. Experimental programs of concentric and eccentric compression loading include rectangular slender and circular noncompact section with the steel yield strength of 746 MPa and 565 MPa, respectively. In the case of rectangular columns, strengthening the slender section with vertical stiffeners was considered to upgrade the section and utilize the full plastic capacity.

Currently, pushing the limitations on material strengths is a key agenda in international code revisions for steel-concrete composite columns. In particular, improved design methods of the CFT are essential for the extension since it is one
the most ideal structural system for the high-strength steel. The present study aims to develop analytical and design models for assessment of the axial-flexural capacity of CFT columns incorporating high-strength materials and slender section. First, existing experimental databases were compiled and applicability of current design methods are investigated. Generally, the structural behavior of rectangular CFTs is featured by the tube local buckling while the hoop tension developed in steel tubes and the resultant concrete confinement are crucial in circular CFTs (AIJ 2008). Based on the characteristics, improved fiber-based models which are essential for the high-strength steels are proposed.

1.3.2 CEFT Columns

Reinforced concrete is often added to CFT columns in the lower floors to enhance the load-carrying capacity as well as the fire resistance. The structural characteristics of such CEFT columns are generally similar to those of conventional CES columns with the I-shaped section. Although satisfactory ductility is expected due to the effect of the inner CFT, since the outer concrete encasement and inner concrete infill are completely separated from each other, early delamination of the outer concrete may happen, leading to the over brittle behavior. In the experimental study on CEFT columns, various encasement details are tested under eccentric compression with considerations of tube shape and yield strength. Based on the experimental results, the applicability of current design codes and analytical estimation in terms of load-carrying capacity and flexural stiffness are investigated.

In steel beam-CFT column joints, semi-rigid connection or simple welding without stiffening plates is preferable from the viewpoint of economy. When
Chapter 1. Introduction

Concrete encasement is added to the composite column, the out-of-plane deformation of the high-strength steel tube may cause the unfavorable concrete cracking. Although the relevant design procedures are well-documented for the CFT structures, studies on the connections using CEFT columns are limited. Thus, the experimental study on steel beam-CEFT column joints focuses on the flexural behavior of the connection. For this purpose, a set of monotonic tension tests on flange-tube connections and cyclic load tests on beam-column exterior joints was carried out. On the basis of the results, strength evaluation using design equations and distortion assessment using 3D FEA simulation are demonstrated.

Although not common, RC beams are connected to CFT columns to enhance the stiffness of the overall structure (Arimatsu et al. 2005). If concrete encasement is available, the CEFT column is more easily mixed with RC beams, in which the beam-column joint can be designed following traditional RC moment frames (Han et al. 2014). Similarly, owing to the presence of PC encasement to the steel tube, PC beams are potentially connected to the CEFT column in the hollow PC construction. Practical concerns for the field application include tube penetration, splice, and anchorage of beam flexural rebars. In this study, connections details such as tube wall opening and coupler splice are applied, considering constructability and earthquake resistance of the connection, respectively. The opening details are feasible because the steel is relatively thin. The seismic performance of the interior and exterior beam-column joints is evaluated through cyclic load testing.
1.4 Thesis Organization

This thesis deals with the structural performance of CFT and CEFT columns with high-strength steel and their applications to beam-column joints (Fig. 1-5). Chapters 3, 4, and 5 mainly focus on the axial-flexural load-carrying capacity of CFT and CEFT columns. For the beam-column joint, the connectivity with steel and concrete beams is considered (Chapters 6 and 7). Specific research objectives of each chapter are summarized as follows.

In Chapter 3, an experimental study for rectangular CFT columns is presented. The study focuses on the evaluation of the axial load-carrying capacity of CFT columns with the high-strength steel slender section. The test parameters include the yield strength of steel, width-to-thickness ratio of steel plates, axial-load eccentricity, and the use of stiffeners. On the basis of the results, the applicability of the current design codes to high-strength steel slender sections is evaluated. To verify the behaviors of steel local buckling and concrete confinement, nonlinear finite element analysis is performed on the specimens. Further, a design method of the vertical stiffener is developed for thin-walled CFT columns.

In order to expand the applicability of high-strength steels in the current design standards, it is essential to clarify the behavioral characteristics accounting for various sectional slenderness as well as material grades. Chapter 4 deals with the assessment of the load-carrying capacities of CFT columns and beam-columns. In particular, careful attention is paid to the effects of not only high-strength materials but also slender tube section, which have often not been addressed in previous
models. First, the effects of design parameters are examined based on extensive databases from literatures. On the basis of the results, the effective peak stress of the rectangular tube is proposed addressing the early concrete crushing with the high-strength steel. For the strength estimation of circular CFTs, a strength degradation model of confined concrete is proposed addressing the effect of axial-load ratio.

In Chapter 5, an experimental study is performed to investigate the axial-flexural load-carrying capacity of thin-walled CFT columns constrained with the concrete encasement (CEFT). Test parameters include tube types (square mild or circular high-strength), reinforcing details, axial-load eccentricity, and the use of concrete encasement. In particular, considering the construction efficiency and structural performance, various reinforcing details for the concrete encasement are studied. The nonlinear behaviors of the specimens are simulated by fiber model analysis addressing the effect of the thin concrete encasement. The applicability of current design codes to the CEFT columns is also evaluated in terms of axial-flexural strength and flexural stiffness.

In Chapter 6, to investigate the flexural behavior of connections between the thin-walled tubular column and steel beam, two types of tests were performed: (1) monotonic tension test on flange-column (CFT and CEFT) connections and (2) cyclic load test on exterior beam-column (CEFT) joints. In flange-column tension test, test parameters were concrete encasement and connection details: increased flange width and tension bars for strengthening. Five specimens were tested to investigate the load-carrying capacity and the failure mode. The experimental results are evaluated by the yield line model and 3D finite element analysis. In cyclic loading test, five exterior beam-column joints were prepared applying square and circular steel tubes. Test parameters were strengthening details such as continuity
plate, tension bars, thickened steel tube, and vertical plate connection.

To evaluate the seismic performance of precast concrete beam-to-CEFT column connections, cyclic load tests were performed (Chapter 7). For penetration of beam flexural rebars through the column tube, connection schemes such as tube wall opening and coupler splice are applied, considering constructability and earthquake resistance, respectively. Four interior and two exterior beam-column joints were tested. The seismic performance of the specimens is evaluated based on the requirements of ACI 374.1-05.
Chapter 2. Literature Review

2.1 Code Provisions for CFT Columns

2.1.1 Limitations and Axial-Flexural Design

Material strength limitations for composite structures are shown in Table 2-1 (CEN 2004; AIJ 2008; AISC 2016). In ANSI/AISC 360 (AISC 2016) and Eurocode 4 (CEN 2004), the compressive strength of the concrete is limited to 70 MPa and 50 MPa, and the yield strength of the steel is limited to 525 MPa and 460 MPa, respectively.

In ANSI/AISC 360, the sectional class of the steel tube is classified into compact \((b/l \leq \lambda_p)\), non-compact \((\lambda_p < b/l \leq \lambda_s)\), and slender \((b/l > \lambda_s)\) sections (Table 2-1). For non-compact and slender sections, the theoretical elastic buckling stress \(F_{cr}\) is used in the design. In Eurocode 4, only compact section is allowed. In the recommendations of the Architectural Institute of Japan (AIJ 2008), a strength degradation factor \(\gamma_s\) is used to account for the local buckling.

Effective material strengths and stress distribution assumed in the cross-section design are also summarized in Table 2-1. Fig. 2-1 illustrates typical rigid-plastic analysis for the cross-sectional design of compact RCFTs. Although the compact section is not uniquely defined in the various international standards, all design codes allow the use of yield strength \(F_y\). Strength factor \(\gamma\) only for infill concrete is different. The AISC code uses \(\gamma = 0.85\) for ordinary reinforced concrete columns, while the Eurocode 4 uses \(\gamma = 1.0\) addressing the lateral confinement effect.
In Eurocode, the slenderness limit for the compact section is stricter than in AISC code. In the AIJ recommendations, a reduction factor $\gamma_c$ addressing the scale effect (Sakino et al. 2004) is used. These factors are generally consistent in the design of axial strength and combined axial-flexural strength.

For slender sections in AISC specifications, a strength reduction factor for the infill concrete is defined as 0.7, addressing the poor lateral confinement. The steel compressive stress is replaced with $F_{cr}$. Also, instead of uniform stress block, triangular stress distribution is assumed for conservative design (AISC 2010). In the case of noncompact sections, the nominal strength is calculated by interpolating the strengths for compact and slender sections. For the construction of $P$-$M$ interaction relationships, a bi-linear model connecting the nominal axial and flexural strengths is recommended (AISC 2016).

In the axial-flexural design of AIJ recommendations, the steel compressive stress is replaced with $\gamma_s F_y$. The effective strength and depth of the concrete stress block, are reduced when either high-strength concrete ($f'_c > 60$ MPa) or large width-to-thickness ratio $[B/t > 2.4\sqrt{(E_s / F_y)}]$ is used (Nakahara and Sakino 2003). Such provision accounts for the brittle behavior of the high-strength concrete or poor confinement of slender tube section.
Chapter 2. Literature Review

In ANSI/AISC 360-10, the nominal compressive strength of RCFT columns with the compact section is defined as follows.

\[ P_{AISC} = P_p = A_c (0.85 f_c') + A_s F_y \]  \hspace{1cm} (2-1)

where \( P_p \) = plastic strength of the composite section; \( A_c \) and \( A_s \) = cross-sectional area of the concrete and steel tube; \( f_c' \) and \( F_y \) = concrete compressive strength and tube yield strength. On the other hand, the nominal strength of the slender section is defined as follows.

\[ P_{AISC} = A_c (0.7 f_c') + A_s F_{cr} \]  \hspace{1cm} (2-2)

where \( F_{cr} \) = elastic local buckling stress. For noncompact sections, the nominal strength is defined by interpolating the values from Eqs. (2-1) and (2-2).
Table 2-1. Structural provisions for RCFT columns

<table>
<thead>
<tr>
<th>Provisions</th>
<th>Eurocode 4</th>
<th>AIJ (CFT guidelines)</th>
<th>ANSI/AISC 360-16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material strengths</td>
<td>$235 \leq F_y \leq 460 \text{ MPa}$</td>
<td>$235 \leq F_y \leq 440 \text{ MPa}$</td>
<td>$F_y \leq 525 \text{ MPa}$</td>
</tr>
<tr>
<td></td>
<td>$20 \leq f_c' \leq 50 \text{ MPa}$</td>
<td>$18 \leq f_c' \leq 90 \text{ MPa}$</td>
<td>$21 \leq f_c' \leq 69 \text{ MPa}$</td>
</tr>
</tbody>
</table>
| Width-to-thickness ratio    | $B/t \leq 1.78\sqrt{(E_s/F_y)}$ | $B/t \leq 2.44\sqrt{(E_s/F_y)}^c$ | Compact: $b/t \leq 2.26\sqrt{(E_s/F_y)} = \lambda_p$
|                             |                             | Non-compact: $b/t \leq 3.00\sqrt{(E_s/F_y)} = \lambda_r$
|                             |                             | Slender: $b/t \leq 5.00\sqrt{(E_s/F_y)}$ |                             |
| Effective material strengths| Concrete: $1.0f_c'$         | Concrete: $\gamma_f f_c'$ | Concrete: $0.85f_c'$ (compact) |
|                             | Steel: $F_y$                | Steel: $\gamma_s F_y$   | $0.7f_c'$ (slender) |
| Stress block$^a$            | Rectangle                   | Rectangle$^d$          | Rectangle (compact) |
|                             |                             |                       | Triangle (slender)$^f$ |

$^a$ Assumed stress block for cross-section design, effective steel strength for compression and $F_y$ for tension

$^b$ Size effect considered for evaluation of test results, $\gamma_c = 1.67D^{-0.112}$ where $D = 2B/\pi$

$^c$ Local buckling considered for $B/t > 2.44\sqrt{(E_s/F_y)}$, $\gamma_s = 1/[0.698+0.128(B/t)^2(F_y/E_s)4.00/6.97]$

$^d$ Effective stress and depth of concrete are reduced if $f_c' > 60 \text{ MPa}$ MPa or $B/t > 2.4\sqrt{(E_s/F_y)}$

$^e$ Critical buckling stress, $F_{cr} = 9E_s/(b/t)^2$

$^f$ Triangular stress distribution assumed for compression
In the AISC predictions, as the strength reduction factor of the infill concrete, 0.85 and 0.7 are used for compact sections and slender sections, respectively. Since the confinement effect is negligible in RCFT columns, the strength factor of 0.85 is used in Eq. (2-1), following the factor of ordinary reinforced concrete columns (ACI 2014). In Eq. (2-2), the smaller factor 0.7 is used to address the effect of local buckling of steel tube (AISC 2010; AISC 2016). The conservative assessment of the concrete contribution is offset by overestimating the contribution of steel [i.e., using the full yield strength \( F_y \) for compact sections and theoretical buckling strength \( F_{cr} \) for slender sections (Lai et al. 2015)].

In the AIJ recommendations, the compressive strength of CFT columns is defined as follows:

\[
P_{Alj} = A_c \left( \gamma_c, f'_c \right) + A_s \left( \gamma_s, F_y \right)
\]

(2-3)

where \( \gamma_c \) and \( \gamma_s \) refer to the material reduction factors for the concrete and steel, respectively. \( \gamma_c \) (= 1.67\(D^{-0.112}\), where \( D = 2B/\sqrt{\pi} \)) is used to address the size effect of concrete (Sakino et al. 2004). The reduction factor \( \gamma_s \) that accounts for the local buckling of the steel tube is defined as follows:

\[
\gamma_s = \frac{1}{0.698 + 0.128 \left( \frac{B}{t} \right)^2 \left( \frac{F_y}{E_s} \right) \frac{4.00}{6.97}}
\]

(2-4)

for \( B/t > 1.5 \frac{23}{\sqrt{F_y/1000}} = 2.44 \sqrt{\frac{E_s}{F_y}} \)
2.1.2 Section Classification

In ANSI/AISC 360 (AISC 2010; 2016), the sectional slenderness for RCFT columns is classified into three categories: compact \((b/t < \lambda_p)\), non-compact \((\lambda_p < b/t < \lambda_r)\), and slender \((b/t > \lambda_r)\) sections, where \(b\) = maximum inner width of the steel tube and \(t\) = thickness of the tube plate. The slenderness limits \(\lambda_p\) and \(\lambda_r\) are defined as follows:

\[
\lambda_p = 2.26 \sqrt{\frac{E_s}{F_y}} \quad (2-5)
\]

\[
\lambda_r = 3.00 \sqrt{\frac{E_s}{F_y}} \quad (2-6)
\]

where \(E_s\) = elastic modulus of the steel and \(F_y\) = yield strength of the tube plate. The plastic strength can be used only for the compact section.

According to the theory of elasticity, elastic local buckling stress \(F_{cr}\) of a plate element is defined as follows.

\[
F_{cr} = k \frac{\pi^2 E_s}{12(1-v^2)(b/t)^2} \quad (2-7)
\]

where \(k\) = factor related to the plate aspect ratio, boundary condition, and stress distribution, \(v\) = Poisson’s ratio, \(b\) = inner width of the tube plate, and \(t\) = thickness of the tube plate. When the tube plate is confined by infill concrete, the \(k\) value is defined as 10.3 (Fig. 2-2, Uy and Bradford 1996). Therefore, from Eq. (2-7), assuming \(v = 0.3\), the width-to-thickness ratio that is required to develop the yield
strength \((F_{cr} = F_y)\) is defined as \(b/t = 3.05\sqrt{(E_s/F_y)}\), which is almost identical to Eq. (2-6). This result indicates that for the ratio \(b/t \leq \lambda_r\), theoretically, elastic local buckling does not occur. However, due to the effect of residual stress and initial imperfection, the possibility of local buckling in the tube plates increases. Considering such effects, the limit value \(\lambda_p\), which is stricter, is used for the compact section. Tort and Hajjar (2003) reported that, under concentric axial loading, the local buckling strains for \(b/t = \lambda_r\) and \(\lambda_p\) correspond to 62\% and 94\% of the yield strain, respectively \([i.e., \varepsilon_{ini}/\varepsilon_y = 3.14\{B/t\sqrt{(F_y/E_s)}\}^{-1.48}]\). When \(b/t = \lambda_r\) or \(B/t\sqrt{(F_y/E_s)} = 3.00\), \(\varepsilon_{ini}/\varepsilon_y = 3.14 \times (3.00)^{-1.48} = 0.62\).

\[
\varepsilon_{ini}/\varepsilon_y = 3.14 \times (3.00)^{-1.48} = 0.62
\]

(a) Effect of concrete filling on buckling mode
(b) Buckling coefficient (Uy and Bradford 1996)

Fig. 2-2. Effect of concrete filling on local buckling coefficient
2.2 Experimental Studies

2.2.1 CFT Columns with High-Strength Steel

Test results for high-strength RCFT members are being updated by latest studies (Table 2-2). In the case of high-strength stub columns, Uy (2001), Mursi and Uy (2004), and Aslani et al. (2015a) utilized high-strength steel up to \( F_y = 784, 761, \) and 701 MPa, respectively. Liu et al. (2003), Han et al. (2005), and Liu and Gho (2005) utilized high-strength concrete up to \( f_c' = 83.8, 82.7, \) and 86.5 MPa (equivalent cylinder strength), respectively. Varma (2000), Nakahara and Sakino (2003), Sakino et al. (2004), and Khan et al. (2013) studied high-strength steel and concrete up to \( (F_y, f_c') = (660, 114.6), (781, 119), (835, 91.1), \) and \((760, 98.0)\) MPa, respectively. Sato et al. (2008) and Sato et al. (2009) used high-strength steel and ultrahigh-strength concrete up to \( (F_y, f_c') = (750, 202) \) and \((814, 150)\) MPa, respectively.

In the case of RCFT flexural members (i.e., beams or beam-columns), Uy (2001) and Mursi and Uy (2004) used high-strength steel up to \( F_y = 784 \) and 761 MPa, respectively. Gho and Liu (2004) and Hardika and Gardner (2004) used high-strength concrete up to \( f_c' = 94.7 \) and 103.2 MPa, respectively. Varma et al. (2002), Nakahara and Sakino (2003), Fujimoto et al. (2004), Inai et al. (2004), Hirade et al. (2008), Kim et al. (2013), Lee et al. (2016b), and Choi et al. (2017) studied high-strength steel and concrete up to \( (F_y, f_c') = (660, 114.6), (781, 119), (834 \) and 80.3), (824 and 94.5), (844 and 103), (913 and 113), (746 and 83.6), and \((703 \) and 99.6) MPa, respectively. Matsumoto et al. (2008), Sato et al. (2008), and Matsumoto et al. (2009) used high-strength steel and ultrahigh-strength concrete up
to \((F_y \text{ and } f_c) = (831 \text{ and } 151), (814 \text{ and } 150), \text{ and } (741 \text{ and } 166) \text{ MPa, respectively.}\)

Table 2-2. Latest experimental studies on RCFTs

<table>
<thead>
<tr>
<th>Research</th>
<th>Country</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>Khan et al. (2013)</td>
<td>Australia</td>
<td>- Stub-columns</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- (F_y) up to 760 MPa, (f_c) up to 98 MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- (b/l = 15 \sim 40) (noncompact)</td>
</tr>
<tr>
<td>Aslani et al. (2015)</td>
<td>Australia</td>
<td>- Stub-columns</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- (F_y) up to 701 MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- (b/l = 14 \sim 38)</td>
</tr>
<tr>
<td>Khan et al. (2017)</td>
<td>Australia</td>
<td>- Stub-columns</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- (F_y) up to 762 MPa, (f_c) up to 113 MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- (b/l = 15 \sim 40) (noncompact)</td>
</tr>
<tr>
<td>Skalomenos et al. (2016)</td>
<td>Japan</td>
<td>- Beam-columns</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- (F_y) up to 788 MPa, (b/l = 23)</td>
</tr>
<tr>
<td>Xiong et al. (2017)a</td>
<td>Singapore</td>
<td>- Stub-columns</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- (F_y) up to 779 MPa, (f_c) up to 164 MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- (b/l = 10 \sim 16.8)</td>
</tr>
<tr>
<td>Xiong et al. (2017)b</td>
<td>Singapore</td>
<td>- Beams</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- (F_y) up to 756 MPa, (f_c) up to 183 MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- (b/l = 14.7)</td>
</tr>
</tbody>
</table>
2.2.2 CEFT Columns with Conventional Details

CEFT columns are often used as underground or lower-floor columns, where concrete encasement is added to increase the structural performance and fire resistance. However, current design guidelines do not provide structural details such as the minimum encasement thickness and reinforcement details to prevent premature failure of the thin concrete encasement. In the design examples given in the Architectural Institute of Japan standard (AIJ 2014), typical thickness of the concrete encasement is 20% of the gross section width, which corresponds to a hollowness ratio of 36%. Similarly, test specimens of CEFT columns in literatures were designed with comparatively small tube sections.

Table 2-3 summarizes the previous tests for rectangular CEFT columns. In early studies, small-scale specimens with relatively low hollowness ratios (the ratio of the hollow section to the gross section area) were tested. Yamada et al. (1981) performed cyclic loading test for a small scale CEFT column (gross section = 160 × 160 mm, steel tube section = 100 × 100 mm, hollowness ratio = 39%, axial load ratio = 0.3). Matsui et al. (1998) performed an axial load test for a CEFT column (gross section = 250 × 250 mm, steel tube section = 150 × 150 mm, hollowness ratio = 36%). Han et al. (2009) performed cyclic loading tests for small-scale CEFT columns (gross section = 150 × 150 mm, steel tube section = 50 × 50 mm, hollowness ratio = 11%, axial load ratios = 0, 0.3, and 0.6). Because of the presence of concrete encasement, the structural performance of the specimens was more similar to that of concrete-encased steel (CES) columns than to concrete-filled steel tubular (CFT) columns [Fig. 2-3(a)]. To fully utilize the structural performance of CFTs, large tube section may be advantageous (Han and An 2014), only if the premature failure of
outer concrete is prevented. As the encasement is completely separated from the core, the whole encasing region may to be vulnerable to early delamination [Fig. 2-3(b)].

Miyauchi et al. (2010) performed a cyclic loading test for CEFT columns with a relatively high hollowness ratio (gross section = 420 × 420 mm, steel tube section = 300 × 300 mm, hollowness ratio = 51%, axial load ratios = 0.07 and 0.21). The test specimens showed excellent performance in terms of load-carrying capacity and deformation capacity. To avoid early spalling of the thin concrete encasement, very closely spaced transverse ties were used (D5, diameter = 5 mm, vertical spacing $s = 60$ mm).

Matsui et al. (1998)
(a) Axial compressive behavior of composite tubular columns
(b) Comparison between SRC and CEFT

Fig. 2-3. Structural characteristics of CEFT columns
Table 2-3. Test parameters of previous studies using square tubes

<table>
<thead>
<tr>
<th>Test program of previous studies</th>
<th>Yamada et al. (1981, Japan)</th>
<th>Matsui et al. (1998, Japan)</th>
<th>Han et al. (2009, China)</th>
<th>Miyauchi et al. (2010, Japan)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Square CEFT specimens</td>
<td>1EA</td>
<td>1EA</td>
<td>3EA</td>
<td>3EA</td>
</tr>
<tr>
<td>Loading</td>
<td>Axial (0.33) + lateral cyclic</td>
<td>Axial</td>
<td>Axial (0.0, 0.3, 0.6) + lateral cyclic</td>
<td>Axial (0.07, 0.21) + lateral cyclic</td>
</tr>
<tr>
<td>Gross section (mm)</td>
<td>160 x 160</td>
<td>250 x 250</td>
<td>150 x 150</td>
<td>420 x 420</td>
</tr>
<tr>
<td>Tube section (thickness, area ratio)</td>
<td>100 x 100 mm (3.2 mm, 4.8%)</td>
<td>150 x 150 mm (3.2 mm, 3.0%)</td>
<td>50 x 50 mm (2.7 mm, 2.3%)</td>
<td>300 x 300 mm (9.5 mm, 6.3%)</td>
</tr>
<tr>
<td>Thickness of concrete encasement (mm)</td>
<td>30</td>
<td>50</td>
<td>50</td>
<td>60</td>
</tr>
<tr>
<td>Hollowness ratio (%)</td>
<td>39</td>
<td>36</td>
<td>11</td>
<td>51</td>
</tr>
<tr>
<td>Longitudinal bars (area ratio)</td>
<td>4-D10 (1.1%)</td>
<td>4-D6 (0.2%)</td>
<td>4-D10 (1.3%)</td>
<td>12-D6 (0.3%)</td>
</tr>
<tr>
<td>Transverse ties (area ratio, s/bc)</td>
<td>Φ6 at 80mm (1.18%, 1/2)</td>
<td>D6 at 65mm (0.98%, 1/3.8)</td>
<td>Φ6 at 100mm (0.64%, 1/1.5)</td>
<td>D5 at 60mm (0.55%, 1/7)</td>
</tr>
</tbody>
</table>

Note: $A_h$ = sectional area of transverse ties; $t_e$ = thickness of concrete encasement; $s$ = spacing of ties.

Table 2-4 also summarizes CEFT columns with the circular tube tested in Japan and China (Matsui et al. 1998; Nakamura et al. 1999; Ueura et al. 1999; Han et al. 2009). Ueura et al. (1999) utilized high-strength steel tubes ($F_y = 560$ MPa, $F_u = 634$ MPa) to improve structural capacities of the CEFT column. Nevertheless, since the former studies were not intended for the hollow PC construction, the hollowness ratio was only 13% - 34%. Also, the steel tubes used in the previous
specimens were compact sections (AISC 2010). In order to enhance the economy of the hollow PC method, the hollowness ratio needs to be greater than 50%, and the tube thickness should be reduced.

<table>
<thead>
<tr>
<th></th>
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</thead>
<tbody>
<tr>
<td>Circular CEFT specimens</td>
<td>1EA</td>
<td>12EA</td>
<td>2EA</td>
<td>3EA</td>
</tr>
<tr>
<td>Loading</td>
<td>Pure axial</td>
<td>Axial (0,0.3) + lateral cyclic</td>
<td>Axial (0.3) + lateral cyclic</td>
<td>Axial (0,0.3,06) + lateral cyclic</td>
</tr>
<tr>
<td>Gross section (mm)</td>
<td>$250 \times 250$</td>
<td>$270 \times 270$</td>
<td>$270 \times 270$</td>
<td>$150 \times 150$</td>
</tr>
<tr>
<td>Tube diameter (thickness, area ratio)</td>
<td>$\phi 165.2$ mm (2.3 mm, 1.9%)</td>
<td>$\phi 165.2$ mm (7.1 mm, 4.8%)</td>
<td>$\phi 165.2$ mm (4.5 mm, 3.1%)</td>
<td>$\phi 60$ mm (2 mm, 1.6%)</td>
</tr>
<tr>
<td>Center-line thickness of concrete encasement (mm)</td>
<td>42.4</td>
<td>52.4 / 77.9</td>
<td>52.4</td>
<td>45</td>
</tr>
<tr>
<td>Hollowness ratio</td>
<td>34%</td>
<td>29% / 14%</td>
<td>29%</td>
<td>13%</td>
</tr>
<tr>
<td>Longitudinal bars (area ratio)</td>
<td>4-D6 (0.2%)</td>
<td>4-D16 (1.1%)</td>
<td>4-D19 (1.6%)</td>
<td>4-D10 (1.3%)</td>
</tr>
<tr>
<td>Transverse ties (area ratio $^a$, s/b_c)</td>
<td>D6 at 75 mm (1.0%, 1/3.3)</td>
<td>2-U6.4 at 30 mm (2.0%, 1/9) / (2.0%, 1/9)</td>
<td>2-U6.4 at 30 mm (2.0%, 1/9)</td>
<td>$\phi 6$ at 100 mm (0.6%, 1/1.5)</td>
</tr>
</tbody>
</table>

Note: $A_h = \text{sectional area of transverse ties}; t_c = \text{thickness of concrete encasement}; s = \text{spacing of ties}.

$^a \rho_h = A_h/(t_c s)$. 

Table 2-4. Test parameters of previous studies using circular tubes
2.2.3 Remarks

According to the existing test results (Table 2-2), by using high-strength materials, the structural performance of RCFT column was definitely enhanced in terms of strength and stiffness, while many test results also revealed that the ultimate strengths were generally overestimated by existing design methods. However, since most studies have been limited to individual test results, the effects of material strengths and sectional slenderness should be further clarified with comprehensive databases.

In order to re-evaluate the code limitations on high-strength materials, very recent studies on the design of CFT members have focused more on high-strength materials (Table 2-5). Aslani et al. (2015) developed a design model for short and slender high-strength CFT columns under concentric axial loading, modifying AS 5100.6 (SA 2004). The authors suggested 0.87 as a constant concrete strength factor for RCFT columns. Lee et al. (2016) refined the strain compatibility method (ACI 2014; AISC 2016) for the design of RCFT compact sections (verified for specimens with \( f_{c}' \) up to 110 MPa and \( F_y \) up to 830 MPa) by defining the concrete crushing strain as a function of material strengths and tube width-to-thickness ratio. Liew et al. (2016) proposed a comprehensive design guide for CFT members with ultrahigh-strength concrete (\( f_{c}' \) up to 190 MPa) and high-strength steel (\( F_y \) up to 550 MPa) based on Eurocode 4 (CEN 2004). To extend the applicability of Eurocode 4 to high-strength concrete, a strength reduction factor was proposed. Strain compatibility of the two materials was also the key idea of the design. However, those previous studies did not properly take into account the effects of width-to-thickness ratio, consequently regulating material strengths as well as sectional slenderness. Lai et al.
(2015) improved the AISC method (AISC 2010) for the beam-column design of non-compact and slender sections, excluding high-strength materials. Finite element analysis for high-strength CFT columns has been limited to the modeling of short columns under pure compression (Tao et al. 2013; Thai et al. 2014; Wang et al. 2017).

As such, a number of experiments and attempts to develop analytical and design models for high-strength materials has been carried out. Because the studies were often limited to compact sections, however, adaptation of the new materials to current design codes, which are also required to accompany noncompact and slender section, inevitably encountered difficulties.

Table 2-5. Latest studies on design and database of RCFTs

<table>
<thead>
<tr>
<th>Research</th>
<th>Country</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>- Limited to short and slender columns</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- 220 short columns</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Concrete strength conversion factors</td>
</tr>
<tr>
<td></td>
<td></td>
<td>: $F_y$ up to 550 MPa, $f_c$ up to 190 MPa</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Extended databases from Goode (2008)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>: 384 short-column specimens</td>
</tr>
<tr>
<td></td>
<td></td>
<td>: 210 beam-column specimens</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Limited to compact section</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Databases</td>
</tr>
<tr>
<td></td>
<td></td>
<td>: 330 short column specimens</td>
</tr>
<tr>
<td></td>
<td></td>
<td>: 125 beam-column specimens</td>
</tr>
<tr>
<td>Lai et al. (2015)</td>
<td>U.S.</td>
<td>- Design of noncompact and slender section</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Adapted in AISC 360 (AISC 2016)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Limited specimens backed-up with FEA</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- High-strength materials excluded</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- Compact section excluded</td>
</tr>
</tbody>
</table>
2.3 Numerical and Analytical Studies

2.3.1 Continuum Finite Element Analysis

2.3.1.1 General Features

Although 3D continuum finite element analysis (FEA) may be time-consuming and not familiar with practitioners, it is advantageous for capturing interactions such as steel local buckling and concrete confinement in a straightforward manner. In this section, previous studies using the finite element method are introduced. Among various commercial FEA programs, ABAQUS has often been utilized in simulating the nonlinear behavior of CFT structures (Varma 2000; Han et al. 2007; Chen et al. 2012; Tao et al. 2013; Lai et al. 2014; Thai et al. 2014; Aslani et al. 2015).

Generally, solid finite element models (Chen et al. 2012; Lai and Varma 2016) showed the more brittle behavior of the infill concrete compared to uniaxial fiber models (Sakino et al. 2004; Popovics 1973).

2.3.1.2 Previous Studies

Varma (2000) conducted experiments for high-strength square CFT column and beam-columns ($F_y$ and $f'_c$ up to 660 and 114.6 MPa, respectively), and developed a finite element model for the column specimens. The numerical results indicated that the confined concrete area could be divided into a core region and surrounding noncore regions, subjected to triaxial and biaxial compression, respectively. Based on the model, fiber stress-strain relationships of the concrete infill and steel plates
were developed and implemented in a nonlinear structural analysis program DRAIN-2DX. In developing the stress-strain curves, the strain was defined as the average strain over the length of the model component which is equal to the section width.

Chen et al. (2012) used solid element (C3D20R) for modeling of steel tube, while shell element (S4R) was utilized in many studies (Varma 2000; Tao et al. 2013; Lai et al. 2014; Thai et al. 2014; Aslani et al. 2015). In addition, the stress-strain curve of concrete obtained by cylinder tests was carefully modified by trial-and-error to implement in ABAQUS. The developed numerical model was validated in detail with experimental observations. The test program included RCFT short columns subjected to axial compression only on steel section and simultaneously on the composite section. Based on the experimental and numerical results, two kinds of confining mechanisms are highlighted: a horizontal arch action that takes place after concrete crushing and vertical arch action that develops after column wall buckling. Due to the combined effect of the two arch actions, the concrete infill started to recover its strength. Since comparatively moderate width-to-thickness ratio was studied \((b/t = 24 \sim 48)\), the local buckling occurred in the post-peak range of concrete.

Tao et al. (2013) developed finite element models for both rectangular and circular CFT columns under axial compression. In the model, geometric imperfection and residual stress of the steel tube were not considered due to their negligible effects. An elastic-perfectly plastic model of steel was used for rectangular tube sections. The numerical results were compared with comprehensive databases from the viewpoint of strength. The accuracy of the developed numerical model was also compared with that of the model developed by Han et al. (2007).

Lai et al. (2014) focused on simulating the nonlinear behavior of RCFT
beam-columns with noncompact and slender sections. Based on the finite element model, Lai et al. (2015) proposed $P-M$ interaction equations for the design of the noncompact and slender sections. Lai et al. (2016) further developed effective stress-strain relationships for fiber model. In the procedure, the strain was estimated as the average axial strain of model component which is equal to three times the section width. The FEA results of Lai et al. (2016) also indicate that strength and ductility of the confined concrete are enhanced as tube width-to-thickness ratio increases (simulated with the range of $b/t = 60 \sim 100$), which is attributed to the confining mechanism provided by buckled steel tube (Chen et al. 2012).

Thai et al. (2014) and Aslani et al. (2015) performed finite element analysis for RCFT short columns with high-strength materials. Aslani et al. (2015) also conducted stub-column tests using high-strength steel compact section ($F_y = 701$ MPa, $B/t = 16 \sim 40$). The numerical model was used to propose equations predicting confining pressure and maximum compressive strength of confined concrete. According to the proposed model, the ratio of the increased strength of confined concrete over cylinder strength of plain concrete is 1.67, 1.40, 1.19, and 1.00 when $B/t = 10, 20, 30,$ and 40, respectively.
Chapter 2. Literature Review

Fig. 2-4. Finite element analysis and development of fiber model (Varma et al. 2005)

Fig. 2-5. Stress distribution of the concrete infill (Chen et al. 2012)
2.3.2 Fiber Model Analysis

2.3.2.1 Stress-Strain Relationships for Concrete Infill

This part introduces two fundamental formula (Popovics 1973; Sakino and Sun 1994) for effective stress-strain relationships of the concrete subjected to axial compression and their applications for the concrete filled in rectangular steel tubes. Also, some issues on concrete strength coefficients addressing size effect and confinement effect are summarized.

(1) Modification of Popovics (1973)

Popovics (1973) proposed a well-known stress-strain curve of concrete subjected to uniaxial compression. The formula is expressed as follows:

\[
\sigma = f'_e \frac{n(\varepsilon_e / \varepsilon_{co})}{n - 1 + (\varepsilon_e / \varepsilon_{co})^n}
\]  

(2-8)

where \( n \) affects the initial slope and the curvature of the ascending branch. The equation for unconfined concrete was later applied in modeling confined concrete of conventional reinforced concrete structures (Mander et al. 1988; Cusson and Paultre 1995).

Eq. (2-8) was also used for the concrete laterally confined by steel tube (Liang et al. 2006; Lai and Varma 2016). Generally, it is believed that interaction between the infill concrete and steel tube is not significant in rectangular tube sections due to the difference in Poisson’s ratio; the interaction is only expected when
concrete stress exceeds $0.7f'_c$. Thus, the compressive strength of the confined concrete is not increased. Instead, improvement of deformation capacity is considered as the width-to-thickness ratio decreases and steel yield strength increases.

Fig. 2-6. Applications of Popovic’s stress-strain model for the concrete laterally confined by reinforcing bars

(2) Modification of Sakino and Sun (1994)

Sakino and Sun (1994) proposed a unified stress-strain relationship for concrete confined by steel tubes or reinforcing bars in rectangular cross-sections. The formula is expressed as follows:

$$Y = \frac{VX + (W - 1)X^2}{1 + (V - 2)X + WX^2}$$  \hspace{1cm} (2-9)

$$X = \frac{\varepsilon_c}{\varepsilon_{co}}; \quad Y = \frac{\sigma_c}{f'_c}$$  \hspace{1cm} (2-10)

where $V$ and $W$ control the ascending and descending branches, respectively. The
model was further refined based on comprehensive test results carried out in U.S.-Japan cooperative research (Sakino et al. 2004; Fujimoto et al. 2004; Inai et al. 2004).

Fig. 2-7. Development of stress-strain relationship for the concrete infill

(3) Concrete size effect

Generally, the concrete infill is not longitudinally or laterally reinforced with deformed bars. Thus, structural behavior of the concrete, especially under pure compression, may be vulnerable to the size effect. This aspect has been emphasized in both analysis and design of CFT structures by some researchers (Blanks and McNamara 1935; Yamamoto et al. 2002; Sakino et al. 2004; AIJ 2008; Liang et al. 2009). In the other studies, however, the size effect is often neglected.
(4) Concrete confinement

Even though the strength improvement of concrete infill is usually not admitted in rectangular tube sections, some previous studies have reported the effectiveness of concrete confinement. On the basis of extensive databases, Yamamoto et al. (2013) reported that the concrete strength increases in very compact tube sections. Aslani et al. (2015) performed both experimental and numerical studies on RCFT short columns using high-strength steel. The results showed that concrete peak strength is enhanced when high-strength steel compact section is used. Based on 3D finite element analysis, Lai et al. (2016) proposed a concrete strength factor ranging from 0.8 to 1.1, which can be used for fiber model analysis.
2.3.2.2 Local Buckling in Rectangular Steel Tube

In this part, existing effective stress-strain relationships for steel tubes are summarized. There are mainly three issues to be considered: peak stress or buckling capacity, strain-hardening effect, and post-peak behavior.

(1) Buckling capacity

Nakahara et al. (1998) defined the buckling strength of concrete-filled steel tube by modifying an empirical equation for hollow tube sections. The buckling equation is also adopted as peak stress in fiber model (Fujimoto et al. 2004). However, the proposed predictions are close to initial buckling strengths and do not take into account the effect of post-buckling behavior. Consequently, the model underestimates the ultimate capacity of the slender section. Also, the maximum strength of RCFT beam-columns with high-strength steel (Garde 780 MPa) was excessively underestimated by the fiber model (Fujimoto et al. 2004).

On the other hand, the steel peak stress considering the post-buckling behavior was proposed by some researchers (Guo et al. 2007; Thai et al. 2015; Lai et al. 2016). Guo et al. (2007) conducted experimental and analytical studies to estimate the ultimate capacity of steel tube. In the study, only the steel section was loaded, and the concrete infill just constrained the inward buckling. On the basis of the finite element analysis, an equation to determine the post-buckling strength of concrete-filled steel tube was proposed. Thai et al. (2015) studied an entire stress-strain curve for steel plates with different boundary conditions. In the model, von Karman’s equation (Karman 1932) was adopted to determine the ultimate stress of thin plates subjected to elastic local buckling.
Chapter 2. Literature Review

Fig. 2-9. A study on influence of the post-buckling behavior (Guo et al. 2007)

(2) Strain-hardening effect

Fig. 2-10, Fig. 2-11, Fig. 2-12, and Fig. 2-13 show test results for tensile coupons of high-strength steel plates applied in RCFT columns. In the case of mild steel, the strain-hardening stage comes after some extent of yield plateau. On the other hand, in the case of high-strength steel, the strain-hardening effect generally appears right after the onset of yielding without apparent yield plateau. However, even among high-strength steel plates, material properties such as yield-tensile strength ratio, yield plateau length, and ultimate strain vary.

Fig. 2-10. Stress-strain relationships of tensile coupons tested in Australia
Fig. 2-11. A500 Grade-80 tested in U.S. (Varma et al. 2002)

Specified yield strength: 700~900 MPa
Specified tensile strength: 780~1,000 MPa

<Hayashi et al. 2015>  <Skalomenos et al. 2016>

Fig. 2-12. H-SA700 tested in Japan

Figure 16. Steel stress-strain relationship

Fig. 2-13. HT780 tested in Japan (Matsumoto et al. 2012)
In RCFT stub-column specimens, obvious overstrength (greater strength than a prediction based on simple superposition) was observed (Nakahara et al. 1998; Yamamoto et al. 2013). Although the cause is not clear, the overstrength may be due to the strain-hardening effect of stocky steel plates (Nakahara et al. 1998; Hajjar et al. 2013) the concrete strength increase due to the lateral confinement effect (Yamamoto et al. 2013). In stress-strain relationships proposed by some researchers (Fujimoto et al. 2004; Lai et al. 2016), the strain-hardening of the compact section is reflected.

On the other hand, another reason for the overstrength can be explained that the concrete strength increase due to the lateral confinement effect (Yamamoto et al. 2013), since under pure compression, the axial compressive strain at the peak load is not large enough for the steel to exhibit strain hardening effect. This can be confirmed from experimental (Matsui et al. 1998; Sakino et al. 2004; Yamamoto et al. 2013) and numerical (Chen et al. 2012) results. Stub-column specimens with $B/t = 17.9$ and 22 reached the peak strength at the axial compressive strain of 0.00749 and 0.0047, respectively (Yamamoto et al. 2013; Matsui et al. 1998).

(3) Post-peak behavior

In the ascending part of the stress-strain relationship, the load-carrying capacity of steel increases linearly up to its peak strength. The descending branch is often described with bi-linear curves (Sakino et al. 2004; Lai and Varma 2016). Sakino et al. (2004) proposed the fiber model based on regression analysis of extensive test results. Lai and Varma (2016) developed the model from the results of finite element analysis. Thai et al. (2015) applied strain energy method (Mursi and Uy 2004) to determine the post-peak behavior as well as inelastic local buckling
strains.

![Diagram](image)

**Fujimoto et al. (2004)**  **Lai and Varma (2016)**

Fig. 2-14. Post-peak behavior of rectangular tube plates using bilinear curves

### 2.3.2.1 Concrete Confinement in Circular Section

In the circular tube section, strength and ductility of the core concrete are significantly affected by the lateral hoop tension of the steel tube, and modeling of the mechanisms is crucial in simulating structural behavior of the filled column. Relevant studies are summarized in several concerns as follows:

1. **Strength model of confined concrete**

   Unlike rectangular sections, circular steel tubes provide sufficient confinement as to develop apparent strength gain of the confined concrete. One of the earliest researchers, Richart et al. (1929) proposed a basic equation for the peak strength of confined concrete (for ordinary reinforced concrete structures), $f'_{cc}$ as follows:

   $$f'_{cc} = f'_c + 4.1 f_t$$  \hspace{1cm} (2-11)
where $f'_c = \text{peak strength of unconfined concrete}$; and $f_l = \text{lateral pressure}$. The equation was further applied to the confined concrete filled in the circular steel tube (Sakino et al. 2004), in which the lateral pressure is theoretically derived as follows (Fig. 2-15):

$$f_l = \frac{2t \cdot 0.19F_y}{D - 2t}$$  \hspace{1cm} (2-12)

where $F_y = \text{tube yield strength}$; $D = \text{tube diameter}$; and $t = \text{tube thickness}$. The factored stress $0.19F_y$ indicates the hoop stress development in the steel tube.

Cusson and Paultre (1995) refined the concept of lateral pressure by distinguishing the nominal lateral pressure $f_l$ and the effective confinement pressure $f'_e$, addressing the effect of geometry due to the arch action in rectangular sections. The authors also proposed a modified strength equation of the confined concrete, introducing the effective confinement index $I'_e$ at peak stress as follows:

$$f'_{cc} = \left(1 + 2.1I'_e\right)f'_c$$  \hspace{1cm} (2-13)

$$I'_e = \frac{f'_e}{f'_c}$$  \hspace{1cm} (2-14)
(2) Steel hoop tension and yield criteria

The hoop tension developed in the steel tube is influential in determining the concrete confinement effect and steel longitudinal stress. As mentioned, the steel horizontal stress was suggested constantly as $0.19F_y$, based on theoretical and experimental investigations, and the relevant models are widely adopted in Japan (AIJ 2008); the peak compressive stress of the steel is limited to $0.89F_y$ while the peak tensile stress is increased to $1.08F_y$ [Fig. 2-16(a)].

In general, effective stress-strain relations of the materials for analytical uses are developed based on the results of pure compression tests, in which lateral expansion of the concrete clearly affects the confining mechanism. On the other hand, when the composite section is subjected to combined axial-flexural forces, the concrete expansion only appears in the compression zone. In the tension side, however, the interaction (or hoop tension) can develop due to the shrinkage of steel tube and the constraint of concrete infill [Fig. 2-16(b)]. As a result, the hoop tension assumed under pure compression may be effective under lower axial compression and greater bending moment.
(3) Effective stress-strain relationships of circular tube

Circular steel tubes possess superior resistance to local buckling when compared with rectangular ones. Therefore, in the material model, the strength degradation due to the local buckling is neglected, and instead, the axial behavior is often described by the effect of hoop tension. Fujimoto et al. (2004) and Lai and Varma (2016) limit the maximum compressive stress of the steel with a constant value ($0.91F_y$ and $0.9F_y$, respectively), and both allow the use of perfectly plastic or even hardening post-peak behavior (Fig. 2-17).
Recently, Katwal et al. (2017) proposed more detailed analytical models based on the results of extensive numerical investigations for stub-columns (Fig. 2-18). The steel stress-strain model particularly captures a temporary drop in compressive stress due to the increasing hoop tension and subsequent strength recovery attributed to the strain-hardening effect.

(4) Effect of axial-load eccentricity on concrete confinement
In the analysis of beam-columns using effective stress-strain models, the ones proposed calibrating experimental results for stub-columns are often utilized for frames composed of RC and CFT members with and without fiber-reinforced polymer (FRP). However, if there is a strain gradient over the cross-section due to the additional flexure, the lateral confining pressure may not be fully developed with limited expansion of the concrete (Fam et al., 2003; El Maaddawy 2009). Considering such circumstances, Fam et al. (2003) proposed a strength model of concrete confined by FRP, where the effect of eccentricity is incorporated as follows (Fig. 2-19):

\[ f_{cc,e} = f_c' + \left( f_{cc} - f_c' \right) \frac{1}{1 + e/D} \] (2-15)

Fig. 2-19. Behavior of confined concrete affected by biaxial stress state in FRP tube (Fam et al. 2003)
2.3.3 Remarks

(1) Numerical insight

In 3D finite element analysis by Chen et al. (2012), the concrete infill recovered its strength in large post-peak deformation. The phenomenon was explained by the vertical arch action (i.e., lateral confinement) due to local buckling distributed along the column height; concrete tends to spread the local buckling over a larger region (Hajjar et al. 2013). These results indicate that the wall distortion leads to additional interaction or lateral confinement between the concrete infill and the steel tube, which is scarcely expected in conventional stocky tubes.

As presented in Chapter 4, current design methods tend to underestimate the strength of CFTs with slender section. This implies that the early local buckling of rectangular tubes does not necessarily degrade the concrete compressive behavior as assumed in the models. While the vertical arch action affected only descending curve in the previous numerical model with moderate width-to-thickness ratio ($b/t = 24 \sim 48$), the earlier interaction in slender section may be beneficial to the peak stress of confined concrete (Lai and Varma 2016). Without the local distortion, the interaction is generally not expected until the axial strain of approximately 0.001, when micro-cracking in the concrete begins to occur and the lateral expansion (or Poisson's ratio) of the concrete increases approaching the constant lateral expansion of the steel.

(2) Model development for rectangular and circular CFTs

The peak strength of rectangular CFTs highly depends on the peak stress of
the concrete infill and steel tube. In the case of concrete, determination of the strength reduction factor or size effect is the critical issue. In the case of steel, determination of the effective local buckling stress or post-local buckling capacity is essential. On the other hand, the effect of early concrete crushing on the high-strength steel is not considered in any previous models.

According to the previous studies, the compressive strength of confined concrete is significantly upgraded in circular CFTs. However, such effect is verified only for short columns subjected to pure compression (Sakino et al. 2004; Lai et al. 2016; Katwal et al. 2017). In such case, it is obvious that the significant interaction between the concrete infill and steel tube (i.e., the lateral hoop tension in the steel tube) develops due to the concrete dilation. However, when the composite section is subjected to the flexural moment, causing a strain gradient, the expansion should limited to the compression zone, affecting the confinement mechanism.
2.4 Studies on Composite Beam-Column Joints

2.4.1 Traditional Steel Beam-CFT Column Connections

2.4.1.1 Steel Beam-CFT Column Connections in Design Codes

The basic design philosophy for steel beam-CEFT column joints may be similar to that for the joints using conventional CFT columns. Although the general design procedures for composite connections are thoroughly provided in ANSI/AISC 341-16 (AISC 2016) and Eurocode 4 (CEN 2004), specific information regarding to the connections to CFT columns, the local flexural design in particular, is not available. On the other hand, the AIJ recommendations (AIJ 2008) and the CIDECT design guide No. 9 (Kurobane et al. 2004) are the ones specialized in the tubular columns and their connections.

(1) AIJ recommendations (AIJ 2008)

In the guidelines, the design procedures of steel beam-CFT column joints involve the shear behavior of joint panels and the local tensile behavior of diaphragm moment connections. Although it is often recommended to ensure the sufficient strength and limit the distortion in the composite joint panel, the shear failure mode exhibits satisfactory deformation and energy dissipation capacities from the viewpoint of seismic design. Thus, some extent of the plasticity is permitted in the panel zone to prevent any stress concentration and subsequent fracture in the beam plastic hinge, and relevant analytical models on load-deformation relationships are adopted (AIJ 2008). A more recent study (Fukumoto 2012) is also available for CEFT columns.
On the other hand, the local flexural (or out-of-plane tensile) failure of the connection occurs in a more brittle manner. Therefore, a sufficient safety is required in the design. Generally, the flexural behavior of the connection is simplified in the planar tensile behavior, conservatively neglecting the slight contribution of the web-tube connection. In the recommendations (AIJ 2008), the ultimate tensile strength $T_u$ of the flange-tube connection is suggested to satisfy the following equation:

$$\beta T_u \geq \alpha \frac{M_{bp}}{d_f}$$  \hspace{1cm} (2-16)

where $\alpha$ = safety margin addressing the effects of strain hardening and material overstrength (1.25 ~ 1.4); $\beta = 0.9$ or 0.8 for rectangular or circular tube (differs due to the accuracy of strength models), respectively; $M_{bp}$ = beam plastic moment capacity; and $d_f$ = center-to-center flange distance. In the recommendations (AIJ 2008), load-deformation relationships for various diaphragm details are introduced (Fukumoto 2007). However, back-up specimens for the equations all belong to the compact section (slenderness coefficient ranges 1.36 ~ 2.23 and 0.031 ~ 0.125 for rectangular and circular sections, respectively).

(2) CIDECT design guide No. 9 (Kurobane et al. 2004)

The design guide No. 9 of CIDECT (Construction with Hollow Steel Sections) deals with connections using tubular steel columns. As in Eurocode 3 (CEN 2005), the connection types are divided into simple, semi-rigid, and rigid one depending on the stiffness (Fig. 2-20). The semi-rigid connection is usually
represented by unstiffened details while the rigid connection indicates rigid full-strength connections with either a through, internal, or external diaphragm. The design guide also covers the tubular columns with concrete infill, in which the three connection types are considered.

The design of rigid connections for CFT columns entails stiffening methods using diaphragm plates that have been popular and extensively studied in Japan. Accordingly, the CIDECT No. 9 also adopts the design methods in AIJ (2001). Nevertheless, the associated specifications were further updated in AIJ (2008). For semi-rigid connections using CFT columns, comparative investigations with the design methods for semi-rigid connections using hollow steel columns are documented rather than unique design equations for the filled column. Due to the filling effect of the concrete, the deformation capacity of the steel tube is limited in the tension side (not enough to allow a yield line mechanism), and the punching shear failure dominates. In experimental results of De Winkel (1998), the ultimate tensile strength attained more than two times the strength for the hollow section due to the concrete filling effect, but was less than the calculated punching shear strength. The design of semi-rigid connections for hollow steel columns in the CIDECT design guide No. 9 (Kurobane et al. 2004) follows that in the CIDECT guide No. 1 (Wardenier et al. 1991).
2.4.1.2 Strength Equations for Semi-Rigid Connections

(1) Fu and Morita (1998)

The tensile strength $T$ of flange plate-circular tube connections is derived applying the concept of virtual internal and external work, based on the yield line mechanism consisting of out-of-plane flexural and punching shear yield lines (Fig. 2-21, Fu and Morita 1998).

\[ T_{nu} = \frac{4D\partial m_u}{X} + \frac{2t(X + t_r)F_u}{\sqrt{1 + 2\cos^2 \theta}} \]

\[ X = \sqrt{\frac{2D\partial m_u\sqrt{1 + 2\cos^2 \theta}}{tF_u}} \]

where left and right terms in Eq. (2-17) indicate the contribution from out-of-plane moment and punching shear yield lines, respectively; $m_u$ = out-of-plane plastic moment capacity per unit wall length (or width) = $\frac{t^2F_u}{4}$; $t_r$ = flange thickness including weld metal; and $X$ is the vertical length between out-of-plane moment yield
lines and determined so that \( T_{yl} \) becomes the minimum. The comparison with test results showed that the prediction is conservative in general (Fu and Morita 1998). When a diaphragm plate (either internal, through, or external) is retrofitted, the contribution can be added to the formula in a similar manner, assuming in-plane yield line.

![Figure 2-21. Assumed failure mechanism in the yield line model (Fu and Morita 1998)](image)

(2) CIDECT design guide No. 1 (Wardenier et al. 2008)

In the design guide for circular hollow section (CHS) joints, the typical failure mode is either chord plastification or chord punching shear. When the transverse plate is connected to the circular hollow section chord (i.e., flange plate-tube connection subjected to axial load), the design strength corresponding to the two limit states are formulated, respectively, as follows:

\[
T_{cp} = Q_a Q_f \frac{F_y t^2}{\sin \theta} \tag{2-19}
\]
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\[ T_{ps} = \frac{2}{\sqrt{3}} F_y t b_f \]  \hspace{1cm} (2-20)

where \( Q_u \) and \( Q_f \) = factors accounting for the connection geometry and chord stress, respectively; \( F_y \) = yield strength of steel tube; \( t \) = tube thickness; and \( b_f \) = flange width at the connection. ANSI/AISC 360-16 (AISC 2016), Eurocode 3 (CEN 2005), and ISO/FDIS 14346 (ISO 2013) have different equations for \( Q_u \) and \( Q_f \). The design equations in 1\(^{st}\) (Wardenier et al. 1991) and 2\(^{nd}\) (Wardenier et al. 2008) edition of the CIDECT design guide No. 1 are implemented in Eurocode 3 and ISO/FDIS 14346, respectively. As mentioned, however, the punching shear failure usually takes place in the filled column, considerably exceeding the prediction of Eq. (2-19). Eq. (2-20) for the punching shear failure is almost consistent among the specifications, and several limitations pertaining to the equation include \( F_y \leq 460 \text{ MPa} \), \( F_y \leq 0.8F_u \), and \( D/t \leq 40 \) (Wardenier et al. 2008).

2.4.1.3 Experimental and Numerical Investigations

(1) Research at the University of Illinois

Analytical (Alostaz and Schneider 1996) and experimental (Schneider and Alostaz 1998) studies were conducted at the University of Illinois (UIUC) to evaluate the seismic performance of steel beam-circular CFT column joints with various connection details [Fig. 2-22(a)] that include simple welding (semi-rigid connection), diaphragm stiffening, continuous web, embedded deformed bar, continuous flange, and continuous girder (i.e., both web and flange). Since the flexural behavior of connections was the main focus, T-shaped configuration was applied, representing exterior beam-column joints. The results indicated that
continuing the entire beam section through the composite column is the most promising detail to realize the ideal rigid connection.

(2) Research at the University of Nebraska-Lincoln

The structural characteristics of the through connection was further investigated at the University of Nebraska-Lincoln (UNL). Six cruciform beam-column joints [Fig. 2-22(b)] were tested to identify possible failure modes of the through connections (Elremaily and Azizinamini 2001b), followed by an analytical study to develop the design provisions (Elremaily and Azizinamini 2001a). The works at UIUC and UNL are referred in the AISC seismic provisions (AISC 2016).

Fig. 2-22. Connection details tested at UIUC and UNL

(a) Test specimens at UIUC
(Azizinamini and Schneider 2004)

(b) Test specimens at UNL
(Elremaily and Azizinamini 2001)
(3) Unstiffened (semi-rigid) connections with circular CFT columns

Fu and Morita (1998) performed monotonic tension tests for flange plate-circular CFT column connections with and without stiffening plates. The experimental variable also included loading type, diameter-to-thickness ratio of steel tube, and tube diameter-to-beam width ratio. The test strengths were compared with the predictions proposed based on the yield line theory.

De Winkel (1998) conducted extensive experimental and numerical studies on multi-planar connections using CHS columns. The connected braces included I-section beams and transverse plates, and the effect of concrete filling in the column was also examined. The test results demonstrated that tensile loading triggered a punching shear failure in the filled column, exceeding the design strength assuming chord plastification.

Chiew et al. (2001) investigated the moment resistance of steel beam-circular CFT column connections. Four unstiffened and four stiffened connections were tested under monotonic static loading. The stiffening methods included cover plates, external diaphragm, and rebars. Based on parametric studies using numerical models, an empirical formula was proposed.

(4) Concrete-filled CHS connections

Currently, design codes are not readily available for concrete-filled CHS connections. Xu et al. (2015) carried out a series of tensile load tests on the connections. The typical failure mode was punching shear mechanism of the concrete-filled chord. The ultimate strengths exceeded the predictions based on the
punching shear equation for CHS section by more than 50% on average. The connection deformation at the ultimate strength was less than 1% of the chord diameter, automatically satisfying the limitation under service loads (Packer et al. 1992). The chord diameter to thickness ratio was 60 and 75 while the yield strength, tensile strength, and the ratio of steel plates were 267 ~ 330 MPa, 398 ~ 485 MPa, and 0.67 ~ 0.70, respectively. The authors further conducted in-plane bending tests and numerical investigations (Xu et al. 2017).

(5) Steel beam-CEFT column joints

The relevant experimental studies were performed mainly in Japan and China (Nakamura et al. 1999; Ueura et al. 1999; Liao et al. 2014). Nakamura et al. (1999) and Ueura et al. (1999) stiffened the connection with external diaphragms, in which the column yielded. Fukuhara et al. (2010) tested steel beam-CEFT column joints using rectangular tubes, in which failure occurred at the beams. In Liao et al. (2014), the beam-column connection was conservatively designed so that the failure was dominated by the flexural strength of beam. For strengthening of the connection, additional details such as the outer diaphragm and stiffening steel plate were often reinforced inside the concrete encasement.
2.4.2 Application of Concrete Beam

2.4.2.1 Experimental Studies

Since the connections between composite tubular columns and concrete beams are not well-documented in standards, the relevant joint systems in literatures were referred to find possible connection details and investigate the structural performance.

(1) Connections between composite tubular columns and RC beams

Arimatsu et al. (2005) performed cyclic loading tests on interior joints using CFT columns and RC beams [Fig. 2-23(a)]. Beam longitudinal rebars (D19, SD490, $f_y = 530$ MPa) penetrate through holes in the steel tube and tightened outside the wall with nuts. Strengthening plates were welded around the holes, which are also expected as shear keys to transfer beam shear force to the column. Since cross-sectional dimensions (350 mm × 350 mm, tube thickness = 9 mm, $f'_{c}$ = 57 MPa) of the CFT column were reduced when compared with the counterpart RC column (450 mm × 450 mm, longitudinal rebars 12-D19, ties 4-D6 at 50 mm spacing, $f'_{c}$ = 61 MPa), to supplement deficient anchorage of beam rebars inside the CFT column, additional nuts were inserted in the joint.

Recently, through-beam connections between CFT columns and RC beams were developed, eliminating the entire or substantial part of the steel tube at the joint (Chen et al. 2014; Tang et al. 2016). These types aim to maximize the structural continuity of RC beams and to get rid of the interference and congestion during connecting the members. To compensate the performance degradation at the
interrupted region, enlarged joint dimensions were used with various configurations of steel cages.

The CEFT column has been utilized in some high-rise buildings and bridges in China, is easily connected to RC beams as well as steel beams (Han et al. 2014). Liao et al. (2014) performed cyclic loading tests on interior joints between CEFT columns and RC beams. Because a small tube section was used, there was no interruption between the tube wall and beam longitudinal rebars. Stiffening steel plates were welded to the core tube to relocate the plastic hinge away from the column thus protecting the connection region and to effectively transfer the beam forces.
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![Figure 2.24. RC beam-CEFT column connections (Liao et al. 2014)](image)

(a) Field application

(b) Specimen details

Fig. 2-24. RC beam-CEFT column connections (Liao et al. 2014)

(2) Precast concrete connection

The earthquake resistance of CEFT column-PC beam joints is expected to be affected by PC construction methods. Previously, a seismic testing was performed for beam-column connections, in which PC was used for the columns and U-shaped beam shells (Im et al. 2013). Various connection details for the interface between the PC column and beam shell, including steel angles for cover concrete, seating length of beam shell, and strengthening with headed bars, were tested. The test results indicated that the stiffness and energy dissipation of the PC connections were inferior to those of the conventional RC connection.
2.4.2.2 Design Specifications

It is expected that the connection between CEFT column and concrete beam can be conservatively designed following design procedures of conventional RC moment frames (Han et al. 2014). Due to the presence of the steel tube, however, design guidelines for composite beam-column joints (ASCE 1994) were also studied.

(1) ACI provisions

Several important aspects of the joint design mainly recommended in ACI 353R-02 (ACI 2002), such as transverse reinforcement, joint shear strength, and rebar development are highlighted as follows.

To transmit member forces to into the joint, adequate lateral confinement should be provided to the joint concrete by means of transverse reinforcement. According to ACI 352R-02 (ACI 2002), the total cross-sectional area $A_h$ of rectangular hoops and cross-ties should satisfy the following equations.

$$A_h \geq 0.3 \frac{sb^t f'_c}{f_{sh}} \left( \frac{A_g}{A_e} - 1 \right) \quad (2-21)$$
where $s = \text{spacing of transverse reinforcement}; f_{yh} = \text{the specified yield strength of hoop and cross-tie reinforcements}; b''_c = \text{core dimension of confined column measured from the outside edges of hoops}; A_c = \text{area of the column core}; h_c = \text{column depth}; d_b = \text{diameter of longitudinal rebars}; \text{and } s_x = 100 + (350 - h_x)/3, \text{where } h_x = \text{maximum center-to-center distance between column longitudinal rebars laterally supported by hoops of cross-ties.}$

The nominal shear strength of the joint $V_{nc}$ is defined as follows (ACI 2002):

$$V_{nc} = 0.083\gamma \sqrt{f'_c b_j h_c} \quad (2-24)$$

$$b_j = \min \left( \frac{b_b + b_c}{2}, b_b + \sum \frac{m h_c}{2}, b_c \right) \quad (2-25)$$

where $b_j = \text{effective joint width}; b_b = \text{beam width}; b_c = \text{column width}; m = \text{factor that accounts for the eccentricity between the beam centerline and the column centroid.} \gamma$ is determined considering the configuration of columns and beams framing into the joint.

To prevent excessive bond slip during earthquake loads, the column depth-to-bar diameter ratio in interior beam-column joints is limited as follows:
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\[ \frac{h_c}{d_b} \geq 20 \quad \text{in ACI 318} \]  \hspace{1cm} (2-26)

\[ \frac{h_c}{d_b} \geq 20 \frac{f_y}{420} \quad \text{in ACI 352} \]  \hspace{1cm} (2-27)

On the other hand, the development length \( l_{dh} \) for exterior beam-column joints is recommended as follows:

\[ l_{dh} \geq \frac{f_y d_b}{4.2 \sqrt{f'_c}} \]  \hspace{1cm} (2-28)

(2) Design guides for RCS joints

Reinforced column-steel beam (RCS) moment frames have been one of the most popular composite structural systems in building constructions worldwide. Since the hybrid joints show a typical and original form of combining design of the structural steel and concrete (ASCE 1994; AIJ 2001), the procedure is useful for understanding general behavior of composite joints.

The shear strength of RCS joints is calculated as the sum of the contributions of the steel web \( V_{wn} \), the inner concrete strut \( V_{csn} \), and the outer concrete compression field \( V_{cfn} \): (b) + (c) + (d), respectively in Fig. 2-26.

\[
\phi \left[ V_{wn} d_f + 0.75 V_{csn} d_w + V_{cfn} (d + d_o) \right] \\
\geq \sum M_p + V_b h_t - V_c d - V_b h_j
\]  \hspace{1cm} (2-29)
where $V_b$ = vertical shear force acting on the beam, $V_c$ = lateral shear force acting on the column, $h_j$ = effective joint depth (recommended conservatively as $0.7h$), $\phi$ = strength reduction factor, $d_f$ = center-to-center distance between the beam flanges, $d_w$ = web depth, and $d_o = 0$ (when neither the steel columns or E-FBPs are present). In Eq. (2-29), the right-hand side is the demand based on the beam flexural capacity. The shear resistances of the three components are defined as follows (ACI 2002; AISC 2010):

$$V_{wn} = 0.6 F_{yw} t_w h_j$$

$$V_{ewn} = 1.7 \sqrt{f'_c b_p h_c} \leq 0.5 f'_c b_p d_w$$

$$V_{effn} = V'_c + V'_s \leq 1.7 \sqrt{f'_c b_o h_c}$$

$$V'_c = 0.4 \sqrt{f'_c b_p h_c}$$

$$V'_s = A_o f_{ysh} 0.9 h/s_h$$

where $F_{yw}$ = web yield strength, $t_w$ = web thickness, $b_p$ = FBP width, and $f_{ysh}$ = tie yield strength. Outer effective width $b_o$ is necessary not only for the calculation of the outer shear resistance, but also for the calculation of the bearing capacity. However, when shear keys such as the steel columns and the E-FBPs are absent, it is regarded as $b_o = 0$ mm (as $d_o = 0$ mm), neglecting the effect of outer joint panel.
Fig. 2-26. Typical design of composite joints between steel beam and RC column
Chapter 3. Axial Testing and Evaluation of CFT Columns with High-Strength Steel

3.1 Introduction

In concrete-filled steel tube (CFT) columns, the concrete crushing is restrained by the lateral confinement of the steel tube, while the local buckling of the steel tube is restrained by the infill concrete. For this reason, CFT columns exhibit excellent structural performance. Further, in the CFT columns, unlike concrete-encased steel columns, the steel plates are located at the perimeter of the cross-section. Thus, the contribution of the steel section to flexural capacity can be maximized, particularly when high-strength steel is used. From the viewpoint of constructability, the use of high-strength steel is also beneficial since the welding and lifting weights can be reduced by using thin plates. When high-strength steel and high strength concrete are utilized together, the cross-section area can be significantly reduced, which is preferable for architectural design and planning. Recently in Japan, high-strength CFT columns with 780 MPa (tensile strength) steel and 150 MPa concrete were used in practice (Aoki et al., 2012; Endo et al., 2011; Matsumoto et al., 2012).

However, when the yield strength of steel tube exceeds 600 MPa (the yield strain of approximately 0.003), the early crushing of concrete can occur before yielding of the high strength steel (Kim et al. 2013). For this reason, current design codes limit the yield strength of steel for composite members. In ANSI/AISC 360 (AISC 2010) and Eurocode 4 (CEN 2004), the yield strength of steel is limited to
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525 MPa and 460 MPa, respectively (Leon et al. 2007). However, in the case of CFT columns, since the infill concrete is laterally confined by the steel tube, the limitation of the yield strength needs to be reconsidered.

For more than a decade, experimental studies have been conducted on rectangular CFT (RCFT) columns and beam-columns using high-strength steel (yield strength ≥ 600 MPa). Despite the advantages of high-strength steel, in order to restrain local buckling, current design codes require thick plates or low width-to-thickness ratios \((b/t)\). Thus, the majority of previous experiments for high-strength RCFT columns were performed for columns with compact or non-compact sections. Uy (2001b) tested columns, beams, and beam-columns with \(b/t = 20.0 \sim 40.0\). Sakino et al. (2004) and Fujimoto et al. (2004) reported extensive experimental studies on columns and beam-columns \((b/t = 16.5 \sim 48.2)\), as parts of U.S.-Japan cooperative research. Varma et al. (2002, 2004) tested beam-columns focusing on monotonic and cyclic behaviors \((b/t = 32.3\) and 48.0). Mursi and Uy (2004) studied effects of column slenderness \((b/t = 22.0 \sim 52.0)\). Sato et al. (2009) tested stub-columns with ultra-high strength concrete (compressive strength \(f'_c = 153 \sim 202\) MPa, \(b/t = 20.5\) and 28.0). More recently, Khan et al. (2013) and Aslani et al. (2015) tested stub-columns to investigate various combinations of concrete strength and width-to-thickness ratio \((b/t = 14.0 \sim 40.0)\).

The existing test results showed that the specimens with thick-walled tubes had the plastic capacity of the composite section. However, the majority of the previous high-strength RCFT specimens belong to compact or non-compact sections (AISC 2010), and only a few test results have been reported for RCFT columns with slender section (Nakahara and Sakino 2003; Mursi and Uy 2004). Thus, further experimental evidence are required to verify the validity of high-strength steel
As the yield strength of steel increases, the thickness of the steel plates needs to be decreased for the economy. In this case, to prevent early local buckling of the slender section, the use of vertical stiffeners can be considered. Tao et al. (2005) conducted axial load tests for RCFT columns strengthened with stiffeners (steel yield strength = 234 MPa, concrete compressive strength = 50.1 ~ 54.8 MPa), and proposed a design equation for the stiffener rigidity. However, the design equation was developed for mild steel tube columns. Thus, the applicability of the equation should be verified to high-strength steel tube columns.

In this chapter, concentric and eccentric axial loading tests were performed on RCFT columns with high-strength steel slender section. To investigate the axial-flexure capacity under high axial load, small values of eccentricity were used for the column specimens. The overall behavior of the specimens was further investigated by performing nonlinear finite element analysis. On the basis of the results, the applicability of the current design codes to high-strength steel slender sections was evaluated, and a design equation for vertical stiffeners was developed.
3.2 Test Plan

3.2.1 Stiffener Rigidity

Vertical stiffeners can be used to improve the structural performance of RCFT columns with the slender section, by restraining the local buckling of the steel tube. The second moment of inertia of the vertical stiffeners, which is required to adequately restrain the local buckling, was proposed by Tao et al. (2005).

\[
I_{s,s} = I_{s,re} = 3.1 \times 10^{-4} \left( \frac{w}{t} \right)^{3.5} \frac{F_y}{280} t^4
\]  

where \(I_{s,s} (= t_s b_s^2/12)\) = second-moment of inertia of the stiffener [Fig. 3-2(c)], \(t_s\) and \(b_s\) = thickness and depth of the stiffener plate, \(I_{s,re}\) = required second-moment of inertia of the stiffener, and \(w\) = width of a subpanel divided by the stiffener, which is defined as half the full width (\(B/2\)) when one stiffener is used per tube plate. Eq. (3-1) was empirically defined from the existing test results of mild steel tube columns [for the tube plates, \(F_y = 234\) MPa (\(t = 2.5\) mm) and for the stiffeners, \(F_{ys} = 234\) MPa (\(t_s = 2.5\) mm) and 311 MPa (\(t_s = 2\) mm), see Fig. 3-1].

Fig. 3-1. Stiffened CFT columns tested by Tao et al. (2005)
3.2.2 Specimens and Test Parameters

Currently, the compactness of the steel sections in the test specimens was defined according to ANSI/AISC 360 (AISC 2010). Table 3-1 and Fig. 3-2 show the test parameters and sectional dimensions of the test specimens, respectively. Five columns were prepared for both concentric and eccentric axial loading tests; the sectional and material properties were identical with the same section number. For high-strength steel, grade 800 steel (actual tensile strength $F_u = 835$ MPa and yield strength $F_y = 746$ MPa) was used. The control specimens C1 and E1 were designed as a square section (□ – 300 × 300 mm) with high-strength steel plates of $t = 5$ mm. Since the tube width-to-thickness ratio $b/t = 58.0$ exceeds $\lambda_r = 49.7$, the tube section was classified as a slender section. For the infill concrete, 70 MPa concrete (average cylinder strength $f_{c'} = 70.5$ MPa) was used.

In ANSI/AISC 360-10, a rectangular section that has both slender plates and compact plates is defined as a slender section. Specimens C2 and E2 were tested to investigate the structural performance of RCFT columns that have both compact plates and slender plates in a cross-section. For this, the specimens were designed as a rectangular section (□ – 300 × 150 mm) using high-strength steel plates of $t = 5$ mm. To maintain the same strength contribution ratio of the steel section as that used for section 1, the nominal compressive strength of the infill concrete was increased to 90 MPa (average cylinder strength $f_{c'} = 83.6$ MPa).

To compare the effect of steel yield strength, specimens C3 and E3 were designed with the mild steel of grade 400 (actual tensile strength $F_u = 466$ MPa and yield strength $F_y = 301$ MPa). The other parameters, such as the plate thickness,
sectional dimensions, and concrete strength, were the same as those of section 1. By using the mild steel, the steel tube of section 3 was classified as a compact section \((b/t = 58.0 < \lambda_p = 59.0)\).

The properties of sections 4 and 5 were the same as those of sections 1 and 2, respectively, except for the use of vertical stiffeners in the former. In specimens C4 and E4, four stiffeners were welded to the tube plates. In specimens C5 and E5 with a rectangular section, two stiffeners were welded to the slender plates. As a stiffener was located at the center of the tube plate, the width-to-thickness ratio of the subpanels was decreased to \(w/t = 29.0\), which is smaller than \(\lambda_p = 37.5\) \((F_y = 746\) MPa) for the compact section. Mild steel \((t_s = 5\) mm and \(F_{ys} = 301\) MPa), which is the same as that used for section 3, was used for the vertical stiffeners. The width of the stiffeners was \(b_s = 60\) mm and their design method is discussed in detail in the section, 3.5 Design of Stiffener.

To investigate the axial-flexure capacity under high axial load (about 80% of concentric axial load capacity), a small value of eccentricity ratio (i.e., the ratio of eccentricity to the dimension of the section) was used for all specimens: \(e/H = 0.133\). For the square-section specimens, the eccentricity was \(e = 40\) mm \((H = 300\) mm), and for the rectangular-section specimens, the eccentricity with respect to the weak axis was \(e = 20\) mm \((H = 150\) mm).

The average compressive strengths of concrete cylinders \((100 \times 200\) mm) are presented in Table 3-1. The concrete cylinders were tested on the 36th day of concrete casting. The stub-column specimens were tested on the 32-33th days while the beam-column specimens were tested on the 34 ~ 36th days. As mentioned, in square-section specimens, the compressive strength of the concrete was \(f_c' = 70.5\)
MPa. In the rectangular-section specimens, the concrete strength was $f'_c = 83.6$ MPa. The yield strength, tensile strength, and elongation of the grade 800 steel were $F_y = 746$ MPa, $F_u = 835$ MPa, and 21.6%, respectively. The properties of the grade 400 steel were $F_y = 301$ MPa, $F_u = 466$ MPa, and 33.8%, respectively. The yield strength of the grade 800 steel was defined using the 0.2% offset method. As shown in Fig. 3-3, the high-strength steel exhibits early strain-hardening behavior without an apparent yield plateau.

Currently, HSA800, which is a grade 800 steel for building structures (KS 2016), was substituted by ATOS80 for mobile structures because HSA800 thin plates suitable for laboratory testing were not available (HSA800 or ATOS80 steel in the Korean Industrial Standard, approximately equivalent to ASTM A514 and S690). In addition to the nature of immediate strain-hardening phenomenon after yielding (Fig. 3-3), the ATOS80 resembles the HSA800 in that $F_y = 746$ MPa and $F_u = 835$ MPa of ATOS80 plates fall in the nominal ranges of HSA800 ($F_y = 650 ~ 770$ MPa and $F_u = 800 ~ 950$ MPa, respectively). Even though the elongation of 21.6% is greater than the typical value of HSA800 around 15%, it is expected that the deformability does not seriously affect the ultimate strength of the composite column under axial compression with small eccentricity. Table 3-2 also compares chemical components in HSA800 and ATOS80. Generally, pre- and post-heating process is not required for ATOS80 thin plates (thickness 2.3 ~ 14 mm).
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Fig. 3-2. Cross sections of test specimens

Fig. 3-3. Stress-strain relationships of tensile coupons
Chapter 3. Axial Testing and Evaluation of CFT Columns with High-Strength Steel

Table 3-1. Test parameters of specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Steel tube</th>
<th>Concrete</th>
<th>Stiffener</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>B (mm)</td>
<td>H (mm)</td>
<td>t (mm)</td>
</tr>
<tr>
<td>C1</td>
<td>300</td>
<td>300</td>
<td>5</td>
</tr>
<tr>
<td>C2</td>
<td>300</td>
<td>150</td>
<td>5</td>
</tr>
<tr>
<td>C3</td>
<td>300</td>
<td>300</td>
<td>5</td>
</tr>
<tr>
<td>C4</td>
<td>300</td>
<td>300</td>
<td>5</td>
</tr>
<tr>
<td>C5</td>
<td>300</td>
<td>150</td>
<td>5</td>
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<tr>
<td>E1</td>
<td>300</td>
<td>300</td>
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<td>E2</td>
<td>300</td>
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<td>E3</td>
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<td>E4</td>
<td>300</td>
<td>300</td>
<td>5</td>
</tr>
<tr>
<td>E5</td>
<td>300</td>
<td>150</td>
<td>5</td>
</tr>
</tbody>
</table>

a Width-to-thickness ratio of the tube flange, where $b = B - 2t$.

b Width-to-thickness ratio of the subpanel, where $w = B/2$.

c Axial load contribution of the steel section $\delta_c = (F_yA_s + F_{ys}A_{ss})/(F_yA_s + F_{ys}A_{ss} + 0.85 f_c A_c)$.

d $w/t$ was considered.

Table 3-2. Comparison of chemical components between HSA800 and ATOS80

<table>
<thead>
<tr>
<th>Grade</th>
<th>C</th>
<th>Mn</th>
<th>P</th>
<th>S</th>
<th>Si</th>
<th>$C_{eq}$</th>
<th>$P_{cm}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSA800a</td>
<td>$\leq 0.20%$</td>
<td>$\leq 3.00%$</td>
<td>$\leq 0.015%$</td>
<td>$\leq 0.006%$</td>
<td>$\leq 0.55%$</td>
<td>$\leq 0.60%$</td>
<td>$\leq 0.30%$</td>
</tr>
<tr>
<td>ATOS80b</td>
<td>$\leq 0.20%$</td>
<td>$\leq 2.00%$</td>
<td>$\leq 0.03%$</td>
<td>$\leq 0.03%$</td>
<td>$\leq 0.40%$</td>
<td>$\leq 0.60%$</td>
<td>$\leq 0.30%$</td>
</tr>
</tbody>
</table>

a KS D 5994 quality standard

b Product of Korean steel maker POSCO
3.2.3 Fabrication and Test Setup

Built-up box sections fabricated by complete-joint-penetration (CJP) groove welding were used for all specimens [Fig. 3-4 (a)]. For grade 800 high-strength steel, flux cored wire of nominal tensile strength $F_{\text{EXX}} = 860$ MPa (AWS E121T-1) was used and 100% CO$_2$ shielding gas was used during the welding operation. For grade 400 mild steel, flux cored wire of nominal tensile strength 580 MPa (AWS E71T-1) was used. When the mild steel vertical stiffeners were welded to the high-strength tube plates, double-fillet welding using the lower strength weld material for mild steel was used [Fig. 3-4 (b)]. In manufacturing stiffened tubes, stiffeners were first welded to individual wall plates, and then box sections were assembled.

In all specimens, the column length was 900 mm which was three times the flange width $B = 300$ (Fig. 3-5). In beam-column specimens, the net column length excluding rigid ends was $L_c = 700$ mm (Fig. 3-6). In order to relieve the stress concentration at the top and bottom of the columns and to prevent the corresponding premature failure, exterior stiffeners of 100 mm depth were welded at the column ends.

Fig. 3-5 shows the test setup for concentric axial loading. The axial force was applied to the top end plate by a 10,000 kN UTM. The loading was controlled by the vertical displacement of 0.003 mm/s. Four LVDTs were used to measure the vertical displacements at the four corners of the endplates. Strain gauges were used to measure the axial and lateral strains of the tube plates.

Fig. 3-6 shows the test setup for eccentric axial loading. An axial force was applied at the top of the column through knife-edges, which were used to realize the
hinge conditions. The column height between the hinges was \( L_e = 1,720 \) mm. The loading was controlled by the vertical displacement of 0.005 mm/s. Two LVDTs were used to measure the vertical displacements in the compression and tension sides, and four LVDTs were used to measure the horizontal displacements of the deflected column.

In actual construction of CFT columns, non-shrinkage concrete is used to avoid the shrinkage of the core concrete. In the test specimens, however, to avoid the complexity of concrete mix design, an ordinary concrete mix design was used. For this reason, shrinkage of the concrete occurred during curing. Therefore, in order to apply the uniform axial load to the steel tube and concrete, the high-strength epoxy was grouted to the casting hole (one \( \Phi 150 \) hole in square specimens and two \( \Phi 100 \) holes in rectangular specimens) for the surface-treatment one week before testing.

![Fig. 3-4. Fabrication of steel tubes for column specimens](image)
Fig. 3-5. Test set-up for concentrically loaded specimens

Fig. 3-6. Test set-up for eccentrically loaded specimens
3.3 Test Results

3.3.1 Stub-Columns Under Pure Compression

Fig. 3-7 shows the axial load-strain relationship of the specimens. The axial strain of the specimens was estimated from two measurements: the local strains at the mid-height section were measured from individual strain gauges attached to the tube walls (dotted lines), and the global strain was calculated from the average value of the measurements of LVDTs (solid lines). In the case of C2, reliable displacements were not measured from the LVDTs. Instead, the behavior of C2 can be seen in Fig. 3-8 showing the UTM-measured axial force-deformation relations of the specimens. Loading was terminated when the load-carrying capacity decreased to 70% of the peak load.

The test results are summarized in Table 3-3. Among C1, C3, and C4 with the same square section, the strength of C4 (high-strength steel + stiffeners) was the highest, and the strength of C3 (low-strength steel) was the lowest. Similarly, in the rectangular sections, the strength of C5 (high-strength steel + stiffeners) was greater than that of C2 (high-strength steel). In all specimens except C1, the test strength reached the plastic axial strength \( P_p \). The deformation capacity of C4 and C5 with stiffeners was greater than that of the specimens without stiffeners. In the case of C2 with high strength concrete \( (f'_c = 83.6 \text{ MPa}) \), the post-peak behavior was brittle, losing the load-carrying capacity immediately after the peak strength (Fig. 3-8). This is because due to the relatively slender steel section, the axial load contribution of the concrete was significantly high and crushing of the infill concrete was not restrained due to the limited lateral confinement of the slender steel tube. Such brittle
failure mode was also reported by an existing study (Varma 2000), where high-strength concrete was used for non-compact CFTs ($f_c' = 110$ MPa, $F_y = 660$ MPa, $b/t = 48$, $\lambda_{coeff} = 2.76$). On the other hand, C5 with stiffeners showed relatively ductile behavior.

Fig. 3-9 shows failure modes of the specimens. Generally, elastic local buckling occurred in the specimens without stiffeners, while local buckling was restrained in the specimens with stiffeners. Such behavior can be confirmed by the longitudinal strains measured by strain gauges (Fig. 3-7). In specimens C1, C2, and C3 without stiffeners, strain reversal occurred at early loading stages, due to the local buckling. On the other hand, in specimens C4 and C5 with stiffeners, there was no sign of local buckling until the peak load. This result indicates that the stiffeners successfully restrained local buckling of the slender plates. As seen in Fig. 3-9, the buckling mode was obviously different in the stiffened specimens. Ultimately, the load-carrying capacity of the specimens began to decrease due to the crushing of the infill concrete.

In square specimens C1 and C4, at 71% and 74% of the peak load after the peak load, respectively, weld fracture occurred at the joint of the tube plates. Thus, special attention is required in the welding of high-strength steel plates. In rectangular specimens, weld fracture did not occur.

In the specimens, the yield strain of the high-strength steel is $F_y/E_s = 0.0037$ which is higher than the crushing strain of concrete ($\varepsilon_{co} = 0.0094\times(f_c')^{0.25} = 0.0028$ under pure compression, where $f_c' = 83.6$ MPa). Thus, if there is no adequate lateral confinement for the infill concrete, crushing of the infill concrete should occur before yielding of the steel tube. However, as shown in Fig. 3-7, the test strength of C3, C4,
and C5 with compact section or stiffeners reached the plastic strength $P_p$. On the other hand, in the case of C1 with the slender section, the test strength was 8% lower than the plastic strength. In the case of C2, unlike C1, the test strength reached the predicted plastic strength. The reason can be explained as follows, assuming the epoxy filling at the top was not perfect. In this case, as shrinkage of the infill concrete occurs, the majority of the axial force is loaded to the steel section at early loading stage. Due to the preloading, despite the large yield strain, the high-strength steel can develop the yield strength before crushing of the infill concrete. Such behavior is confirmed by the low stiffness of C2 in Fig. 3-7(b): the initial stiffness of C2 was not fully developed when compared to that of C5. Since the yield strength of high-strength steel can be developed, the effect of pre-loading could be beneficial for CFT columns with high strength steel. On the contrary, Han and Yao (2003) reported that, in the case of CFT columns with conventional steel ($F_y = 340$ MPa), pre-loading decreased the strength.

Fig. 3-7 also shows residual strengths $P_r$ predicted by Yamamoto et al. (2015). The residual strengths were calculated as $P_r = P_p(\delta_s)^{a}$, where $a = 132F_y/E_s+0.22$. In general, the predictions agreed with the test residual strengths.
Fig. 3-7. Axial load-strain relationships of stub-column specimens
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Fig. 3-8. UTM-measured axial load-displacement relationship of stub-column specimens

Fig. 3-9. Failure modes of stub-column specimens at the end of the test

Table 3-3. Strength evaluation of stub-column specimens

<table>
<thead>
<tr>
<th>Spec.</th>
<th>Test</th>
<th>AISC</th>
<th>AIJ</th>
<th>FEA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(P_{test})  (kN)</td>
<td>(P_{AISC})  (kN)</td>
<td>(P_{test} / P_{AISC})</td>
<td>(P_{test} / P_{AIJ})</td>
</tr>
<tr>
<td>C1</td>
<td>8,686</td>
<td>7,307</td>
<td>1.19</td>
<td>7,809</td>
</tr>
<tr>
<td>C2</td>
<td>6,152&lt;sup&gt;a&lt;/sup&gt;</td>
<td>-</td>
<td>-</td>
<td>4,730</td>
</tr>
<tr>
<td>C3</td>
<td>6,602</td>
<td>6,816</td>
<td>0.97</td>
<td>6,934</td>
</tr>
<tr>
<td>C4</td>
<td>9,726&lt;sup&gt;b&lt;/sup&gt;</td>
<td>9,730&lt;sup&gt;b&lt;/sup&gt;</td>
<td>1.00</td>
<td>9,847&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
<tr>
<td>C5</td>
<td>6,253&lt;sup&gt;b&lt;/sup&gt;</td>
<td>6,305&lt;sup&gt;b&lt;/sup&gt;</td>
<td>0.99</td>
<td>6,372&lt;sup&gt;b&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<sup>a</sup> Test strength of C2 was exaggerated due to unideal testing condition

<sup>b</sup> Width-to-thickness ratio of the subpanel w/t considered
3.3.2 Beam-Columns Under Eccentric Axial Loading

Fig. 3-10 and Fig. 3-12 show the axial load-lateral deflection relationships of the specimens. The lateral deflection was measured at the mid-height of the columns. Loading was terminated when the load-carrying capacity decreased to 70% of the peak load. The test results are summarized in Table 3-4.

In the square columns E1, E3, and E4, the peak strength was the greatest in E4 (high-strength steel + stiffeners), and the lowest in E3 (mild steel). In particular, when compared to E1 (high-strength steel) and E4, E3 showed significant early stiffness degradation due to local buckling and early yielding of the tube flange, although the section was classified as a compact section. In the rectangular specimens, the load-carrying capacity of E5 (high-strength steel + stiffeners) was greater than that of E2 (high-strength steel). The test result indicates that the vertical stiffeners increased the load carrying capacity by restraining the plate local buckling and providing confinement to the filled concrete.

E1 attained the peak load at the mid-height deflection, $\Delta = 3.63$ mm, and E4 with stiffeners attained the peak load at $\Delta = 4.64$ mm. In E3 with mild steel, the peak load occurred at $\Delta = 4.42$ mm. In the rectangular specimens, E2 and E5, the peak loads occurred at $\Delta = 8.53$ and 8.96 mm, respectively.

Yield deflection $\Delta_y$ was defined by the secant stiffness connecting the origin and 75% of the peak load, and maximum deflection $\Delta_u$ was defined as the post-peak deflection corresponding to 85% of the peak strength (Fig. 3-11). Ductility was defined as $\mu = \Delta_u/\Delta_y$ (Park 1988). In the square specimens, the ductility was the greatest in E4 with $\mu = 4.97$. The ductility of E1 and E3 was $\mu = 3.85$ and 2.01,
respectively. In the rectangular specimens, E2 and E5, the ductility was \( \mu = 1.97 \) and 2.71, respectively.

Fig. 3-13 shows the failure modes of the specimens. In E1, E2, and E4, local buckling occurred near the center of the columns. On the other hand, in E3 and E5, local buckling occurred in the upper region of the columns. The local buckling initiated in the compressive flange and propagated to the neighboring webs.

In E3 with mild steel, local buckling of the compression flange occurred at \( P = 2,000 \) kN with yielding of the plate. Afterwards, the tangential stiffness significantly decreased. On the other hand, in E1 and E2 with high-strength steel, local buckling occurred in the slender flange plates at \( P = 4,000 \) and 1,500 kN, respectively. However, the load-carrying capacities continued to increase without significant stiffness degradation. In E4 and E5 with vertical stiffeners, local buckling occurred at \( P = 7,000 \) and 4,000 kN, respectively, which was close to the peak load. As shown in Fig. 3-13, the buckling width and length were decreased by the effect of the stiffeners. In E2, local buckling did not occur in the compact web plates. In all the specimens, failure occurred due to the crushing of the infill concrete, which was detected from the sound during testing.

In E1 and E4, at 77% of the peak load in the post-peak behavior, weld fracture occurred at the joint of the compressive flange and web plates. This result indicates that careful attention is required when welding high-strength steel plates.
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Fig. 3-10. Axial load versus mid-height lateral deflection relationship of beam-column specimens

Fig. 3-11. Definitions of yield and maximum deflections
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(a) E1
(b) E2
(c) E3
(d) E4
(e) E5

Axial load (kN) vs. Mid-height deflection (mm)

- Local buckling
- (Stiffened)
- Test result
- Fiber model analysis result

Axial strains of compressive flange
- Tube yielding $F_y/E_s$
  - $0.003$
  - $0.005$

(400MPa steel)
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**Fig. 3-12.** Comparisons of test results and fiber model analysis

**Fig. 3-13.** Failure modes of the specimens at the end of the test

Table 3-4. Test results of beam-column specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>$\Delta_y$ (mm)</th>
<th>$P_{test}/\Delta_y$ (kN/mm)</th>
<th>$\Delta$ (mm)</th>
<th>$P_{test}$ (kN)</th>
<th>$\Delta_u$ (mm)</th>
<th>$\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>E1</td>
<td>2.12</td>
<td>3,062</td>
<td>3.63</td>
<td>6,491</td>
<td>8.17</td>
<td>3.85</td>
</tr>
<tr>
<td>E2</td>
<td>5.68</td>
<td>682</td>
<td>8.53</td>
<td>3,873</td>
<td>11.18</td>
<td>1.97</td>
</tr>
<tr>
<td>E3</td>
<td>3.44</td>
<td>1,324</td>
<td>4.42</td>
<td>4,553</td>
<td>6.92</td>
<td>2.01</td>
</tr>
<tr>
<td>E4</td>
<td>2.67</td>
<td>2,671</td>
<td>4.64</td>
<td>7,131</td>
<td>13.27</td>
<td>4.97</td>
</tr>
<tr>
<td>E5</td>
<td>5.85</td>
<td>739</td>
<td>8.96</td>
<td>4,322</td>
<td>15.88</td>
<td>2.71</td>
</tr>
</tbody>
</table>
3.3.3 Strains of Steel Tubes

In the present study, initial imperfections such as residual stress and out-of-flatness of tube plates were not measured. Instead, initial buckling strains of the tube plates were evaluated using an existing empirical equation (Tort and Hajjar 2003), which was calibrated from the test results of concentrically loaded columns implicitly taking into account the influence of geometric imperfections and residual stress.

\[
\frac{\varepsilon_{\text{ini}}}{\varepsilon_y} = 3.14 \left( \frac{B}{t \sqrt{\frac{F_y}{E_s}}} \right)^{-1.48}
\]

(3-2)

In the present study, initial local buckling was detected by bare eyes and confirmed with the reversal of measured strains. For high-strength steel tubes C1, C2, C4, and C5, the predictions agreed with the test results (Fig. 3-14). On the other hand, the prediction overestimated the test result of mild steel tube C3. In the evaluation of C4 and C5, \( B \) was defined as the width of the subpanel \( w (= B/2) \), considering the effect of vertical stiffeners.
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Fig. 3-14. Initial local buckling strains in stub-column specimens

Fig. 3-15 shows the strain distributions at the mid-height tube section of E4 and E5 with stiffeners. In the case of specimens without stiffeners, the strains were not accurately measured due to local buckling at early loading. In Fig. 3-15, the horizontal axis refers to the distance from the center of the cross-section. Until 80% of the peak load, the strains were linearly distributed in both E4 and E5. At the peak load, the strains showed nonlinear distribution due to local buckling and yielding. However, the strains at the flange are apparently greater than 0.003, which is the ultimate compressive strain of ordinary concrete under axial compression. This result indicates that the deformation capacity of the concrete infill increased due to the lateral confinement of the steel tube. In E4 and E5, the local buckling strains were estimated as 0.0041 \((P = 7,000 \text{ kN})\) and 0.0034 \((P = 4,000 \text{ kN})\), respectively, which are comparable to the yield strain of 0.0036 \((= \sigma_y/E_s)\). This result indicates that the stiffeners successfully restrained the elastic local buckling, thus developing the yield strain. Fig. 3-16 shows a comparison between the measured buckling strains and the predictions based on Eq. (3-2).
Fig. 3-15. Distribution of axial strain in mid-height section of E4 and E5

Fig. 3-16. Flange local buckling strains of beam-column specimens
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3.4 Evaluation of Test Results

3.4.1 Finite Element Analysis

3.4.1.1 Modeling Overview

To investigate the nonlinear response and detailed structural behavior such as local buckling of the steel tube and confinement of the infill concrete, numerical studies were conducted using a finite element analysis program (ABAQUS). Fig. 3-17 shows the finite element model of the specimen. Considering the symmetry of the column cross-section and boundary conditions, a 1/8 model (1/8 of the volume of the test specimen) was used. Four-node reduced integration shell elements (S4R) and eight-node reduced integration brick elements (C3D8R) were used for the steel tube and concrete, respectively. Both material nonlinearity and geometric nonlinearity were considered.

Fig. 3-18 shows the uniaxial stress-strain relationships for the infill concrete without lateral confinement. The ascending part was defined by the Mander’s model (Mander et al. 1988), and the descending part was defined as a bi-linear model (Chen et al. 2012). The multi-axial behavior of the infill concrete with lateral confinement (i.e., due to the steel tube) was idealized by the damaged plasticity model (Lee and Fenves 1998). For the constitutive laws of tube plates, elastic-perfectly plastic relationships with the Mises yield surface were used because the effect of post-yield hardening behavior on the peak strength of the RCFT column is limited (Tao et al. 2013).

For the contact properties between concrete and steel plates, the “Hard”
contact option was used in the normal direction, which allows separation between the steel tube and concrete without penetration. The frictional coefficient in the tangential direction was taken as 0.25 (Thai et al. 2014). Considering computational efficiency and accuracy, the mesh sizes were chosen as $B/16$ ($B = 300$ mm) in the cross-section and $L/30$ ($L = 900$ mm) in the longitudinal direction. Clamped boundary conditions were applied at the loading plane using rigid body constraints, to consider the effect of welding between the steel tube and endplate. In actual CFT columns, imperfection (out-of-flatness) and residual stress are inherent in the steel tube. However, such imperfections were not considered in the present study.

![Fig. 3-17. Finite element model of column specimen](image-url)
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3.4.1.2 Evaluation of FEA results

Fig. 3-19 shows the axial load-strain relationships of FEA results. In the figure, the contributions of the infill concrete and steel tube are also shown. The response of each material was evaluated at the mid-height section of the column. Table 3-3 presents the peak strengths and the corresponding average strain obtained from the FE analysis. Generally, the numerical analysis results showed good agreement with the test results. In particular, the stiffness, strength, and post-peak behavior of specimens C4 and C5 with stiffeners were well-simulated by the numerical analysis. However, the strength of the specimens without stiffeners was overestimated, which shows that local buckling of steel plates occurred earlier than the expectation. Nevertheless, Fig. 3-19(a) and Fig. 3-19(c) show that the contribution of the high-strength steel of C1 was apparently greater than that of mild steel of C3 ($P_{test}/P_{FEA} = 0.94$ and 0.87, respectively).

In the numerical results, when the high-strength steel was used, the peak load of the specimens was attained when the contribution of the steel tube section reached...
the peak, at an axial strain slightly greater than the crushing strain (that corresponds to the peak strength) of the concrete. In C4 with stiffeners, the average axial strain corresponding to the peak strength was greater than that of C1 ($\varepsilon = 0.00326$ and 0.00288, respectively). On the other hand, the peak load of C3 with 300 MPa steel occurred when the axial strain reached the crushing strain $\varepsilon_{co}$ of the concrete ($\varepsilon_{co}$ was assumed as 0.0027 in Fig. 3-18).

In the FEA models, local buckling of steel plates occurred after yielding of the tube in case of the mild steel (C3) and after concrete expansion in case of the high-strength steel (C1), respectively. In the case of the specimens with stiffeners (C4 and C5), although the full yield capacity was not developed due to the local buckling after concrete crushing [Fig. 3-19(d) and Fig. 3-19(e)], significant strength degradation did not occur. In the FEA model, local buckling occurred in the middle of the column length where concrete crushing and the corresponding lateral expansion took place. Fig. 3-20 shows the outward deflection of the mid-height tube plates in C1 and C4. The FEA results are presented in several axial load levels before and after the peak load. In both specimens, out-of-plane deformations were not significant before the peak load. At the peak load, the center deflection of C1 was greater than that of C4 (1.79 and 1.18 mm, respectively). In C4 (with stiffeners), after the peak load, the out-of-plane deformation was limited when compared to that of C1 (without stiffeners), maintaining the large contribution of the tube plates as shown in Fig. 3-19(d).

Generally, in the design of RCFT members, the confinement effect of the infill concrete is neglected. This is confirmed from the FE analysis results (Fig. 3-19): the strength increase of the confined concrete did not occur even in the stiffened specimens (the peak strength of the concrete was close to $1.0f_c'$), though the residual
strength was maintained due to the confinement effect of the steel tube. The ductile behavior of specimens C4 and C5 was mainly attributed to the restrained local buckling of the stiffened steel tube, rather than the confinement effect of the infill concrete.

Fig. 3-19. Axial response of FEA models
Fig. 3-20. Out-of-plane deformation of tube wall in C1 and C4 (FEA results)
### 3.4.2 Section Analysis Using Fiber Models

In order to evaluate the overall behavior of the specimens, sectional analysis using fiber elements was performed, and the analysis results were compared with the test results. For this study, a constitutive model of laterally confined concrete developed by Sakino and Sun (1994) and Nakahara et al. (1999) was used. As is well known, the confinement effect of the rectangular steel tube is not significant when compared to that of circular steel tube. Thus, in the constitutive model, the confinement effect does not increase the peak strength, though it increases the deformation capacity. The concrete stress-strain relationship is defined as follows:

\[
Y = \frac{VX + (W - 1)X^2}{1 + (V - 2)X + WX^2} \quad (3-3)
\]

\[
X = \frac{\varepsilon_c}{\varepsilon_{co}} \quad ; \quad Y = \frac{\sigma_c}{f'_c} \quad (3-4)
\]

\[
V = \frac{E_c \varepsilon_{co}}{f'_c} \quad (3-5)
\]

\[
W = 1.5 - 0.0171f'_c + 2.39\sqrt{f_{rc}} \quad (3-6)
\]

\[
f_{rc} = \frac{1}{2} \rho_y F_y \left(\frac{t}{b}\right) \quad (3-7)
\]

where \(\varepsilon_c\) and \(\sigma_c\) = axial strain and stress of the concrete, respectively, and \(\varepsilon_{co}\) = axial strain corresponding to the concrete compressive strength \(f'_c\). \(E_c\) = elastic modulus of concrete. \(W\) = parameter related to descending branch of the stress-strain relationship.
and $f_{ce} =$ effective confining pressure. $\rho_t =$ volumetric ratio of the steel tube to concrete $= 4(B - t)t/b^2$.

Fig. 3-12(f) shows the stress-strain relationship of the concrete infill for the test specimens. In the case of E4 and E5, the effect of stiffeners was addressed by replacing $t/b$ in Eq. (3-7) with $t/w$ (Fig. 3-21). Strength degradation due to the size effect was neglected in the model.

For the uniaxial stress-strain relationship of steel, a trilinear model was used [Fig. 3-12(f)]. For the slender plates of E1 and E2, the elastic local buckling stress $F_{cr} = 9E_s(b/t)^2 = 535$ MPa (AISC 2010) was used as the peak stress. In the case of E3, since the tube section is compact, it was assumed that the yield strength could be developed. In the analysis of E4 and E5, considering the effect of stiffeners, the tube plates were regarded as a compact section which can develop the yield strength. The descending branch representing the inelastic buckling mechanism was described based on the analytical model of Fujimoto et al. (2004).

Using the material constitutive models and linear strain distribution, fiber model sectional analysis was performed for the mid-height section of the specimens. To address the column slenderness effect, the relationship $\phi = \Delta(\pi/L_e)^2$ between the mid-height curvature $\phi$ and deflection $\Delta$ was assumed, relating the local response to the global response. Iterative calculations were performed to satisfy the force equilibrium at the mid-height section, considering the second-order effect.

Fig. 3-12 compares the results of the test and fiber model analysis. The figure also shows the reference points of the axial strains of the compressive flange that represent the main events: the crushing strain (0.003) of the unconfined concrete; the yield strain ($F_y/E_s = 0.0036$ for grade 800 steel and 0.0015 for grade 400 steel); and
the crushing strain (0.005) of the confined concrete for CFTs (Tomii and Sakino 1979). Generally, the numerical analysis results showed good agreement with the test results in terms of initial stiffness and strength. Except for E3, the test results were 98% ~ 101% of the predictions. In E3, on the other hand, the prediction overestimated the stiffness and strength of the test specimen. This indicates that in the case of mild steel, even considering the post-buckling deterioration of the tube plates [Fig. 3-12(f)], the analysis results still significantly overestimate the test results. It seems that the performance degradation of concrete infill also occurred due to the combined effect of local buckling and yielding of the mild steel. For conservative design, as mentioned in previous studies (Fujimoto et al. 2004; Sakino et al. 2004; Liang 2009), a strength reduction factor which accounts for the size effect might be necessary for the modeling of concrete.

In the numerical analysis of all specimens, the peak loads occurred when the extreme compressive strain exceeded 0.003. In E4 and E5 with stiffeners, the peak loads occurred at the extreme compressive strain = 0.00464 (mid-height deflection = 4.60 mm) and 0.0037 (mid-height deflection = 7.13 mm), respectively. The corresponding mid-height deflections were smaller than the measured deflections, 4.64 and 8.96 mm of the specimens.

![Fig. 3-21. Effect of stiffeners on arch action in the cross section](image)
3.4.3 Strength Prediction of Current Design Codes

The test strengths were compared with the predictions of current design codes (CEN 2004; SA 2004; AIJ 2008; AISC 2010). In Eurocode 4 (CEN 2004), only a design method for compact sections (rigid-plastic approach) is provided, and the slenderness requirement for the compact section \( [B/t \leq 1.78\sqrt{(E_s/F_y)}] \) is stricter than that of ANSI/AISC 360-10 \( [b/t \leq 2.26\sqrt{(E_s/F_y)}] \), recommending different concrete strength factors 1.0 and 0.85, respectively. The slenderness requirement of AS 5100 (SA 2004) is even stricter not considering the beneficial effects of the infill concrete. Both Eurocode 4 and AS 5100 do not clearly specify design methods for CFT columns with plate slenderness exceeding the slenderness requirement. On the other hand, AIJ recommendations specify design methods for both compact section and slender section.

Table 3-3 presents the ratios of the test peak load \( P_{test} \) to the predictions based on current design codes. In C1 and C2 with slender sections, the test strengths exceeded the AISC predictions by a considerable margin \( (P_{test}/P_{AISC} = 119\% \text{ and } 130\%) \). In C4 and C5 with stiffeners, the test strengths reached the plastic capacity \( (P_{test}/P_{AISC} = 100\% \text{ and } 99\%) \) where the yield contribution \( (A_{ps}F_{ys}) \) of stiffeners was included in the calculation. In the case of the AIJ predictions, the ratios of the test results to the predictions were \( P_{test}/P_{AIJ} = 95\% \text{ ~125\%} \).

Fig. 3-22 compares the axial-flexural capacity of the beam-column specimens and the predictions of ANSI/AISC 360-10. Considering the second-order effect, the mid-height flexural moment was calculated as \( M = P(e + \Delta) \). In the
specification, the plastic stress distribution method (Method 2) is permitted only for compact sections, and the equivalent steel column method (Method 1) is recommended for non-compact and slender sections.

In ANSI/AISC 360-10, considering the length effects, the axial capacity of each reference point in the capacity curve is degraded. Thus, the interaction diagrams in Fig. 3-22 indicates the nominal beam-column capacity of specimens. Consequently, 3% of the axial capacity was decreased in E1 and E4, 2% in E3, and 12% in E2 and E5. The reduction was the greatest in the rectangular specimens, E2 and E5, because of the greater slenderness ratio.

In Fig. 3-22, all specimens except E3 with mild steel satisfied the nominal axial-flexural strength predicted by AISC specification. E4 and E5 with stiffeners (high strength steel compact sections) reached their plastic strengths (Method 2). E1 and E2 (high strength steel slender sections) were close to the plastic strength, significantly exceeding the nominal strength (Method 1). In the case of E3, although the mild steel compact section was used, the test strength was smaller than the prediction of Method 2.

As mentioned previously, Method 1 of ANSI/AISC 360-10 significantly underestimated the ultimate strength of the RCFT columns E1 and E2 with high-strength steel slender sections. The over conservatism of Method 1 was also reported in previous studies for normal strength steel (Lai et al. 2015).

Fig. 3-22 also shows the interaction curve based on the recommendations of Architectural Institute of Japan (AIJ) (Nakahara and Sakino 2003; AIJ 2008). In the method, which is similar to the rigid-plastic method for the steel tube, a uniform effective buckling stress is used for the compression zone, and a uniform yield
strength is used for the tension zone. For the compressive stress of concrete, the stress distribution varies according to the strength of the concrete. The predictions showed good agreement with the test results for the unstiffened CFT columns. In the case of E1 and E2, the prediction is much greater than that of Method 1 for the high-strength steel slender section, while being conservative against the test strengths. In the case of E3, the prediction is comparable to that of Method 2 for the mild steel compact section.

Fig. 3-22. Axial-flexural strengths of specimens
3.5 Design of Stiffener Rigidity

Former experimental (Tao et al. 2005) and numerical (Lee and Yoo 2012) studies on the design of stiffened RCFT columns were conducted only for mild steel. Thus, the design methods of the stiffener need to be further studied for high-strength steel RCFT columns. Also, in previous studies (Tao et al. 2005; Petrus et al. 2010), the steel grade of stiffeners was the same as that of the tube plate or even higher. On the other hand, in the present study, the steel grade of stiffeners was lower than that of the tube plate, using mild steel for the stiffeners. In this section, a theoretical design method of stiffeners is developed for high-strength steel RCFT columns.

Fig. 3-23 illustrates the deformed buckling shape of concrete-filled steel tubes. According to experiments by Uy (1998), due to the rigidity of the boundaries and the concrete infill, the effective length of a buckled plate in the elastic state was close to the tube width \( B/2 \) (Fig. 3-23), which agrees with the theoretical results (Uy and Bradford 1996). Thus, the effective buckling length \( KL \) of the stiffener plate can be assumed to be \( B/2 \), and the corresponding buckling resistance can be defined as follows:

\[
P_{cr,s} = \frac{\pi^2 E_s I_{s,s}}{(KL)^2}
\]

where \( I_{s,s} = \) second-moment of inertia of the stiffener (= \( t_s b_s^3/12 \)) and \( E_s = \) tangent modulus of the steel. In the specimens tested in the present study, the stiffeners of the mild steel yielded prior to the high-strength steel tube. Thus, the elastic modulus
$E_s$ should not be used for $E_t$ in Eq. (3-8).

The uniaxial stress-strain behavior of mild steel is generally idealized with three regions: elastic region, yield plateau, and hardening region. The apparent tangent modulus in the yield plateau is almost zero (Fig. 3-3). However, according to the slip theory (White 1960; Lay 1965), the tangent modulus ranges from the elastic modulus $E_s$ to the initial hardening modulus $E_{st}$. This is because the yield deformation of mild steel is caused by a series of discontinuous slips. In order to define the inelastic modulus, the slip theory was applied as follows (Lay 1965):

$$E_s = \left[ \frac{1}{E_{st}} - \frac{1}{E_s} \left( \frac{\varepsilon - \varepsilon_y}{\varepsilon_{st} - \varepsilon_y} \right) + \frac{1}{E_s} \right]^{-1}$$  \hspace{1cm} (3-9)

Fig. 3-23. Deformed shape and length of tube local buckling
where $\varepsilon_{st}$ = strain corresponding to the initial strain hardening and $\varepsilon$ = the measured strain of the tensile coupon. The strain $\varepsilon$ varies between $\varepsilon_y$ and $\varepsilon_{st}$. Eq. (3-9) was derived by addressing contributions of the plastic deformation of the slip planes and the elastic deformation of the regions without slip. Fig. 3-24 shows an example of the tangent modulus for the mild steel (Fig. 3-3). As the strain $\varepsilon$ increases, the tangent modulus $E_t$ decreases.

![Diagram](image)

(a) Result of coupon test

![Diagram](image)

(b) Tangent modulus

Fig. 3-24. Tangent modulus of Grade 400 steel (stiffener)
Even for an identical steel grade (i.e., the same yield strength), the post-yield properties such as $E_{st}$ and $\varepsilon_{st}$ differ depending on the manufacturing process. The properties of mild steel generally belong to the range of $E_{st} = 1/50 \sim 1/33$ times $E_s$ and $\varepsilon_{st} = 7 \sim 11$ times $\varepsilon_y$ (Park 2015).

Generally, in RCFT columns, the peak loads are attained when the axial strain reaches the crushing strain of the filled concrete. In the present study, following Tomii and Sakino (1979), the crushing strain of the infill concrete at the peak strength was defined as 0.005. Using the compressive strain of 0.005, $F_{ys} = 300$ MPa, $E_{st} = 1/50 \sim 1/33E_s$, and $\varepsilon_{st} = 7 \sim 11\varepsilon_y$, the tangent modulus of a stiffener is assumed as $E_t = 0.05 \sim 0.12E_s$.

The buckled wall area to which the stiffener should provide stiffness was estimated using effective width method (Fig. 3-25). The axial compression on the region $P_d$ (i.e., the demand force for the stiffener) is calculated as follows:

$$P_d = (b - b_e) t F_y$$  \hspace{1cm} (3-10)

$$\frac{b_e}{b} = \alpha \sqrt{\frac{F_{cr}}{F_y}}$$  \hspace{1cm} (3-11)

where $b_e = $ effective width of the steel tube, $F_{cr} = $ elastic local buckling stress = $9E_s/(b/t)^2$, and $\alpha$ is the reduction coefficient that accounts for residual stress and imperfection, which is taken as 0.65 (SA 1998; Uy 2001).
By using $P_{cr,s} = P_d$ in Eqs. (3-8) and (3-10), the required second moment of inertia $I_{s,s}$ of the stiffener can be calculated. When $E_t = 0.05E_s$ and $B = b$ are assumed, the required stiffener rigidity is expressed as follows:

$$I_{s,s} = 5.67 \times 10^{-3} \left( \frac{w}{l} \right)^3 \frac{F_y}{280} t^4 - 3.69 \times 10^{-3} \left( \frac{w}{l} \right)^3 \sqrt{\frac{F_{cr,s} F_y}{280}} t^4 \quad (3-12)$$

The first term accounts for the required stiffener rigidity corresponding to the entire width $b$ of the tube plate. The second term is used to reduce the stiffener rigidity considering the effective width of the tube plate. The first term of Eq. (3-12) is similar to the empirical Eq. (3-1) proposed by Tao et al. (2005).

In Fig. 3-26, the proposed method was compared with existing design methods. In the comparison, the mild steel of $F_y = 300$ MPa and high-strength steel of $F_y = 746$ MPa were considered for the tubes. The proposed design range of $E_t =$
0.05 ~ 0.12E_s for the mild steel stiffener was used. In the prediction of stiffener rigidity, the width B was limited to satisfy \( b/t > \lambda_p \) (non-compact or slender tube plates) and \( w/t < \lambda_p \) (compact subpanels) because stiffeners are required for non-compact sections and slender sections.

For both mild steel and high strength steel tubes, the results of the proposed method were close to those of Eq. (3-1). On the other hand, the predictions by Lee and Yoo (2012) were significantly higher than the others. This is because the numerical study was performed for highly slender subpanels \( (F_y = 345, w/t = 57.1) \). When \( w/t = 57.1 \), even with stiffeners, the subpanel was classified as a non-compact section \( (w/t > \lambda_p = 54.4) \).

In Fig. 3-26(b), the rigidity of the stiffener required for the test specimens \( (B = 300 \text{ mm}) \) was \( I_{s,re} = 47,800 ~ 114,700 \text{ mm}^4 \). The corresponding width of the stiffener was \( b_s = 49 ~ 65 \text{ mm} \). This result agreed with the stiffener of \( b_s = 60 \text{ mm} \), which was used for the test specimens. The stiffeners successfully restrained local buckling of the test specimens with the slender section.

![Graph showing comparison of existing methods and proposed method for stiffener rigidity](image-url)
3.6 Discussion

In the present study, concentric and eccentric axial loading tests were performed on RCFT columns with high-strength steel slender section. The test parameters included the yield strength of steel (746 MPa and 301 MPa), the width-to-thickness ratio (slender section and compact section), axial-load eccentricity, and the use of stiffeners. The major findings of this study are summarized as follows:

1) The structural performance of the test specimens was quite consistent regardless of the axial-load eccentricity probably due to its low value.

2) In specimens with 400 MPa mild steel, although the section was classified as a compact section, early local buckling occurred, followed immediately by yielding. At this point, the stiffness was degraded. In the case of eccentrically loaded specimen, the peak strength did not reach the plastic capacity. In the case of stub-column specimen, however, it was confirmed that the peak load agreed with the predicted plastic strength of the composite section. This is because the influence of the strength contribution of the steel tube was limited due to the low yield strength (i.e. low strength contribution).

3) In the slender section specimens with 800 MPa high-strength steel, although early elastic local buckling occurred, the load carrying capacity continued to increase without significant decrease of the stiffness. When compared with the plastic prediction, the shortage of peak loads was less than 8%.

4) In the compact section specimens strengthened with stiffeners, elastic local
buckling was restrained, and the load carrying capacity reached the plastic strength of the composite section, showing relatively ductile behavior.

5) The results of FE analysis showed that the ductile behavior of C4 and C5 is mainly attributed to the restrained local buckling of the stiffened steel tube rather than the effect of the confined infill concrete. The FEA results slightly overestimated the strength of specimens without stiffeners, and agreed with the strengths of specimens with stiffeners.

6) The results of FE analysis showed that when the high-strength steel was used, the peak load of the specimens was attained when the contribution of the steel tube reached the peak. On the other hand, when the 300 MPa steel was used, the peak load occurred when the axial strain reached the crushing strain of the concrete.

7) The predictions of fiber model numerical analysis agreed with the test peak strengths of the high strength specimens. On the other hand, for E3 with mild steel, the prediction overestimated the test peak strength and stiffness.

8) The test strength of the specimens was compared with the predictions by ANSI/AISC 360-10. Although E3 with mild steel was classified as a compact section, the test strength of E3 did not reach the plastic strength of Method 2. The test strengths of E4 and E5 with stiffeners and high-strength steel agreed with the plastic strength of Method 2. On the other hand, the equivalent steel section method (Method 1) significantly underestimated the test strengths of slender sections E1 and E2 with high strength steel.

9) A design method for stiffeners was theoretically developed for high-strength steel CFT columns. The proposed method agreed with an existing empirical
equation and satisfactorily predicted the required rigidity of stiffeners of the test specimens.
Chapter 4. Development of Analytical and Design Models for CFT Columns

4.1 Introduction

Available material strengths for composite structures are limited in current design codes (Table 2-1). The high-strength concrete is not recommended owing to its brittle behavior. In case of high-strength steel, the restriction mainly comes from the incompatibility between yield strain of the steel and crushing strain of the concrete: the early crushing of the concrete may occur prior to the development of plasticity in the steel components. However, such concerns have been partly solved by experimental evidences, and the results are being reflected to new design codes. Recently, most research topics on CFTs are concentrated on the high-strength materials (Table 2-2 and Table 2-5), aiming to break the limitations.

According to the existing test results (Table 2-2), by using high-strength materials, the structural performance of RCFT column was definitely enhanced in terms of strength and stiffness, while many test results also revealed that the ultimate strengths were generally overestimated by existing design methods. However, since most studies have been limited to individual test results, the effects of material strengths and sectional slenderness should be further clarified with comprehensive databases. In order to re-evaluate the code limitations on high-strength materials, very recent studies on the design of CFT members have focused more on high-strength materials (Table 2-5).
As such, a number of experiments and attempts to develop analytical and design models for high-strength materials has been carried out. Because the studies were often limited to compact sections, however, adaptation of the new materials to current design codes, which are also required to accompany noncompact and slender section, inevitably encountered difficulties.

In order to expand the applicability of high-strength steels in the current design standards, it is essential to clarify the behavioral characteristics accounting for various sectional slenderness as well as material grades. This chapter focuses on assessing the load-carrying capacities of CFT columns and beam-columns. In particular, careful attention was paid to the effects of not only high-strength materials but also slender tube section, which have often not been addressed in previous studies. First, the applicability of current design codes is examined based on extensive databases of RCFTs in literatures, and effects of the design parameters are discussed. For better strength assessment, improved strain-based fiber models which are essential for the high-strength steels are proposed.
4.2 Applicability of Current Design Codes

4.2.1 Selected Test Specimens

In the present study, to extend the applicability of current design codes for RCFT columns covering wide ranges of material strengths \( f_c' > 70 \text{ MPa}, F_y > 525 \text{ MPa} \) (AISC 2010) and sectional slenderness, relevant test results were extensively collected. The constructed database is categorized into two parts: 1) short columns for axial capacity and 2) beam and beam-columns for flexural and combined axial-flexural capacity. The properties of existing test specimens are summarized in Table 4-1.

In selecting test specimens, careful attention was paid to obtain reasonable results: 1) a wide range of material strengths and sectional slenderness were included, 2) exceptionally small specimens were excluded, and 3) the compressive strength of concrete was modified considering the shape and size of the material test specimens. Small-scale specimens with \( B \times H < 180^2 \text{ mm}^2 \) were excluded to minimize the size effect (Nakahara et al. 1998).

195 RCFT columns were collected from 36 studies in literature. The ranges of the test parameters were as follows: \( 180^2 \leq B \times H \leq 500^2 \text{ mm}^2 \), \( 211 \leq F_y \leq 835 \text{ MPa} \), and \( 20 \leq f_c' \leq 202 \text{ MPa} \), \( 2.8 \leq L/H \leq 4.8 \), \( 0.47 \leq \text{slenderness coefficient } \lambda_{coeff} \leq 4.94 \), and \( 10.4 \leq b/t \leq 131.7 \).

210 RCFT beam-columns were collected from 39 studies in literature. The ranges of the test parameters were as follows: \( 180^2 \leq B \times H \leq 600^2 \text{ mm}^2 \), \( 211 \leq F_y \leq 913 \text{ MPa} \), and \( 20 \leq f_c' \leq 183 \text{ MPa} \), \( 4.0 \leq L/H \leq 24.0 \), \( 0.65 \leq \text{slenderness coefficient } \lambda_{coeff} \leq 4.94 \).
\[ \lambda_{\text{coeff}} \leq 4.46, \text{ and } 13.6 \leq b/t \leq 106.7. \]

A variety of test set-ups were used for existing beam-column test specimens. Fig. 4-2 shows typical loading conditions applied in literature. The test strengths measured at the critical section, considering the additional moment due to the axial force (i.e., second-order effect), were used for the strength evaluation of cross-section.

The concrete compressive strength \( f'_c \) was defined as the strength of an equivalent concrete cylinder with 100 mm diameter. According to Yi et al. (2006), for normal-strength concrete \( (f'_c \leq 70 \text{ MPa}) \), an equivalent cylinder (100 mm diameter) strength can be obtained by using modification factors of 1.03, 0.85, and 0.91 for cylinder with 150 mm diameter, cube with 100 mm width, and cube with 150 mm width, respectively. In the case of high-strength concrete \( (f'_c > 70 \text{ MPa}) \), the modification factors are 1.04, 0.96, and 1.02, respectively. In this study, concrete strength greater than 70 MPa was regarded as a high strength.

![Fig. 4-1. Section width of existing test specimens](image)

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Chapter 4. Development of Analytical and Design Models for CFT Columns

Fig. 4-2. Test set-ups for beam-column specimens

Table 4-1. Parameter ranges of existing test specimens

<table>
<thead>
<tr>
<th>Test parameters</th>
<th>Stub-columns</th>
<th>Beam-columns</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of specimens</td>
<td>195</td>
<td>210</td>
</tr>
<tr>
<td>$B$</td>
<td>180 ~ 500 mm</td>
<td>149 ~ 600 mm</td>
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<tr>
<td>$H$</td>
<td>135 ~ 500 mm</td>
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<td>$b/t$</td>
<td>10.4 ~ 131.7</td>
<td>13.6 ~ 106.7</td>
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<td>$F_y$</td>
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<td>211 ~ 913 MPa</td>
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<td>$(b/t)\sqrt{(F_y/E_s)}$</td>
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<td>0.65 ~ 4.46</td>
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<tr>
<td>$f_c'$</td>
<td>20.0 ~ 202.0 MPa</td>
<td>20.0 ~ 183.0 MPa</td>
</tr>
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</table>
Table 4-2. Test parameters of existing RCFT short columns

<table>
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<tr>
<th>Reference</th>
<th>Number of specimens</th>
<th>$B^a$ (mm)</th>
<th>$H^a$ (mm)</th>
<th>$b/t$</th>
<th>$F_y$ (MPa)</th>
<th>$\lambda_{\text{coeff}}$</th>
<th>$f_c'$ (MPa)</th>
<th>$L/H^b$</th>
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<td>200-251</td>
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<td>200-250</td>
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<td>135-500</td>
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\( a \geq H \)

\( b \) Both column ends fixed (i.e. \( KL = 0.5L \))
# Table 4-3. Test parameters of existing RCFT beam-columns

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<tr>
<th>Reference</th>
<th>Number of specimens</th>
<th>$B$ (mm)</th>
<th>$H$ (mm)</th>
<th>$b/l^c$</th>
<th>$F_y$ (MPa)</th>
<th>$\lambda_{\text{coeff}}$</th>
<th>$f_c$ (MPa)</th>
<th>$KL/H$</th>
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<tr>
<td>Lu &amp; Kennedy (1992)$^b$</td>
<td>8</td>
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<td>252-253</td>
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### Chapter 4. Development of Analytical and Design Models for CFT Columns

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<th>Study</th>
<th>Samples</th>
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<th>Flange Thickness (mm)</th>
<th>Axial Load (kN)</th>
<th>Axial Displacement (mm)</th>
<th>Flange Width-to-Thickness Ratio</th>
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<td>70.5-83.6</td>
<td>5.7-11.5</td>
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<td>Wang et al. (2017)</td>
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<td>150-600</td>
<td>13.6-106.7</td>
<td>211-913</td>
<td>0.65-4.46</td>
<td>20.0-183.0</td>
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a Flange width-to-thickness ratio

b Contains specimens with $H > B$

c $0$ indicates pure-flexural testing (no axial force)
4.2.2 Strength Evaluation

Firstly, test strengths of stub-column and beam-column specimens were compared with the predictions of current design codes (AISC 2016; AIJ 2008; CEN 2004). On the basis of the results, effects of design parameters such as material strengths and width-to-thickness ratio were examined.

Fig. 4-3 shows the comparison between test strengths and predictions for stub-columns. Generally, the AISC code shows unsafe predictions for noncompact specimens with high-strength materials. Thus, in the AISC code, the use of high-strength materials is restricted. On the other hand, the conservative nature of the AISC predictions significantly increases as the slenderness ratio increases. Although overall accuracy of the AIJ predictions seems better, conservatism of the high-strength materials in noncompact and slender sections is still inadequate. It was confirmed that Eurocode 4 is only applicable for test specimens with specified material strengths \((F_y \leq 460 \text{ and } f'_c \leq 50 \text{ MPa})\) and sectional slenderness \([B/t \leq 1.78\sqrt{(E_s/F_y)}]\).
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Fig. 4-3. Strength evaluation of stub-column specimens

Fig. 4-4 shows typical behavior of slender columns. In general, even after the lateral load starts to decrease (or after the eccentric axial load starts to decrease), which indicates the instability of the slender column, the moment demand at the critical section may increase in the case of slender columns, which indicates the failure of the cross section (Perea et al. 2014). Therefore, for such slender columns (currently assumed as aspect ratio $KL/H > 20$), the test strengths measured at the peak moment were used for the strength evaluation of cross-section. The slenderness
effect is more prominent in circular CFT (CCFT) columns than in RCFT columns. Thus, as far as $KL/H \leq 20$, the test strengths measured at the peak lateral load (or the peak eccentric axial load) were accepted as the cross-section capacity. It is also noted that test results obtained from experimental set-ups, which may be affected by base confinement, were omitted in strength evaluation (AIJ, 2008). Although Roeder et al. (2010) reported that the eccentrically loaded tests had greater scatter than the other test types, the eccentrically loaded specimens were included in the evaluation. The strength ratio between the test results and predictions were evaluated as described in Fig. 4-5.

The results showed similar tendency as in the case of stub-column specimens. The AISC predictions significantly underestimated the beam-column specimens with slender sections [Fig. 4-6(a)]. On the other hand, the AIJ method showed satisfactory accuracy irrespective of material combination and sectional slenderness (Table 4-4). Nevertheless, in the range of axial-load ratio 0.2 ~ 0.6 (with respect to nominal compressive strength of each method), the load-carrying capacity of specimens with high-strength steel tended to be overestimated [Fig. 4-7(c)].
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Fig. 4-4. Cross-sectional strength of slender columns

(a) Eccentric axial compression testing

(b) Flexural testing with constant axial compression

Fig. 4-5. Calculation of strength ratio

Test result/Prediction = (a+b)/a
Fig. 4-6. Strength evaluation of beam-column specimens
Fig. 4-7. Effect of axial-load ratio on strength evaluation of beam-column specimens
Table 4-4. Strength evaluation of existing test specimens

<table>
<thead>
<tr>
<th>Concrete strength (MPa)</th>
<th>Steel strength (MPa)</th>
<th>No. of stub-column specimens</th>
<th>Design models</th>
<th>Test strength / Prediction (Mean / Standard deviation)</th>
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<tbody>
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<td>$F_y \leq 525$</td>
<td>$f_c' \leq 70$</td>
<td>Eurocode 4&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.96 / 0.098</td>
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<tr>
<td>All</td>
<td>$F_y &gt; 525$</td>
<td>$f_c' \leq 70$</td>
<td>AIJ&lt;sup&gt;b&lt;/sup&gt;</td>
<td>1.05 / 0.096</td>
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<td>$F_y \leq 525$</td>
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<td>ANSI/AISC 360-16&lt;sup&gt;c&lt;/sup&gt;</td>
<td>1.08 / 0.131</td>
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<td>All</td>
<td>$F_y &gt; 525$</td>
<td>$f_c' &gt; 70$</td>
<td>Proposed Eq. (4-3)</td>
<td>1.03 / 0.083</td>
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</tbody>
</table>

<table>
<thead>
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<th>No. of beam-column specimens</th>
<th>Design models</th>
<th>Test strength / Prediction (Mean / Standard deviation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>104</td>
<td>Eurocode 4&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.95 / 0.114</td>
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<tr>
<td>60</td>
<td>AIJ&lt;sup&gt;b&lt;/sup&gt;</td>
<td>1.05 / 0.088</td>
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<tr>
<td>18</td>
<td>ANSI/AISC 360-16&lt;sup&gt;c&lt;/sup&gt;</td>
<td>1.12 / 0.192</td>
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<tr>
<td>16</td>
<td>Proposed (continuous)</td>
<td>1.04 / 0.084</td>
</tr>
<tr>
<td></td>
<td>Proposed (discontinuous)</td>
<td>1.06 / 0.090</td>
</tr>
</tbody>
</table>

<sup>a</sup> Recommended for $F_y \leq 460$ MPa, $f_c' \leq 50$ MPa, and $B/t \leq 1.78\sqrt{(E_s/F_y)}$

<sup>b</sup> Recommended for $F_y \leq 440$ MPa, $f_c' \leq 90$ MPa, and $B/t \leq 2.44\sqrt{(E_s/F_y)}$

<sup>c</sup> Recommended for $F_y \leq 525$ MPa, $f_c' \leq 69$ MPa, and $b/t \leq 5.00\sqrt{(E_s/F_y)}$
Chapter 4. Development of Analytical and Design Models for CFT Columns

4.3 Effects of Design Parameters on Rectangular CFTs

4.3.1 Effect of High-Strength Materials

Generally, the structural behavior of rectangular CFTs is affected by the tube local buckling (i.e., performance degradation of the buckled wall itself is critical). As seen in Fig. 4-3(a), Fig. 4-3(b), and Fig. 4-3(c) (or Fig. 4-8), such effect becomes remarkable when the high-strength steel is used, exhibiting smaller strength ratio in comparison with specimens with the mild steel. This is because the peak steel stress expected in the design is not actually attained due to the early concrete crushing, as discussed in 3.4.1 (Fig. 3-19). Since the relative contribution of high-strength steel in the composite section is substantial, the load-carrying capacity of high-strength steel needs to be properly assessed.

It seems that the use of high-strength concrete also has the negative effect. The cause is supposed to be premature failure related to different sizes between cylinders (diameter of 100 mm) and column specimens (width greater than 180 mm).

![Fig. 4-8. Strength evaluation for stub-column specimens based on AIJ predictions](image)
4.3.2 Effect of Large Width-to-Thickness Ratio

On the other hand, the AISC method is significantly conservative for both short columns and beam-columns with slender section [Fig. 4-3(a) and Fig. 4-6(a)]. In this section, relevant test results were investigated focusing of the effect of large width-to-thickness ratio \( b/t > 60 \) on concrete confinement. For this, individual test series containing the specimens with \( b/t > 60 \) were summarized in Table 4-5 and Table 4-6. The predicted strengths \( P_p \) and \( M_p \) were calculated assuming plastic stress distribution (for pure compression, \( P_p = F_y A_s + 0.85 f'_c A_c \)). It is noted that the experimental conditions may be slightly different in every study (i.e., testing machine, curing condition, tube fabrication, boundary condition, concrete mixture, and etc.).

In the case of short columns, when normal-strength materials were used [Fig. 4-9(a)], the strength ratio \( P_{test}/P_p \) does not decrease as \( b/t \) increases. Only test results of Huang et al. (2002), where \( b/t = 38.0 \) and 68 were tested, and Yamada et al. (1984), where rectangular cross-sections were used, showed clear strength degradation. In the case of beam-column specimens [Fig. 4-9(b)], only beam specimens in Uy (2000) showed the strength deterioration. However, in those test series, larger cross-sections were used for greater sectional slenderness, potentially affecting the size effect. Generally, the strength ratios were consistent and conservative (or greater than 1.0), demonstrating excessive conservatism of current design methods recommending underestimation of concrete resistance (AIJ 2008; AISC 2016). The results are attributed to beneficial concrete confinement provided by buckled tube wall, which is explained in section 4.5.
In the case of high-strength concrete (Nakahara and Sakino 2003; Sakino et al. 2004), however, the strength ratio degrades as the wall slenderness increases (Table 4-5 and Table 4-6). The result indicates the poor steel-concrete interaction due to the low dilatation of the high-strength concrete as well as the size effect, promoting the brittle failure. In the case of high-strength steel, the test specimens with \( b/t > 60 \) were extremely limited. As seen in Table 4-5, although the specimens CR6-D in Sakino et al. (2004) are categorized as noncompact section (\( \lambda_{\text{coeff}} = 2.68 \)), the width-to-thickness ratio was only \( b/t = 48.2 \) due to high yield strength (\( F_y = 618 \) MPa).
Fig. 4-9. Strength evaluation for test series with normal-strength materials and $b/t > 60$

Table 4-5. Individual test series containing stub-columns of $b/t > 60$

<table>
<thead>
<tr>
<th>Reference</th>
<th>Specimen ID</th>
<th>B (mm)</th>
<th>H (mm)</th>
<th>$b/t$</th>
<th>$F_c$ (MPa)</th>
<th>$f'_{c}$ (MPa)</th>
<th>$P_{test}/P_{p}$</th>
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<td>150</td>
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<td>58.0</td>
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<td>1.70</td>
<td>41.1</td>
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</table>
### Chapter 4. Development of Analytical and Design Models for CFT Columns

| CR4-C-8  | 215 | 215 | 47.1 | 262 | 1.70 | 80.3 | 0.99 |
| CR4-D-2  | 323 | 323 | 71.7 | 262 | 2.60 | 25.4 | 0.94 |
| CR4-D-4-1| 323 | 323 | 71.7 | 262 | 2.60 | 41.1 | 1.01 |
| CR4-D-4-2| 323 | 323 | 71.7 | 262 | 2.60 | 41.1 | 0.98 |
| CR4-D-8  | 324 | 324 | 72.0 | 262 | 2.60 | 80.3 | 0.91 |
| CR6-C-2  | 211 | 211 | 31.2 | 618 | 1.73 | 25.4 | 0.96 |
| CR6-C-4-1| 211 | 211 | 31.2 | 618 | 1.73 | 40.5 | 0.97 |
| CR6-C-4-2| 211 | 211 | 31.2 | 618 | 1.73 | 40.5 | 0.98 |
| CR6-C-8  | 211 | 211 | 31.2 | 618 | 1.73 | 77.0 | 0.99 |
| CR6-D-2  | 319 | 319 | 48.2 | 618 | 2.68 | 25.4 | 0.91 |
| CR6-D-4-1| 319 | 319 | 48.2 | 618 | 2.68 | 41.1 | 0.95 |
| CR6-D-4-2| 318 | 318 | 48.0 | 618 | 2.67 | 41.1 | 0.92 |
| CR6-D-8  | 319 | 319 | 48.2 | 618 | 2.68 | 85.1 | 0.89 |
| Han et al. (2005) |       |       |       |      |      |      |      |
| SA4-1    | 200 | 200 | 105.0 | 282 | 3.94 | 82.7 | 0.93 |
| SA4-2    | 200 | 200 | 105.0 | 282 | 3.94 | 82.7 | 0.93 |
| SA5-1    | 250 | 250 | 131.7 | 282 | 4.94 | 54.5 | 0.99 |
| SA5-2    | 250 | 250 | 131.7 | 282 | 4.94 | 54.5 | 1.02 |
| SB4-1    | 200 | 200 | 98.0  | 404  | 4.40 | 46.3 | 0.93 |
| SB4-2    | 200 | 200 | 98.0  | 404  | 4.40 | 46.3 | 0.95 |
| Tao et al. (2008) |       |       |       |      |      |      |      |
| UNC25a   | 250 | 250 | 98.0  | 342  | 4.05 | 60.9 | 1.03 |
| UNC25b   | 250 | 250 | 98.0  | 342  | 4.05 | 60.9 | 1.02 |
| UNC19a   | 190 | 190 | 74.0  | 342  | 3.06 | 60.9 | 1.03 |
| UNC19b   | 190 | 190 | 74.0  | 342  | 3.06 | 60.9 | 1.01 |
| UNC25c   | 250 | 250 | 98.0  | 270  | 3.60 | 53.0 | 1.04 |
| UNC19c   | 190 | 190 | 74.0  | 270  | 2.72 | 53.0 | 1.04 |
| Ding et al. (2016) |       |       |       |      |      |      |      |
| A1-a     | 240 | 240 | 58.0  | 257  | 2.08 | 31.8 | 1.07 |
| A1-b     | 240 | 240 | 58.0  | 257  | 2.08 | 31.8 | 1.13 |
| A1-c     | 240 | 240 | 58.0  | 257  | 2.08 | 31.8 | 0.95 |
| B1-a     | 500 | 500 | 81.3  | 300  | 3.15 | 57.1 | 1.10 |
| B1-b     | 500 | 500 | 81.3  | 300  | 3.15 | 57.1 | 1.10 |
| B5       | 500 | 500 | 60.5  | 280  | 2.26 | 57.1 | 1.14 |
Table 4-6. Individual test series containing beam-columns of $b/t > 60$

<table>
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<tr>
<th>Reference</th>
<th>Specimen ID</th>
<th>$B$ (mm)</th>
<th>$H$ (mm)</th>
<th>$b/t$</th>
<th>$F_y$ (MPa)</th>
<th>$f_{c'}$ (MPa)</th>
<th>$\lambda_{c_{eff}}$</th>
<th>$M_{test}/M_p$</th>
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Chapter 4. Development of Analytical and Design Models for CFT Columns

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</tr>
<tr>
<td>BRA4-6-5-02</td>
<td>200</td>
<td>200</td>
<td>31.7</td>
<td>320</td>
<td>1.27</td>
<td>47.6</td>
<td>0.20</td>
<td>1.02</td>
</tr>
<tr>
<td>BRA4-6-5-04</td>
<td>200</td>
<td>200</td>
<td>31.7</td>
<td>320</td>
<td>1.27</td>
<td>47.6</td>
<td>0.39</td>
<td>1.05</td>
</tr>
<tr>
<td>BRA4-4-5-02</td>
<td>200</td>
<td>200</td>
<td>45.1</td>
<td>211</td>
<td>1.46</td>
<td>47.6</td>
<td>0.19</td>
<td>1.08</td>
</tr>
<tr>
<td>BRA4-4-5-04</td>
<td>200</td>
<td>200</td>
<td>45.1</td>
<td>211</td>
<td>1.46</td>
<td>47.6</td>
<td>0.39</td>
<td>1.10</td>
</tr>
<tr>
<td>BRA4-2-5-02</td>
<td>200</td>
<td>200</td>
<td>96.0</td>
<td>253</td>
<td>3.42</td>
<td>47.6</td>
<td>0.19</td>
<td>1.06</td>
</tr>
<tr>
<td>BRA4-2-5-04</td>
<td>200</td>
<td>200</td>
<td>96.0</td>
<td>253</td>
<td>3.42</td>
<td>47.6</td>
<td>0.39</td>
<td>1.01</td>
</tr>
</tbody>
</table>

Fujimoto et al. (2004)

| ER4-C-4-21    | 215 | 215 | 47.1 | 262 | 1.70 | 41.1 | 0.24 | 1.02 |
| ER4-C-4-38    | 215 | 215 | 47.1 | 262 | 1.70 | 41.1 | 0.42 | 0.94 |
| ER4-C-4-51    | 215 | 215 | 47.1 | 262 | 1.70 | 41.1 | 0.56 | 0.92 |
| ER4-D-4-27    | 323 | 323 | 71.7 | 262 | 2.60 | 41.1 | 0.30 | 0.96 |
| ER4-D-4-60    | 323 | 323 | 71.7 | 262 | 2.60 | 41.1 | 0.67 | 0.95 |
| ER6-C-4-44    | 210 | 210 | 31.0 | 618 | 1.72 | 41.1 | 0.46 | 0.95 |
| ER6-C-4-57    | 209 | 209 | 30.9 | 618 | 1.72 | 41.1 | 0.59 | 0.95 |
| ER6-D-4-47    | 319 | 319 | 48.2 | 618 | 2.68 | 41.1 | 0.49 | 0.82 |

Wu et al. (2012)

| S-T3N2M0      | 300 | 300 | 106.7 | 350 | 4.46 | 43.8 | 0.19 | 1.28 |
| S-T3N4M0      | 300 | 300 | 106.7 | 350 | 4.46 | 43.8 | 0.38 | 1.44 |
| S-T6N4M0      | 300 | 300 | 52.5 | 270 | 1.93 | 43.8 | 0.37 | 1.48 |
4.3.3 Concrete Confinement with Local Buckling

The capacity of the slender section was fairly underestimated by existing design methods. This aspect has not been emphasized in previous studies since the specimens with non-compact and slender sections \((b/t > \lambda_p)\) were often omitted in the evaluations. This part examines the reason for such outperformance of the specimens.

Generally, it is known that confining pressure on the infill concrete decreases as the wall thickness decreases. Such concept is reflected to the ductility in stress-strain relationships for fiber model analysis. Also in design, further performance deterioration of concrete is usually recommended for the slender section because when the local buckling takes place, it is assumed no more interaction is expected (Fig. 4-10). According to the AIJ model, the performance deterioration of normal-strength concrete should be considered due to the large width-to-thickness ratio as well as the AISC specification and Hardika and Gardner (2004).

However, aforementioned assumptions are valid when the interaction is governed by passive confinement mechanism. On the other hand, when tube local buckling takes place in slender sections, the concrete confinement is affected by a different mechanism, which may be called active confinement. Obviously, since buckling mode of the steel tube changes with the presence of concrete (Fig. 4-11), that constraint should react as confinement to the concrete.
4.3.3.1 Passive Confinement

3D finite element analysis was performed to investigate the effect of tube local buckling on the concrete confinement. First, to separately evaluate the confining mechanisms with and without local buckling, two types of numerical models were tested: 1) composite column with axial compression only on the concrete infill and 2) composite column with axial compression only on the steel tube. In the analysis, the compressive strength of the concrete was 70 MPa, and yield strength of the steel was 300 MPa.

When only the core concrete is loaded, where the steel tube acts as an external jacket, so-called passive confinement develops from the corners [Fig. 4-12(a)]. This mechanism agrees with the results of existing numerical studies (Varma 2000). As
seen in Fig. 4-12(b), the concrete compressive stress is greater in a more stocky steel tube.

In Fig. 4-12(c), the vertical axis, confining pressure, means average normal stress acting on concrete façade along the column height. Development of the confining pressure can be divided into three stages: 1) at early loading (before reaching the axial strain of 0.0015 ~ 0.002), the confining pressure linearly increases due to a constant Poisson’s ratio of the concrete and elasticity of the steel jacket, 2) the rate of growth suddenly explodes as the Poisson’s ratio becomes greater with inelasticity of the concrete, and 3) after the axial strain of 0.004, the growth rate is alleviated because of yielding of the steel tube. It is noted that the confining pressure showed good accordance with compressive stress of the concrete infill; the greater confining pressure is usually coupled with the greater compressive stress at any axial strain.

Fig. 4-12. Passive confining mechanism with axial compression on concrete infill
4.3.3.2 Active Confinement

Fig. 4-13 shows the results of finite element analysis in which only the tube section is loaded. Unlike the previous investigation on passive confinement, the confining pressure was detected from the mid-surface, which was caused by buckled tube plates at the region. This mechanism may be called active confinement. It should be noted that the confining pressure is much smaller than that developed by passive confinement because the lateral expansion of the concrete infill is not addressed in Fig. 4-13.

As seen in Fig. 4-13(c), no interaction was measured until the onset of inelastic local buckling [Fig. 4-13(b)]. Therefore, as the tube section is more slender, the active confinement takes place faster. Also, at the beginning of the plate collapse
mechanism, the confining pressure rises steeply as the buckling distortion becomes remarkable. After some extent of the distortion, where the steel tube itself approaches the residual strength, however, the growth of confinement showed degradation as seen in the case of $b/t = 60$.

### 4.3.3.3 Combined Mechanism

Fig. 4-14(c) shows the results of the numerical analysis in which the entire composite section is subjected to axial compression. The comparatively large width-to-thickness ratio was included in the analysis ($b/t$ up to 70), to induce early local buckling and investigate combined effects of the two confining mechanisms. According to ANSI/AISC 360-10 (AISC 2016), such combination is feasible as mild steel slender section.

Although the difference is not noticeable, the concrete peak stress is greater in more slender tube section ($b/t = 30 \sim 40$ vs. $60 \sim 70$). The compressive capacity and the post-peak behavior, under multi-axial stress states, is actually more susceptible to input properties for the concrete damaged plasticity model in ABAQUS than the steel-concrete interaction. At least, however, the numerical observation indicates that greater confining pressure may develop at the peak compressive stress of concrete, in more slender tube section.

These results imply that, in the real response of the slender section, the active confining mechanism may be influential so that poorness in the passive confinement can be compensated. This beneficial effect of local buckling may be the reason for the outperformance of existing test specimens with the slender section. On the other hand, CFTs with the large width-to-thickness ratio ($b/t > 60$), that is prone to early
local buckling, are naturally limited to mild steel [Fig. 4-14(a)]. It is noted that the onset of elastic local buckling of tube plates is only affected by the width-to-thickness ratio and plate out-of-flatness.

In contrast, when high-strength steel compact section is used, the actual width-to-thickness ratio is very small. Thus, in such case, early local buckling can be prevented. Fig. 4-14(b) shows the numerical results for RCFT columns with the high-strength steel. Due to the high yield strength, the passive confining mechanism of compact tube section generally governed. In particular, when \( b/t = 20 \), strength and ductility of the concrete infill is clearly enhanced. The results demonstrate the effectiveness of concrete confinement in high-strength steel compact section (Aslani et al. 2015).
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Fig. 4-14. Concrete confinement in high-strength steel compact section
4.4 Proposed Local Buckling Model for Rectangular Tubes

4.4.1 Effective Peak Stress

From the viewpoint of strength estimation, defining peak stresses of the steel and concrete is the most crucial part. Thus, effective peak stress of the high-strength steel tube was proposed and applied to strain-based fiber analysis. In the proposed model, the steel peak stress $\beta_1 F_y$ was basically defined using an existing post-buckling model [Eq. (4-1), Fig. 4-15, Guo et al. 2007]. However, the buckling strength $\beta_1 F_y$ does not take into account the influence of concrete dilation when the effective yield strain of steel $\beta_1 \varepsilon_y$ is greater than the crushing strain of concrete $\varepsilon_{co} (= 0.94(f'_c)^{0.25} \times 10^{-3})$. As confirmed in numerical results [stiffened plates in Fig. 3-19(d) and Fig. 3-19(e)], actual peak stress of the high-strength steel is affected by the rapid expansion of the crushed concrete (Nakahara et al. 1998). Thus, the actual peak stress of steel $\beta_2 F_y$ should be restrained in between limit strains of the two materials [Fig. 4-16(b)], which was empirically proposed based on the test results [Eq. (4-2)].

$$\beta_1 = 1.32 \lambda_b^{0.51} \leq 1.0 \quad (4-1)$$

$$\beta_2 \varepsilon_y = \varepsilon_{co} + \left( \frac{30}{B/t} \right)^{1.0} (\beta_1 \varepsilon_y - \varepsilon_{co}) \leq \beta_1 \varepsilon_y \quad (4-2)$$

where $\lambda_B = B/t\sqrt{F_y/E_s}$. According to Eq. (4-2), the yield capacity $F_y$ of the high-strength steel is available if the plate aspect ratio is sufficiently small ($B/t \leq 30$). This reflects the fact that the strength enhancement of the concrete infill is expected when the stocky tube section is used together with the high-strength steel [Fig. 4-14(b)].
For simplicity, the confinement effect is not considered directly in modelling of the concrete infill [Fig. 4-16(a)].

Fig. 4-15 compares Eq. (4-1) with the buckling capacities recommended in the AISC and AIJ specifications (AISC 2016; AIJ 2008). The buckling strength of ANSI/AISC 360-16 is based solely on elastic plate theory, and it does not take into account the effects of geometric imperfections and residual stress as well as post-buckling behavior. The AIJ model also neglects the beneficial effect of the post-buckling reserve. Since the axial-load contribution of high-strength steel even thin plate is substantial, the post-buckling reserve needs to be considered.

The descending branch of stress-strain relationship was simply defined with a bi-linear model [Fig. 4-16(b), \( F_{y2} = (1.19 - 0.207\lambda_B)F_y \), Fujimoto et al. 2004]. On the other hand, effective stress-strain relationship of the concrete infill followed Sakino et al. (2004), except for strength reduction addressing the size effect [Fig. 4-16(a)].

![Normalized buckling strength vs. slenderness coefficient](image)
Fig. 4-16. Uniaxial stress-strain relationships for section analysis

Fig. 4-17 compares the analytical results with test results for stub-column specimens (Fujimoto et al. 1997). Each unit (a) ~ (i) contains three steel tubes with different grades (400, 600, and 800), and the units are categorized with respect to sectional slenderness (very compact, compact, and noncompact) and concrete strengths (class 20, 40, and 80). Although satisfactory accordance was achieved from the viewpoints of strength and ductility, irrespective of the design parameters, some noteworthy observations are discussed as follows.

First, apparent overstrength was exhibited in very compact section [Fig. 4-17(a)]. It is supposed that the enhanced performance was attributed to the concrete confinement effect since the steel axial stress should decrease due to the interaction with the concrete (Yamamoto et al. 2013; Katwal et al. 2017) rather than the strain-hardening effect of very stocky tube wall in the case of bare steel (Fujimoto et al. 1997). Similarly, Fig. 4-17(h) indicates that the confinement effect on strength and deformability of low-strength concrete is more pronounced when the high-strength steel is used.

Second, from comparisons Fig. 4-17(e) vs Fig. 4-17(h) and Fig. 4-17(f) vs...
Fig. 4-17(i), the existence of size effect (width 211 ~ 215 mm vs 319 ~ 323 mm) was not confirmed. On the other hand, the load-carrying capacity of composite columns with the high-strength concrete was generally underestimated by the analytical predictions [Fig. 4-17(b), Fig. 4-17(d) and Fig. 4-17(g)], implying the likelihood of premature failure. Therefore, reasonable reduction factors are required for the high-strength concrete (Liew et al. 2016), which will be introduced in the next section for development of a simplified design method.
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(a) Very compact section with $f_c = 39.1$ MPa

(b) Very compact section with $f_c = 91.1$ MPa

(c) Compact section with $f_c = 39.1$ MPa
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(d) Compact section with $f_c = 91.1$ MPa

(e) Compact section with $f_c = 25.4$ MPa

(f) Compact section with $f_c = 41$ MPa
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Fig. 4-17. Comparison between analytical and experimental (Fujimoto et al. 1997) results

(g) Compact section with $f_c = 80$ MPa

(h) Noncompact section with $f_c = 25.4$ MPa

(i) Noncompact section with $f_c = 41.1$ MPa
4.4.2 Simplified Method for $P$-$M$ Interactions

For practical uses, a simplified method to construct the $P$-$M$ interaction curve was developed, assuming equivalent rectangular stress blocks (Fig. 4-18). In the simplified method, the peak compressive stresses $\alpha f'_c$ and $\beta_2 F_y$ are applied. The strength coefficient $\alpha$ for high-strength concrete ($f'_c > 50$ MPa) is defined as $\alpha = 1 - (f'_c - 50)/200 \geq 0.8$ following Eurocode 2 (CEN 2004). In the case of $f'_c \leq 50$ MPa, $\alpha = 1.0$. The steel peak stress $\beta_2 F_y$ is calculated by Eqs. (4-1) and (4-2). Based on parametric studies using fiber model analysis ($b/t = 15 ~ 80$, $f'_c = 20 ~ 150$ MPa, $F_y = 200 ~ 900$ MPa), validity of the simplified method was examined.

![Fig. 4-18. Comparison between fiber model analysis and simplified method](image)

(1) Pure compressive strength $P_A$

The performance points for pure compression and pure flexure (points A and B in Fig. 4-18) are the fundamental in the construction of $P$-$M$ relationships. The pure compressive capacity the composite section is expressed as follows:
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\[ P_A = A_t \left( \alpha f_y' \right) + A_f \left( \beta F_y \right) \]  \hspace{1cm} (4-3)

The strain incompatibility of peak stresses between steel and concrete may be significant when high-strength steel (i.e., early failure of concrete) or conventional steel slender section (i.e., early failure of steel) is used. According to the results of parametric studies (also seen in Fig. 4-17), in the case of high-strength steel, the peak load of the composite section was generally governed by the failure of the steel component. In the case of conventional steel slender section, the peak load was governed by the failure of infill concrete. In both cases, the effect of strain incompatibility on the pure compression of the composite section was marginal [unconservatism of Eq. (4-3) compared to the analytical result was less than 2%] mainly due to the ductile behavior of filled concrete.

Consequently, when compared with existing test results, Eq. (4-3) still gives conservative predictions [Fig. 4-21(a) and Fig. 4-21(b)]. Further, from Fig. 4-21(c) and Fig. 4-21(d), it was confirmed that the strength coefficient \( \alpha \) provides reasonable predictions for the high-strength concrete. Among the strength evaluation methods, the proposed equation showed the best accuracy (Table 4-4).

(2) Pure flexural strength \( M_B \)

For pure flexure (Fig. 4-19), depth of the neutral axial \( a_B \) (so that \( P_B = 0 \)) and the resultant moment capacity \( M_B \) are expressed as follows:

\[ a_B = \frac{2F_yHt + \alpha f_y'bt + (1 - \beta)F_ybt}{2F_yt + \alpha f_y'b + 2\beta F_yt} \]  \hspace{1cm} (4-4)
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\[
M_b = \left( bt \right) \left( \frac{1 + \beta}{2} F_y \right) (H - t) + \left( 2ta_h \right) \left( \frac{1 + \beta}{2} F_y \right) (H - a_h) \\
+ b \left( a_b - t \right) \left( \alpha f_c' \right) \left( \frac{H - t - a_b}{2} \right)
\]  

(4-5)

According to the results of parametric studies (Fig. 4-20), the simplified method generally yields \( M_b \) comparable or smaller than that calculated by the fiber model analysis, which is conservative for practical uses. This is because, in the simplified method, depth of the neutral axis is comparatively small, so that the majority of steel plastic stress is ineffective for flexural action (Fig. 4-19). Also the analytical model assumes increased tensile yield stress of \( 1.1F_y \), accounting for the strain-hardening effect (Fujimoto et al. 2004). On the other hand, when high-strength steel thin plates are used [Fig. 4-20(b)], the proposed predictions tend to be greater than those of the analytical model. Since the overestimation is greater than 2% when slender section (\( \lambda_{coeff} \geq 3.0 \)), high-strength steel (\( F_y \geq 800 \text{ MPa} \)) and low-strength concrete (\( f'_c \leq 30 \text{ MPa} \)) are used together, the simplified method is not recommended for such combination.

<table>
<thead>
<tr>
<th>Axial force ( P = 0 )</th>
<th>( \beta F_y )</th>
<th>( \alpha f_c' )</th>
<th>( \beta F_y )</th>
<th>( f_c' )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strain</td>
<td>Steel stress</td>
<td>Concrete stress</td>
<td>Steel stress</td>
<td>Concrete stress</td>
</tr>
<tr>
<td></td>
<td>( \beta F_y )</td>
<td>( \alpha f_c' )</td>
<td>( \beta F_y )</td>
<td>( f_c' )</td>
</tr>
<tr>
<td>Tensile strain</td>
<td>Ineffective</td>
<td>for flexure</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 4-19. Stress distributions under pure flexure
(3) Combined axial-flexural strength

In the range of medium axial-load ratio, the predictions of the simplified method generally overestimates the analytical results (Fig. 4-18), indicating full stress blocks of \( \alpha f_c \) and \( \beta F_y \) is not applicable for determining the entire curve. Therefore, an additional stress reduction factor \( \eta \) addressing the neutral axis depth \( a \) (> \( H/2 \)) was proposed as follows:

\[
\eta = 1 - 0.1(H - a)(a - H/2)/(H/4)^2
\]  

(4-6)

Fig. 4-20. Comparison between analytical and simplified flexural strengths

Fig. 4-21 and Fig. 4-22 compare the proposed predictions with existing test
results of stub-column and beam-column specimens. The simplified method provided reasonable predictions regardless of material strengths and sectional slenderness (Table 4-4). In the case of beam-columns, the conservatism of the predictions was also consistent regardless of axial load ratio (Fig. 4-23).

Fig. 4-21. Verification of simplified method for stub-columns
Fig. 4-22. Verification of simplified method for beam-columns

Fig. 4-23. Effect of axial-load ratio on strength assessment of beam-columns
(4) Discontinuous approach (tri-linear relationships)

The proposed method can be further simplified with four sample points (A ~ D in Fig. 4-24). The points C (neutral axis depth $a_C = H-t$) and D ($a_D = H/2$) are expressed as follows:

$$M_D = \frac{1 + \beta_2}{2} F_y Z_s + \alpha f'_c Z_c'$$  \hspace{1cm} (4-7)

$$P_D = \frac{A_y}{2} (\beta_2 - 1) F_y + \frac{A_y}{2} \alpha f'_c$$  \hspace{1cm} (4-8)

$$M_C = B t \frac{1 + \eta \beta_2}{2} F_y (H - t)$$  \hspace{1cm} (4-9)

$$P_C = A_y \eta \beta_2 F_y - B t (1 + \eta \beta_2) F_y + A_y \eta \alpha f'_c$$  \hspace{1cm} (4-10)

$$M_B = M_D - \frac{1 + \beta_2}{2} F_y Z_{sn} - \alpha f'_c Z_{cn}'$$  \hspace{1cm} (4-11)

where $Z$ is plastic section modulus expressed as $Z_s = (BH^2 - bh^2)/4$, $Z_c = bh^2/4$, $Z_{sn} = 2t(H - 2a_B)^2/4$, and $Z_{cn} = b(H - 2a_B)^2/4$. The predictions of discontinuous approach are still satisfactory (Fig. 4-25 and Table 4-4).
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Fig. 4-24. Tri-linear simplification of proposed method

Fig. 4-25. Verification of tri-linear method for beam-column specimens
4.4.3 Design Conformance with Eurocode 4

In the proposed model for the steel peak stress [Eq. (4-2)], the yield capacity $F_y$ of the high-strength steel is available if the plate aspect ratio is sufficiently small ($B/t \leq 30$). In fact, the high-strength tube plate ($F_y > 650$ MPa) with $\beta_1 = 1.0$ by Eq. (4-1) or $\lambda_B = B/t \sqrt{(F_y/E_s)} = 1.71$, automatically satisfies $B/t \leq 30$ ($B/t = 33.4$, 31.2, and 30.0 for $F_y = 525$, 600, and 650 MPa, respectively, see Table 1-1). This indicates that when the high-strength steel is defined as $\beta_1 = 1.0$, the effect of early concrete crushing or Eq. (4-2) does not further degrade the steel contribution [i.e., Eq. (4-1) governs].

In such case, the proposed Eq. (4-3) for pure compression capacity is identical to the prediction of Eurocode 4 in which the limitation on the tube aspect ratio is similar ($\lambda_B \leq 1.78$, see Table 2-1). As seen in Fig. 4-3(b) or Fig. 4-21(b), when $F_y > 525$ MPa, $f_c' \leq 70$ MPa, and $\lambda_B \leq 1.71$, the Eurocode method is generally conservative against the test results of stub-columns.

In the case of beam-column specimens [Fig. 4-6(b)], however, the predictions based on the plastic stress distribution method (Table 2-1) overestimate the test results. This may be the reason why Eurocode 4 cannot be extended to allow the use of high-strength steel ($F_y > 440$ MPa). To accommodate the high-strength steel, it is recommended to consider the stress degradation as illustrated in Fig. 4-18.
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4.5 Proposed Concrete Confinement Model for Circular Tubes

4.5.1 Confined Strength Degradation with Axial Load Ratio

The strength and deformation capacity of the filled concrete are significantly enhanced in the circular tube due to the lateral confinement effect provided by hoop tension (in contrast, the steel axial compressive stress degrades). Thus, it is important to reflect the interaction properly in structural modeling. Generally, effective stress-strain relationships of the materials are developed on the basis of the experimental or numerical response of short columns subjected to pure compression (Sakino et al. 2004; Lai et al. 2016; Katwal et al. 2017). The material models are often applied in frame analysis of beam-column members without modification. This section deals with analytical assessment of the load-carrying capacity of circular CFT beam-columns, in which a strength degradation model of confined concrete was proposed accounting for axial-load ratio (Fig. 4-26).

Under pure compression, it is obvious that the significant interaction between the concrete infill and steel tube (i.e., the lateral hoop tension in the steel tube) develops due to the concrete dilation. However, when the composite section is subjected to the flexural moment, causing a strain gradient, the expansion is limited to the compression zone. In the tension side, the interaction can still develop due to the shrinkage of steel tube and the constraint of concrete infill. Nevertheless, predicting the magnitude of hoop tension and the resultant confinement effect of concrete infill is more complicated than in the case of pure compression.

As seen in Fig. 5-25, although the analysis result was in good agreement with
the test result of EC2 (eccentricity = 180 mm), it clearly underestimated the test result of EC1 (eccentricity = 60 mm). This implies that the greater confined strength of concrete infill was exhibited with smaller eccentricity. In the Eurocode 4 (CEN 2004), the beneficial interaction is recognized only when the eccentricity ratio is less than 0.1. According to the test results of O'Shea and Bridge (2000), where comparatively large width-to-thickness ratio was used ranging $D/t = 60 \sim 220$, the effect was confirmed when the eccentricity ratio was less than 0.05 and the concrete compressive strength was less than 80 MPa. According to the experimental investigation of Elremaily and Azizinamini (2002), the hoop tension in the steel tube tended to decrease with the reduction of axial-load ratio. These results indicates that the lateral confinement effect of concrete needs to be degraded when the eccentricity is large (or the axial-load ratio is small).

![Diagram](image)

**Fig. 4-26. Degradation of confined concrete strength and resultant $P$-$M$ interaction**

Following the previous observations, it was assumed that the confined concrete strength $f_{ccn}'$ decreases as the axial-load ratio $n$ decreases; the modified
strength $f_{ccn}'$ is identical to the original one $f_{cc}'$ when $n = 1.0$ (pure compression) while the strength increase is neglected when $n = 0$ (pure flexure).

$$f_{ccn}' = f_{cc}' + (f_{cc}' - f_{c}) n^{0.5} \quad (4-12)$$

The proposed Eq. (4-12) was applied to existing constitutive equations. Among various analytical models (see 2.3.2.1), the stress-strain relationships of Katwal et al. (2017) were utilized for the strength estimation of composite sections (Fig. 4-26). Except for the Eq. (4-12), key design parameters such as the strain at peak stress and the residual strength followed the original model. It is also noted that the concrete tensile stress is neglected in this study.

Fig. 4-27 shows effective stress-strain relationships of circular steel tube subjected to axial compression and tension. The compressive behavior followed the original model (Katwal et al. 2017), which was validated by numerical results of stub-columns. Due to the increasing hoop tension after the concrete crushing, the compressive stress temporarily decreases until the strain-hardening. On the other hand, the tensile behavior followed the one by Fujimoto et al. (2004), which is characterized by the yield point of $1.08F_y$ considering the bi-axial tensile stress and linear strain-hardening up to ultimate tensile strength $F_u$. 
Fig. 4-27. Effective stress-strain relations of steel subjected to compression and tension
4.5.2 Effect of High-Strength Steel on Ultimate State

Fig. 4-28 compares the test results and predictions using the original and modified strength models. Smaller $P-M$ interaction curves indicate the modified ones. The capacity curves (or collection of peak moment points) were constructed by performing moment-curvature analysis at the interval of $n = 0.1$. Because of optimized use of the steel and concrete, the load-carrying capacity of circular CFT columns generally keep increasing until a very large inelastic deformation. In calculating the predictions, the limit state was prescribed as either dimensionless curvature (= the product of curvature and the tube diameter) of 8% and ultimate tensile strain. According to Roeder et al. (2010), at the dimensionless curvature of 7.7%, the calculated flexural capacity approaches above 97% of the analytical peak.

Graphs in Fig. 4-28 covers three steel grades ($F_y = 283, 579, \text{ and } 834 \text{ MPa}$) and two kinds of sectional slenderness (roughly noncompact and slender). It should be noted that the effect of confined concrete strength is conspicuous when the high-strength steels ($F_y = 579$ and 834 MPa) are used. In the case of mild steel ($F_y = 283$ MPa), the reduction in flexural moment was less than 10%. This implies that the confinement effect (confined strength-to-unconfined strength ratio) is significantly large when the high-strength steel is used, and that the enhanced performance of concrete in the compression zone seriously influences the optimized utilization of the high-strength steel as tensile components.

Degrading the confined concrete strength for low axial-load ratio conservatively amended the strength assessment. The modified predictions turned out to be effective especially for specimens with the high-strength steels. This again
emphasizes that the confinement effect with the presence of flexural moment needs to be properly estimated when the high-strength steel is used.

It was clear from the analytical investigation that the flexural capacity of circular CFTs with the high-strength steel was sensitive to the degree of confined concrete strength. In the future, further experimental back-ups are necessary though. Currently, comparison between the analytical and experimental results was limited for the following reasons. 1) The filled concrete may be susceptible to the size effect because of the absence of reinforcing bars. In fact, small-scale specimens seem to exhibit superior performance when compared with large-section ones. Since definition of the size effect makes the problem more complex, to minimize such uncertainty, specimens with the diameter smaller than 200 mm were excluded in the evaluation. Beyond the size limit, the size effect still exists (Blanks and McNamara 1935), but the effect is not significant. 2) In general, when beam-column specimens are fixed with the foundation, the structural performance improves drastically owing to the additional confinement provided by the foundation (AIJ 2008; Lai et al. 2015). Also, even in the case of simply-supported beam-columns with either one-point or two-point shear loading, unexpected constraint may occur if the loading point is strongly constrained with a rigid stub (Elremaily and Azizinamini 2002) or the distance between the two loading points is close ($< 3D$). Because the strength evaluation should focus on pure axial-flexural resistance of the composite section (AIJ 2008), the specimens mentioned above were not selected. Consequently, available experimental data are very limited.
Chapter 4. Development of Analytical and Design Models for CFT Columns

(a) Noncompact section with $f_c = 25.4$ MPa

(b) Noncompact section with $f_c = 40.7$ MPa
Fig. 4-28. Confined concrete stress with axial-load ratio and resultant $P$-$M$ interaction curve
4.5.3 Deformation Limit States

In the analysis, the maximum moment tended to be determined by the dimensionless curvature limit of 8% as the contribution of the steel section and axial-load ratio become large and small, respectively. On the other hand, no cases were governed by the limit of ultimate tensile strain. However, the curvature limit state of 8% (= the product of curvature and the tube diameter) may be too large to be realistic from the viewpoint of practical design. In Roeder et al. (2010), the strength was evaluated at the curvature limit state of 3%.

Although test specimens with the premature failure such as weld-affected fracture (Fujimoto et al. 2004) were omitted in the strength evaluation, such early failure due to the lack of deformability was often observed especially when the high-strength steels were applied. Therefore, to avoid such failure mode, it is recommended to define the limit state more conservatively.
4.6 Discussion

This section focused on assessing the load-carrying capacities of CFT columns and beam-columns. In particular, careful attention was paid to the effects of not only high-strength materials but also slender tube section, which have often not been addressed in previous models. Major findings of this study is summarized as follows:

1) It was confirmed that the current design codes (ANSI/AISC 360-16, Eurocode 4, AIJ recommendations) safely predict the test strengths of RCFT columns and beam-columns with conventional material strengths and width-to-thickness ratios. In particular, the test specimens with normal-strength materials and large sectional slenderness significantly outperformed the nominal strengths by the AISC specifications, attaining the plastic capacity of composite section. This result is attributed to beneficial confinement of buckled tube plates.

2) For section analysis of RCFTs, the effect peak stress of steel was proposed addressing the effects of post-buckling behavior and concrete crushing, which is reasonable for the high-strength steel. A simple superposition of the peak stresses of steel and concrete also gave conservative estimations for existing test results regardless of material strengths and sectional slenderness. When constructing the $P$-$M$ interaction curve, a conventional rigid-plastic method can be simply modified with peak compressive stresses of steel and concrete. A strength reduction factor is suggested for medium axial-load ratio.

3) For strength estimation of circular CFT beam-columns, an analytical
model addressing the effect of axial-load eccentricity on the load-carrying capacity of the composite section was proposed. Degrading the confined concrete strength for low axial-load ratio conservatively amended the strength assessment. The modified predictions turned out to be effective especially for specimens with the high-strength steels. This indicates that the flexural capacity of circular CFTs with the high-strength steel was sensitive to the degree of confined concrete strength.
Chapter 5. Axial Testing and Evaluation of CEFT Columns

5.1 Introduction

Recently, the use of large columns has increased in the construction of wholesale stores and warehouse buildings that have large spans and story heights. In the construction of such large columns, the use of conventional reinforced concrete may not be economical because of the difficulties in the rebar fabrication and formwork. Furthermore, the use of precast concrete (PC) columns may also not be advantageous because of difficulties in the transportation and lifting of the large-weight columns.

Alternatively, hollow PC columns, with reduced weight, can be used for the construction of such large columns. After erecting the hollow PC column on the construction site, concrete can be filled into the hollow core. In manufacturing, however, due to the requirement of cross-ties, it is difficult to install and remove the inner form that is required to form the hollow section. In existing methods, various techniques have been attempted to avoid this interference: spinning molds using centrifugal force (Kono et al. 1995; Matsumoto et al. 1997; Hamada et al. 1998; Choi et al. 2000; Hosoya and Asano 2000; Hosoya and Hukuyama 2008; Seo et al. 2008; Lee et al. 2015); internal molds or air-tubes (Kajihara et al. 1999; Hagiwara et al. 2001; Zavliaris 2014); sequential concrete placements rotating a section (Iso et al. 1999; Kim et al. 2016); and a circular corrugated steel tube (encased-type) with diagonal cross ties. However, in many cases, such methods are uneconomical (in
terms of production time, weight efficiency, and non-structural inner mold) or inapplicable for various column dimensions.

Considering the difficulty of the use of an inner form, a permanent thin steel tube can be used for the hollow PC column (Fig. 5-1). In the concrete-encased-and-filled tubular column (CEFT column), cross-ties are not necessary, and the thin steel tube can be used as a structural element, resisting member forces. Furthermore, the concrete encasement can develop additional strength and stiffness, providing local-buckling restraint and fire resistance to the steel tube (Xu and Liu 2013). Because of the local buckling restraint, a relatively thin steel tube can be used.

Currently, both square and circular steel tubes were considered for the proposed CEFT column. Square sections are more commonly used in practice and advantageous in increasing the hollow section of large-scale columns [Fig. 5-1(a)]. Although the weight efficiency may be limited with circular tubes [Fig. 5-1(b)], circular CFT columns usually exhibit superior structural performance to rectangular CFT columns.

![Cross section of proposed composite column](image)

Fig. 5-1. Cross section of proposed composite column
In the present study, concentric and eccentric axial load tests were performed on CEFT columns. In the design of test specimens, the following aspects required by engineers and construction companies were considered. 1) The weight of the hollow section significantly affects the economy of the construction method (lifting weight and transportation). Thus, to reduce the weight of the hollow section, relatively large hollowness ratios were used, using thin concrete encasement. 2) For economy, thin-walled steel tubes (width-to-thickness ratio $b/t = 38.0$ and $54.7$ for square tube and diameter-to-thickness ratio $D/t = 58.1$ for circular tube which are both noncompact sections) were used. 3) To verify the load-carrying capacity of the columns under high axial loading, concentric axial load or eccentric axial load with low eccentricity was used. 4) Relatively large specimens (gross section $= 480 \times 480$ mm) were used. 5) The structural performance of CEFT columns depends strongly on the integrity between the concrete filled steel tube and the concrete encasement. Thus, various connection details (including the details of the concrete encasement) were tested to assess their effect on the integrity and structural performance of the columns.


5.2 Test Program

5.2.1 Test Parameters and Specimen Details

5.2.1.1 Specimens with Square Tube

The test specimens were half-scale models of prototype CEFT columns with a cross-section of $760 \times 760$ mm or $960 \times 960$ mm. Six eccentrically loaded specimens, ERE1–ERE6, and a concentrically loaded specimen, CRE, were prepared for testing. Fig. 5-2 and Table 5-1 show the details and parameters of the test specimens. The major test parameters were the reinforcement details of the concrete encasement to restrain early spalling of the concrete encasement. Furthermore, the effects of the axial load eccentricity and column length were also studied.

In the eccentrically loaded specimens, the dimensions of the cross-section were $480 \times 480$ mm. On the other hand, the dimensions of the cross-section of the concentrically loaded specimen, CRE, were reduced to $380 \times 380$ mm, considering the loading capacity of UTM. In all the specimens, the thickness of the concrete encasement was 70 mm, which was determined considering 1) the cover concrete thickness and re-bar arrangement and 2) the minimum thickness to prevent cracking during handling and shipping. The hollowness ratios of the cross-sections were about 50% for the eccentrically loaded columns and 40% for the concentrically loaded column. In the case of full-size prototype columns (gross section = $760 \times 760$ mm or $960 \times 960$ mm), the hollowness ratio can be increased to 73% and 67% if the same thickness of the concrete encasement (70 mm) is used.
The steel tube was a built-up section of 340 × 340 mm for the eccentrically loaded specimens and 240 × 240 mm for the concentrically loaded specimen. The thickness of the steel tube was \( t = 6 \) mm. The width-to-thickness ratio of the steel tube for \( \text{ERE1–ERE6} \) was \( b/t = 54.7 \), which slightly exceeded the limitation \( [\lambda_p = 2.26\sqrt{(E_s/F_y)}] \) of the compact section for CFT columns specified in American National Standards Institute ANSI/AISC 360 (AISC 2010). In \( \text{CRE} \), the ratio was \( b/t = 38.0 \), which belongs to the compact section.

Fig. 5-5 shows the eccentrically loaded columns. The axial load eccentricity of \( \text{ERE1–ERE6} \), except \( \text{ERE2} \), was \( e = 60 \) mm, corresponding to an eccentricity ratio \( e/h_c = 0.125 \). The eccentricity of \( \text{ERE2} \) was \( e = 180 \) mm \( (e/h_c = 0.375) \). The effective length (= length between the top and bottom hinges) of \( \text{ERE1–ERE6} \), except \( \text{ERE3} \), was \( L_e = 2,880 \) mm, including the rigid zones and knife-edges. According to Eurocode 4 (CEN 2004), the relative slenderness of the specimens ranged from \( \lambda_c = 0.27 \) to 0.29, in which \( \lambda_c = \sqrt{(P_{no}/P_{cr})} \); \( P_{no} = 0.85f'_cA_c + F_yA_s + F_yrA_r \); \( P_{cr} = \pi^2EI_{eff}/L_e^2 \); and \( EI_{eff} = E_sI_s + E_rI_r + 0.6E_{cr}I_c \). The net column length, excluding the rigid zones and the knife-edges, was \( L_c = 1,360 \) mm. To evaluate the slenderness effect of the composite column, the effective length of \( \text{ERE3} \) was increased to \( L_e = 4,320 \) mm \( (L_c = 2,800 \) mm). The corresponding relative slenderness was \( \lambda_c = 0.41 \). In the concentrically loaded column, \( \text{CRE} \), the net column length and the effective length (= length between the top and bottom loading plates) were \( L_c = 1,050 \) mm and \( L_e = 1,500 \) mm, respectively.

In the Japanese standard (AIJ 2014), the lateral reinforcement ratio, \( \rho_h \), of conventional ties for concrete-encased steel tubes is specified as \( \rho_h \geq 0.2\% \), in which \( \rho_h = A_h/(t_c s) \), \( A_h \) = sectional area of ties, \( t_c \) = thickness of the concrete encasement, and \( s \) = vertical spacing of ties. In the test specimens, the lateral reinforcement ratios
were $\rho_h = 0.85\% \ (s = 120 \text{ mm})$ and $0.42\% \ (s = 240 \text{ mm})$. The spacing was one-fourth and one-half of the cross-section dimension and was 1.7 and 3.4 times the thickness of the concrete encasement, respectively.

In specimens ERE2, ERE3, and CRE, shear studs were welded to the exterior surface of the steel tube to restrain early spalling of the concrete encasement. Fig. 5-2(a) shows the details of ERE2 and ERE3. Four longitudinal D16 bars (diameter = 16 mm, $A_r = 199 \text{ mm}^2$) were placed at the corners of the cross-section. D10 bars (diameter = 10 mm, $A_h = 71 \text{ mm}^2$) were used for the ties. The vertical spacing was $s = 120 \text{ mm}$. Instead of a 135° hook anchorage, a 90° hook lap splice was used for the ties, because of the interference with the steel tube. Two studs (diameter = 13 mm, length = 40 mm) were welded at each face of the steel tube. The vertical spacing of the studs was 240 mm. The cover concrete thickness for the studs and ties was 30 mm. In addition to the four corner longitudinal re-bars, two longitudinal D13 bars (diameter = 13 mm, $A_r = 127 \text{ mm}^2$) were used at each side of the cross-section. The D13 bars were tack-welded to the studs to prevent buckling of the re-bars and to provide better resistance to the concrete spalling. In the concentrically loaded column, CRE, the reinforcement details of the concrete encasement were the same as those of ERE2 and ERE3, although the dimensions of the cross-section were smaller.

The details of ERE1 were almost the same as those of ERE2. However, steel fiber-reinforced concrete was used to restrain the premature spalling of the thin concrete encasement. The aspect ratio of the hooked steel fiber was 60 (diameter = 0.5 mm, length = 30 mm). The volumetric ratio was limited to 0.8% for concrete workability in the thin concrete encasement. In the concrete-encased steel tube specimen tested by Yamashita et al. (2005), the steel fiber ratio of the concrete
The details of ERE4 were the same as those of ERE2, except for the re-bar details of the concrete encasement. The concrete encasement of ERE4 was reinforced by dense and fine welded wire mesh (WWM, bar diameter = 5 mm, grid dimensions = 50 × 50 mm). To provide a bond between the concrete encasement and steel tube, two D25 longitudinal re-bars (diameter = 25 mm, $A_r = 507$ mm$^2$) were welded intermittently to each side of the steel tube [Fig. 5-2(b)]. The WWM was then tack-welded to the longitudinal rebars. The longitudinal rebars can increase the flexural capacity of the column, and provide buckling resistance for the steel tube after spalling of the concrete encasement.

In ERE5, the steel tube, longitudinal bars, and transverse ties were the same as those of ERE2. However, instead of using studs, U-shaped ties, which penetrate through the steel tube, were used to connect the concrete encasement to the inner filled concrete. To insert the U-cross ties through the steel tube, holes of 16 mm diameter were drilled in the steel tube. The U-shaped ties were D10 bars, and the vertical spacing was 240 mm, which was the same as the spacing of the studs in ERE1, ERE2, ERE3, and CRE. The embedded length inside the filled concrete was 160 mm. The vertical spacing of the perimeter ties in the concrete encasement was $s = 120$ mm.

The properties of ERE6 were the same as those of ERE5, including the U-shaped ties. However, the vertical spacing of the perimeter ties in the concrete encasement was increased to twice that of ERE5: $s = 240$ mm.

The material properties of the concrete, steel, and re-bars are presented in Table 5-1. All the values indicate the average of results obtained from three tension
or compression tests. Tensile coupons of steel plates and re-bars were taken from the same sheets and re-bars used in the specimens. The maximum size of coarse aggregate was limited to 19 mm, considering the thickness of the concrete encasement. The compressive strengths of concrete cylinders were measured on the day of testing.

In **ERE1** with FRC, compressive strengths of concrete encasement (FRC) and filled concrete were $f_{ce} = 28.7$ and $f_{cf} = 21.9$ MPa, respectively. In **ERE2, ERE3, and ERE4** with ordinary concrete, the compressive strengths of the concrete encasement and filled concrete were $f_{ce} = 37.5$ and $f_{cf} = 21.1$ MPa, respectively. In **ERE5 and ERE6** with U-cross ties, the same concrete strength was used for both the concrete encasement and the filled concrete. The compressive strengths were $f_{ce} = f_{cf} = 31.7$ MPa in **ERE5** and $f_{ce} = f_{cf} = 24.3$ MPa in **ERE6**. In **CRE**, the concentrically loaded specimen, compressive strengths of the concrete encasement and filled concrete were $f_{ce} = 38.7$ and $f_{cf} = 21.9$ MPa, respectively. In **ERE1, ERE2, ERE3, ERE4, and CRE**, the yield strength of the steel tube was $F_y = 409$ MPa. On the other hand, the steel yield strengths were $F_y = 442$ MPa in **ERE5** and $F_y = 387$ MPa in **ERE6**.

The thin concrete encasement contributes greatly to the axial load and moment capacities of the specimens when plastic stress distribution is assumed. The axial load contribution of the concrete encasement, including the longitudinal bars, was estimated as 38% ~ 48% in **ERE1-ERE6** and 55% in **CRE** (Table 5-1).
Table 5-1. Properties of test specimens with the square tube

<table>
<thead>
<tr>
<th>Test parameters</th>
<th>ERE1</th>
<th>ERE2</th>
<th>ERE3</th>
<th>ERE4</th>
<th>ERE5</th>
<th>ERE6</th>
<th>CRE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Details for concrete encasement</td>
<td>FRC</td>
<td>Stud</td>
<td>Stud</td>
<td>WWM</td>
<td>U-cross tie</td>
<td>U-cross tie</td>
<td>Stud</td>
</tr>
<tr>
<td>Dimensions of cross section (mm)</td>
<td>480 x 480</td>
<td>480 x 480</td>
<td>480 x 480</td>
<td>480 x 480</td>
<td>480 x 480</td>
<td>480 x 480</td>
<td>380 x 380</td>
</tr>
<tr>
<td>Thickness of concrete encasement (mm) (hollowness ratio)</td>
<td>70 (50%)</td>
<td>70 (50%)</td>
<td>70 (50%)</td>
<td>70 (50%)</td>
<td>70 (50%)</td>
<td>70 (50%)</td>
<td>70 (40%)</td>
</tr>
<tr>
<td>Eccentricity e (mm) (e/hc)</td>
<td>60 (0.125)</td>
<td>180 (0.375)</td>
<td>60 (0.125)</td>
<td>60 (0.125)</td>
<td>60 (0.125)</td>
<td>60 (0.125)</td>
<td>0</td>
</tr>
<tr>
<td>Effective length Le (mm) (λca)</td>
<td>2880 (0.27)</td>
<td>2880 (0.27)</td>
<td>4320 (0.41)</td>
<td>2880 (0.27)</td>
<td>2880 (0.29)</td>
<td>2880 (0.27)</td>
<td>1500b</td>
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<tr>
<td>Concrete encasement</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete strength fce (MPa)</td>
<td>28.7</td>
<td>37.5</td>
<td>37.5</td>
<td>37.5</td>
<td>31.7</td>
<td>24.3</td>
<td>38.7</td>
</tr>
<tr>
<td>Longitudinal bars</td>
<td>4-D16</td>
<td>4-D16</td>
<td>4-D16</td>
<td>8-D25</td>
<td>4-D16</td>
<td>4-D16</td>
<td>4-D16</td>
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<tr>
<td>Bar yield strength Fy (MPa)</td>
<td>489/518</td>
<td>489/518</td>
<td>489/518</td>
<td>308</td>
<td>496/473</td>
<td>496/473</td>
<td>489/518</td>
</tr>
<tr>
<td>Bar area ratio (%)</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>1.8</td>
<td>0.9</td>
<td>0.9</td>
<td>1.5</td>
</tr>
<tr>
<td>Axial load contribution δec (%)</td>
<td>41</td>
<td>46</td>
<td>46</td>
<td>48</td>
<td>38</td>
<td>38</td>
<td>55</td>
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<td>Filled concrete strength fcf (MPa)</td>
<td>21.9</td>
<td>21.1</td>
<td>21.1</td>
<td>21.1</td>
<td>31.7</td>
<td>24.3</td>
<td>21.9</td>
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<td>Steel tube (mm)</td>
<td>340x340x6</td>
<td>340x340x6</td>
<td>340x340x6</td>
<td>340x340x6</td>
<td>340x340x6</td>
<td>340x340x6</td>
<td>240x240x6</td>
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<tr>
<td>Yield strength Fy (MPa)</td>
<td>409</td>
<td>409</td>
<td>409</td>
<td>409</td>
<td>442</td>
<td>387</td>
<td>409</td>
</tr>
<tr>
<td>Width-to-thickness ratio blt</td>
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<td>54.7</td>
<td>54.7</td>
<td>54.7</td>
<td>54.7</td>
<td>54.7</td>
<td>38.0</td>
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<tr>
<td>Area ratio (%)</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
<td>3.5</td>
</tr>
<tr>
<td>Ties</td>
<td>D10@120</td>
<td>D10@120</td>
<td>D10@120</td>
<td>WWM^d</td>
<td>D10@120</td>
<td>D10@240</td>
<td>D10@120</td>
</tr>
<tr>
<td>Yield strength (MPa)</td>
<td>443</td>
<td>443</td>
<td>443</td>
<td>500</td>
<td>496</td>
<td>496</td>
<td>443</td>
</tr>
<tr>
<td>Area ratio (%)</td>
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<td>0.85</td>
<td>0.85</td>
<td>0.56</td>
<td>0.85</td>
<td>0.42</td>
<td>0.85</td>
</tr>
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</table>
Chapter 5. Axial Testing and Evaluation of CEFT Columns

<table>
<thead>
<tr>
<th>Studs</th>
<th>Φ13@240</th>
<th>Φ13@240</th>
<th>Φ13@240</th>
<th>-</th>
<th>-</th>
<th>-</th>
<th>Φ13@240</th>
</tr>
</thead>
<tbody>
<tr>
<td>U-cross ties</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>D10@240</td>
<td>D10@240</td>
<td>-</td>
</tr>
</tbody>
</table>

\( a \) Relative slenderness (CEN 2004).

\( b \) Length of the specimen itself.

\( c \) Axial load contribution of concrete encasement \( \delta_c = (0.85 f'_{ce} A_{ce} + F_{yr} A_r)/(0.85 f'_{ce} A_{ce} + F_{yr} A_r + 0.85 f_y A_s + 0.85 f_y A_{st}) \)

\( d \) Welded Wire Mesh, grid of 50 × 50mm and bar diameter of 5 mm.

Fig. 5-2. Details of test specimens with the square tube
5.2.1.2 Specimens with Circular Tubes

Fig. 5-3 and Table 5-4 show details and properties of test specimens with circular tubes, respectively. The main variables were an axial-load eccentricity, the spacing of ties, and the use of concrete encasement. Specimens ECE1 and ECE2, with prototype details, have eccentricity of $e = 60$ and $180$ mm (eccentricity ratio of $e/h_c = 0.125$ and $0.375$), respectively. The vertical spacing of ties was $s = 240$ mm in both specimens. The spacing was reduced to $s = 120$ mm in ECE3, whose eccentricity was the same as ECE1 ($e = 60$ mm). CCE with the prototype details is a concentrically loaded specimen. Specimens EC1 and EC2 are conventional CCFT columns without concrete encasement. The eccentricity was the same as ECE1 and ECE2 ($e = 60$ and $180$ mm), respectively.

Circular steel tubes were fabricated from Grade 590 steel (STKT590, Korean standard, nominal yield strength = 440 MPa, actual yield strength $F_y = 565$ MPa, tensile strength $F_u = 681$ MPa, elongation = 36%). Tube diameter and thickness were $D = 406.4$ mm and $t = 7$ mm, respectively. The corresponding diameter-to-thickness ratio was $D/t = 58.1$, which is classified as non-compact section according to ANSI/AISC 360-10 ($\lambda_p = 54.4, \lambda_r = 68.9$). Table 5-3 summarizes tensile coupon test results of the STKT590 plates made of $t = 7, 9$, and $17$ mm. Generally, the yield strength, tensile strength, and elongation of longitudinal direction is greater than those of circumferential direction. The tubes with $t = 9$ and $17$ mm will be utilized in testing for beam-column connections.

Currently, circular tubes were fabricated through roll forming. Although roll bending and press bending are popular for cold-forming large-diameter tubes in Korea, the roll forming is often cheaper for small-diameter tubes. In a local fabricator,
Chapter 5. Axial Testing and Evaluation of CEFT Columns

The method is applicable for outer-diameter of 19 ~ 610 mm and thickness of 1.7 ~ 22 mm. After the forming, electric resistance welding (ERW) was applied longitudinally.

Table 5-2 compares chemical components of STKT590 steel (high-strength steel tube for tower structures, KS 2013) with similar steel grades STK590 (carbon steel tube for general structures, KS 2016) and STKN570B (carbon steel tube for building structure, KS 2016). Generally, STKT50 is expected to have a good weldability due to the low carbon content (C ≤ 0.12%) and carbon equivalent value (C_{eq} ≤ 0.40%), which are comparable to C ≤ 0.18% and C_{eq} ≤ 0.46% of STKN570B. It is also noted that the nominal tensile strength of STKT590 is limited to F_u ≤ 740 as STKN570B while such limitation is absent in STK590.

Cross-section dimensions of CEFT columns were 480 mm × 480 mm, which are the half scale of the prototype column. Longitudinal reinforcements 4-D16 (SD400) and 8-D13 (SD400) were placed at four corners of the cross-section (area ratio 0.78%). For transverse ties, D10 bars were used with a spacing of s = 240 mm (= b_c/2) in ECE1, ECE2, and CCE, and s = 120 mm (= b_c/4) in ECE3.

The concrete encasement thickness at the column center-line was 36.8 mm in order to maximize the hollowness ratio, and transverse ties were in contact with the tube surface. Current design specifications generally require some extent of spacing between the concrete-encased steel and perimeter ties for the purpose of restraining steel local buckling after the spalling of cover concrete. However, since resistance against the local buckling is excellent in concrete-filled circular tubes, additional confinement from the concrete encasement was not considered in the proposed CEFT column. Although the hollowness ratio of CEFT specimens is about
56%, in the prototype column (960 mm × 960 mm), the ratio is expected to surpass 60% if the concrete encasement is designed with the same concept.

Instead of ordinary cross-ties passing through the whole section, U-shaped stirrups were locally applied in the corner regions. The new cross-ties, whose legs were inserted into the tube wall, are intended to provide buckling constraint to longitudinal bars and to prevent early failure of the vulnerable concrete encasement. For penetration of D10 bars through the tube wall, 16 mm-diameter holes were punched. The cross-ties were buried in the core concrete with the anchorage length of 180 mm. The vertical spacing of U-cross ties was 240 mm in all CEFT specimens. In ECE1, ECE2, and CCE, considering alternate placement of conventional ties and U-cross ties, the effective tie spacing is reduced to 120 mm. In the case of ECE3, where the spacing of conventional ties was 120 mm, the largest distance between buckling constraint is still 120 mm.

The compressive strength of the concrete infill and encasement was 26.6 MPa in all CEFT specimens except for CCE, in which the concrete strength was 31.7 MPa. On the other hand, the compressive strength of the concrete infill of CFT specimens was 31.7 MPa.

Table 5-2. Chemical components of STKT590, STK590, and STKN570B

<table>
<thead>
<tr>
<th>Grade</th>
<th>C</th>
<th>Mn</th>
<th>P</th>
<th>S</th>
<th>Si</th>
<th>C_eq</th>
</tr>
</thead>
<tbody>
<tr>
<td>STKT590^a</td>
<td>≤ 0.12%</td>
<td>≤ 2.00%</td>
<td>≤ 0.030%</td>
<td>≤ 0.030%</td>
<td>≤ 0.40%</td>
<td>≤ 0.40%</td>
</tr>
<tr>
<td>STK590^b</td>
<td>≤ 0.30%</td>
<td>≤ 2.00%</td>
<td>≤ 0.040%</td>
<td>≤ 0.040%</td>
<td>≤ 0.40%</td>
<td>-</td>
</tr>
<tr>
<td>STKN570B^c</td>
<td>≤ 0.18%</td>
<td>≤ 1.60%</td>
<td>≤ 0.030%</td>
<td>≤ 0.015%</td>
<td>≤ 0.55%</td>
<td>≤ 0.46%</td>
</tr>
</tbody>
</table>

^a KS D 3780 quality standard
^b KS D 3566 quality standard
^c KS D 3632 quality standard
Table 5-3. Summary of tensile coupon tests for STKT590 steel

<table>
<thead>
<tr>
<th>$t$ (mm)</th>
<th>$D$ (mm)</th>
<th>$D/t$</th>
<th>Note</th>
<th>$F_y$ (MPa)</th>
<th>$F_u$ (MPa)</th>
<th>Elongation (%)</th>
<th>$F_y/F_u$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>406.4</td>
<td>58.1</td>
<td>flat</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>longitudinal</td>
<td>565</td>
<td>681</td>
<td>36</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>circumferential</td>
<td>546</td>
<td>658</td>
<td>25</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ERW$^a$</td>
<td>506</td>
<td>589</td>
<td>40</td>
<td>86</td>
</tr>
<tr>
<td>9</td>
<td>609.6</td>
<td>67.7</td>
<td>flat</td>
<td>586</td>
<td>674</td>
<td>28</td>
<td>87</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>longitudinal</td>
<td>572</td>
<td>670</td>
<td>38</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>circumferential</td>
<td>498</td>
<td>661</td>
<td>28</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ERW$^a$</td>
<td>560</td>
<td>649</td>
<td>34</td>
<td>86</td>
</tr>
<tr>
<td>17</td>
<td>609.6</td>
<td>35.9</td>
<td>flat</td>
<td>524</td>
<td>608</td>
<td>32</td>
<td>86</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>longitudinal</td>
<td>583</td>
<td>682</td>
<td>43</td>
<td>86</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>circumferential</td>
<td>556</td>
<td>657</td>
<td>34</td>
<td>85</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ERW$^a$</td>
<td>520</td>
<td>615</td>
<td>53</td>
<td>85</td>
</tr>
</tbody>
</table>

$^a$ Longitudinal direction of electric resistance welding region

Fig. 5-3. Details of test specimens with the circular tube
Table 5-4. Properties of test specimens with the circular tube

<table>
<thead>
<tr>
<th>Test parameters</th>
<th>ECE1</th>
<th>ECE2</th>
<th>ECE3</th>
<th>CCE</th>
<th>EC1</th>
<th>EC2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gross section ( b_c \times h_c ) (mm)</td>
<td>480 \times 480</td>
<td>480 \times 480</td>
<td>480 \times 480</td>
<td>480 \times 480</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Thickness of concrete encasement (mm)</td>
<td>36.8 (56%)</td>
<td>36.8 (56%)</td>
<td>36.8 (56%)</td>
<td>36.8 (56%)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Eccentricity ( e ) (mm) ((e/h_c))</td>
<td>60 (0.125)</td>
<td>180 (0.375)</td>
<td>60 (0.125)</td>
<td>-</td>
<td>60 (0.148)</td>
<td>180 (0.443)</td>
</tr>
<tr>
<td>Effective length ( L_e ) (mm)</td>
<td>2,880</td>
<td>2,880</td>
<td>2,880</td>
<td>1,500</td>
<td>2,880</td>
<td>2,880</td>
</tr>
</tbody>
</table>

Concrete encasement

- Concrete strength \( f_{ce}' \) (MPa) 26.6 | 26.6 | 26.6 | 31.7 | -   | -   |
- Longitudinal bars
  - 4-D16 | 4-D16 | 4-D16 | 4-D16 | -   | -   |
- Bar yield strength \( F_{yr} \) (MPa) 496/473 | 496/473 | 496/473 | 496/473 | -   | -   |
- Bar area ratio (%) 0.78 | 0.78 | 0.78 | 0.78 | -   | -   |
- Axial load contribution \( \delta_e^a \) (%) 29 | 29 | 29 | 30 | -   | -   |

Filled concrete strength \( f_{cf}' \) (MPa) 26.6 | 26.6 | 26.6 | 31.7 | 31.7 | 31.7 |

Steel tube (mm) \( \phi 406.4 \) (7T)

- Yield strength \( F_y \) (MPa) 565 | 565 | 565 | 565 | 565 | 565 |
- Width-to-thickness ratio \( D/t \) 58.1 | 58.1 | 58.1 | 58.1 | 58.1 |
- Area ratio (%) 3.8 | 3.8 | 3.8 | 3.8 | 6.8 | 6.8 |

Ties

- \( \text{D10@240} \)
- Yield strength (MPa) 496 | 496 | 496 | 496 | -   | -   |
- Area ratio (%) 0.8 | 0.8 | 1.6 | 0.8 | -   | -   |

U-cross ties

\( \text{D10@240} \)

\( a \) Axial load contribution of concrete encasement \( \delta_e = (0.85 f_{ce} A_{ce} + F_{yr} A_r) / (0.85 f_{ce} A_{ce} + F_{yr} A_r + F_y A_s + 0.85 f_{cf} A_{cf}) \)
5.2.2 Specimen Construction and Test Setup

After assembling of steel tubes and reinforcing bars [Fig. 5-4(a)], in which all the longitudinal components were welded to end plates to transfer tensile stress, the specimens were covered with outer molds for concrete pouring. Even though outer precast concrete is poured in a state where the column is laid down at factories, due to the existence of rigid ends in the test specimens, the member was upright during the casting. Both outer and inner concrete was dropped from different holes in the upper end plate [Fig. 5-4(b)]. After curing, surface-treatment was made for uniform contact of the loading plate [Fig. 5-4(c)].

Fig. 5-5(a) shows the test setup of the eccentrically loaded columns. To provide accurate hinge conditions in the direction of eccentricity, knife-edges were used at the top and bottom of the specimens. The effective column length between the hinges was $L_e = 2,880$ mm (effective length factor $K = 1.0$), and the net column length was $L_c = 1,360$ mm. On the other hand, in the case of concentrically loaded specimens, the axial load was applied directly by the UTM, without using knife-edges [Fig. 5-5(b)]. The length between the end plates was $L_e = 1,500$ mm ($K = 0.5$), in which the net column length excluding rigid boundaries was $L_c = 1,050$ mm. The axial force was controlled by a vertical displacement of 0.01 mm/s for the eccentrically loaded specimens and 0.005 mm/sec for the concentrically loaded specimen.

Fig. 5-5 also shows the instruments for measurement. Four LVDTs were used to measure the vertical displacements at the corners of the end plates, and two LVDTs were used to measure the horizontal displacements at the center of the column. Strain
gauges were used to measure the strains of the steel tube and rebars.

Fig. 5-4. Construction of test specimens
Fig. 5-5. Test set-up for eccentrically loaded specimens
5.3 Test Results

5.3.1 CEFT Columns with Square Tube

Fig. 5-6 shows the axial load-displacement relationships of the specimens. The vertical axis indicates the magnitude of the axial load, and the horizontal axis indicates the mid-height transverse deflection for ERE1−ERE6 and the axial shortening for CRE. The testing was terminated when the axial load decreased to 70% of the peak load. In the figures, the main events related to the damage of the specimens are indicated as A−F.

The test results including the peak axial load $P_{\text{test}}$, yield displacement $\Delta_Y$, and maximum displacement $\Delta_u$ are summarized in Table 5-5. Following the recommendation of Park (1988), the yield displacement was defined by the secant stiffness corresponding to 75% of the peak load (Fig. 5-6). The maximum displacement was defined as the post-peak displacement that corresponded to 80% of the peak load. Specimens ERE1, ERE4, ERE5, and ERE6, with an eccentricity ratio of $e/h_c = 0.125$, reached the peak load at transverse deflection of $\Delta = 4.33 \sim 6.08$ mm. However, in ERE3, the transverse deflection at the peak load was increased to $\Delta = 12.07$ mm, due to the larger column length. In ERE2, with the higher eccentricity ratio of $e/h_c = 0.375$, the transverse deflection at the peak load was $\Delta = 8.99$ mm. In the concentrically loaded specimen CRE, the axial shortening at the peak load was $\Delta = 3.20$ mm.

Fig. 5-6(a) shows the test result of ERE1, in which steel fiber-reinforced concrete (FRC) was used for the concrete encasement in addition to the conventional
ties (s = 120 mm) and studs. The load-carrying capacity decreased gradually after the peak load, showing ductile behavior, indicating that FRC with high toughness could be used when high ductility is required for the seismic design of columns.

Fig. 5-6(b) and Fig. 5-6(c) show the test results of ERE2 and ERE3, in which ordinary reinforced concrete and studs were used for the concrete encasement. In the case of ERE3, unlike ERE1, the load-carrying capacity decreased quickly after the peak load, due to early spalling of the concrete encasement. After spalling of the concrete encasement, the load-carrying capacity of ERE3 decreased further due to the second-order effect of the slender column. Also, in CRE subjected to pure axial loading [Fig. 5-6(g)], the load-carrying capacity decreased quickly after the peak load. However, in ERE2, with greater eccentricity, the decrease in strength was not significant. This is because, in ERE2 subjected to a smaller axial load, the strength contribution of the concrete encasement was less than that of ERE3 and CRE.

Fig. 5-6(d) shows the test result of ERE4 using welded wire mesh for the concrete encasement. Like ERE3 and CRE, the load-carrying capacity of ERE4 decreased quickly after the peak load, due to spalling of the concrete encasement.

Fig. 5-6(e) shows the test result of ERE5, in which conventional ties (s = 120 mm) and U-cross ties were used. Due to the greater strength of the steel plate and the filled concrete, the peak load was the greatest among the eccentrically loaded specimens. Despite the high peak load, the load-carrying capacity did not quickly decrease after the peak load. In ERE6, in which conventional ties of spacing s = 240 mm and U-cross ties were used, the peak load was significantly less than that of ERE5, because of the lower material strengths. Despite the lower peak load, the load-carrying capacity decreased immediately after the peak load, due to premature
spalling of the concrete encasement and subsequent buckling of longitudinal re-bars. These results indicated the importance of the spacing of conventional ties in restraining early concrete spalling and re-bar buckling.

Table 5-5. Test results

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Maximum load</th>
<th>Yield displacement</th>
<th>Maximum displacement</th>
<th>Ductility, $\mu = \phi_u/\phi_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$P_{test}$ (kN)</td>
<td>$\Delta$ (mm)$^a$</td>
<td>$\phi$ (1/m)</td>
<td>$\Delta_y$ (mm)$^a$</td>
</tr>
<tr>
<td>ERE1</td>
<td>6,844</td>
<td>6.08</td>
<td>0.00644</td>
<td>2.92</td>
</tr>
<tr>
<td>ERE2</td>
<td>4,195</td>
<td>8.99</td>
<td>0.00979</td>
<td>6.68</td>
</tr>
<tr>
<td>ERE3</td>
<td>6,883</td>
<td>12.07</td>
<td>0.00515</td>
<td>7.82</td>
</tr>
<tr>
<td>ERE4</td>
<td>7,674</td>
<td>5.65</td>
<td>0.00585</td>
<td>3.59</td>
</tr>
<tr>
<td>ERE5</td>
<td>7,730</td>
<td>5.91</td>
<td>0.00649</td>
<td>4.09</td>
</tr>
<tr>
<td>ERE6</td>
<td>6,269</td>
<td>4.33</td>
<td>0.00522</td>
<td>2.52</td>
</tr>
<tr>
<td>CRE</td>
<td>8,322</td>
<td>3.20</td>
<td>-</td>
<td>2.17</td>
</tr>
</tbody>
</table>

$^a$ Mid-height transverse deflection for ERE1–ERE6 and axial shortening for CRE
Fig. 5-6. Axial load-displacement relationships of specimens with rectangular tube
The major damage events A–F of the test specimens are presented in Fig. 5-6. Fig. 5-7 shows the failure modes of the specimens at the end of the test. The damage occurred at different locations in the specimens: at the bottom of the column in ERE1 and ERE5, at the top of the column in ERE2 and ERE6, and at the center of the column in ERE3, ERE4, and CRE. In the damage or failure region, local buckling of the steel tube was observed after testing. The spalling of the concrete encasement is not clearly visible in Fig. 5-7 because the specimens were wrapped in thin plastic sheets during testing.

In ERE1, the sequence of the damage modes was B (compressive cracking); C (yielding of compressive tube flange); F (spalling of concrete encasement); A (tensile cracking); E (yielding of ties). Although premature compressive cracking occurred in the cover concrete, the load-carrying capacity continued to increase, as shown in Fig. 5-6(a). Due to the effect of FRC, many fine cracks developed in the damaged region [Fig. 5-8(a)], and yielding of the ties occurred after a relatively large inelastic deformation. After removing the concrete encasement, local buckling was observed in both the longitudinal re-bars and steel plates.
In ERE2, with ordinary concrete and a greater eccentricity, the sequence of the damage modes was A (tensile cracking); C (yielding of compressive tube flange); B (compressive cracking); F (spalling of concrete encasement); D (yielding of tensile tube flange). In ERE3, with greater column length, the sequence of the damage modes was C (yielding of compressive tube flange); B (compressive cracking); F (spalling of concrete encasement); A (tensile cracking). In CRE, subjected to a concentric axial load, the sequence of the damage modes was C (yielding of compressive tube flange); B (compressive cracking); F (spalling of concrete encasement). In CRE, because of the smaller axial deformation and compactness of the steel plate \( (b/t = 38.0) \), local buckling of the longitudinal re-bars and steel tube was not seen clearly even after the end of the test.

In ERE4, reinforced with welded wire mesh in the concrete encasement, the sequence of the damage modes was the same as that of ERE3, except that transverse bars of the WWM yielded after spalling of the concrete encasement. As shown in Fig. 5-7, due to the dense WWM, the cover concrete was delaminated over a large part of the column surface. After the end of the test, fracture and local buckling were observed in the WWM [Fig. 5-8(b)].

In ERE5, with U-cross ties, the damage mode was similar to that of ERE4. In ERE6, which had U-cross ties and a greater vertical spacing of ties \( (s = 240 \text{ mm}) \), delamination of the concrete encasement occurred suddenly. As shown in Fig. 5-8(c), the concrete encasement was damaged significantly at the corners of the cross-section, and local buckling of the longitudinal re-bars occurred. This result again indicates the importance of the spacing of the ties to restrain the buckling of the longitudinal bars.
Chapter 5. Axial Testing and Evaluation of CEFT Columns

Fig. 5-7. Failure modes of square-tube specimens at the end of the test

Fig. 5-8. Damage after removal of the cover concrete
5.3.2 CEFT Columns with Circular Tube

Fig. 5-9 shows damaged specimens after the end of testing. Failure of ECE1 and ECE2 ultimately occurred at an upper and bottom part of the column, respectively. On the other hand, ECE3 was damaged at the mid-height region. The failure regions were confirmed from the spalling of the concrete encasement. Because the concrete of specimens was wrapped during the test, the concrete spalling is not clearly shown in Fig. 5-9. The local buckling of both longitudinal bars and steel tube was also detected after the testing. In EC1 and EC2, significant inelasticity occurred at the mid-height region. Axial load to mid-height deflection relationships are shown in Fig. 5-10, with major failure events A-G.

In ECE1 and ECE3, with an eccentricity of 60 mm, failures occurred in sequences of B (compressive cracking) - C (compressive yielding of the tube) - F (delamination of PC cover) - A (tensile cracking) - E (yielding of the hoop) - D (tensile yielding of the tube). Although early vertical cracking was observed at the column center-line where the concrete encasement is the thinnest (point B), the spalling of the concrete encasement (point F) did not happen until compressive tube fiber reaches yield strain 0.0028 (point C). The local buckling of the steel tube and longitudinal bars was observed after the breakup of the specimens.

Failure process of ECE2, with an eccentricity of 180 mm, followed A (tensile cracking) - B (compressive cracking) - C (compressive yielding of the tube) - F (delamination of PC cover) - E (yielding of the hoop) - D (tensile yielding of the tube). Even with the larger eccentricity, early spalling of the concrete encasement did not take place.
Local buckling was less remarkable in the circular tubes than in longitudinal bars. The distortion of tube wall took place at the height where U-cross ties were placed. Even though CEFT specimens exhibited excellent deformation capacity, loading was terminated due to the excessive failure of the concrete encasement. In the case of CCE, loading was terminated due to the limited capacity of UTM (Table 3). While yielding nor local buckling of reinforcing bars and the tube happened, tiny cracks were observed along the column center-line.

Specimens EC1 (eccentricity of 60 mm) and EC2 (eccentricity of 60 mm) followed C (compressive yielding of the tube) - D (tensile yielding of the tube) - G (tube local buckling). The load-carrying capacity started to decrease in both specimens as tube local buckling took place.

Fig. 5-9. Failure modes of circular-tube specimens at the end of testing
In load-displacement relationships (Fig. 5-10), CEFT specimens maintained residual strength without brittle failure even though sudden strength degradation happened after cover crushing. This indicates that the concrete failure was limited to the cover region outside the ties and the load-carrying capacity of the inner CCFT column increased further. This is contrary to the previous test results (Park et al. 2015), where CEFT columns with the square steel tube experienced substantial
deterioration in axial load due to the complete delamination of the concrete encasement and subsequent local buckling of the flange plate (hollowness ratio of 50%, axial-load contribution of concrete encasement of 38%).

Table 5-6 shows the peak load and the corresponding deflection of each specimen. Stiffness was defined as secant slope connecting the origin and 75% of the peak load (Park 1988). The peak load of ECE3 (tie spacing of 120 mm) was slightly greater than that of ECE1 (tie spacing of 240 mm) by 3%. Also, the corresponding deflection ECE3 was larger than that of ECE1 (7.1 and 6.1 mm, respectively). After the peak load, strength reduction was limited in ECE3 than in ECE1. However, in large inelastic deformation, the concrete encasement was completely damaged, and the load-carrying capacity was similar in the specimens.

The peak load of EC1 and EC2 was similar to that of ECE1 and ECE2 with the same eccentricity, respectively (96% and 89% of CEFT specimens, respectively). This is mainly attributed to the concrete strength higher in CFT specimens (31.7 and 26.6 MPa in CFT and CEFT specimens, respectively). On the other hand, the stiffness of CFT specimens was significantly less than that of CEFT specimens by more than 50%. Due to the effects of well-confined concrete and high-strength steel, the peak load of CFT specimens was attained in quite large deflections (28.7 and 34.9 mm, respectively). The load-carrying capacity gradually decreased due to the local buckling.

In the axial load-strain relationship of CCE (Fig. 5-10), the mean value of strains was used, which was measured by strain gauges attached to longitudinal bars and the tube wall. The loading was terminated at a load of 9,844 kN (an average axial strain of 0.0017) due to insufficient capacity of UTM. The peak load was about 84%
of the nominal strength 11,761 kN, which was calculated assuming a concrete strength coefficient as 0.85. The test result was almost consistent with the fiber analysis result using the stress-strain relationship obtained from the material test.

Table 5-6. Summary of test results for specimens with circular tube

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Peak load (kN)</th>
<th>Deflection (mm)</th>
<th>Stiffness (kN/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ECE1</td>
<td>7,237</td>
<td>6.1</td>
<td>1,841</td>
</tr>
<tr>
<td>ECE2</td>
<td>4,223</td>
<td>10.7</td>
<td>541</td>
</tr>
<tr>
<td>ECE3</td>
<td>7,487</td>
<td>7.1</td>
<td>1,797</td>
</tr>
<tr>
<td>CCE</td>
<td>9,844&lt;sup&gt;a&lt;/sup&gt;</td>
<td>0.0017&lt;sup&gt;b&lt;/sup&gt;</td>
<td>-</td>
</tr>
<tr>
<td>EC1</td>
<td>6,913</td>
<td>28.7</td>
<td>787</td>
</tr>
<tr>
<td>EC2</td>
<td>3,752</td>
<td>34.9</td>
<td>255</td>
</tr>
</tbody>
</table>

<sup>a</sup> Test was terminated due to the limited capacity of UTM

<sup>b</sup> Measured average axial strain at the peak load (mm/mm)
5.3.3 Moment-Curvature Relationships

Fig. 5-11 shows the moment $M$ -curvature $\phi$ relationships of eccentrically loaded specimens ERE1–ERE6. The vertical axis indicates the moment capacity at the mid-height section, which was estimated considering a second-order effect.

$$M = P(e + \Delta)$$  \hspace{1cm} (5-1)

where $M =$ moment at the mid-height section, $P =$ axial load, $e =$ initial eccentricity, and $\Delta =$ measured lateral deflection at the mid-height section. The horizontal axis refers to the average curvature over the length of the specimen, which was calculated as $\phi = (\Delta_t - \Delta_c)/(lx)$, where $\Delta_t$ and $\Delta_c =$ displacements of the vertical LVDTs measured at the tension side and the compression side, respectively (Fig. 5-5). The original length of the vertical LVDTs was $l = 2,060$ mm (= net column length $1,360$ mm + $2 \times$ rigid end length $390$ mm − $2 \times$ end plate thickness $40$ mm) for ERE1, ERE2, ERE4, ERE5, and ERE6, and $l = 3,500$ mm for ERE3. The horizontal distance between the two LVDTs on the compression and tension sides was $x = 1,200$ mm. In the calculation of the curvature, the strains measured from the strain gauges in the longitudinal re-bars and steel tube at the mid-height section could be used. However, in the present study, the strains were not used because they were not readable in large inelastic deformations due to local buckling.

As shown in Fig. 5-11, specimen ERE1 maintained a uniform moment capacity after yielding, showing relatively large ductility. ERE2, with a greater eccentricity, showed the greatest moment capacity ($M_{test} = 789$ kN-m) among the
specimens. In all specimens, the shapes of the moment-curvature relationships were similar to those of the axial load-displacement relationships. The moment capacity of all specimens except ERE1 and ERE5 decreased quickly after the peak strength due to the spalling of the concrete encasement. However, the residual moment capacity was maintained by the concrete-filled steel tube.

To evaluate the post-peak behavior of the specimens, the curvature ductility $\mu_\phi$ was estimated, as follows:

$$\mu_\phi = \frac{\phi_u}{\phi_y}$$  \hspace{1cm} (5-2)

where $\phi_y$ and $\phi_u$ = yield curvature and maximum curvature, respectively, which correspond to the yield and maximum displacements (Table 5-1). In Fig. 5-11, the maximum curvature and ductility of ERE1 with steel fiber-reinforced concrete were $\phi_u = 0.01790$ and $\mu_\phi = 5.19$, which was the highest among the specimens. The ductility of ERE2 and ERE3 was the lowest, $\mu_\phi = 2.13$ and 2.26, respectively. In ERE4, having WWM and longitudinal re-bars welded to the steel tube, the ductility $\mu_\phi = 2.49$ was similar to that of ERE2 and ERE3. The ductility of ERE5 and ERE6 with U-cross ties were $\mu_\phi = 3.15$ and 2.35, respectively.
Fig. 5-11. Moment-curvature relationships of square-tube specimens

Fig. 5-12. Moment-curvature relationships of circular-tube specimens
5.3.4 Measured Local Response

(1) Transverse ties

Fig. 5-13 shows the strains of the ties measured at different heights of the columns. In the eccentrically loaded specimens, three strains were measured at the compression side of the cross-section: at the top, middle, and bottom of the column. The vertical axis indicates the locations of the strain gauges and the horizontal axis indicates the measured strains at 45% of the peak strength, the peak strength, and 80% of the peak strength. Crack patterns of the compression surface at the end of the test are also shown in the figure.

In all specimens, as the deformation increased, the strains of the ties increased. Particularly, after the peak load, the strains increased significantly in ERE1, ERE4, and ERE5, exceeding the yield strain. The strains showed the greatest values in the region of concrete crushing.

(2) Concrete encasement

Fig. 5-14 shows measured longitudinal strains at the extreme compressive fiber of CEFT specimens. The strain gauges were attached to the mid-height surface 150 mm away from the center-line. In all CEFT specimens, the axial strain at the peak load exceeded 0.003, which means that the concrete encasement was effective in resisting to the compression force without premature failure. Particularly, the peak strain was the greatest in ECE2 with large eccentricity.
Fig. 5-13. Strains of the ties at compressive face

Fig. 5-14. Measured strain of concrete at the extreme compressive fiber
(3) Circular steel tubes

In order to assess the concrete confinement effect provided by circular tubes, the steel longitudinal and circumferential strains measured at compressive end of mid-height section were investigated (Fig. 5-15). In the horizontal axis, positive and negative values indicate compressive and tensile strains, respectively. The specimen ECE3 with concrete encasement reached the peak load (i.e. crushing of the encasement) after the tube yielding [Fig. 5-15(a)]. At the peak load of CFT specimens EC1 and EC2, the corresponding strains were greater.

Fig. 5-16 shows relationships between the circumferential and longitudinal strains. Until around the yield strain \( \frac{F_y}{E_s} = \frac{565 \text{ MPa}}{200,000 \text{ MPa}} = 0.00283 \) the relationships are almost linear regardless of the design parameters, and the measured circumferential strains are comparable to the predicted value assuming plane stress with uniaxial compression (i.e., lateral expansion governed solely by the poisson’s ratio 0.3). Thus, it is supposed that the negligible hoop tension developed at this stage. After this point, the transverse strain increases substantially, mainly due to the inelastic expansion of crushed concrete. The strain was greater in EC1 (eccentricity = 60 mm) than in EC2 (eccentricity = 180 mm), which implies that greater interaction between the steel tube and concrete infill occurred in EC1.
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Fig. 5-15. Measured tube strain in longitudinal and circumferential directions

Fig. 5-16. Circumferential-longitudinal strain relationships of tube wall
5.4 Predictions of Current Design Codes

5.4.1 Axial-Flexural Capacity

To examine the applicability of current design codes to the test specimens, the test peak strengths were compared with the predictions of American Concrete Institute (ACI) 318 (2011), ANSI/AISC 360 (AISC 2010), Eurocode 4 (CEN 2004), and AIJ (2014). For the prediction, the properties of the concrete, steel, and re-bars measured from the material tests (Table 2-3) were used. The tensile contribution of the concrete was neglected. The stress-strain relationships of the steel and re-bars were assumed to be elastic-perfectly plastic.

In the strain compatibility method of ACI 318 and ANSI/AISC 360, the axial-flexural capacity of a cross-section is calculated using linear strain distribution and the ultimate compressive strain of concrete (= 0.003). For the compressive stress of concrete, Whitney’s rectangular stress block was used. The concrete stress blocks of the filled concrete and the concrete encasement were defined differently, considering the different concrete strengths, as shown in Fig. 5-17.

Eurocode 4 and Method 2 of ANSI/AISC 360 use plastic stress distribution. In the plastic stress distribution method, the compressive stress distribution of concrete is assumed as a stress block with 0.85 $f'_c$, and uniform yield strengths are used for both re-bars and steel tube. For a simple and conservative design, Method 1 of ANSI/AISC 360 uses the axial load $P$-moment $M$ relationship with two linear lines, assuming an equivalent steel column.

Exclusively, the AIJ standard includes a design method for concrete-encased-
and-filled tubular columns (CEFT columns). As shown in Fig. 5-18, the P-M relationship of a cross-section is calculated by superposing the plastic strengths of the concrete encasement, steel tube, and core concrete. For simplicity, the core concrete is assumed to contribute only to the axial strength, neglecting any contribution to the flexural strength.

Fig. 5-19 shows the P-M relationships of the specimens predicted by the current design codes. Generally, the plastic stress distribution method showed the greatest strength, and method 1 of ANSI/AISC 360 showed the lowest. The AIJ standard prediction is close to the strain compatibility method. The predictions of the strain compatibility method agreed with the test strengths of all the specimens. Method 1 of ANSI/AISC 360 significantly underestimated the test strengths due to the relatively small contribution of the steel section. The AIJ standard slightly underestimated the test strength of the specimens with the low eccentricity ratio \( e/h_c = 0.125 \) (ERE1, ERE3–ERE6), by neglecting the flexural contribution of the core concrete. The plastic stress distribution method overestimated the test strength of ERE2 with greater eccentricity \( e/h_c = 0.375 \).

For the strength prediction of CRE, the nominal compressive strength \( P_{no} \) was calculated as follows:

\[
P_{no} = 0.85f'_c A_c + F_y A_s + F_{yr} A_r
\]

Where \( f'_c \) = cylinder strength of the concrete; \( F_y \) and \( F_{yr} \) = yield strength of the steel tube and longitudinal re-bars respectively; and \( A_c, A_s, \) and \( A_r \) = cross-sectional area of the concrete, steel tube, and longitudinal re-bars. In the calculation, a reduction...
factor of 0.85 was used for both the filled concrete and concrete encasement. Fig. 5-6(g) compares the test result and the prediction. The peak strength of the test was 19% higher than the prediction. This is mainly because the compressive strength of concrete was underestimated by using the factor of 0.85 in Eq. (5-3).

![Diagram of strain distribution and equivalent stress block](image)

Fig. 5-17. Equivalent stress blocks of concrete

![Diagram of axial load-moment relationship of AIJ SRC provision](image)

Fig. 5-18. Axial load-moment relationship of AIJ SRC provision (adapted from AIJ 2014)
Fig. 5-19. Axial load-moment relationships of specimens

Fig. 5-20 compares the test results and the predictions of the current design codes. While the plastic stress distribution method overestimated the strengths of CEFT specimens probably due to the use of high-strength steel, Method 1 of ANSI/AISC 360-10 significantly underestimated the test results. Generally, the strain compatibility method estimated the test strengths with good accuracy although specimen ECE1, with the tie spacing of 240 mm, slightly underperformed the
prediction. These results indicate that the thin concrete encasement in the CEFT column can be considered as a structural component if proper reinforcement details are provided. All CEFT specimens substantially outperformed the interaction curve after the peak axial load.

The AIJ prediction overestimated the capacity of ECE2 with large eccentricity. It is because that the axial-load contribution of the whole infill section was considered in the design. In conventional CEFT columns with small tubular sections, the contribution of the core concrete to the axial compression and flexure is not significant. However, a comparatively large tube section was used in this study. Consequently, it was dangerous to consider the axial resistance of the entire core section even when its flexural contribution was neglected.

![Graph showing axial load-moment relationships of CEFT columns with circular tube](image)

Fig. 5-20. Axial load-moment relationships of CEFT columns with circular tube
5.4.2 Effective Flexural Stiffness

In the design of columns, effective flexural stiffness is required to evaluate the buckling strength and the second-order effect. In Mirza and Tikka (1999), the effective flexural stiffness was defined at the peak load of the composite section. Following the definition of Mirza and Tikka, the test results at the peak load were used to evaluate the effective flexural stiffness of the test specimens.

In the test specimens, it was assumed that a bending action occurred only in the column part, and the rigid ends and knife-edges have infinite flexural stiffness. Considering the effect of the rigid ends and knife-edges, the mid-height deflection $\Delta$ was calculated as follows (Fig. 5-21).

$$\Delta = y_0 + y_c$$

(5-4)

where $\Delta =$ mid-height deflection at the peak load, $y_0 =$ lateral deflection at the end of the column part, and $y_c =$ mid-height deflection of the column part measured from its end position. Assuming that the flexural stiffness of the column part is uniform as $EI = EI_{eff}$ in the entire length of the column part, $y_0$ and $y_c$ were estimated using the theory of elastic stability, as follows:

$$y_0 = \frac{(1 - \cos kL_e)kL_0e}{\sin kL_0 + (\cos kL_e - 1)kL_0}$$

(5-5)

$$y_c = \left(\frac{\sec \frac{kL_e}{2} - 1}{e + y_0}\right)(e + y_0)$$

(5-6)
where $k = \sqrt{\frac{P_{\text{test}}}{EI_{\text{eff}}}}$, $L_c = \text{net column length}$ (2,800 mm for ERE3 and 1,360 mm for the other specimens), and $L_o = \text{total length of the rigid end and knife-edge}$ (760 mm). Using Eqs. (5-4)−(5-6), the effective flexural stiffness $EI_{\text{eff}}$ of the specimens corresponding to the test peak load and deflection was calculated.

The estimated effective flexural stiffness are presented in Table 5-7. The effective flexural stiffness ranged from $EI_{\text{eff}} = 54,900$ (ERE1) to 82,300 kN·m² (ERE3). Despite the same sectional profiles, the effective flexural stiffness of ERE3 ($EI_{\text{eff}} = 82,300$ kN·m²) was significantly higher than that of ERE2 ($EI_{\text{eff}} = 65,700$ kN·m²). This is because ERE2 was subjected to tensile cracking of the concrete encasement at peak load due to the higher eccentricity, which reduced the contribution of the concrete encasement to flexural stiffness.

In current design codes, the effective flexural stiffness of composite members is defined in Eqs. (5-7)−(5-9) by current design codes ACI (2011), AISC (2010), and CEN (2004), respectively, as follows:

\[
EI_{\text{eff}} = 0.2E_cI_c + E_sI_s \quad (5-7)
\]

\[
EI_{\text{eff}} = E_cI_c + 0.5E_rI_r + c_1E_cI_c \quad (5-8)
\]

\[
EI_{\text{eff}} = 0.9\left(E_cI_c + E_rI_r + 0.5E_cI_c\right) \quad (5-9)
\]

where $E_c$, $E_s$, and $E_r = \text{the elastic modulus of concrete, steel, and re-bar respectively}$, $I_c$, $I_s$, and $I_r = \text{second-order moment of inertia of concrete, steel, and re-bar respectively}$, and $c_1 = 0.1 + 2A_s/(A_c + A_s) \leq 0.3$. 

\[
\text{Figure 5.27: Measurement setup for ERE2.}
\]
Table 5-7 compares the effective flexural stiffness estimated from the test results and the predictions from the current design codes [Eqs. (5-7)–(5-9)]. ACI 318 and ANSI/AISC 360 underestimated the test results by 20% and 17%, on average, respectively. However, Eurocode 4 overestimated the test results by 33%, on average. This is because in Eq. (5-9), the contribution of the concrete is greater than that of the other design codes.

Fig. 5-22 shows the ratio of the code predictions to the test results. The ratios of **ERE1** were the highest because the peak load occurred at a large transverse deflection of $\Delta = 6.08$ mm. On the other hand, the ratios of **ERE3** and **ERE6** were relatively low. This result indicates that **ERE3** and **ERE6** failed due to early delamination of the concrete encasement.

In **ECE3** with smaller tie spacing, the measured stiffness was the lowest since the peak load was attained at greater displacement. The predictions of Eurocode 4 are greater than those of ANSI/AISC 360-10 because of a higher coefficient in Eurocode 4. ANSI/AISC 360-10 slightly overestimated the test results by 2%~13% while Eurocode 4 significantly overestimated by 52% ~ 68%.
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Fig. 5-21. Deformed shape of eccentrically loaded specimens

Fig. 5-22. Predictions of effective flexural stiffness by current design codes

Table 5-7. Effective flexural stiffness of specimens

<table>
<thead>
<tr>
<th>Code predictions</th>
<th>ERE1</th>
<th>ERE2</th>
<th>ERE3</th>
<th>ERE4</th>
<th>ERE5</th>
<th>ERE6</th>
<th>Average</th>
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<tr>
<td>Test result</td>
<td>5.49</td>
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<td>6.62</td>
<td>6.40</td>
<td>6.93</td>
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<td>5.41</td>
<td>5.41</td>
<td>5.33</td>
<td>5.07</td>
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</tr>
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<td>(94%)</td>
<td>(82%)</td>
<td>(66%)</td>
<td>(82%)</td>
<td>(83%)</td>
<td>(73%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>ANSI/AISC 360</strong></td>
<td>5.34</td>
<td>5.54</td>
<td>5.54</td>
<td>5.87</td>
<td>5.47</td>
<td>5.24</td>
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<td>(84%)</td>
<td>(67%)</td>
<td>(89%)</td>
<td>(85%)</td>
<td>(76%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Eurocode 4</strong></td>
<td>8.40</td>
<td>8.93</td>
<td>8.93</td>
<td>9.51</td>
<td>8.73</td>
<td>8.14</td>
<td>133%</td>
</tr>
<tr>
<td>(153%)</td>
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<td>(108%)</td>
<td>(144%)</td>
<td>(136%)</td>
<td>(117%)</td>
<td></td>
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</tbody>
</table>
Chapter 5. Axial Testing and Evaluation of CEFT Columns

5.5 Analytical Investigation

The CEFT specimens attained the peak load with concrete crushing and exhibited ductile behavior after a small drop in strength. In this section, the overall behavior of both CEFT and CFT columns was simulated by sectional fiber model analysis. Based on the results, the actual contribution of the thin concrete encasement was evaluated.

For the concrete infill of CEFT and CFT specimens, the effective stress-strain relationship proposed by Fujimoto et al. (2004) was applied. In general, strength and deformation capacity of the concrete infill significantly increase in CCFTs owing to the hoop action. As shown in Fig. 5-23(b), the strength of the confined concrete considerably increased from cylinder strength of 26.6 MPa to 38.4 MPa with improved ductility. Following the former studies (Fujimoto et al. 2004; Liang 2009), strength reduction considering the size effect was applied to the concrete model. For the stress-strain relationship of the steel tube, a bilinear model with strain-hardening was used for circular tubes. The effect of local buckling was neglected (Fujimoto et al. 2004).

For CEFT columns, a stress-strain relationship of the concrete encasement was defined, modifying an existing uniaxial model for unconfined and confined concrete (Cusson and Paultre 1995; Legeron and Paultre 2003). Although the concrete encasement was laterally confined by conventional ties and U-cross ties, it was still expected to be vulnerable to early spalling. Thus, an unconfined model was applied for the entire encasement, neglecting the lateral confining pressure of the ties.
In this study, the compressive strain $\varepsilon_{co}$ corresponding to the peak compressive strength was assumed to be 0.0022 according to the cylinder test result.

The peak load and the post-peak behavior of CEFT columns are sensitive to the descending branch of the stress-strain relationship for the concrete encasement. The descending branch is defined by two parameters; the compressive strain $\varepsilon_{cu}$ corresponding to 50% of the peak strength in the post-peak range and a factor $k_2$. While Cusson and Paultre (1995) recommend $\varepsilon_{cu} = 0.004$, Sheikh et al. (1994) suggested using smaller values for concrete of higher strength. Kim et al. (2012) used $\varepsilon_{cu} = 0.003$ for 100 MPa concrete. Because the thin concrete encasement is completely separated from the core concrete, $\varepsilon_{cu} = 0.003$ was used to simulate the brittle behavior conservatively. Another parameter was defined as $k_2 = 1.5$ (Cusson and Paultre 1995). The resultant stress-strain relationship is shown in Fig. 5-23(b).

For longitudinal bars, elastic-perfectly plastic behavior was assumed in tension while buckling deterioration was considered in compression. As shown in Fig. 5-23(a), it was assumed that the local buckling initiates when the compressive strain reaches $\varepsilon_{cu}$ (Chen and Lin 2006). A descending branch after the buckling follows a model proposed by Morino et al. (1986).

By using the effective stress-strain relationships, the fiber analysis was performed assuming strain compatibility at the mid-height section. For consideration of the column slenderness, it was assumed that a deflected shape of the column resembles a sine curve, where the mid-height deflection $\Delta$ and curvature $\phi$ are coupled as $\phi = \Delta(\pi/L_c)^2$ (Fig. 5-24). Iterative calculations were performed to attain force equilibrium at the fiber section, addressing the second-order effect under eccentric axial compression.
Fig. 5-25 compares the experimental and analytical results in terms of load-deflection relationships. In addition to a total resistance of the composite section, the contribution of each material (steel tube, inner concrete, outer concrete, and longitudinal bars) is shown. In the analytical results for CEFT specimens, axial load contribution was the greatest in the concrete encasement (outer concrete and longitudinal bars) until the peak load. In other words, the peak load was attained when the strength of the concrete encasement is the maximum. After the peak load, the load-carrying capacity suddenly decreases due to the concrete crushing while contributions of the steel tube and inner concrete continue to increase. The additional strength enhancement of the CCFT was the main cause of stable response of CEFT columns even with the failure of concrete encasement. In particular, due to the effect of high-strength steel tube ($F_y = 565$ MPa), the performance of the CCFT was excellent.

Fig. 5-25 indicates that the structural behavior of EC2 with large eccentricity was well simulated by the fiber analysis. On the other hand, the overall performance of EC1 with small eccentricity was underestimated. It is because that concrete confinement effect is greater with smaller eccentricity (CEN 2004) although an identical constitutive relationship was assumed for the inner concrete. This trend was also observed in CEFT specimens. While ECE2 with large eccentricity showed good accordance between the test and analysis results, ECE1 and ECE3 with small eccentricity did not. The amount of strength difference between the results of test and analysis was similar in CFT and CEFT specimens. Therefore, inaccuracy of the analytical model for ECE1 and ECE3 comes from the stress-strain relationship of inner concrete which depends on the degree of eccentricity. Further study is needed to improve the fiber model of the concrete filled in circular steel tubes.
The fiber model proposed for the concrete encasement seems to be appropriate. In the analysis, CEFT specimens attained the peak load slightly earlier (0.0028 in ECE1 and ECE3, 0.0031 in ECE2) than in the experiment (Fig. 5-14) in terms of extreme compressive fiber strain as well as mid-height deflection. This result demonstrates that actual deformability of the outer concrete was greater than expected in the fiber model. Thus, the effective stress-strain relationship of unconfined concrete can be conservatively applied for the thin concrete encasement in CEFT columns.

Fig. 5-23. Uniaxial compressive stress-strain relationships for fiber analysis

Fig. 5-24. Deformed shape of specimens subjected to eccentric axial compression
Fig. 5-25. Comparison between analytical and experimental results
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5.6 Discussion

Axial load tests were performed to evaluate the axial-flexural behavior of concrete-encased-and-concrete-filled steel tubular (CEFT) columns with thin-walled square or circular tubes. The present study focused on the details of the thin concrete encasement, which is vulnerable to premature spalling under high axial load. The major results of the present study are summarized as follows:

1) In the specimens with square tube, the peak strengths agreed with the predictions of the strain compatibility method (maximum compressive strain of concrete = 0.003). After the peak strength, the axial load-carrying capacity decreased, due to spalling of the concrete encasement. However, the residual moment capacity of the specimens was maintained due to the effect of the concrete-filled steel tube.

2) In specimens, ERE2 and ERE3 where conventional ties (vertical spacing \( s = 120 \text{ mm} \) and shear studs were used for the concrete encasement, spalling of the concrete encasement occurred immediately after the peak load. Thus, the ductility was limited.

3) In specimen ERE1, the steel fiber-reinforced concrete, which was used for the concrete encasement (in addition to the ties and shear studs), effectively restrained spalling of the concrete encasement in the post-peak behavior. The welded wire mesh (ERE4) and U-cross ties (ERE5 and ERE6, vertical spacing \( s = 240 \text{ mm} \)) were not as effective as the steel fiber-reinforced concrete in restraining early concrete spalling.
4) In specimen **ERE6**, as the vertical spacing of the ties increased to \( s = 240 \) mm (the requirement for conventional composite columns), early concrete spalling and re-bar buckling occurred immediately after the peak load. This result indicates that in CEFT columns, the spacing of ties should be decreased, considering the thin concrete encasement.

5) The effective flexural stiffness of the rectangular CEFT columns was compared with the predictions of current design codes. ACI 318 and ANSI/AISC 360 underestimated the effective flexural stiffness of the test specimens by 20% and 17% on average, respectively. However, Eurocode 4 overestimated the test results by 33% on average. The evaluation of the flexural stiffness is limited to short-term loading effect. Further studies are required to evaluate long-term loading effect on the flexural stiffness.

6) In the specimens with circular tube, although early vertical cracking occurred at the compressive face along the column center-line, the concrete encasement contributed to stiffness and strength of the CEFT column without premature failure. Particularly, the stiffness of CEFT specimens was significantly enhanced compared to that of CFT specimens by more than 50%. The CEFT specimens showed consistent performance regardless of the degree of eccentricity.

7) Reducing the spacing of conventional ties from \( b_c/2 \) to \( b_c/4 \) was effective in delaying failure of the concrete encasement and slightly increasing the strength. However, in large inelastic deformation, the tie spacing had limited effect because the contribution of concrete encasement is almost lost.

8) The CEFT specimens showed substantial ductility even with the failure of concrete encasement. This is attributed to the increased resistance of the circular steel
Chapter 5. Axial Testing and Evaluation of CEFT Columns

tube and confined core concrete. In hollow PC construction, where reduction of the lifting weight is crucial, CEFT columns with thin concrete encasement are applicable, also considering the fire resistance and improved stiffness. Particularly, the use of circular steel tube is recommended when high ductility is required.

9) The overall behavior of CEFT columns was simulated by sectional analysis using fiber models. The analysis results agreed with the test results, capturing unique limit states of the concrete encasement such as crushing, spalling, and rebar buckling.

10) Strength and flexural stiffness of the proposed CEFT columns were evaluated by predictions of current design codes. The combined axial-flexural strength of the specimens was the most reasonably predicted by the strain compatibility method. On the other hand, the flexural stiffness corresponding to the peak load was slightly overestimated by ANSI/AISC 360-10.

These conclusions are limited to the specimens tested in the present study. To confirm these conclusions, further studies need to be performed. Further, under cyclic loading, CEFT columns are expected to be more vulnerable to the premature spalling of the concrete encasement. Thus, the seismic performance of CEFT members needs to be verified under cyclic lateral loading.
Chapter 6. Connection Testing and Evaluation of CEFT Columns with Steel Beams

6.1 Introduction

In steel beam-CFT column connections, the local tensile mechanism should be prevented to avoid the brittle failure. When concrete encasement is added to the composite column, the out-of-plane deformation of the connection arises as another concern in regard to the unfavorable concrete cracking. Although the relevant design procedures are well-documented for the CFT structures, studies on the connections using CEFT columns are limited.

Experimental studies on beam-column joints using CEFT columns were performed mainly in Japan and China (Nakamura et al. 1999; Ueura et al. 1999; Liao et al. 2014). In the previous studies, the diameter of the steel tube was relatively small compared to the whole column section, and the concrete-filled tube section was also classified as compact (AISC 2016). For the strengthening, rigid connection details such as the outer diaphragm and stiffening steel plates were often used within the concrete encasement. Therefore, structural integrity and robustness of the concrete encasement were maintained throughout the test. In the case of the CEFT column of this study, however, the relatively thin concrete encasement and tube wall should be used for economy of the hollow PC construction. In addition, non-diaphragm connections are preferred considering concrete casting as well as less amount of welding. In such conditions, careful attention should be paid to restrain the significant distortion of the thin tube wall and the associated concrete damage.
In this study, the flexural behavior of the connection between the steel beam and CEFT column is examined. Among various connection details available for the composite tubular columns (AIJ 2008; AISC 2016), weld connections with or without strengthening details are adapted. A set of monotonic tension tests on flange plate-to-circular tube semi-rigid connections without diaphragm plates is conducted [Fig. 6-1(a)]. The load-distortion relationships are simulated by 3D finite element analysis. Cyclic load tests are then carried out for exterior beam-column joints with stiffened square or circular tubes. Based on the test results, failure modes and flexural resistance of the connections are investigated.

Fig. 6-1. Local behavior of steel beam-CFT column connection
6.2 Tension Testing of Flange-Column Connections

Previous studies on CEFT column-to-steel beam joints have applied external diaphragms, which are commonly used for circular CFT constructions (Nakamura et al. 1999; Ueura et al. 1999; AIJ 2008). However, the outer diaphragm is not appropriate for the CEFT column in this study because of the space limitation in the thin concrete encasement. Instead, a simple connection method was applied, directly welding the steel beam to the tube face, which is beneficial from the viewpoint of saving in material and welding. Since such connections are vulnerable to out-of-plane deformation of the steel tube and the subsequent local fracture takes place in a brittle manner (Schneider and Alostaz 1998), the semi-rigid connection often requires careful design. In particular, the local distortion of the high-strength steel tube may be detrimental to the concrete encasement. Thus, the characteristics of the tensile behavior of flange-tube connections need to be identified first, and appropriate strengthening details should be presented.

6.2.1 Test Program

To investigate the effects of the concrete encasement, both CEFT and CFT columns were included in the connection test. Table 6-1 and Fig. 6-2 show the test parameters and specimen details, respectively. Specimens TE-200 and TE-350 are connections using the CEFT columns, while specimens T-200, T-350, and T-350R indicate CFT column connections.

Fig. 6-2(a) illustrates the dimensions of connection specimens and details of
reinforcing bars. The test specimens correspond to 2/3 scale of the actual structure, and the cross-section of the CEFT column is 700 mm × 700 mm. In order to induce the connection failure and prevent the flange thickness from becoming excessively thick, high-strength steel HSA800 (Korean standard, nominal yield strength = 650 MPa) plates were applied (25 mm thick), maintaining the flange to be elastic up to the maximum load.

The length of the column was 1,600 mm so that the length of the column did not affect the local behavior of the connection. Instead of conventional cross-ties, U-shaped bars (D13) were applied in the same manner as the column compression test, in which the axial load-carrying capacity of the CEFT column was investigated. The reinforcing detail was proposed to ensure structural resistance of the concrete encasement preventing the early spalling when subjected to axial compression. To insert the U-cross ties through the steel tube, holes of 19-mm diameter were drilled in the tube wall. The ties were alternately arranged with the details of conventional hoop bars (D13), and the spacing was 300 mm for each type. In the vicinity of the flange edge, two consecutive layers of the U-cross ties were used to improve the lateral confinement for the concrete encasement.

Fig. 6-2(b) shows details of the connection between the steel tube and flange. STKT590 (Korean standard, nominal yield strength = 440 MPa) steel tube with an outer diameter of \( D = 609.6 \) mm and a thickness of \( t = 9 \) mm was used. The diameter-to-thickness ratio was \( D/t = 67.7 \), which corresponds to the non-compact section (AISC 2016). For weld between the tube wall and flange plate, flux cored wire of nominal tensile strength 630 MPa (AWS E81T1) was used matching the tube grade. The flange was connected to the tube through the K-groove CJP welding. The thickness of fillet welds was measured as 13 mm (Fig. 6-4).
A specimen **T-200** with a flange width of 200 mm was directly welded to the steel tube. The flange width of a specimen **T-350** was increased at the connection to improve the strength and stiffness. The tapered transition of the flange width was applied to avoid unexpected stress concentration. In a specimen **T-350R**, additional tension bars (4-D19) were welded to the flange to supplement the tensile resistance of the connection. Flare-bevel-groove welding was used for the weld connection. On the other hand, welding was not applied between the bars and tube (Fig. 6-4). The strengthening bars were penetrated through the steel tube without welding between the bars and the tube. Details of specimens **TE-200** and **TE-350** are identical to those of **T-200** and **T-350**, respectively, except for the additional concrete encasement.

Table 6-2 shows the material test results. Yield and tensile strengths of STKT590 steel tube in the longitudinal direction after bending were 572 and 670 MPa, respectively. The yield and tensile strengths of tension bars (D19) for **T-350R** were 518 and 634 MPa, respectively. The same concrete was poured for both concrete infill and encasement, whose compressive strength was 22.0 MPa at 28th day after casting. The connection tests were carried out at 25th to 28th days of age.

Fig. 6-3 shows the experimental setup and the measurement plan. A UTM of 3,000 kN capacity was used for the experiment. The distance from the UTM fastening part to the column surface was set to 500 mm so that tensile force was uniformly transmitted through the flange. LVDTs were installed to measure the deformation of the connection and the out-of-plane deformation of the column. Strain gauges were attached to the tube wall, reinforcing bars, and concrete surface to analyze the local responses.
## Chapter 6. Connection Testing and Evaluation of CEFT Columns with Steel Beams

### Table 6-1. Properties of specimens for flange-column connection test

<table>
<thead>
<tr>
<th>Test parameters</th>
<th>TE-200</th>
<th>TE-350</th>
<th>T-200</th>
<th>T-350</th>
<th>T-350R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column</td>
<td>CEFT</td>
<td>CEFT</td>
<td>CFT</td>
<td>CFT</td>
<td>CFT</td>
</tr>
<tr>
<td>CEFT section width $b_c$ (mm)</td>
<td>700</td>
<td>700</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Concrete encasement thickness (mm)</td>
<td>45.2</td>
<td>45.2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>(60%)$^a$</td>
<td>(60%)$^a$</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Longitudinal bars</td>
<td>4-D25</td>
<td>4-D25</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>8-D16</td>
<td>8-D16</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Tube diameter $D$ (mm)</td>
<td>609.6</td>
<td>609.6</td>
<td>609.6</td>
<td>609.6</td>
<td>609.6</td>
</tr>
<tr>
<td>Tube thickness $t$ (mm)</td>
<td>9</td>
<td>9</td>
<td>9</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>Diameter-to-thickness ratio $D/t$</td>
<td>$67.7^b$</td>
<td>$67.7^b$</td>
<td>$67.7^b$</td>
<td>$67.7^b$</td>
<td>$67.7^b$</td>
</tr>
<tr>
<td>Tube area ratio</td>
<td>3.5%</td>
<td>3.5%</td>
<td>5.8%</td>
<td>5.8%</td>
<td>5.8%</td>
</tr>
<tr>
<td>Conventional ties</td>
<td>D13</td>
<td>D13</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>@300</td>
<td>@300</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>U-cross ties</td>
<td>D13</td>
<td>D13</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>@300</td>
<td>@300</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Flange width (mm)</td>
<td>200</td>
<td>350</td>
<td>200</td>
<td>350</td>
<td>350</td>
</tr>
<tr>
<td>Tension bars</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>4-D19</td>
</tr>
</tbody>
</table>

$^a$Hollowness ratio  
$^b$Categorized as noncompact section (AISC 2016)

### Table 6-2. Material properties for connection test

<table>
<thead>
<tr>
<th>Tensile coupon</th>
<th>Application</th>
<th>Yield strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9T(STKT590)$^a$</td>
<td>Col. tube</td>
<td>572</td>
<td>670</td>
<td>38.1</td>
</tr>
<tr>
<td>25T(HSA800)</td>
<td>Flange</td>
<td>725</td>
<td>868</td>
<td>20.4</td>
</tr>
<tr>
<td>D25(SD500)</td>
<td>Long. bar</td>
<td>509</td>
<td>633</td>
<td>35.8</td>
</tr>
<tr>
<td>D19(SD500)</td>
<td>Tension bar</td>
<td>518</td>
<td>634</td>
<td>33.2</td>
</tr>
<tr>
<td>D16(SD500)</td>
<td>Long. bar</td>
<td>508</td>
<td>622</td>
<td>33.3</td>
</tr>
<tr>
<td>D13(SD400)</td>
<td>Tie</td>
<td>511</td>
<td>619</td>
<td>29.3</td>
</tr>
</tbody>
</table>

$^a$ Further information such as the mechanical characteristics, chemical compositions, and forming process is given in 5.2.1.2.
Fig. 6-2. Details of flange-column connections for tension test

Fig. 6-3. Test set-up and LVDT measurement for tension test
Fig. 6-4. Construction of flange-column connections with circular tubes
6.2.2 Load-Deformation Relationships and Failure Modes

Fig. 6-5 shows the load-deformation relationships of connection specimens. The deformation in the horizontal axis is the elongation occurring between the points 50 mm away from the both sides the column surface. Black lines indicate CEFT specimens and gray lines indicate CFT specimens. In all specimens, the maximum strength was attained by the fracture at the connection or tension bars (D19), and then the load rapidly decreased. Since the post-peak behavior of the connection is useless in the actual design, it was omitted.

Table 6-3 summarizes the experimental results. The yield point was defined as the point at which the tangential slope decreases to 1/3 of the initial stiffness (AIJ 2008). From the experimental results, there was no significant difference between CFT and CEFT specimens. It should also be noted that the peak strength and corresponding deformation of TE-350 with concrete encasement were smaller than those of T-350 without the encasement. This is probably because the holes drilled for penetration of U-cross ties were located at the edges of flange-tube connection that are prone to the stress concentration, thereby promoting the failure mode of the connection (i.e., edge tearing of the tube).

Fig. 6-6 shows the failure mode of connection specimens. In the specimens except for T-350R, the steel tube was finally torn at the heat-affected zone by welding, resulting in a decrease of load-carrying capacity. In these specimens, the initiation of tube fracture was almost coincident with the maximum strength. In the case of T-350R, the load started to drop due to the tensile fracture of strengthening bars (4-D19), followed by edge tearing of the flange-tube connection. The ultimate
failure mode of connections was similar in all specimens [Fig. 6-6(a)]. Early weld fracture or flange yielding did not occur.

In the case of the CEFT specimens TE-200 and TE-350, fine concrete cracks spread radially from the flange-embedded region at the beginning of the test, and finally the concrete encasement was extensively damaged [Fig. 6-6(b)]. The maximum crack width at the yield load (Table 6-3) was 0.3 mm. In the case of TE-350, the flange force was transmitted to the steel tube in a wide width, so that longitudinal cracking was also observed at the center-line of column surfaces where the flange was not connected.

Table 6-3. Results of connection test

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Initial stiffness (kN/mm)</th>
<th>Yield point Load (kN)</th>
<th>Def. (mm)</th>
<th>Peak point Load (kN)</th>
<th>Def. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TE-200</td>
<td>285</td>
<td>670</td>
<td>3.85</td>
<td>1,044</td>
<td>15.44 (1.3%)</td>
</tr>
<tr>
<td>TE-350</td>
<td>523</td>
<td>1,208</td>
<td>3.77</td>
<td>1,600</td>
<td>10.48 (0.9%)</td>
</tr>
<tr>
<td>T-200</td>
<td>231</td>
<td>702</td>
<td>4.84</td>
<td>1,090</td>
<td>14.13 (1.2%)</td>
</tr>
<tr>
<td>T-350</td>
<td>360</td>
<td>1,269</td>
<td>5.10</td>
<td>1,629</td>
<td>12.89 (1.1%)</td>
</tr>
<tr>
<td>T-350R</td>
<td>692</td>
<td>1,662</td>
<td>3.59</td>
<td>2,198</td>
<td>10.78 (0.9%)</td>
</tr>
</tbody>
</table>

*aDeformation divided by 2D

![Fig. 6-5. Load-deformation relationships of tension test](image-url)
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Fig. 6-6. Failure modes of connection specimens

(a) Specimen T-200  (b) Specimen TE-200

Fig. 6-7. Strains of U-cross ties
6.2.3 Effects of Test Parameters

Experimental results show that the increased flange width (by 1.75 times) improved the strength and stiffness of the connection by 1.61 and 1.56 times, respectively. Tension bars (4-D19) for the connection enhanced the performances additionally by 1.35 and 1.92 times, respectively. The thin concrete encasement contributed only to the increase of the initial stiffness, but the effect on the ultimate tensile strength was negligible. In practice, it is desirable to ensure sufficient capacity of the connection so that the damage of the steel tube and concrete encasement is controlled.

The maximum strength of T-350R reinforced with tension bars (4-D19) was 569 kN higher than that of T-350. The value is close to the yield capacity of the reinforcements (595 kN, \( f_y = 518 \) MPa). However, considering that the tensile strength (728 kN, \( f_u = 634 \) MPa) was actually reached immediately before the fracture, it is estimated that the flange-tube connection strength was partly degraded by drilling holes (for tension bars) on the tube wall.

Fig. 6-7 shows strains of the U-shaped ties installed in CEFT specimens TE-200 and TE-350. The strain gauges were attached to the upper and lower bars near the flange. The ties in TE-350 with the extended flange approached the yield strain at the maximum strength due to the out-of-plane deformation of the steel tube. The results support the effectiveness of U-cross ties for lateral confinement of the concrete encasement. However, as mentioned above, the resistance of the steel tube was degraded by puncturing the tube wall where the stress concentration occurred. Consequently, the beneficial effect of the U-cross ties on the connection strength was not clear. The arrangement of U-cross ties needs to be designed carefully.
6.2.4 Local Strains of Tube Wall

To estimate the vertical length between out-of-plane flexural yield lines (Fig. 6-28), local bending profiles of the tube wall were examined. Fig. 6-8 shows distribution of longitudinal strains along the tube surface. Horizontal axis refers the measured strain that is positive in tension and negative in compression. The vertical axis refers to the distance from flange face. Weld thickness (= 13 mm) outside the flange is also illustrated in the graphs. Fig. 6-8 shows apparent double curvature, in which peak points in compression (around the location 75 mm away from the flange) and tension (weld end or 13 mm away from the flange) should be the location of yield lines. In the specimens T-200 and TE-200 with connection width 200 mm, the vertical length between experimental yield lines is supposed to be 37 (= 50 − 13) ~ 62 (= 75 − 13) mm. In the specimens T-350 and TE-350 with connection width 350 mm, the vertical length is supposed to be greater with 62 ~ 87 (= 100 − 13) mm. The values are slightly greater than the calculated lengths 39 and 51 mm (Fu and Morita 1998), respectively.

As mentioned, due to strengthening with tension rebars, the peak load of T-350R increased by 569 kN that is comparable to bar yield strength and less than the bar ultimate tensile strength, respectively. On the other hand, because of tube holes for the bar penetration, resistance of the flange-tube connection should be degraded. Fig. 6-9 compares longitudinal strains at 50 mm away from the flange in T-350 and T-350R. As the gauge spot is supposed to be close to the inflection point (or zero moment), the strain of T-350 was almost neutral or positive in small deformation. On the other hand, the strain of T-350R at the same location was clearly negative, indicating the bending action. The results back up that flexural stiffness of the flange-
tube connection decreased in T-350R even though the overall performance was upgraded with the stiffening bars.

Fig. 6-10 shows circumferential strains measured besides the flange edges where punching shear failure ultimately occurred. A consistent but large variation in the measured strains at the four locations indicates the significant stress concentration as well as complicated stress state at the area.

Fig. 6-8. Strain distribution along tube longitudinal direction
Fig. 6-9. Comparison between specimens T-305 and T-350R (effect of tube holes)

Fig. 6-10. Circumferential strain at connection edges
6.3 3D Finite Element Analysis

6.3.1 Modeling Overview

The experimental results of flange-column connections were simulated by 3D finite element analysis using ABAQUS. In the numerical model, the same dimensions and material properties as applied in test specimens were implemented. Considering computational accuracy and efficiency of the analysis, fine and course mesh discretization was properly applied (Fig. 6-11). Eight-node reduced integration brick elements (C3D8R) were used for both tube wall and concrete infill. Material nonlinearity and geometric nonlinearity were also considered. However, geometric imperfection and weld effects were not addressed in the model.

For the constitutive laws of steel tubes, a multi-linear relationship with strain-hardening effect was used, assuming the Mises yield criteria. Nonlinear behavior of the concrete infill was idealized by the damaged plasticity model (Lee and Fenves 1998). The interaction between the tube wall and concrete infill was described by defining contact properties; the “Hard” contact option was used in the normal direction, and the penalty friction formulation with the Coulomb friction model was used in the tangential direction (frictional coefficient = 0.6, maximum shear stress = 0.4 MPa).
6.3.2 Numerical Results and Validation

Fig. 6-12 compares the tensile force-deformation relationships of test specimens and numerical models. The nonlinear responses of T-200 and T-350 were well-simulated by the analysis. In the finite element models, however, the deformation capacity was significantly greater than in the test results, and the corresponding load-carrying capacity continued to increase. To capture the fracture of tube wall and assess the peak strength, further development in the modeling is required, one of which is a careful consideration of the weld effects.

The load-deformation relationship of T-350R was simulated by modifying the numerical results for T-350. It was assumed that the elongation of reinforcing bars (4-D19) was uniformly distributed over the tube diameter (i.e., bar tensile strain is calculated as the deformation divided by 609.6 mm), and the resultant bar tensile force was simply added to the numerical results of T-350. Elastic-perfectly plastic behavior (yield strength = 518 MPa) of the rebar was also assumed. The analytical result accorded well with the test result. However, it should be mentioned that actual bar elongation is localized especially under inelastic stress, and actual peak contribution of the tensile rebars should be greater at the moment of fracture (tensile
strength = 634 MPa). This implies that tensile contribution of the flange-tube connection was somewhat degraded due to the holes for the bar penetration. Thus, when different configurations such as tube dimensions, rebar diameter, and concrete strength are combined, careful considerations on the superposition are necessary.

Fig. 6-12 also shows the numerical prediction for J-350T which is a beam-column joint specimen (see the next section). By increasing the tube thickness from 9 mm to 17 mm, initial stiffness significantly increased, comparable to the numerical results of T-350R. The elastic range and expected peak strength are even greater.

Fig. 6-13 shows the deformed shape and stress distribution of tube wall in T-200. At the early displacement of 2.2 mm, yielding of the steel tube initiated. At the displacement of 14.4 mm, when the test specimen attained the peak load, the yielding propagated toward surrounding area of the connection. On the other hand, concrete failure did not occur until a very large displacement (37.4 mm, 1,390 kN).

The longitudinal strains in tube walls of the numerical model were also compared with the test results (Fig. 6-14). The local accordance seems to be satisfactory in the critical area between flexural yield lines.
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Fig. 6-12. Global response validation of numerical results

Fig. 6-13. Failure modes in numerical model
Fig. 6-14. Local response validation of numerical results
6.4 Cyclic Flexure Testing of Beam-Column Joints

In cyclic load tests of beam-column joints, both circular and square tubes were applied. Since the flexural behavior of connections was the main focus, an exterior beam-column configuration was adopted (Schneider and Alostaz 1998).

6.4.1 Test Program

6.4.1.1 Test Specimens with Circular Tube

Based on the results of monotonic tension test, a cyclic load test on the beam-to-column joints was carried out. The purpose of this experiment is to verify the seismic performance of the joints with respect to the flexural behavior of connection and the structural integrity of concrete encasement. Three exterior beam-column joints with different connection details were considered.

Fig. 6-15 and Fig. 6-16 show the experimental set-up and details of the beam-column joint specimens, respectively. In the cyclic load tests on beam-column joints, since the flexural behavior of the connection was the main focus, not the panel shear behavior, an exterior configuration of the joint was planned. The dimensions and material properties of the joint specimens were almost the same as those used in the connection test. Although the same steel tube was applied for the CEFT column \((D = 609.6 \text{ mm}, t = 9 \text{ mm})\), the overall width of the column was slightly reduced to 690 \(\times\) 690 mm because of the restriction of the test set-up. A built-up section of \(H-600\times200\times11\times17\) (H-height\(\times\)width\(\times\)web thickness\(\times\)flange thickness) was used for the steel beam, where the flange width was equal to that in the connection test.
The CEFT columns were manufactured with a length of 2,000 mm including both end plates. In order to prevent damage to the column ends due to the stress concentration, lateral reinforcement was intensively placed at the regions (D13 at 75mm). For longitudinal reinforcement of the concrete encasement, 4-D25 and 8-D16 rebars were used as in the connection specimens.

Table 6-4 shows test parameters of the joint specimens. The connection of all joint specimens was conservatively designed to induce flexural yielding of the beam and to limit the damage of concrete encasement. In specimen J-350R, tension bars (4-D19) and extended flange width (350 mm) were applied [Fig. 6-16(a)]. In specimen J-350T, an increased tube thickness (17 mm) was used instead of the tension bars [Fig. 6-16(b)]. The connection of specimen J-200C was strengthened with vertical steel plates passing through the steel tube [Fig. 6-16(c)], which were excluded in the connection test. Although the continuation of the full beam section through the column is an ideal method from the viewpoint of structural performance (Schneider and Alostaz 1998), such detail is not promising in terms of concrete filling.

The connection details of J-350R are almost identical to those of T-350R for the connection test except that the outer concrete is added. In J-350T, the connection strength was augmented by increasing the thickness of the steel tube to 17 mm (STKT590 steel tube having the same outer diameter of 609.6 mm). The increased tube thickness was applied to the length of 1,000 mm, 200 mm away from the upper and lower beam flanges (Fig. 6-18). CJP welding was used for the tube splice. In practice, it was planned that on-site welding is required for the upper tube splice, while the lower tube splice can be shop welded. In joint specimens J-350R and J-350T, as in the tension test, the conventional hoop reinforcements and the U-cross
ties were alternately arranged. Although both joint specimens are expected to possess the superior flexural capacity to the expected demand, less damage was anticipated in J-350T with greater stiffness (Fig. 6-17). Generally, the flexural design of beam-column connections is simplified with the planar models (AIJ 2008), conservatively neglecting contribution of the web-tube connection.

In J-200C with vertical steel plates, the continuous web section was designed to take both shear and flexural moment that correspond to the beam plastic capacity. The cross-shaped configuration was considered for the case of transverse beams. In manufacturing, the cruciform vertical plates were fabricated first, inserted into the steel tube where four vertical slots were cut, and fillet-welded to the external tube surface (Fig. 6-19). During the manufacturing, out-of-plane imperfection of the steel beam was inevitable, whose effect will be addressed later in the test results. Firstly, a difficulty in the fabrication arose during gas cutting of the tube for plate penetration, in which some extent of tube distortion was caused by the inherent residual stress because of no heat treatment. Another challenge would be excessive welding in vertical plates-tube and vertical plates-beam connections, by which the frame distortion is triggered. Considering the cross-section configuration of vertical steel plates, the L-shaped ties were arranged in the corners of the joint and butt-welded to the vertical plates.

The column-tree construction with bracket and on-site beam splice was applied to joint specimens, following the actual construction process. In J-350R and J-350T, the flange of beam bracket was connected to the column tube with CJP welding, and the web was connected by fillet welding. The expected plastic hinge of the specimens is located 200 mm away from the column surface (Fig. 6-16). The beam connection with the bracket was planned at a distance of 300 mm away from
(= \(d/2\), where \(d\) = beam depth = 600 mm) the potential plastic hinge. The flange plates were spliced with CJP welding, and the web plates were spliced with bolt connection. In J-200C, beam flanges were connected to the vertical plate with CJP welding, and web plate was connected with both-side fillet welding. The critical section of the beam is located at 300 mm away from the column face, where the vertical steel plate ends. The longitudinal stress acting on the beam flange is designed to be transmitted to the vertical steel plate through the welding length of 300 mm. The flange plates end at the column face.

Table 6-5 summarizes the material test results for joint specimens. The mechanical properties of STKT590 steel tube with a thickness of 9 mm were the same as those used in the connection test. The yield and tensile strength of a 17 mm-thick steel tube in the longitudinal direction after bending were 583 and 682 MPa, respectively. The yield and tensile strength of D19 rebars were 520 and 666 MPa, respectively, which were similar to those used for the connection specimen T-350R. The same concrete was cast in the inner and outer parts, whose compressive strength at 31st day if curing was 31.7 MPa; the joint cyclic load tests were performed on 32th to 38th days of age.

The cyclic loading applied vertically at the end of the steel beam. The effective length between the horizontal and vertical reaction forces of the column is 3,500 mm, and the effective length between the column center line the vertical loading is 3,515 mm. The equivalent story drift ratio can be calculated as the net vertical displacement at the beam end divided by 3,515 mm. In order to prevent lateral torsional buckling of beams during the test, lateral supports were installed. According to the AISC standard (AISC 2016), the hysteresis loading was repeated for 6 cycles at 0.375%, 0.5% and 0.75% drift ratios; for 4 cycles at 1.0% drift ratio;
for 2 cycles at 1.5%, 2.0%, 3.0%, 4.0%, 5.0%, and 6.0% drift ratios (Table 6-6). The loading was first applied in an upward direction which induces positive moment at the connection. LVDTs were installed to measure the displacement at loading point, the rigid-body motion of joint specimens, and shear deformation of joint panel. Strain gauges were also embedded to analyze the local deformation of steel plates and reinforcing bars.

Fig. 6-15. Test set-up for cyclic loading on exterior beam-column joints (dimensions in mm)
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### Table 6-4. Properties of beam-column joint specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Connection details</th>
<th>Member details</th>
</tr>
</thead>
<tbody>
<tr>
<td>J-350R</td>
<td>Wide flange</td>
<td><strong>CEFT column</strong>&lt;br&gt;Cross-section : 690×690 (mm)&lt;br&gt;Steel tube : Ø609.6 (9T, STKT590)&lt;br&gt;Longitudinal bars : 4-D25, 8-D16&lt;br&gt;Concrete strength : 31.7MPa&lt;br&gt;Flexural strength : 2,726kN·m</td>
</tr>
<tr>
<td>J-350T</td>
<td>Wide flange</td>
<td><strong>Steel beam</strong>&lt;br&gt;H-section : 600×200×11×17 (mm)&lt;br&gt;Flexural strength : 1,042kN·m</td>
</tr>
<tr>
<td>J-200C</td>
<td>Vertical plate</td>
<td></td>
</tr>
</tbody>
</table>

### Table 6-5. Material properties for beam-column joint specimens

<table>
<thead>
<tr>
<th>Tensile coupon</th>
<th>Application</th>
<th>Yield strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>17T(STKT590)²</td>
<td>Joint tube ¹</td>
<td>583</td>
<td>682</td>
<td>43.3</td>
</tr>
<tr>
<td>11T(SM490)</td>
<td>Beam web</td>
<td>355</td>
<td>531</td>
<td>35.2</td>
</tr>
<tr>
<td>17T(SM490)</td>
<td>Beam flange</td>
<td>368</td>
<td>540</td>
<td>36.3</td>
</tr>
<tr>
<td>22T(SM490)</td>
<td>Vertical plate ³</td>
<td>416</td>
<td>502</td>
<td>31.8</td>
</tr>
<tr>
<td>D25(SD500)</td>
<td>Long. bar</td>
<td>550</td>
<td>694</td>
<td>35.1</td>
</tr>
<tr>
<td>D19(SD500)</td>
<td>Tension bar ⁴</td>
<td>520</td>
<td>666</td>
<td>35.3</td>
</tr>
<tr>
<td>D16(SD500)</td>
<td>Long. bar</td>
<td>538</td>
<td>667</td>
<td>31.6</td>
</tr>
<tr>
<td>D13(SD400)</td>
<td>Tie</td>
<td>483</td>
<td>603</td>
<td>32.9</td>
</tr>
</tbody>
</table>

¹ Further information such as the mechanical characteristics, chemical compositions, and forming process is given in 5.2.1.2.

² Applied for J-350T

³ Applied for J-200C

⁴ Applied for J-350R
Table 6-6. Cyclic loading program for exterior beam-column joints

<table>
<thead>
<tr>
<th>Number of cycles</th>
<th>Drift ratio (%)</th>
<th>Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 cycles</td>
<td>±0.375</td>
<td>13.2</td>
</tr>
<tr>
<td>6 cycles</td>
<td>±0.5</td>
<td>17.6</td>
</tr>
<tr>
<td>6 cycles</td>
<td>±0.75</td>
<td>26.4</td>
</tr>
<tr>
<td>4 cycles</td>
<td>±1.0</td>
<td>35.2</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±1.5</td>
<td>52.7</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±2.0</td>
<td>70.3</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±3.0</td>
<td>105.5</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±4.0</td>
<td>140.6</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±5.0</td>
<td>175.8</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±6.0</td>
<td>210.9</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±7.0</td>
<td>246.1</td>
</tr>
</tbody>
</table>

Fig. 6-16. Details of beam-column connections with circular tube
Chapter 6. Connection Testing and Evaluation of CEFT Columns with Steel Beams

Fig. 6-17. FEA prediction for expected planar behavior

Fig. 6-18. Manufacturing process of J-350T with thickened tube
Fig. 6-19. Manufacturing process of J-200C with vertical plates
6.4.1.2 Test Specimens with Square Tube

Table 6-7 and Fig. 6-20 show the test parameter and specimen details, respectively. The test parameter was the beam depth (H-488 × 300 × 11 × 18 or H-588 × 300 × 12 × 20, where H-beam depth × flange width × web thickness × flange thickness) while identical CEFT columns were used for all the specimens. The cross-section of the tested columns (670 × 670 mm) was a two-thirds scale model of the prototype columns (1,000 × 1,000 mm). On the other hand, the dimensions of the prototype beams were used for the tested beams, to evaluate the strength of the beam-column joints considering unfavorable design conditions.

Since a steel tube is used, design considerations for the CEFT column are more similar to those for the CFT column than to those for the CES column. Therefore, continuity plates, which are commonly used for the rectangular tube, were applied in specimens J-D500 and J-D600 to transfer the tensile force of the steel beam flanges to the joint. The thickness of the continuity plate was 25 mm, which was greater than the beam flange thickness of 18 mm and 20 mm for J-D500 and J-D600, respectively. To ensure good concrete filling, the continuity plate had a square hole of 200 × 200 mm at the center, and 40 mm diameter holes at the corners of the plate. The top and bottom continuity plate (thickness = 25 mm) and a web plate (thickness = 11 and 12 mm for J-D500 and J-D600, respectively) were extended to 500 mm from the column face to form brackets, which were connected to the beams using welding for the flanges and bolting for the webs. To avoid early spalling of the concrete encasement, ties were closely spaced (Fig. 6-20) at the top and bottom of the column framed into the joint.
In the CEFT column, a square built-up steel tube with 8 mm thick steel plates was used. The thickness of the concrete encasement was 110 mm. The ratio of the hollow area to gross section area of the CEFT column was 0.45. In the concrete encasement, four D25 longitudinal bars were placed at the corners of the section, and D13 bars were used for the ties at the vertical spacing of 180 mm. To enhance the bond between the steel tube and concrete encasement, shear studs (ϕ13) were welded to the tube plates. The proposed CEFT column was expected to be susceptible to early cracking and spalling of the thin concrete encasement. Thus, to prevent such failure, eight D16 longitudinal bars were placed and tack-welded to the shear studs. The longitudinal bars were welded to the beam flanges, which provide additional resistance to the flange bearing. The thickness of the cover concrete for the ties and studs was 30 mm.

The material properties of the concrete, steel, and re-bars are also presented in Table 6-7. All the values indicate the average of the results obtained from three compression or tension tests. The maximum size of coarse aggregates was limited to 19 mm, considering the thickness of the concrete encasement. The compressive strength of the concrete cylinders was measured on the day of testing. The compressive strengths of concrete were 35.0 MPa and 43.5 MPa in the columns of J-D500 and J-D600, respectively. The yield strengths of steel plates were 430 MPa for 8 mm thick tubes; 437 MPa (409 MPa) for 11 mm (12 mm) thick web plates; 382 MPa (406 MPa) for 18 mm (20 mm) thick flange plates; and 421 MPa for the continuity plates. The yield strengths of re-bars were 479 MPa for D13 bars.

The moment capacities in Table 6-7 were calculated using the plastic stress distribution for the steel beams and the strain compatibility method for the CEFT columns. In the exterior beam-column connections, the moment capacity ratios (2.26
~ 3.15) of the columns to the beams were relatively high. Thus, the load-carrying capacity of the specimens was expected to be determined by the moment capacity of the beam if the early failure of the connections (such as fracture of the continuity plates) did not occur.

The column tube was connected to the continuity plate and web of the bracket by complete-joint-penetration (CJP) groove welding. The flanges and the web of the bracket were connected to the beam by CJP groove welding and bolting, respectively. Because of the change of the flange thickness, a plastic hinge may occur at the splice between the bracket and the beam, which may cause fracture of the splice or excessive slip of the bolt connection. This was investigated on the basis of the test results. The column longitudinal bars (two D16) and the joint ties (D13, spacing = 180 mm) were connected to the flanges and the web of the bracket, respectively by external fillet welding. The electrode E71T-1 with a tensile strength of 584 MPa and CVN toughness of 86J at 0°C was used. The concrete encasement and the filled concrete were placed at the same time after the fabrication of steel and re-bars.

The test set-up and loading program are shown in Fig. 6-15 and Table 6-6, respectively. The loading was first applied in the downward direction (negative loading) which caused the negative moment in the beam.
Table 6-7. Properties of exterior beam-column joint specimens with square tube

<table>
<thead>
<tr>
<th>Property</th>
<th>Specimens</th>
<th>J-D500</th>
<th>J-D600</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>H-section steel</td>
<td>488x300x11x18</td>
<td>588x300x12x20</td>
</tr>
<tr>
<td>Nominal dimensions (mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yield/tensile strength of flange (MPa)</td>
<td></td>
<td>382/507</td>
<td>406/555</td>
</tr>
<tr>
<td>Yield/tensile strength of web (MPa)</td>
<td></td>
<td>437/541</td>
<td>409/557</td>
</tr>
<tr>
<td>Yield/tensile strength of continuity plate (MPa)</td>
<td>421/542</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive flexural capacity (kN·m)</td>
<td>1,166</td>
<td></td>
<td>1,688</td>
</tr>
<tr>
<td>Negative flexural capacity (kN·m)</td>
<td>(1,675 for bracket) (2,115 for bracket)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column</td>
<td>CEFT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gross section dimensions (mm)</td>
<td></td>
<td>670x670</td>
<td></td>
</tr>
<tr>
<td>Tube section dimensions (mm)</td>
<td></td>
<td>450x450x8</td>
<td></td>
</tr>
<tr>
<td>Yield/tensile strength of steel tube (MPa)</td>
<td>430/538</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal re-bars</td>
<td>4-D25, 8-D16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete strength (MPa)</td>
<td>35.0</td>
<td>43.5</td>
<td></td>
</tr>
<tr>
<td>Flexural capacity (kN·m)</td>
<td>1,834</td>
<td>1,905</td>
<td></td>
</tr>
<tr>
<td>Column to beam moment ratio</td>
<td>3.15</td>
<td>2.26</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 6-20. Dimensions and details of specimens J-D500 and J-D600 (dimensions in mm)
6.4.2 Experimental Results

6.4.2.1 Test Specimens with Circular Tube

Fig. 6-21 and Fig. 6-22 show the moment-drift angle relationship and the failure modes of joint specimens, respectively. All the joint specimens were failed by a clear flexural yielding of the beam and exhibited excellent energy dissipation capacity. The maximum applied moment at the column surface was 118% ~ 129% of the beam plastic moment (Table 6-8). The stiffness of J-200C was the greatest among the specimens due to the enhanced second moment of inertia by the vertical steel section. In specimens J-350R and J-350T, the stiffness may be slightly degraded because of the slip in bolt connections.

In J-350R with tension bars, fine diagonal cracking in the panel zone initiated at 0.75% drift ratio but did not develop much until the end of the experiment. At 1% drift angle, cracks occurred radially when the flange was subjected to tensile force. A slip in the web bolt connection was detected by the sound early at a drift angle of 1.5% when the specimen reached the beam yield moment. At 2% drift angle, the specimen attained the maximum strength, and the concrete cracking did not develop anymore. The bolt slip was further facilitated, and local buckling at the flange splice was observed visually at 3% drift angle. At 4% drift angle, the stability loss of the beam was confirmed by a large friction noise between the steel beam and transverse support. Finally, the test was terminated by a tensile fracture in the heat-affected region of the flange splice at 5% drift angle. Until then, there was no drop in the load-carrying capacity, and the specimen failed in a brittle manner afterward. At the end of the test, no significant damage was observed in the joint panel zone, but minor
cover exfoliation due to the flange tensile force was observed on the connection surface (Fig. 6-23).

In the case of \textbf{J-350T} with increased steel tube thickness in the joint region, the overall behavior and failure modes were similar to those of \textbf{J-350R}. However, due to the greater out-of-plane stiffness (Fig. 6-23), the associated cracking was more limited than in \textbf{J-350R}. Also, increased shear resistance of the thickened tube further eliminated cracks at the panel zone. In \textbf{J-350T}, the bolt fastening force was increased from 660 N·m in \textbf{J-350R} to 911 N·m to minimize the influence of slip at the bolt connection. However, the local buckling at the flange joint and its fracture were delayed only from 3% to 4% and from 5% to 6% drift ratio, respectively. The maximum load was increased by 2% and 5% in positive and negative direction, respectively. In spite of the unexpected failure at the beam splice, the specimens \textbf{J-350R} and \textbf{J-350T} were well above the beam plastic capacity and showed a deformation capacity of more than 4% drift angle.

In \textbf{J-200C} with vertical through plates, diagonal cracking did not develop in the panel zone, and only flexural cracks were observed in the column. A frictional noise at the lateral support initiated early at 1.5% drift ratio, which was attributed to the initial imperfection during the manufacturing process as well as limited flexural rigidity about the weak axis; out-of-plane displacement of the beam end was approximately 40 mm. At 3% drift angle, the local buckling of the flange was visually captured, and the maximum load was reached. At 4% drift angle, the distortion at the plastic hinge propagated toward the vertical plate, and the load-carrying capacity decreased. However, no failure occurred at the weld connection between the vertical plate and beam plates until the test ended at 5% drift angle. At the end of testing, minor concrete cracking was observed at the contact region.
between the beam flange and the column surface.

Table 6-8. Strength evaluation of joint specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Beam plastic strength (kN-m)</th>
<th>Test results (kN-m) (+) loading</th>
<th>Test results (kN-m) (-) loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>J-350R</td>
<td>1,042</td>
<td>1,230 (118%)</td>
<td>-1,137 (109%)</td>
</tr>
<tr>
<td>J-350T</td>
<td></td>
<td>1,254 (120%)</td>
<td>-1,195 (115%)</td>
</tr>
<tr>
<td>J-200C</td>
<td></td>
<td>1,343 (129%)</td>
<td>-1,318 (126%)</td>
</tr>
</tbody>
</table>

(a) J-350R

(b) J-350T

(c) J-200C

(c) Envelope curves

Fig. 6-21. Moment-drift angle relationships of joint specimens under cyclic loading
6.4.2.2 Test Specimens with Square Tube

Fig. 6-24 shows the moment-drift angle relationship of the specimens. In the figure, the drift ratio was calculated by dividing the vertical displacement by the effective beam length 3,515 mm (from the column center to the loading point). In J-D500 and J-D600, the negative loading of the beam attained the maximum strength at 4% and 3% drift ratio, respectively. On the other hand, the positive loading reached the peak load at 3% drift ratio in both specimens. After the peak strength, the load-carrying capacity quickly decreased due to the failure of the connection between the steel beam and column tube.
Fig. 6-24. Load-displacement relationships of specimens J-D500 and J-D600

Fig. 6-24 also shows the nominal strength based on the plastic moment of the beam $M_{bp}$. The peak moment $M_{b_{test}}$ of J-D500 with the smaller steel beam was obviously greater than the beam plastic capacity $M_{bp}$, while in J-D600 with the larger steel beam, the peak loads were comparable to $M_{bp}$. In both cases, the experimental peak loads were determined by the strength of the continuity plate-to-tube connection rather than by sufficient plastification of the beam, which would be discussed in the section 6.5.1.

Fig. 6-25 shows the damage of the specimens at the end of the test, and Fig. 6-26 shows the failure modes of the specimens. The concrete encasement of J-D500 and J-D600 was severely damaged, which indicates that large deformation occurred in the steel tube. At the beginning of the test (0.375% drift ratio), fine diagonal cracks of less than 0.1 mm occurred in both the joint faces parallel and orthogonal to the beam direction. As the displacement increased, the number of cracks increased. At 1.5% drift ratio, initial yielding of the beam flange of J-D500 occurred. In J-D500 and J-D600, the concrete cracks were less than 0.3 mm at 1.5% and 1.0% drift ratios.
respectively. At 2.0% and 1.5% drift ratios in J-D500 and J-D600, respectively, local fracture occurred at the corners of the transition between the continuity plate and the bracket flange, which was detected by the strain gauge measurement. For this reason, the concrete cracks significantly increased particularly at the joint face connected to the beam. The maximum crack width was 0.6 mm. However, even after the local fracture, the load-carrying capacity increased due to the effect of stress redistribution (Fig. 6-24). At 3% and 2% drift ratios in J-D500 and J-D600, respectively, the spalling of cover concrete occurred [Fig. 6-26(a) and Fig. 6-26(b)]. Further, at 4% and 3% drift ratios in J-D500 and J-D600, respectively, the concrete encasement was completely delaminated, the inner steel tube yielded, and the specimens exhibited the maximum load. The steel tube showed a significant out-of-plane deformation in the beam direction.

Fig. 6-25. Damage after the end of the test (J-D500 and J-D600)
Fig. 6-26. Failure modes of specimens J-D500 and J-D600

Fig. 6-27 shows strains of the re-bars and steel plates. In Fig. 6-27(a), the bracket flange of J-D500 [No. 1 in Fig. 6-27(a)] remained elastic under cyclic loading, while the beam flange (No. 2) at the splice experienced large inelastic deformation. Yielding of the beam flange occurred at 1.5% drift ratio. In Fig. 6-27(d), both the bracket and beam flange of J-D600 remained almost in the elastic range until the end of the test. Fig. 6-27(c) and Fig. 6-27(f) show the strains of the column tube plate near the continuity plate. The strains were significantly greater than the yield strain, which indicates that the steel tube experienced a significant out-of-plane deformation in the beam direction.

Fig. 6-27(b) and Fig. 6-27(e) exhibit the strains of the hoops. The strains gradually increased as the cyclic loading proceeded. The strain significantly increased when diagonal cracking occurred in the joint and reached the yield strain.
Fig. 6-27. Measured strains of rebars and tube flanges
6.5 Evaluation of Test Results

6.5.1 Connection Strength

(1) Connection tensile strengths by existing models

The final failure mode of the connection tension test was the steel tube tear out implying the punching shear mechanism in the vicinity of the weld joint (Fig. 6-6). Currently, the test strengths were compared with the predictions of Eq. (2-17) based on the yield line theory (Fu and Morita 1998, see Fig. 6-28) and Eq. (2-20) or the punching shear model of CIDECT No. 1 (Wardenier et al. 2008). On the basis of the results, two modified yield line models were studied.

Table 6-9 compares the predictions of Eqs. (2-17) and (2-20) with the test results. In the case of flange-column connection with tension bars (T-350R), the connection strength was calculated by adding the yield strength of the through tension bars to the equations for the semi-rigid connection. It is also assumed that the effect of the concrete encasement is negligible in the connection strength.

Even though the yield line theory is an upper-bound technique based on the mechanism approach, the calculated strengths $T_{YL}$ underestimated the test results $T_{test}$ (Table 6-9). Some assumptions in the simplified model can be considered as reasons for the conservatism. The flat plate of width equivalent to the circumferential length of tube wall was assumed in the original model to estimate the out-of-plane flexural resistance. However, the curved tube wall apparently possesses the greater stiffness. This could be the reason why the cap was greater in T-350 ($T_{test} / T_{YL} = 1.36$) than in T-200 ($T_{test} / T_{YL} = 1.08$). Also, the effective tube thickness subjected to the punching
shear seemed greater than \( t \) (Fig. 6-29).

On the other hand, the CIDECT predictions \( T_{PS} \) overestimated the test strengths (Table 6-9), which accords with the previous study on filled columns (De Winkel 1998). It should be noted that the steel tubes used in this study are characterized by high yield strength, yield ratio, and diameter-to-thickness ratio (\( F_y > 460 \text{ MPa}, F_y/F_u > 0.8 \), and \( D/t > 55 \), respectively), exceeding the limitations.

Table 6-9. Evaluation of connection tensile strength

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test results</th>
<th>Mixed yield line</th>
<th>Punching shear</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( T_{test} ) (kN)</td>
<td>( X ) (mm) by Eq. (2-18)</td>
<td>( T_{TYL} ) (kN) by Eq. (2-17)</td>
</tr>
<tr>
<td>TE-200(^a)</td>
<td>1,044</td>
<td>39</td>
<td>934</td>
</tr>
<tr>
<td>TE-350(^a)</td>
<td>1,600</td>
<td>51</td>
<td>1,201</td>
</tr>
<tr>
<td>T-200</td>
<td>1,009</td>
<td>39</td>
<td>934</td>
</tr>
<tr>
<td>T-350</td>
<td>1,629</td>
<td>51</td>
<td>1,201</td>
</tr>
<tr>
<td>T-350R(^b)</td>
<td>2,198</td>
<td>51</td>
<td>1,796</td>
</tr>
</tbody>
</table>

\(^a\) Contribution of concrete encasement neglected
\(^b\) Yield strength of tension bars 595 kN (4-D19, \( f_y = 518 \text{ MPa} \))
(2) Connection tensile strengths by modified models

In this study, the original yield line model Eq. (2-17) was modified for better agreement with the test results. One alternative model (named as shear modification) was derived with the corrected stress decomposition in the shear yield lines [Fig. 6-30(a)], addressing the shear modulus $G_s = \frac{E_s}{2(1+\nu)}$, where $\nu = 0.3$. The resultant model is proposed as follows:

$$T_{yls} = \frac{4D\partial m_u}{X} + \frac{t(X + t_v)F_u(1.6\sin^2 \theta + 1)}{1.3\sqrt{1-0.56\cos^2 \theta}}$$  \hspace{1cm} (6-1)

$$X = \sqrt{\frac{5.2D\partial m_u\sqrt{1-0.56\cos^2 \theta}}{tF_u(1.6\sin^2 \theta + 1)}}$$  \hspace{1cm} (6-2)

Another modification (named as flexure modification) was examined applying the increased flat-plat thickness to account for the greater flexural rigidity of the arc segment [Fig. 6-30(b)]. The equivalent thickness was calculated, equating the moment of inertia of the curved and flat plates.
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\[ t_{eq} = \frac{3(D-t)^3 t}{2b_f} \left( \frac{2\theta + \sin 2\theta}{2} - \frac{1-\cos 2\theta}{\theta} \right) \]  \hspace{1cm} (6-3)

The modified approach is only different from the original model in that \( t_{eq} \) by Eq. (6-3) is used to calculate \( m_{ul} (= t_{eq}^2 F_u d / 4) \) in Eqs. (2-17) and (2-18).

Fig. 6-31 compares the predictions of modified and original models. By correcting the stress decomposition in the shear yield lines, the accuracy was slightly enhanced, maintaining the conservatism. On the other hand, the prediction with the flexure modification significantly overestimated the strength of T-350.

Fig. 6-30. Assumptions in modified yield line models
(3) Flexural strength of connections $\textbf{J-350R}$ and $\textbf{J-350T}$

Based on the yield line model, the tensile capacity of $\textbf{J-350R}$ can be calculated as 1,798 kN, which is the sum of the semi-rigid connection strength 1,201 kN by Eq. (2-17) and bar (4-D19) tensile strength 597 kN. The corresponding flexural resistance is calculated as 1,048 kN·m ($= 1,798 \text{ kN} \times 0.583 \text{ m}$), by simply assuming the moment arm as flange center-to-center depth. In the experiment, the maximum moment acting on the column surface of $\textbf{J-350R}$ was +1,230 kN·m, which is about 117% of the calculated flexural strength. However, no significant damage on the concrete encasement was observed except for the minor cover spalling at the column surface. In the case of $\textbf{J-350T}$ with increased tube thickness, the calculated connection tensile and flexural strengths were 2,671 kN and 1,577 kN·m, respectively. The experimental peak moment was +1,254 kN·m, which is about 81% of the calculated flexural resistance. Consequently, the cracking was more limited in $\textbf{J-350T}$ than in $\textbf{J-350R}$.

The results of the tensile test and the cyclic load test were compared to assess
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The connection strength. Fig. 6-32(a) and Fig. 6-32(b) shows the measured strains of the tension bars in T-350R and J-350R, respectively. The average strain at the peak strength of J-350R was 0.00117, which was about 76% of the average strain 0.00153 measured in T-350R (an elastic state in both cases). These results indicate that the beam flexural stress is transmitted through the web-tube connection as well as the flange-tube connection. In the joint design of tubular structures (Wardenier et al. 2008), the contribution of the part between the flanges (i.e., web plates in I-section or box-section braces) is not considered.

Table 6-10. Evaluation of connection flexural strength

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test results</th>
<th>Mixed yield line</th>
<th>Punching shear</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_{\text{test}}$ (kN·m)</td>
<td>$T_{\text{YL}}$ (kN) by Eq. (2-17)</td>
<td>$M_{\text{test}}$</td>
</tr>
<tr>
<td>J-350R$^{abc}$</td>
<td>(+) 1,230</td>
<td>1,798</td>
<td>1,048</td>
</tr>
<tr>
<td>J-350T$^{bc}$</td>
<td>(+) 1,254</td>
<td>2,671</td>
<td>1,557</td>
</tr>
</tbody>
</table>

$^a$ Contribution of concrete encasement neglected
$^b$ Yield strength of tension bars 597 kN (4-D19, $f_y$ =520 MPa)
$^c$ Test strength determined by beam failure

![Fig. 6-32. Measured strains of tension bars in J-350R and T-350R](image)

(4) Yield criteria of connection J-200C
The vertical through plate at the critical section (column face) is required to transmit the shear force and bending moment of the beam to the column. According to the yield criteria of Tresca and von Mises (Horne 2014), the plate can be designed to satisfy the following equation:

\[
\frac{M_{bcr}}{M_{bp}} + \frac{3}{4} \left( \frac{V_b}{V_{bp}} \right)^2 \leq 1
\]

where \(M_{bcr}\) and \(V_b\) denote moment and shear force acting on the critical section, respectively, and \(M_{bp}\) and \(V_{bp}\) are the plastic capacities of the vertical steel plate [\(M_{bp} = F_{yv}(t,h_v^2)/4 = 2,200\) kN·m, \(V_{bp} = 0.577F_{yv}(t,h_v) = 5,078\) kN, where \(F_{yv}\), \(t_v\), and \(h_v\) are yield strength, thickness, and height of the vertical plate, respectively]. By substituting the test results of \(M_{bcr} = 1,343\) kN·m and \(V_b = 424\) kN, the left side of Eq. (6-4) becomes 0.62, indicating that the vertical steel sheet is in an elastic state.

In J-200C with the vertical through plates, unlike J-350R and J-350T, since the beam vertical load is directly transferred to the vertical plate, the concrete damage was the most limited. As shown in Fig. 6-33(b), the beam flange exhibited a large plastic strain at the plastic hinge region, while it remains elastic at the welded transfer region. In the vertical plate [Fig. 6-33(a)], the strain remained almost elastic but considerably increased after the maximum strength, which is affected by the significant distortion at the plastic hinge (or lateral buckling).
(5) Flexural strength of connections J-D500 and J-D600

The test results of J-D500 and J-D600 showed that failure occurred at the connection between the column tube and the continuity plate. To evaluate the safety of the connection, an existing design method for CFT columns was applied to the CEFT columns, neglecting the contribution of the concrete encasement which showed early spalling. In the Recommendations for Design and Construction of Concrete-Filled Steel Tubular Structures (AIJ 2008), a design model (Fukumoto 2007) based on yield line theory was recommended for the design of the steel beam-to-tube connection. Fig. 6-34 shows the load transfer mechanism of the beam-to-tube connection using a continuity plate. The total tensile strength $T_{AU}$ of the connection
is defined as the sum of the following contributions: the strength of the continuity plate $T_c$, the out-of-plane strength of the tube flange in the horizontal and vertical direction $T_x$ and $T_y$, respectively, and the strength of the tube web $T_w$.

$$T_{AIJ} = T_y + \min(T_x, T_w) + T_c$$ (6-5)

$T_x$ and $T_w$ act in the same load transfer path. Thus, the smaller of $T_x$ and $T_w$ was used. In Fig. 6-34(b), the resistance of the continuity plate is divided into area A subjected to direct tension and area B subjected to less tensile deformation. In the present study, considering the air holes at the corner of the continuity plate, only area A was considered.

Table 6-12 presents the calculated strengths of the connections according to the AIJ recommendations. For the ultimate state, the total tensile strength of the beam flange-to-tube connection was $T_{AIJu} = 2,976$ kN in both J-D500 and J-D600. The majority of the resistance was attributed to $T_{cu} = 2,709$ kN of the continuity plate because relatively thin steel plates were used for the column tube. The resultant moment capacity of the connection was calculated as $M_{AIJu} = T_{AIJu}d$, where $d =$ depth of the steel beam: $M_{AIJu} = 1,452$ kN·m for J-D500 and $M_{AIJu} = 1,750$ kN·m for J-D600. As presented in Table 6-12, the predicted strengths agreed with the ultimate strength of the specimens: $M_{btest} = 1,447$ kN·m for J-D500 and $M_{btest} = 1,714$ kN·m for J-D600. This result indicates that in the evaluation of the strength of steel beam-CEFT column connection, the contribution of the concrete encasement should be neglected.
Chapter 6. Connection Testing and Evaluation of CEFT Columns with Steel Beams

Fig. 6-34. Load transfer mechanism between beam flange and tube (adapted from Fukumoto 2007)

Fig. 6-35. Moment capacity and demand of J-D500 and J-D600
Table 6-11. Summary of test results for exterior beam-column joints with square tube

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Beam plastic capacity $M_{bp}$, kN-m</th>
<th>Load-carrying capacity</th>
<th>Deformation capacity</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_{btesty}$, kN-m</td>
<td>$M_{btestu}$, kN-m</td>
<td>Yield displ. $\Delta_y$, mm (%)</td>
<td>Ultimate displ. $\Delta_u$, mm (%)</td>
</tr>
<tr>
<td>J-D500 (+)</td>
<td>1,166</td>
<td>1,294</td>
<td>1,447</td>
<td>63.2 (1.8)</td>
</tr>
<tr>
<td>J-D500 (−)</td>
<td>-1,166</td>
<td>-1,243</td>
<td>-1,425</td>
<td>-70.6 (2.0)</td>
</tr>
<tr>
<td>J-D600 (+)</td>
<td>1,688</td>
<td>1,418</td>
<td>1,714</td>
<td>59.2 (1.7)</td>
</tr>
<tr>
<td>J-D600 (−)</td>
<td>-1,688</td>
<td>-1,439</td>
<td>-1,657</td>
<td>-55.6 (1.6)</td>
</tr>
</tbody>
</table>

Table 6-12. Strength evaluation of beam flange-to-tube connections with square tube

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Geometry, mm</th>
<th>Predictions (Fukumoto 2007)</th>
<th>Test results</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$l_y$</td>
<td>$l_x$</td>
<td>$l_w$</td>
</tr>
<tr>
<td>J-D500 Yield</td>
<td>109</td>
<td>71</td>
<td>45</td>
</tr>
<tr>
<td>J-D500 Max.</td>
<td>109</td>
<td>71</td>
<td>143</td>
</tr>
<tr>
<td>J-D600 Yield</td>
<td>109</td>
<td>71</td>
<td>45</td>
</tr>
<tr>
<td>J-D600 Max.</td>
<td>109</td>
<td>71</td>
<td>143</td>
</tr>
</tbody>
</table>
Fig. 6-35 shows a comparison between the moment capacities and demands of J-D500 and J-D600, which were based on the strengths of the beam and the beam-tube connection. The moment in the vertical axis was normalized by the beam plastic moment. In J-D500, the moment based on the connection strength was slightly higher than the moment demand corresponding to the beam plastic hinge moment. This result indicates that the initial yielding occurred at the beam plastic hinge zone, and the tensile failure of the connection can occur as the plastic hinge moment of J-D500 is increased by the cyclic strain hardening effect. The failure mode agrees with the actual failure mode of J-D500. In J-D600, on the other hand, the moment based on the connection strength is significantly smaller than the moment corresponding to the beam plastic moment capacity. Thus, the failure mode of J-D600 is determined by the connection strength. The failure mode agrees with the test result.

However, in the actual design, such failure modes of J-D500 and J-D600 should be avoided because the failure at the connection causes severe spalling of concrete encasement and inelastic deformation of steel tube in the joint, which is difficult to repair, and decreases the deformation capacity of the beam-column joint. Under repeated cyclic loading, the plastic moment of the beam can be increased by the cyclic strain hardening effect. Thus, the connection between the beam and tube needs to be conservatively designed with greater strength. In steel beams, the strain hardening factor is generally taken as 1.1 [ANSI/AISC 341-10 (AISC 2010)].
6.5.2 Out-of-Plane Distortion

(1) Flange-column connections

In Fig. 6-5, the peak load of all connections was attained far before reaching deformation limit of 3% of the diameter \((2 \times 3\% \times 609.6 \text{ mm} = 36.6 \text{ mm})\) that is recommended for circular hollow section connections (Wardenier et al. 2008). The low deformability is mainly attributed to the effect of concrete infill. In the test specimens, the out-of-plane distortion at the maximum strength was \(0.9\% \sim 1.3\%\) of the diameter (Table 6-3). In a previous test specimen with similar parameters \((D = 609.6 \text{ mm}, t = 9.5 \text{ mm}, F_y = 451 \text{ MPa in longitudinal direction}, h_f = 300 \text{ mm}, t_f = 19 \text{ mm})\), the peak strength and corresponding distortion were 1,119 kN and 13.44 mm \((1.1\%D)\), respectively (Ito et al. 1995).

It is also noted that T-350R strengthened by tension bars exhibited the peak load at the smallest deformation among the connection types. This is contrary to the results of Schneider and Alostaz (1998), where the tension bars obviously improved the deformability. The reason is supposed that yielding of the tension bars tends to be localized irrespective of the tube size. In the previous study, the diameter of tubes was 355.6 mm \((6.4 \text{ mm thick})\), and four D19 bars were used for strengthening. The compatibility issue may be more crucial for greater tube dimensions.

(2) Beam-column connections

Fig. 6-36(a) and Fig. 6-36(b) show the measured strains of the U-cross ties and longitudinal rebar of J-350R, respectively. In the vicinity of flange-tube connection, the strain of a longitudinal bar [the lower one in Fig. 6-36(b)] exhibited
noticeable elongation when the flange was subjected to tensile force, which is attributed to the out-of-plane distortion of the steel tube. As a result, U-cross ties also experienced considerable tensile strain (0.00150 mm/mm) indicating confinement towards the concrete encasement. Minor cracking developed in the joint concrete of **J-350R** demonstrates the structural integrity of the concrete encasement. In the counterpart specimen **TE-350**, the peak strain of a U-cross tie at tension failure was 0.00194 mm/mm (Fig. 6-7).

![Graph of measured strains](image)

**Fig. 6-36.** Measured strains of reinforcing bars in J-350R
6.5.3 Shear Distortion of Joint Panel

Fig. 6-37 shows the shear deformation measured outside the joint panel using LVDTs. The average shear strain of the panel zone was calculated as \( \gamma_p = (\delta_1 - \delta_2) \sqrt{a^2 + b^2/(2ab)} \), where \( \delta_1 \) and \( \delta_2 \) are diagonal deformations of the LVDTs in the joint panel, and \( a \) and \( b \) are vertical and horizontal distances between the LVDTs. Since the relatively large cross-section of the column was used, the shear strength of the joints was sufficient and the shear mechanism was insignificant in all joint specimens. Especially, J-350T and J-200C exhibited lower shear deformation than J-350R. In the case of J-350T, the increased tube thickness was the main reason for the enhancement of the shear capacity. In the case of J-200C, the vertical plate that penetrates the steel tube directly resisted to the shear force. If smaller column section and interior joint configuration are used, due to the lower shear capacity and the greater shear demand, the shear behavior of the joint panel may affect the overall joint behavior. In such case, reinforcing methods like the vertical through plates and increased tube plate are recommended to improve the joint shear resistance effectively.
Fig. 6-38(a) shows the average shear strain $\gamma_p$ of the joint panel with square tubes. The shear strain has positive values under the positive loading of the beam. The joint shear behavior of J-D500 and J-D600 was similar in both the positive and negative directions due to the symmetric section of the beam.

To evaluate the joint shear deformation of J-D500 and J-D600, with a steel beam, the joint shear force-shear strain relationship was compared with the result of an existing prediction model [Fig. 6-38(b)]. Fukumoto and Morita (2005) proposed a prediction model that can describe the elasto-plastic behavior of the joint panel in steel beam-CFT column connections. The model is adopted in Recommendations for Design and Construction of Concrete-Filled Steel Tubular Structures (AIJ 2008). Fukumoto (2012) also proposed a prediction model for the joint panel of steel beam-CEFT column connections by modifying the existing model.

Fig. 6-38(b) compares the joint shear strains of J-D500 and J-D600 with the predictions (Fukumoto and Morita 2005 and Fukumoto 2012). In the predictions of the model for CFT column (Fukumoto and Morita 2005), the average joint shear strain and joint shear strength at the yield point were predicted as (0.00378, 2,641 kN) for J-D500 and (0.00378, 2,777 kN) for J-D600. In the predictions of the model for CEFT column (Fukumoto 2012), the yield points were predicted as (0.00378, 4,877 kN) and (0.00378, 5,511 kN). This result indicates that the joint shear strength was significantly increased by the concrete encasement. However, in the tests, failures of J-D500 and J-D600 occurred due to fracture of the connection between the continuity plate and the tube, rather than due to the shear strength of the joint panel. Thus, in Fig. 6-38(b), in the range of the experimental shear strains, the
predictions showed only the elastic behavior. Nevertheless, Fig. 6-38(b) shows that the initial stiffness of the test specimens agreed with the predictions.

![Graph](image_url)

(a) Average shear strains of joint specimens

(b) Comparisons of test results and predictions

Fig. 6-38. Shear strain of joint
6.5.4 Bracket-Beam Bolt Connection

Column-tree (or connection-stub) construction with bolt connection was applied in J-350R and J-350T, following actual field process. Although both specimens satisfied the beam plastic moment and exhibited a deformation capacity exceeding 4% of the rotation angle, early bolt slip and corresponding fracture at the flange splice are not desirable. As seen in Fig. 10, the buckling distortion was not significant at the expected plastic hinge because the yielding propagated away toward the beam splice with insufficient connection rigidity. The results indicate that considerable plastic deformation enough to undergo strain hardening was not concentrated at the expected plastic hinge.

The cause of the early failure of the beam splice is the bolt slip at the web connection. After the bolt slip, excessive deformation was concentrated at the flange splice, resulting in fracture of the heat affected zone after repeated local buckling and plastic elongation. In addition, the flange splice is considered to be vulnerable to early local buckling as the scallop radius at the web connection was 50 mm, which is larger than the standard detail (35 mm).

In order to prevent such premature failure, it is desirable to locate the splice further away from the expected plastic hinge point. In current test specimens, the splice was 300 mm (half of the beam height) away from the critical section. However, it is recommended that the distance is more than the beam depth. The AISC seismic provisions (AISC 2016) suggest that the protected zone should be more than half the beam height from the plastic hinge.
6.6 Discussion

In this chapter, the flexural behavior of the connection between the steel beam and thin-walled tubular column was studied. For this purpose, a set of monotonic tension tests on flange-tube connections and cyclic load tests on beam-column exterior joints was carried out. On the basis of the results, strength evaluation using design equations and distortion assessment using 3D FEA simulation are demonstrated. The results are summarized as follows:

1) When the flange width at the flange-column connection was increased from 200 mm to 350 mm, the strength and stiffness of the semi-rigid connection were enhanced by 1.61 and 1.56 times, respectively. Tension bars (4-D19) additionally improved the strength and stiffness by 1.35 and 1.92 times, respectively. An increased tube thickness improves not only the connection strength but also the panel shear resistance.

2) The concrete encasement in the CEFT columns contributes only to the increase of the initial stiffness, but the effect on the ultimate tensile strength was negligible. The U-cross ties penetrating the steel tube are effective in confining the concrete encasement when the tube wall is subjected to out-of-plane distortion. However, the holes for the ties may be vulnerable to the stress concentration and deteriorate the connection strengths. Thus, the location of the holes should be carefully designed.

3) The tensile force-deformation relationships of the specimens were well-simulated using 3D finite element analysis. The behavior of the specimen T-350R
was well-simulated by simple superposition of the calculated rebar force and numerical result without the reinforcements. The failure of concrete infill was not detected.

4) The final failure mode of the connection test was the steel tube tear out due to the punching shear mechanism in the vicinity of the weld joint, exceeding the design strength assuming chord plastification. The tensile strengths of flange-to-column connections were conservatively assessed by a modified yield line model. When tension bars are added, the strength can be increased by the bar yield strength.

5) The ultimate strength of the connection between the flange plate and tube wall should be greater than the expected maximum demand force which includes the effects of the material overstrength and strain hardening of steel beam under cyclic loading. The required tensile strength is expressed as $1.1M_{bp,exp}/d$, where $M_{bp,exp}$ is the expected flexural moment at the column face corresponding to the beam plastic moment and the factor 1.1 indicates a strain hardening effect of the steel beam.

6) In the beam-column joint specimen J-200C with vertical through plates at the joint, the flexural stiffness of the beam was improved due to the increased moment of inertia by the vertical plate. The vertical steel plate is also effective in strengthening the shear resistance of the joint panel.

7) Ultimate failure modes of specimens J-350R and J-350T were the bolt slip at the web splice and subsequent fracture at the flange splice. In order to avoid such failure modes, it is desirable to locate the beam splice away from the expected plastic hinge by at least the beam section height.
Chapter 7. Connection Testing and Evaluation of CEFT Columns with Concrete Beams

7.1 Introduction

The composite tubular columns have been commonly mixed with steel beams following the traditional steel construction. Accordingly, prior research and design guidelines for composite columns have also focused on connections with the steel beam (AISC 341-16). If the concrete encasement is added, however, the CEFT column is more easily mixed with RC beams, in which the beam-column joint can be conservatively designed following traditional RC moment frames (Han et al. 2014; Liao et al. 2014). The concrete beams can be combined with CFT columns to enhance the stiffness of the overall structure as well as the economy (Arimatsu et al. 2005).

Similarly, owing to the presence of PC encasement to the steel tube, PC beams can be connected to the CEFT column in the hollow PC construction. Currently, the use of U-shaped PC beam shells was considered (Fig. 7-1). The PC moment frame is constructed by erecting the concrete-encased steel tubular column first; seating the PC beam shell on the concrete encasement of column; placing beam reinforcing bars; and then pouring concrete to integrate the column and beam. Potential concerns for the field application are the tube penetration and anchorage of beam flexural rebars. In the ordinary CEFT column, since the inner tube is small, the interference with the beam rebars may not be a problem.
For conventional RC beam-CFT column joints, a simplified connection detail was proposed, eliminating the entire or substantial part of the steel tube at the joint (Chen et al. 2014; Tang et al. 2016). This method aims to maximize the structural continuity of RC beams and to get rid of field problems such as the rebar interference and congestion. However, it seems not advantageous in that the significant amount of rebars needs to be retrofitted for the compensation. Further, the concrete mass at the joint becomes enormous.

In this study, potential connection details for PC beam-to-CEFT column joints are investigated. For penetration and anchorage of beam flexural rebars at the joint, connection details such as partial wall opening and coupler splice are applied, considering constructability and earthquake resistance, respectively. The opening details are feasible because the steel is relatively thin. To evaluate the seismic performance of the connections, cyclic load tests are conducted. On the basis of the test results, the load-carrying capacity, deformation capacity, energy dissipation, and failure modes are investigated.

Fig. 7-1. Connection between CEFT column and PC beams
Chapter 7. Connection Testing and Evaluation of CEFT Columns with Concrete Beams

7.2 Test Program

7.2.1 Connection Details

In construction sites, it is not easy to pass beam longitudinal reinforcements through the holes in steel tubes. Instead, couplers can be retrofited outside the tube wall for the penetration and splice of the rebars. In this case, strengthening details may be necessary to protect the connection region (or the interface between the PC beam and joint). The coupler connection can also be an alternative for the anchorage of rebars in exterior beam-column joints. However, it should be noted that precise construction is required to place rebars at the right position. From the viewpoint of workability, the expanded opening in the steel tube is a simple option to accommodate the construction error, in which reinforcing plates are indispensable. Such detail also enables anchorage hook for in the exterior joint.

In current study, opening and coupler details were applied to interior and exterior beam-column joints, respectively. For the PC construction, U-shaped PC shells were used for the beams (Im et al. 2013). In Korea and several other countries, the U-shaped PC beam is frequently used to enhance the integrity of the beam and the joint by using cast-in-place concrete and to reduce the lift weight. The concrete casting in the PC beam-CEFT column joints was performed following the actual practice (Fig. 7-5): first, concrete was cast for the concrete encasement of the lower column and the PC beam shell; the remaining concrete was then placed after assembly of the lower column and the PC shell.
7.2.2 Specimens and Test Set-ups

7.2.2.1 Interior Beam-Column Joints

(1) Connection details

For interior beam-column joints, opening details of the tube wall were applied to improve the rebar placement at the field. However, in order to ensure the continuity of the steel tube as a longitudinal member, the cross-sectional opening was minimized, and the strengthening plates were welded to the tube wall. Fig. 7-2 shows the connection details of four test specimens. In order to flexibly respond to field requirements, two types of connection details A and B with different size of tube wall opening were considered.

In specimens **JP-A** and **JP-AT**, with small openings in the steel tube, the openings (150 mm × 150 mm) for penetration of the top and bottom flexural bars were separated. In order to make up for the cross-sectional loss, 75 mm-wide steel plates (of the same steel sheet as for the tube) was longitudinally welded to both sides of the opening. Due to the limited size of the opening, some of the top rebars (8-D29) needs to be placed outside the tube while the beam bottom rebars (4-D29) are accommodated in the opening. The advantage of type A is that the small opening ensures the stable stress transfer in the tube wall as long as appropriate reinforcing plates are added. However, it is required to settle the PC beam shell on the column after the placement of bottom rebars is complete.

In specimens **JP-B** and **JP-BR**, the openings were unified, and the size was increased to 210 mm × 480 mm. Accordingly, the strengthening plates were also
widen (105 mm). The increased opening has more potential to accommodate beam rebars inside the tube, and the bottom rebars can be easily placed after the PC shell settlement. While the structural integrity of concrete is enhanced at the joint region, in type B, the shear contribution of the panel web plates may not be expected.

If there is a cross-beam in the joint, the wall openings are also required in the perpendicular direction. However, in the absence of the cross-beam, undamaged web panels are expected to better contribute to the shear mechanism. Also, opening type B enables construction of hook bars (Fig. 7-3).

![Fig. 7-2. Opening details for PC beam-CEFT column connections](image1)

![Fig. 7-3. Wall opening schemes in exterior beam-column joints](image2)
### Table 7-1. Properties of test specimens

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam ((b_b \times d))</td>
<td>U-shaped PC shell + CIP slab ((600 \times 700 \text{ mm}))</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete strength(^a) (MPa)</td>
<td>26.5 for PC, 35.2 for CIP</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top re-bars(^b) (%)</td>
<td>8-D29 (1.40)</td>
<td>8-D29 (1.40)</td>
<td>8-D29 (1.42)</td>
<td>10-D29 (1.76)</td>
</tr>
<tr>
<td>Bottom re-bars(^c) (%)</td>
<td>4-D29 (0.97)</td>
<td>4-D29 (0.97)</td>
<td>4-D29 (0.97)</td>
<td>6-D29 (1.46)</td>
</tr>
<tr>
<td>Stirrup</td>
<td>D13 at 100 mm spacing</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative moment(^d) (M_{by}^-) (kN-m)</td>
<td>1,201</td>
<td>1,201</td>
<td>1,180</td>
<td>1,431</td>
</tr>
<tr>
<td>Positive moment(^d) (M_{by}^+) (kN-m)</td>
<td>485</td>
<td>485</td>
<td>485</td>
<td>715</td>
</tr>
<tr>
<td>Column ((b_c \times h_c))</td>
<td>CEFT column ((\square - 670 \times 670 \text{ mm}))</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel tube(^e) (mm)</td>
<td>(\square - 450 \times 450 \times 8.4) (3.31%)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete strength(^a) (MPa)</td>
<td>26.5 for encasement / 35.2 for infill</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal re-bars(^f) (%)</td>
<td>12-D25 (1.36)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tie</td>
<td>D13 at 200 mm spacing</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nominal flexural moment(^d) (kN-m)</td>
<td>2,126</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column-to-beam moment ratio</td>
<td>2.52</td>
<td>2.52</td>
<td>2.55</td>
<td>1.98</td>
</tr>
<tr>
<td>Beam-column connection</td>
<td>Small opening</td>
<td>Small opening</td>
<td>Large opening</td>
<td>Large opening</td>
</tr>
<tr>
<td>Concrete strength(^a) (MPa)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Column depth-to-bar diameter ratio(^g)</td>
<td>20.7</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear demand (V_j^h) (kN)</td>
<td>2,753</td>
<td>2,753</td>
<td>2,762</td>
<td>3,712</td>
</tr>
<tr>
<td>Shear capacity (V_{nc}^i) (kN)</td>
<td>3,752</td>
<td>2,814</td>
<td>3,752</td>
<td>3,752</td>
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<tr>
<td>Shear capacity (V_{ns}^j) (kN)</td>
<td>620</td>
<td>1,860</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>(V_{nc}/V_j)</td>
<td>1.36</td>
<td>1.02</td>
<td>1.36</td>
<td>1.01</td>
</tr>
<tr>
<td>((V_{nc} + V_{ns})/V_j)</td>
<td>1.59</td>
<td>1.70</td>
<td>1.36</td>
<td>1.01</td>
</tr>
</tbody>
</table>

\(^a\) Measured at 28\(^{th}\) day after casting.

\(^b\) \(\rho = A_r/b_b d_1\); \(d_1\) is effective depth for negative moment; D29 re-bars in slab included.

\(^c\) \(\rho = A_r/b_b d_2\); \(d_2\) is effective depth for positive moment.

\(^d\) Capacity of cross section based on actual material strengths; slab reinforcement neglected.

\(^e\) \(\rho = A_s/b_c h_c\).

\(^f\) \(\rho = A_r/b_c h_c\).

\(^g\) \((h_c - 2s)/d_b\); \(s\) is seating length of PC shell \((35 \text{ mm})\).

\(^h\) Joint shear demand based on flexural yielding of beams.

\(^i\) Joint shear capacity based on reduced joint depth \((h_c - 2s)\).
(2) Design of specimens

In order to evaluate the seismic performance of the proposed PC beam-to-CEFT column connections, cyclic load tests were performed on cruciform beam-column connections simulating interior beam-column joints. Variables of the test specimens are shown in Table 7-1. The cross-sectional dimensions of the column are 670 mm × 670 mm, and the cross-sectional dimensions of the beam including the slab thickness (160 mm) is 600 mm × 700 mm. Moment capacity ratio of columns to beams based on actual material strength is 1.98 ~ 2.55. Basically, all specimens were designed to fail by flexural yielding of the beams.

The main test variables include the size of the tube openings, the presence of the cross-beams, and flexural capacity of the beam. As mentioned earlier, **JP-A** and **JP-AT** are the specimens with small openings in the tube wall while specimens **JP-B** and **JP-BR** have a larger opening. All test specimens except for **JP-AT** have cross-beams, and that is the difference between **JP-A** and **JP-AT**. Connection details of **JP-B** and **JP-A** are generally the same except for the opening design. When compared with the specimen **JP-B**, the beam flexural capacity of **JP-BR** was enhanced to increase the shear demand at the joint.

The joint shear strength $V_{nc}$, $V_{ns}$ of the concrete and steel can be obtained as follows (ACI 2002, AISC 2016):

\[
V_{nc} = 0.083 \gamma \sqrt{f'_c b_j h_c} \quad (7-1)
\]

\[
V_{ns} = 0.6 F_{yw} t_w h_j \quad (7-2)
\]
where $\gamma = 20$ and 15 for the beam-column connections with and without cross-beams, respectively, $b_j = (b_b+b_c)/2$ indicates the joint effective width, and $h_c$ is the column depth. $F_{yw}$, $t_w$ and $h_j$ are yield strength, effective thickness ($=2t_w$), and effective depth of the panel web, respectively. $V_j$ is the shear demand at the joint while $C_{b2} (= A_{r2} \alpha f_y)$ and $T_{b1} (= A_{r1} \alpha f_y)$ can be calculated as the tensile yield strength of the bottom and top reinforcements of the beam, respectively (ACI 2002). $V_c$ is the shear force acting on the column, which is equal to the predicted strength $V_{cnb}$ assuming the flexural yielding of the beam.

In specimens JP-A, JP-AT, and JP-B, rebars 8-D29 and 4-D29 were used for top and bottom reinforcements, respectively. The required shear forces were $V_j = 2,753$ kN, 2,753 kN, and 2,762 kN, respectively based on the actual bar yield strength. On the other hand, the shear demand of JP-BR with increased reinforcements (10-D29 and 6-D29) was $V_j = 3,712$ kN. The shear strength of concrete was $V_{nc} = 3,752$ kN for JP-A, JP-B, and JP-BR with cross-beams, and $V_{nc} = 2,814$ kN for JP-AT without cross-beams. In the calculation, the column depth $h_c$ was reduced to $(h_c - 2s)$ considering the seated length $s$ (35 mm) of the PC beam shell (Im et al. 2013). As a result, the ratio of the concrete shear strength to the required shear force was $V_{nc}/V_j = 1.01 \sim 1.36$. If the contribution of the steel is simply added, the capacity ratio becomes $(V_{nc}+V_{ns})/V_j = 1.01 \sim 1.70$.

In the test specimens, D29 reinforcing bars were used as flexural bars, and the column depth-bar diameter ratio was 23.1 ($= h_c/d_b = 670/29$). If the seated length of the PC shell is removed from the column depth, the depth-to-diameter ratio is
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calculated as 20.7 \[= (h_c-2s)/d_b = 600/29\] (about 10% reduction). In ACI 318-14 (ACI 2014) and ACI 352R-02 (ACI 2002), to prevent excessive bond-slip during earthquake loading, it is required to design the depth-diameter ratio to be greater than 20 and 20\(f_y/420\) (= 21.1), respectively.

In specimens **JP-A**, **JP-B**, and **JP-BR**, the openings were made on four sides of the steel tube for the cross-beams. Although the cross-beams were fabricated through the slab width of 1,300 mm, D10 bars were used for the top reinforcement considering no out-of-plane moment. On the other hand, the PC part of the cross-beam was manufactured to be a solid rectangular cross-section (600 mm × 550 mm) instead of a U-shaped configuration. Thus, the bottom reinforcements could not be placed. Such construction process would degrade the integrity of concrete and confinement effect of the cross-beams at the joint. In the specimens with cross-beams, the transverse reinforcements inside the joints were omitted for ease of construction.

In the case of **JP-AT** without cross-beams, although the panel web plates were intact, three layers of transverse rebars (D13) were arranged in order to prevent brittle failure of the concrete encasement at the joint. The U-shaped bars were used at the joint.

Fig. 7-4 shows the overall details of the test specimens focusing on the column and beam. Square steel tubes were manufactured by cold-forming 8 mm-thick steel plates, and the outer width was 450 mm (steel area ratio 3.31%). The thickness of the concrete encasement was designed to be 110 mm considering the seated length of the PC shell. Therefore, the cross-sectional dimensions of the CEFT column become 670 mm × 670 mm. 12-D25 (SD400, area ratio 1.36%) bars were used for the longitudinal reinforcement. The bending capacity of the column was
2,126 kN·m, according to the plastic stress distribution method.

The lateral reinforcement (D13) of the columns was placed at a spacing of 200 mm (KCI 2012). The shear resistance of the tube web plates is 1,790 kN, well above the predicted shear demand of 849 kN assuming beam flexural yielding in JP-BR. Seven layers of studs (8-ϕ16) were welded to the upper and lower column tubes, respectively, to ensure full composite action of the steel and concrete encasement. Based on one tube plate, the shear capacity of the stud connections (totally 14 studs) was 1,127 kN, and the demand force was 1,388 kN (KBC 2016, Table 7-2). The vertical spacing of the column ties and studs is basically 200 mm, but the spacing was reduced near the hinges to prevent the premature damage at the loading point due to the stress concentration and to satisfy the number of studs for the composite action. Except for the specimen JP-AT, no transverse reinforcement was applied at the joint. The studs were not used at the joint in all specimens.

As mentioned, for the purpose of increasing the shear demand at the joint, the longitudinal reinforcements in the beam were increased in the specimen JP-BR (the ratio equals 1.76% and 1.46% for top and bottom rebars, respectively). In particular, the top rebar ratio was comparable to the maximum value allowed by the design code (KCI 2012), which was calculated as 1.74% assuming that the bar tensile strain reaches twice the actual yield strain.

U-shaped PC beams were applied to all test specimens. The U-shaped PC beam has the advantages of reducing the PC weight and improving the integrity of concrete at the joint. In the PC shell, the bottom reinforcing bars 3-D29 were used for the splice under a positive moment. The top reinforcing bars 4-D16 were used to prevent cracking during transportation and installation, 2-D16 of which were
embedded in the cast-in-place slab without penetrating the column). Those bars including slab longitudinal reinforcements (D10) were not included in the calculation of beam flexural strength.

The spacing of beam shear reinforcements D13 was designed to be 100 mm to prevent the shear failure. The shear reinforcement consists of a U-shaped stirrup embedded in the PC shell and a cap bar with 90- and 135-degree hooks. The longitudinal and transverse slab reinforcements were arranged at a spacing of 200 mm for the purpose of preventing cracks. At the potential plastic hinge region, however, the spacing of transverse bars was reduced to 100 mm so that the stress of the top reinforcing bar D29 in the slab could be transferred to the column.

Table 7-2. Material test results

<table>
<thead>
<tr>
<th>Tensile coupon</th>
<th>Application</th>
<th>Yield strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Elongation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D29 (SD400)</td>
<td>Beam longitudinal</td>
<td>444</td>
<td>571</td>
<td>-</td>
</tr>
<tr>
<td>D25 (SD400)</td>
<td>Column longitudinal</td>
<td>520</td>
<td>654</td>
<td>-</td>
</tr>
<tr>
<td>D13 (SD400)</td>
<td>Beam stirrup &amp; Column tie</td>
<td>437</td>
<td>614</td>
<td>-</td>
</tr>
<tr>
<td>D10 (SD400)</td>
<td>Slab longitudinal &amp; transverse</td>
<td>454</td>
<td>589</td>
<td>-</td>
</tr>
<tr>
<td>8T (SM490)</td>
<td>Column tube</td>
<td>410</td>
<td>551</td>
<td>33.0</td>
</tr>
</tbody>
</table>
Chapter 7. Connection Testing and Evaluation of CEFT Columns with Concrete Beams

Fig. 7-4. Dimensions and reinforcement details of specimen JP-AT

(3) Fabrication of test specimens

Fig. 7-5 shows the manufacturing process of test specimens. The steel tube was fabricated by cold-forming C-shaped sections and connecting them by single V butt welding (CJP). In all specimens, the weld connection is located on the joint panel side. The electrode CSF-71T (E71T-1) with the yield strength of 517 MPa,
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tensile strength of 574 MPa, elongation of 29%, and CVN toughness of 90 J at 0°C was used.

In the test specimens, the seated length of the PC shell (width 600 mm) on the concrete encasement of the column was designed to be 35 mm considering the reinforcing details and cover thickness of the column. On the other hand, the PC part of cross-beams (width 600 mm) also has a seated length of 35 mm. In this case, the corners (35 mm × 35 mm) of the column (670 mm × 670 mm) are completely isolated. Since those edges are considered to be vulnerable to premature spalling and unlikely to contribute to the joint resistance at ultimate state, the corner concrete was not supplemented. In practice, however, it is strongly recommended to use special materials such as high-strength epoxy with excellent adhesion. Alternatively, it is preferable to avoid such situation by properly designing the dimensions each member.

The test specimens were fabricated through three times of concrete casting following actual process in practice. In particular, it is essential to realize the interface between the PC beam shell and joint concrete. First, the first casting was carried out for the concrete encasement of the lower column and PC beam shells, followed by seating the PC shells on the encasement of the column. The second concrete (or cast-in-place concrete) was poured in the infill part of the lower column, joint region, and the rest of beams up to the slab level. Finally, the third casting was completed for the upper column. The design strength of concrete was 30 MPa, and the test results were analyzed based on the actual cylinder strengths at the age of 28 days. The cyclic testing for the connections was conducted at 54 to 57 days after the first casting, 41 to 44 days after the second casting, and 32 to 35 days after the third casting.
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Fig. 7-5. Construction of interior beam-column joints
(4) Test set-up

Fig. 7-6 shows the loading configuration and the location of LVDT measurement. The end of each member was constrained by a hinge condition. The beam horizontal length between the hinges was 5,760 mm, and the column vertical length was 2,860 mm. The cyclic loading was applied at the upper tip of the column. Four LVDTs were installed at the column and beam ends to measure the lateral displacement at the column tip and the rigid body rotation of the test specimens. The shear distortion of the joint panel was measured only in JP-AT without cross-beams. A number of strain gauges were embedded to measure the local strain of steel and steel tube. The cyclic loading program is presented in Table 7-3 following recommendations for composite beam-column frames (AISC 2016). Story drift ratio was calculated by dividing the column lateral displacement by the column effective length of 2,860 mm.

Fig. 7-6. Test set-up for interior beam-column joints
Table 7-3. Cyclic loading program for interior beam-column joints

<table>
<thead>
<tr>
<th>Number of cycles</th>
<th>Story drift ratio (%)</th>
<th>Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 cycles</td>
<td>±0.375</td>
<td>10.7</td>
</tr>
<tr>
<td>6 cycles</td>
<td>±0.5</td>
<td>14.3</td>
</tr>
<tr>
<td>6 cycles</td>
<td>±0.75</td>
<td>21.5</td>
</tr>
<tr>
<td>4 cycles</td>
<td>±1.0</td>
<td>28.6</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±1.5</td>
<td>42.9</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±2.0</td>
<td>57.2</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±3.0</td>
<td>85.8</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±4.0</td>
<td>114.4</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±5.0</td>
<td>143.0</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±6.0</td>
<td>171.6</td>
</tr>
</tbody>
</table>

7.2.2.2 Exterior Beam-Column Joints

Table 7-4 and Fig. 7-7 show test parameters and specimen details, respectively. In exterior beam-column joints, coupler details were used for the anchorage, splice, and tube penetration of the beam rebars. The anchorage part of the beam longitudinal bars was prefabricated with the steel tube column by inserting into the steel tube and fixing with couplers before concrete casting. The beam longitudinal bars were then connected to the anchorage bars by the couplers. The couplers not only connect the beam longitudinal bars but also transfer the bar force to the steel tube by direct bearing. Further, a part of the bar force is transferred to the core concrete by the bar bond. To supplement the loss of the steel tube due to the drilled holes for the re-bar penetration, 10 mm thick steel plates were fillet-welded to the surface of the steel tube in the joint. In the case of interior beam-column joints, the beam longitudinal bars can be continuous through the column tube, without using couplers. In this case, like ordinary RC beam-column joints, the forces of the beam longitudinal bars can be transferred to the joint by the bond of the deformed bars.
For convenience in fabrication, transportation, and lifting, the PC beam was connected to the CEFT column at the column face. Thus, shear studs of $4 - \phi 19$ (sectional area = $283 \text{ mm}^2$) were welded to the steel tube to transfer the beam shear force in the column surface as well as to protect the coupler connection (or strengthen the interface between the PC beam and joint). The nominal shear capacity of the studs was 453 kN. To alleviate the stress concentration at the head of the shear studs, the upper and lower studs were designed with different lengths: 250 mm and 200 mm.

The test parameter was the flexural re-bar ratio of PC beams (1.1% or 1.5% for the negative moment) while identical CEFT columns were used. In specimen JP-E, six D29 and four D29 bars were used at the top and bottom of the beam cross-section, respectively. In JP-ER, eight D29 and six D29 bars were used. In order to restrain local buckling of the longitudinal re-bars under cyclic loading, D13 hoop bars were placed at 100 mm spacing in the region of twice the beam depth (i.e., in the potential plastic hinge zone), which satisfied the special moment frame requirements in ACI 318-11 (ACI 2011). The thickness of the top cover concrete was 35 mm.

The cross-section of the tested columns ($670 \times 670 \text{ mm}$) was a two-thirds scale model of the prototype columns ($1,000 \times 1,000 \text{ mm}$). In the CEFT column, a rectangular built-up steel tube with 8 mm thick steel plates was used. The thickness of the concrete encasement was 110 mm. The ratio of the hollow area to gross section area of the CEFT column was 0.45. In the concrete encasement, four D25 longitudinal bars were placed at the corners of the section, and D13 bars were used for the ties at the vertical spacing of 180 mm. To enhance the bond between the steel tube and concrete encasement, shear studs (\( \phi 13 \)) were welded to the tube plates. The
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The proposed CEFT column was expected to be susceptible to early cracking and spalling of the thin concrete encasement. Thus, to prevent such failure, eight D16 longitudinal bars were placed and tack-welded to the shear studs. The thickness of the cover concrete for the ties and studs was 30 mm. To restrain spalling of the concrete encasement, ties were closely spaced at the top and bottom of the lower and upper columns. The seated length of the PC shells on the concrete encasement of the lower column was 40 mm (Fig. 7-9).

The material properties of the concrete, steel, and re-bars are also presented in Table 7-4. All the values indicate the average of the results obtained from three compression or tension tests. The maximum size of coarse aggregates was limited to 19 mm, considering the thickness of the concrete encasement. The compressive strength of the concrete cylinders was measured on the day of testing. The compressive strengths of concrete were 43.5 MPa and 39.2 MPa in the 1st casting (lower column and U-shell beam) and 2nd casting (joint, upper column, and core of the beam), respectively. The yield strengths of steel plates were 430 MPa for 8 mm thick tubes. The yield strengths of re-bars were 479 MPa for D13 bars and 543 MPa for D29 bars.

The moment capacities in Table 7-4 were calculated using the strain compatibility method for the PC beams and CEFT columns. In the exterior beam-column connections, the moment capacity ratios (2.47 ~ 3.12) of the columns to the beams were relatively high. Thus, the load-carrying capacity of the specimens was expected to be determined by the moment capacity of the beam if the early failure of the connections (such as excessive bar bond-slip) did not occur.

As shown in Fig. 7-8, cyclic vertical loading was applied at the beam end
using an actuator. The length of the beam between the column center and the vertical loading point was 3,515 mm while the net beam length was 3,180 mm. The length of the column between the top and bottom hinges was 3,500 mm. The net column height excluding the rigid ends was 2,000 mm. The loading was first applied in the downward direction (negative loading) which caused the negative moment in the beam. In order to restrain the out-of-plane distortion under cyclic loading, lateral supports were provided. LVDTs were used to measure the displacement of the loading point, rigid-body motion, and deformation of the joint. Uniaxial strain gauges were used to measure the strains of the steel plates and re-bars.

Table 7-5 presents the displacement-control loading program for the test, which is specified in the Korean Building Code 2009 (AIK 2009) for steel structures. The loading program is the same as the seismic qualification loading sequence in section K2.4 b of ANSI/AISC 341-10 (AISC 2010).
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Table 7-4. Properties of exterior beam-column joint specimens with square tube

<table>
<thead>
<tr>
<th>Property</th>
<th>JP-E</th>
<th>JP-ER</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam</td>
<td>PC beams (U-type shell)</td>
<td></td>
</tr>
<tr>
<td>Nominal dimensions (mm)</td>
<td>500 × 700</td>
<td></td>
</tr>
<tr>
<td>Concrete strength (MPa)</td>
<td>43.5 (1st casting), 39.2 (2nd casting)</td>
<td></td>
</tr>
<tr>
<td>Top re-bars</td>
<td>6-D29</td>
<td>8-D29</td>
</tr>
<tr>
<td>Bottom re-bars</td>
<td>4-D29</td>
<td>6-D29</td>
</tr>
<tr>
<td>Yield/tensile strength of re-bars (MPa)</td>
<td>543/676</td>
<td></td>
</tr>
<tr>
<td>Positive flexural capacity(^a) (kN·m)</td>
<td>591</td>
<td>843</td>
</tr>
<tr>
<td>Negative flexural capacity(^a) (kN·m)</td>
<td>1,179</td>
<td>1,491</td>
</tr>
<tr>
<td>Column</td>
<td>CEFT column (□ − 670 × 670 mm)</td>
<td></td>
</tr>
<tr>
<td>Steel tube (mm)</td>
<td>□ − 450 × 450 (8 mm thickness)</td>
<td></td>
</tr>
<tr>
<td>Concrete strength (MPa)</td>
<td>43.5 (1st casting), 39.2 (2nd casting)</td>
<td></td>
</tr>
<tr>
<td>Yield/tensile strength of steel tube (MPa)</td>
<td>430/538</td>
<td></td>
</tr>
<tr>
<td>Longitudinal re-bars</td>
<td>4-D25, 8-D16</td>
<td></td>
</tr>
<tr>
<td>Flexural capacity(^a) (kN·m)</td>
<td>1,841</td>
<td></td>
</tr>
<tr>
<td>Column to beam moment ratio(^b)</td>
<td>3.12(^b)</td>
<td>2.47(^b)</td>
</tr>
</tbody>
</table>

\(^a\) Calculated using actual material strengths.

\(^b\) Negative plastic moment of the beam was considered.

Table 7-5. Cyclic loading program for exterior beam-column joints

<table>
<thead>
<tr>
<th>Number of cycles</th>
<th>Drift ratio (%)</th>
<th>Displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 cycles</td>
<td>±0.375</td>
<td>13.2</td>
</tr>
<tr>
<td>6 cycles</td>
<td>±0.5</td>
<td>17.6</td>
</tr>
<tr>
<td>6 cycles</td>
<td>±0.75</td>
<td>26.4</td>
</tr>
<tr>
<td>4 cycles</td>
<td>±1.0</td>
<td>35.2</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±1.5</td>
<td>52.7</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±2.0</td>
<td>70.3</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±3.0</td>
<td>105.5</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±4.0</td>
<td>140.6</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±5.0</td>
<td>175.8</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±6.0</td>
<td>210.9</td>
</tr>
<tr>
<td>2 cycles</td>
<td>±7.0</td>
<td>246.1</td>
</tr>
</tbody>
</table>
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Fig. 7-7. Dimensions and details of exterior joint specimens (dimensions in mm)

Fig. 7-8. Test set-up for cyclic loading on exterior beam-column joints (dimensions in mm)
Fig. 7-9. Specimen details of exterior beam-column joints
7.3 Load-Displacement Relations and Damage Patterns

7.3.1 Force Equilibrium in Joint Specimens

(1) Prediction based on beam flexural failure, $V_{cnb}$

In Fig. 7-10, the column shear force $V_c$ ($= V_{c1} = V_{c2}$ assumed) and beam flexural moments $M_{b1}$ (negative) and $M_{b2}$ (positive) are presented as follows:

$$ V_c = \frac{L_b}{2} \left( V_{b1} + V_{b2} \right) $$

(7-4)

$$ M_{b1} = V_{b1} \frac{L_b - h_c}{2} $$

$$ M_{b2} = V_{b2} \frac{L_b - h_c}{2} $$

(7-5)

where $V_{b1}$ and $V_{b2}$ are the beam shear forces that yield negative and positive moment at the interface, respectively; $L_c$ (= 2,860 mm) is the vertical distance between the column inflection points; $L_b$ (= 5,760 mm) is the horizontal distance between the beam inflection points; and $h_c$ (= 670 mm) is the column depth. Assuming the beam flexural yielding ($M_{b1} = M_{by1}$, $M_{b2} = M_{by2}$, $V_{b1} = V_{by1}$, and $V_{b2} = V_{by2}$), the corresponding column lateral load $V_{cnb}$ is obtained as follows:

$$ V_{cnb} = \left( M_{by1} + M_{by2} \right) \frac{L_b}{(L_b - h_c)L_c} $$

(7-6)

where $M_{by1}$ and $M_{by2}$ indicate the yield capacity of the beam under negative and
positive moment, respectively.

\[ V_j = C_{b2} + T_{b1} - V_c = \frac{M_{b2}}{j_2 d_2} + \frac{M_{b1}}{j_1 d_1} - V_c \]  \hspace{1cm} (7-7)

\[ V_{b1} = V_c \frac{L_c}{L_b} \frac{2M_{by1}}{M_{by1} + M_{by2}} \]

\[ V_{b2} = V_c \frac{L_c}{L_b} \frac{2M_{by2}}{M_{by1} + M_{by2}} \]  \hspace{1cm} (7-8)

where \( C_{b2} \) is the compressive force that composes the positive moment; \( T_{b1} \) is the...

Fig. 7-10. Global equilibrium in interior beam-column joint
The tensile force that composes the negative moment; and \(j_2d_2\) and \(j_1d_1\) are the internal moment arms in the positive and negative moment, respectively. In Eq. (7-8), it is assumed that the ratio of beam shear forces \(V_{b1}\) to \(V_{b2}\) is proportional to the ratio of yield capacity \(M_{by1}\) to \(M_{by2}\). From Eqs. (7-5), (7-7), and (7-8), the joint shear force \(V_j\) is expressed as follows:

\[
V_j = V_c \left\{ \frac{L_b(L_b-h)}{L_b(M_{pl}+M_{p2})} \left( \frac{M_{p2}}{j_2d_2} + \frac{M_{pl}}{j_1d_1} \right) - 1 \right\} 
\]

(7-9)

where \(A_{r2}f_y\) and \(A_{r1}f_y\) are the tensile yield force of bottom and top reinforcing bars, respectively; and \(V_{cnb}\) is defined by Eq. (7-6). In Eq. (7-9), if the joint shear force \(V_j\) is set as the nominal joint shear capacity \(V_{nc}\) (ACI 2002), the predicted column lateral load \(V_{cnj}\) corresponding to the joint failure is obtained as follows.

\[
V_{cnj} = \frac{V_{nc}}{A_{r1}f_y + A_{r2}f_y - V_{cnb}} - 1
\]

(7-10)

Further, \(V_{cnj}\) divided by \(V_{cnb}\) is equivalent to the ratio of joint shear capacity to demand \(V_{nc}/V_j\) (Table 7-1).

\[
\frac{V_{cnj}}{V_{cnb}} = \frac{V_{nc}}{A_{r1}f_y + A_{r2}f_y - V_{cnb}} = \frac{V_{nc}}{V_j}
\]

(7-11)
The predicted beam shear for exterior beam-column joints based on the beam flexural failure can be calculated as $V_{by} = M_{by} / L_{b0}$, where $M_{by}$ = the moment capacity of the beam and $L_{b0} = 3,180$ mm. In the case of exterior joint specimens, because the shear demand only comes from either top or bottom rebars, it is expected that the joint panel possesses sufficient overstrength against the demand force. Instead, local failure modes associated with coupler anchorage and bond-slip were payed attention.
7.3.2 Interior Beam-Column Joints

Fig. 7-12 shows the lateral load-drift ratio relationships of test specimens. The key experimental results are summarized in Table 4. The test specimens exhibited the strengths 115% ~ 127% of the predicted load and showed stable deformation capacity up to drift ratio of 4% or more. The predictions were calculated assuming beam flexural yielding at the column surface. However, the pinching phenomenon of the hysteresis curve was remarkable.

Fig. 7-13 shows the envelop curves for positive and negative loading. The envelop curve consists of the maximum displacement points in the first load cycle of each drift ratio. The maximum strengths of all specimens except for JP-BR were almost similar. However, while the maximum strengths of JP-A and JP-AT were attained at 2% drift ratio, the maximum strengths of JP-B and JP-BR were exhibited at 3% drift ratio. Also, the stiffness of JP-B was smaller than that of JP-A and JP-AT. These differences may be attributed to the size of opening in the tube wall.

Fig. 7-14 demonstrates the strength degradation caused by cyclic loading. The strength ratio in the vertical axis was defined as the load at the peak displacement in the first cycle divided by the one in the second cycle at the same drift level. It can be seen that the strength deterioration becomes noticeable at 1.5% drift ratio when the beam flexural strength was reached. The strength ratio in positive loading dropped under 90% at 3% drift ratio while in negative loading, the strength reduction became serious at 4% drift ratio. The main causes of such degradation are bond-slip of the beam flexural rebars and the compressive failure of the PC beam shell under negative moment.
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Fig. 7-12. Lateral load-drift ratio relationships of test specimens

Fig. 7-13. Envelope curves of lateral load-drift ratio relationships
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Fig. 7-14. Cyclic strength degradation of test specimens

Table 7-6. Summary of test results for interior beam-column joints

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Test result</th>
<th>Strength, kN</th>
<th>Stiffness, kN/mm</th>
<th>Energy dissipation, kN·mm</th>
<th>Elasto-plastic energy $E_{d0}$</th>
<th>Actual energy $E_d \beta = E_d / E_{d0}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_{c_{test}}$</td>
<td>$V_{c_{cab}}$</td>
<td>$V_{c_{cab}}$</td>
<td>$k_i$</td>
<td>$k_{3.5}$</td>
<td>$k_{3.5}/k_i$</td>
</tr>
<tr>
<td>JP-A</td>
<td>(+) 809</td>
<td>667</td>
<td>1.21</td>
<td>32.3</td>
<td>1.61</td>
<td>0.050</td>
</tr>
<tr>
<td></td>
<td>(−) −801</td>
<td>−667</td>
<td>1.20</td>
<td>34.2</td>
<td>0.86</td>
<td>0.025</td>
</tr>
<tr>
<td>JP-AT</td>
<td>(+) 832</td>
<td>667</td>
<td>1.25</td>
<td>36.6</td>
<td>1.05</td>
<td>0.029</td>
</tr>
<tr>
<td></td>
<td>(−) −794</td>
<td>−667</td>
<td>1.19</td>
<td>29.6</td>
<td>0.71</td>
<td>0.024</td>
</tr>
<tr>
<td>JP-B</td>
<td>(+) 837</td>
<td>659</td>
<td>1.27</td>
<td>29.3</td>
<td>2.17</td>
<td>0.074</td>
</tr>
<tr>
<td></td>
<td>(−) −802</td>
<td>−659</td>
<td>1.22</td>
<td>31.7</td>
<td>1.28</td>
<td>0.040</td>
</tr>
<tr>
<td>JP-BR</td>
<td>(+) 974</td>
<td>849</td>
<td>1.15</td>
<td>35.8</td>
<td>4.49</td>
<td>0.126</td>
</tr>
<tr>
<td></td>
<td>(−) −951</td>
<td>−849</td>
<td>1.12</td>
<td>32.1</td>
<td>2.02</td>
<td>0.063</td>
</tr>
</tbody>
</table>

Fig. 7-15 shows the failure modes of test specimen after the end of 4% drift ratio. All specimens reached the maximum strength by crushing failure of the PC beam shells under negative moment after the flexural reinforcements of the beam yielded. In this process, however, the slip of the PC shells was accelerated due to the bond-slip failure of beam flexural rebars at the joint.

Fig. 7-16 shows the edge cracking in the slab and the slip of the PC beam.
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The edge cracking in the slab was the most prominent part among the cracks during the negative moment. The cracking started to be noticeable at 0.75% drift ratio (maximum crack width of 0.65mm in JP-B). At 1.0% drift ratio, the maximum crack width was measured as 0.7 mm, 0.4 mm and 0.9 mm in JP-A, JP-AT, and JP-B, respectively. At 1.5% drift ratio, the maximum crack width exceeded 1 mm (1.5 mm, 1.2 mm, 1.9 mm, and 1.5 mm in JP-A, JP-AT, JP-B, and JP-BR, respectively). The local failure was generally located inside the beam-column interface. Under the beam positive moment, a substantial portion of the displacement was generated by the slip of the PC shell. In the case of specimen JP-B, the slip was observed as about 4 mm, 6 mm, and 10 mm at 0.75%, 1.0%, and 1.5% drift ratio, respectively.

Fig. 7-17 shows concrete spalling at the PC shell and column corners. The compressive failure the PC shell occurred at 2% to 3% drift ratio, when the specimen reached the peak load. On the other hand, the crushing of slab concrete was limited until the end of testing. In the case of JP-A, at 4% drift ratio, concrete spalling initiated at four corners of the lower column, and column longitudinal reinforcements were exposed at 5% drift ratio. This is because, due to the absence cross-sectional area (35 mm × 35 mm) at four corners of the joint and cover spalling of the PC shell (the seated part on the column), the concrete compressive stress of the column subjected to flexure was concentrated at the contact surface with cross-beams (i.e., sliding failure). In JP-AT without area loss and cross-beams, such failure mode did not occur. JP-B and JP-BR exhibited the similar failure pattern but not as obvious as JP-A. This was inferred from the fact that, in the case of JP-B and JP-BR, the concrete compressive stress was alleviated due to the effect of strengthening plates located at tube corners.

Fig. 7-18 shows the failure modes of the joint panel zone. In the specimens
with cross-beams, cracking was observed only in the slab. As the drift ratio increased, however, the lower part of the cross-beam was clearly separated from the joint. This indicates that the joint concrete and cross-beams were not unified and the confinement effect toward the joint concrete was also limited. In manufacturing the test specimens, the PC part of cross-beams was cast with the solid rectangular section, and accordingly, bottom flexural reinforcements were not arranged in the cross-beams. Thus, if the cross-beams are properly fabricated, the confinement effect to the joint panel would improve the overall structural performance.

In the case of JP-AT without cross-beams, diagonal cracks were observed in the panel zone at early loading. The maximum crack width was 0.4 mm at 0.5% drift ratio and gradually increased to 0.5 mm, 0.6 mm, 0.8 mm, and 1.0 mm at 0.75%, 1.0%, 1.5%, and 2.0% drift ratio, respectively. This indicates that even when the panel web plates are intact, the concrete encasement contributes significantly to the shear resistance of the joint, demonstrating the necessity of transverse reinforcements in the joint to prevent premature failure of the concrete encasement.

Fig. 7-19 shows the flexural and diagonal cracks in the slab under negative and positive loading, respectively. It is considered that the diagonal cracking was caused by the strut-tie mechanism for stress transfer of slab longitudinal rebar D29. The diagonal cracks did not occur in JP-B without the slab reinforcement.
Fig. 7-15. Failure modes of test specimens at the end of 4% drift ratio

Fig. 7-16. Slab edge cracking and PC beam slip at 1.5% drift ratio
Fig. 7-17. Concrete spalling in specimen JP-A

Fig. 7-18. Failure modes in joint panel

Fig. 7-19. Cracking in slab
7.3.3 Exterior Beam-Column Joints

Fig. 7-20 shows the vertical load-displacement relationship of the specimens at the loading point. In the figure, the drift ratio was calculated by dividing the vertical displacement by the effective beam length 3,515 mm (from the column center to the loading point). Generally, the specimens showed ductile behavior due to the development of a full plastic hinge zone at beam end. Pinching is also observed in the cyclic response, which indicates that unfavorable bar bond-slip occurred in the beam-column joint.

In the exterior beam-column joints, the negative loading of the beam reached the peak strength at 3% drift ratio and reached the maximum deformation (corresponding to 80% of the peak strength) at 6% drift ratio. On the other hand, the positive loading reached the peak load at 4% and 3% drift ratio, respectively, and reached the maximum post-peak deformation at 6% drift ratio when local buckling occurred in the top bars of the beam. The test peak strengths agreed with the predictions assuming the beam flexural failure.

Table 7-7 presents the summary of the test results. The yield displacement $\Delta_y$ was defined using the secant stiffness connecting the origin and 75% of the peak load (Park 1988). Maximum displacement $\Delta_u$ was defined as the post-peak displacement corresponding to 80% of the peak strength (Park 1988). The maximum drift ratio ranged 5.5% ~ 6.2%, and the ductility ($\mu = \Delta_u/\Delta_y$) ranged $\mu = 3.60 ~ 5.72$.

Fig. 7-21 shows the strength degradation due to cyclic loading at each drift ratio. The strength ratio was defined as the ratio of the 2nd-load cycle strength to the 1st-load cycle strength at each drift ratio. Generally, as the drift ratio increased, the
strength ratio decreased. In the exterior joints, the strength under the negative loading gradually decreased as the drift ratio increased, due to the bond-slip of re-bars and spalling of the cover concrete at the bottom of the beam [Fig. 7-21(a)]. On the other hand, in the positive loading direction, the strength degradation was not significant [Fig. 7-21(b)].

![Graphs showing load-displacement relationships and strength degradation ratios](image)

**Fig. 7-20.** Vertical load-displacement relationships of specimens

**Fig. 7-21.** Cyclic strength degradation at 2nd load cycle
Table 7-7. Summary of test results for exterior beam-column joints

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load-carrying capacity</th>
<th>Deformation capacity</th>
<th>Yield stiffness $k_y = P_y / \Delta_y$, kN/mm</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Predicted strength $V_{br}$, kN</td>
<td>Maximum strength $V_{test}$, kN</td>
<td>Yield displacement $\Delta_y$, mm (%)</td>
<td>Maximum displacement $\Delta_u$, mm (%)</td>
</tr>
<tr>
<td>JP-E (+)</td>
<td>186</td>
<td>242 (4.0)</td>
<td>38.3 (1.1)</td>
<td>219 (6.2)</td>
</tr>
<tr>
<td>JP-E (−)</td>
<td>-370</td>
<td>-409 (3.0)</td>
<td>-46.2 (1.3)</td>
<td>210 (6.0)</td>
</tr>
<tr>
<td>JP-ER (+)</td>
<td>256</td>
<td>340 (3.1)</td>
<td>54.1 (1.5)</td>
<td>195 (5.5)</td>
</tr>
<tr>
<td>JP-ER (−)</td>
<td>-469</td>
<td>-528 (3.0)</td>
<td>-54.5 (1.6)</td>
<td>213 (6.1)</td>
</tr>
</tbody>
</table>


<table>
<thead>
<tr>
<th>Specimen</th>
<th>Stiffness, kN/mm</th>
<th>Energy dissipation, kN-mm</th>
<th>Elasto-plastic energy $E_{d0}$</th>
<th>Actual energy $E_d$</th>
<th>$\beta = E_d / E_{d0}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$k_i$</td>
<td>$0.05k_i$</td>
<td>$k_{3.5}$ at 4.0%</td>
<td>$k_{3.5} / 0.05k_i$</td>
<td></td>
</tr>
<tr>
<td>JP-E (+)</td>
<td>8.62</td>
<td>0.431</td>
<td>1.53</td>
<td>3.55</td>
<td>133,059</td>
</tr>
<tr>
<td>JP-E (−)</td>
<td>12.3</td>
<td>0.625</td>
<td>1.55</td>
<td>2.48</td>
<td></td>
</tr>
<tr>
<td>JP-ER (+)</td>
<td>9.28</td>
<td>0.464</td>
<td>2.22</td>
<td>4.78</td>
<td>163,544</td>
</tr>
<tr>
<td>JP-ER (−)</td>
<td>13.8</td>
<td>0.690</td>
<td>2.10</td>
<td>3.04</td>
<td></td>
</tr>
</tbody>
</table>
As shown in Fig. 7-22, the top of the beam was severely damaged due to local buckling of the top longitudinal bars. Furthermore, flexural cracks developed more at the top of the beam than at the bottom of the beam. At the bottom of the beam, the beam-column interface was locally damaged.

At the beginning of loading (0.375% drift ratio), a few flexural cracks of less than 0.1 mm developed within the distance of 1,000 mm from the column face. The number of cracks gradually increased, and the cracks were propagated toward the joint as the displacement increased. At 0.75% drift ratio, a gap was observed at the joint interface where the PC beam shell was seated on the concrete encasement of the lower column. The development of the gap is attributed to the bond-slip of the beam bottom bars. At 1.0% drift ratio, the top and bottom bars first yielded. The flexural cracks in the beam grew to 0.7 mm. The maximum crack width in the beam-column joint was 0.3 mm. After yielding of the flexural bars, cracks were concentrated at the beam end. As the shear force in the joint was resisted mainly by the inner CFT, shear cracks in the joint face did not further propagate. At 3% drift ratio, spalling of concrete occurred at the bottom of the PC beam shell. The spalling of the bottom cover concrete was the main cause of the degradation of the negative load-carrying capacity. At 3% drift ratio, the negative load-carrying capacity reached the maximum load, and the maximum crack width exceeded 1.0 mm.

Ultimately at 6% drift ratio, the load-carrying capacity significantly decreased due to the local buckling of the top flexural bars. Since the bottom bars had a relatively thick concrete cover of the PC shell, local buckling of them did not occur. Failure of the couplers did not occur until the end of the test.
In the exterior joint face at the opposite side of the beam connection, spalling of the cover concrete occurred at the location of the bottom flexural bars, and the bottom couplers were exposed [5% drift ratio, Fig. 7-22(b)]. This result indicates that the couplers did not effectively prevent bond-slip of the flexural bars subjected to compression. Under repeated cyclic loading, residual tensile strain increased, particularly in the bottom bars which had less area than the top bars. Due to the force equilibrium in the cross-section, the residual strain of the bottom bars was greater than that of the top bars having the greater area. Thus, as the bar length increased in the joint, the couplers were not completely in contact with the steel tube, and only the bar bond provided resistance against the compressive force. This result indicates that when high ductility is required under cyclic loading, the compression development length of the bottom bar needs to be satisfied in the joint.

Fig. 7-22. Failure modes of exterior PC beam-column joints
7.4 Strains of Rebars and Steel Plates

7.4.1 Interior Beam-Column Joints

7.4.1.1 Beam Longitudinal Bars

Fig. 7-23(a) shows measured strains of the beam bottom rebars. The gauges 2 and 4 were installed at 200 mm outside the column face and 110 mm inside the column face, respectively. From the early loading, larger strain occurred inside the joint (gauge 4) and the location yielded at 3% drift ratio. The gauge 2 also exceeded the yield strain significantly at the 4% drift ratio. Fig. 7-23(b) shows the strain of the top flexural reinforcements. As in the case of bottom rebars, the yielding of the top rebars took place first inside the joint (gauge 4). However, a plastic deformation was not significant outside the joint (gauge 2).

(a) Bottom rebars
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(b) Top rebars

Fig. 7-23. Strains of beam longitudinal reinforcements

(a) Positive loading

(b) Negative loading

Fig. 7-24. Strain distribution in bottom longitudinal reinforcements
Fig. 7-24 shows the strain distribution in the bottom rebar at different drift ratios. The horizontal axis represents the horizontal distance from the center of the column to the measurement point, and the vertical axis represents the strain at the maximum displacement of the first cycle. In the case of JP-A, the largest tensile strain was observed at the column surface (gauge 3). The tendency was similar in JP-AT. Fig. 7-25 shows the strain distribution of the top rebar. It can be seen that the largest plastic deformation occurred inside the joint (gauge 4).

Fig. 7-25. Strain distribution in top longitudinal reinforcement

On the other hand, yielding of the top reinforcing bars (peak elongation smaller than 2%, see Fig. 7-25) was not as significant as that of the bottom
reinforcing bars (peak elongation greater than 2%, see Fig. 7-24). This is because, under the negative moment, the strain demand at tensile reinforcements is smaller than in the case of the positive moment. When specimens JP-B and JP-BR are compared (Fig. 7-25), the strain of the reinforcing bars is smaller in JP-BR (less than 1%) with greater reinforcement ratio.

Fig. 7-26. Strain distribution in slab longitudinal reinforcements

Fig. 7-26 shows the strain distribution in the flexural reinforcements (D29) in the slab. As in the case of top rebars penetrating the column (Fig. 7-25), the tensile strain of the slab rebar in JP-AT was the greatest inside the joint (gauge 4). On the other hand, the strain of the slab rebar in JP-A was considerably smaller than JP-
AT. This implies that the cross-beams contribute to the increase of flexural stiffness at the joint.

Fig. 7-27 shows the strain of slab transverse reinforcements (D10). General, the strain was greater in the positive moment than in negative moment and approached the yield strain. This may be attributed to the diagonal strut formed from slab longitudinal reinforcement (D29) to the column, rather than the confining effect of the compression zone. In the case of JP-B without the slab rebar, the lateral strain was the lowest.

Fig. 7-27. Strains of slab transverse reinforcements
7.4.1.2 Tube Plates

Fig. 7-28 shows the strain tube flanges. The gauges TL1 and TL5 were installed at 200 mm and 50 mm outside and inside (center of strengthening plates) the joint, respectively. Generally, the strain was greater inside the joint, but the strain remained elastic until the end of testing. The reason for a large difference of compressive strains between the locations was supposed to be owing to the bearing effect by beam flexure. The strains in JP-B were greater than those in JP-A even though their peak loads were similar. This was because the strain in JP-B was measured closer to the tube corner where the stress tends to be concentrated.

Fig. 7-29 shows the strain of panel web plates. The vertical axis represents the von Mises stress calculated from measured strains of the rosette gauge. In JP-AT without web openings, the web plate remained elastic. On the other hand, in JP-A and JP-B with openings, the web plates yielded at 3% drift ratio.
Fig. 7-29. Mises stress of panel web plates

7.4.2 Exterior Beam-Column Joints

Fig. 7-30 shows strains of the re-bars and steel plates of JP-ER. The strains of JP-E were similar to those of JP-ER. In JP-ER [Fig. 7-30(a)], both the top and bottom longitudinal bars exhibited large inelastic strains. In the figure, the strain of the top bar decreased when it was subjected to compression under reversed loading. On the other hand, the strain of the bottom bar did not significantly decrease under reversed loading. This result indicates that significant bond-slip occurred in the bottom bars inside the joint. The maximum tensile strain of the top flexural bars, which failed in the local buckling, was about 4.7%. Fig. 7-30(b) exhibits the strains of the hoops. The strains gradually increased as the cyclic loading proceeded. The
strain significantly increased when diagonal cracking occurred in the joint, and the peak value was close to the yield strain. Fig. 7-30(c) shows the strain histories of the shear studs installed at the column surface. Ultimately, the strains were close to the yield strain.

Fig. 7-30. Strains measured in exterior joint specimens
7.5 Structural Performance of Beam-Column Joints

7.5.1 Seismic Design Code Conformance

For evaluation of the seismic performance, ACI 374.1-05 suggests following requirements with respect to the strength, stiffness and energy dissipation at the third load cycle of 3.5% drift ratio (Fig. 7-31).

(a) The load-carrying capacity exceeds 75% of the maximum strength.

(b) The secant stiffness $k_{3.5}$ between $-0.35\%$ and $+0.35\%$ drift ratios exceeds $5\%$ of initial stiffness $k_i$.

(c) Energy dissipation $E_d$ exceeds $12.5\%$ of the idealized energy dissipation $E_{d0}$ assuming elastoplastic behavior.

In this study, load steps were planned according to the standards for composite structures. Thus, the seismic performance was evaluated at a second load cycle of $4\%$ drift ratio.

Table 7-6 shows the evaluation for seismic performance of the interior joint specimens. All specimens satisfied the ACI requirements with respect to the strength and energy dissipation. In JP-AT without cross-beams, the energy dissipation ratio was the poorest with 0.166 even though the panel web plates were intact and the transverse reinforcements were used in the joint. The degradation implies that bond-slip failure of the beam flexural rebars was the most serious in JP-AT. On the other hand, in JP-BR with increased beam flexural strength, the ratio was the greatest with 0.245.
On the other hand, in terms of the stiffness, only **JP-BR** satisfied the requirement. In the end, **JP-BR** with the increased beam capacity exhibited the best seismic performance among the test specimens. The reason can be explained that the inelastic deformation of the plastic hinge was relatively limited in **JP-BR** and the bond-slip failure of beam flexural reinforcements in the joint was not serious compared to other specimens. Instead of less flexural deformation and slip of the beam, flexural deformation of the column and shear distortion of the joint would have increased. However, if the reinforcement ratio is high as in **JP-BR**, crushing of the beam compression zone may take place early, and the load-carrying capacity may decrease rapidly.

In the current specimens, substantial bond slip occurred in the beam-column joint even though the column-depth-to-bar-diameter ratio was greater than the requirements of ACI 352R-02 (ACI 2002) and ACI 318-14 (ACI 2014). There are mainly two causes. First, the joint depth was decreased by 10% due to seating length of the PC beam shells (Fig. 7-33). The reduced joint depth could affect the panel shear performance and depth-to-bar diameter ratio (23.1 reduced to 20.7). Second, the reinforcing bars (4-D16 and 3-D29) in the PC beam shell increased the flexural capacity of the beam. Because of the discontinuity, yielding of the beam flexural bars should take place at the interface between the PC beam shell and the joint (35 mm inward from the column surface) or the critical section. The plastic deformation then propagated toward the joint rather than toward the beam with greater strength. Eventually, the yielding of the beam flexural bars inside the joint caused the bond slip.

Table 7-8 presents the evaluation results for the exterior joints. Unlike the interior joint specimens, both specimens sufficiently satisfied the acceptance criteria
for both stiffness and energy dissipation. The results indicate that the couplers provided adequate tension anchorage for the beam longitudinal bars, and the headed studs protected the interface between the PC shell and the joint, pushing the beam plastic hinge away from the connection. Consequently, the exterior joint specimens exhibited satisfactory seismic performance.

The seismic performance of interior beam-column joints proposed in this study deteriorated due to the bond-slip failure of beam flexural reinforcements. To improve the seismic capacity, it is required to increase the depth-to-diameter ratio \((h_c/d_b)\) by increasing the column depth or using smaller-diameter rebars. Alternatively, additional details, such as couplers for anchorage of rebars and headed studs for strengthening the interface between the PC beam shell and joint, are necessary.

![Fig. 7-31. Acceptance criteria for stiffness and energy dissipation in ACI 374.1-05](image)

**Fig. 7-31. Acceptance criteria for stiffness and energy dissipation in ACI 374.1-05**

\[
\begin{align*}
\text{k}_{3.5} & \geq 0.05 k_i \text{ (or } k_i') \text{ at 3.5% drift ratio} \\
E_d & \geq 0.125 E_{d0}
\end{align*}
\]
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Fig. 7-32. Energy dissipation ratio in 2\textsuperscript{nd} load cycle

Fig. 7-33. Causes of bond-slip in the precast concrete beam-column connection

7.5.2 Panel Shear Distortion

Fig. 7-34 shows the strain of the transverse reinforcing bars at the joint of JP-AT. The transverse reinforcement at the center yielded at 2\% drift ratio, and the one at the top yielded at 3\% drift ratio. Ultimately, the strain was the greatest at the
top, where the damage of panel concrete was the most severe.

Fig. 7-35 shows the shear strain calculated using LVDTs. The shear distortion increases linearly with the drift ratio even after the specimen reached the peak load at 2% drift ratio. The difference between shear strains under the positive and negative loading was marginal. The shear strain of JP-AT exceeding 1% implies that the composite joint panel reached the ultimate state. According to Parra-Montesinos and Wight (2003), the joint panel composed of steel beams and RC columns exhibits the maximum strength at a shear strain of 1.2%, when yielding of the web plate has spread all over the panel zone (yield strain $0.6F_{yw}/G_s = 0.32\%$). However, the potential shear strength ($V_{nc}+V_{ns}$), which is assessed as the sum of contributions of the concrete and web plates, was 1.36 times the actual shear demand \[ V_{cnj}/V_{ctest} = (V_{cnj}/V_{cnb})/(V_{ctest}/V_{cnb}) = (1.70)/(1.25), \text{ see Table 7-1 and Table 7-6}. \] Also, the web plates remained elastic (Fig. 7-29). These results imply a likelihood of early failure of the concrete encasement at the joint. Further study is needed on the shear strength of beam-column joints addressing effects of the column tube panel (Fig. 7-36).

On the other hand, in the exterior joints JP-E and JP-ER, the shear distortion was significantly smaller than in the interior joints. This is because the shear capacity-to-demand ratio is greater in the exterior joint specimens. Meanwhile, the shear strains show greater values in the negative direction because the negative moment capacity of the PC beams was greater than the positive moment capacity (i.e. the joint shear force under the negative loading was greater).
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Fig. 7-34. Strains of joint ties in JP-AT

Fig. 7-35. Panel shear strain in JP-AT

Fig. 7-36. Effect of web plates on panel shear behavior
7.6 Discussion

In this study, potential connection details for PC beam-to-CEFT column joints were studied. For penetration and anchorage of beam flexural rebars at the joint, connections details such as partial wall opening and coupler splice were applied, considering constructability and seismic reinforcement, respectively. To evaluate the seismic performance of the connections, cyclic load tests are conducted. Major findings from this research are summarized as follows:

1) In interior joint specimens with opening details, the peak loads were attained by beam flexural yielding, exhibiting 115% ~ 127% of the predicted beam strength. The load-carrying capacity was maintained up to 4% drift ratio. However, after 2% ~ 3% drift ratio, when the specimens reached the peak loads, the strength degradation due to cyclic loading exceeded 10%.

2) Despite the satisfactory strength and deformation capacity, considerable bond-slip failure of the beam flexural reinforcements occurred at the joint, causing significant pinching in hysteresis curves. With respect to the seismic performance conformance with ACI 374.1-05, all specimens satisfied the requirements for the strength and energy dissipation capacity (energy dissipation ratio 0.166 ~ 0.245). However, the stiffness requirement was only met by JP-BR.

3) The opening details are recommended for the use in ordinary or intermediate moment-resisting frames. In order to apply the connection to special moment frames, it is required to increase the depth-to-diameter ratio \((h_c/d_b)\) by increasing the column depth or reducing the rebar diameter.
4) The shear failure of interior joints did not occur even when the shear demand was 89% ~ 123% of the shear strength of joint concrete. However, further study is needed to assess the shear strength considering the contribution of column web plates.

5) If beam-column joints are confined by cross-beams, transverse reinforcements may be omitted for better workability. However, when cross-beams are not available, the transverse reinforcements are necessary to prevent premature failure of the concrete encasement.

6) Exterior joints with couplers and headed studs exhibited enhanced seismic performance. To protect the coupler connection and strengthen the interface between the PC shell and joint, the use of multiple headed studs is recommended. The welded studs are also effective as shear keys.

7) In the exterior joints, the couplers can provide adequate tension anchorage for the beam longitudinal bars. However, since couplers cannot provide compression anchorage, compression bar development length should be satisfied, particularly when high ductility is required under cyclic loading.

8) To enhance the deformation capacity of the PC beams, local buckling of the top flexural bars, which is likely to occur in exterior joints with the coupler details, should be restrained. In the bottom bars having a relatively thick concrete cover of the PC shell, local buckling does not occur.
Chapter 8. Summary and Conclusions

In this dissertation, various experimental and analytical studies were carried out on the columns and beam-column connections for practical applications of concrete-filled steel tubular (CFT) and concrete-encased-and-filled steel tubular (CEFT) columns especially with high-strength steel. For CFT columns, to extend the material availability in design codes, experimental and analytical studies were performed to estimate the axial-flexural strength of high-strength CFTs incorporating slender section. In the experimental study on CEFT columns, various encasement details are tested under eccentric compression with considerations of tube shape and yield strength. For connectivity with the CEFT columns, steel and precast concrete (PC) beams were considered. In the connection with the steel beams, local tensile and flexural behavior of the connection was investigated. In the study of beam-column joints with the PC beams, connection details considering the construction efficiency and seismic behavior were devised, and the associated performance was evaluated through cyclic loading tests. The major findings and conclusions of each chapter are summarized as follows:

8.1 Axial Testing and Evaluation of CFT Columns with High-Strength Steel

Concentric and eccentric axial loading tests were performed on RCFT columns with high-strength steel slender section. The test parameters included the
yield strength of steel (746 MPa and 301 MPa), the width-to-thickness ratio (slender section and compact section), axial-load eccentricity, and the use of stiffeners. The major findings of this study are summarized as follows:

1) In the specimens with the same width-to-thickness ratio \((b/t = 58)\) and different steel yield strengths \((F_y = 301\) and 746 MPa), the peak loads were both attained after local buckling. In the specimens with the mild steel, although the section was classified as a compact section, the early local buckling occurred, followed immediately by yielding. At this point, the stiffness was degraded. In the slender section specimens with high-strength steel, although early elastic local buckling occurred, the load carrying capacity continued to increase without significant decrease of the stiffness. The structural performance of the test specimens was quite consistent regardless of the axial-load eccentricity probably due to its low value.

2) In spite of the early local buckling, the strengths of the specimens with the mild steel were comparable to the predicted plastic capacity. This is because the influence of the strength contribution of the steel tube was limited due to the low yield strength (i.e. low strength contribution). On the other hand, those of the specimens with the high-strength steel were slightly less than the predictions. Nevertheless, the test results of the high-strength steel slender section sufficiently satisfied the predictions by ANSI/AISC 360.

3) In the compact section specimens strengthened with stiffeners, elastic local buckling was restrained, and the load carrying capacity reached the plastic strength of the composite section, showing relatively ductile behavior. Further, a design method for stiffeners was theoretically developed for high-strength steel CFT.
columns. The proposed method agreed with an existing empirical equation and satisfactorily predicted the required rigidity of stiffeners of the test specimens.

4) The results of FE analysis showed that when the high-strength steel was used, the peak load of the specimens was attained when the contribution of the steel tube reached the peak. On the other hand, when the 300 MPa steel was used, the peak load occurred when the axial strain reached the crushing strain of the concrete. It should also be noted that even the stiffened plates (w/t = 30) hardly exhibited the yield capacity due to the dilation of the crushed concrete.

8.2 Development of Analytical and Design Models for CFT Columns

This section focused on assessing the load-carrying capacities of CFT columns and beam-columns with various material strengths and sectional slenderness. Major findings of this study is summarized as follows:

1) It was confirmed that the current design codes (ANSI/AISC 360-16, Eurocode 4, AIJ recommendations) safely predict the test strengths of RCFT columns and beam-columns with conventional material strengths and width-to-thickness ratios. However, the strength predictions for the specimens with high-strength materials tended to be unconservative.

2) The unconservatism of the high-strength steel comes from improper estimation of the buckling capacity (which is not critical to low-strength steel) as well as the early concrete crushing. The high-strength concrete may be susceptible to premature failure affected by size effect. On the other hand, the test specimens
with normal-strength materials and large sectional slenderness did not show clear strength degradation, reaching the plastic capacity of the composite section. This result is attributed to the beneficial confinement of buckled tube plates.

3) Based on the numerical observations, the effective peak stress of the rectangular tube was proposed addressing the effects of post-buckling behavior and concrete crushing, which is reasonable for the high-strength steel. For construction of $P-M$ interactions, a simplified method assuming equivalent stress blocks was suggested, in which a strength reduction factor is defined by the neutral axis depth.

4) For strength estimation of circular CFT beam-columns, a strength degradation model of confined concrete was proposed addressing the effect of axial-load ratio. The modified model conservatively amended the strength assessment, which is essential for the high-strength steels.

8.3 Axial Testing and Evaluation of CEFT Columns

Axial load tests were performed to evaluate the axial-flexural strength of concrete-encased-and-filled steel tubular (CEFT) columns with thin-walled square and circular tubes. The present study focused on the details of the thin concrete encasement, which may be vulnerable to premature spalling under high axial load. The major results of the present study are summarized as follows:

1) Generally, the axial load-carrying capacity of CEFT columns with the rectangular tube decreased immediately after the peak load, due to spalling of the concrete encasement as well as the small axial-load eccentricity and tube local
buckling. However, the residual moment capacity of the specimens was maintained owing to the effect of inner CFT. The ductility of the composite column was significantly enhanced if the steel fiber was added to the concrete encasement.

2) Compared with the rectangular tube, CEFT specimens with the circular tube showed superior ductility even with the failure of concrete encasement. This is attributed to the increased resistance of the circular steel tube and confined core concrete.

3) Strength and flexural stiffness of the proposed CEFT columns were evaluated by predictions of current design codes. The combined axial-flexural strength of the specimens was the most reasonably predicted by the strain compatibility method (maximum compressive strain of concrete = 0.003 assumed and also verified by measurement in the test). For strength conformance with the method, the tie spacing of \( b_{lc}/4 \) is recommended on the basis of the test results. The overall behavior of CEFT columns was simulated in detail by fiber-based section analysis.

### 8.4 Connection Testing and Evaluation of CEFT Columns with Steel Beams

The flexural behavior of the connection between the steel beam and thin-walled tubular column was studied. For this purpose, a set of monotonic tension tests on flange-tube connections and cyclic load tests on beam-column exterior joints was carried out. The results are summarized as follows:

1) In the planar tension test, semi-rigid connections between the flange plate
and the circular tube were studied. The test results indicated that widened flange and tension bars are effective in enhancing strength and stiffness. The concrete encasement in the CEFT columns contributes only to the initial stiffness, but the effect on the ultimate tensile strength was negligible. At the yield point, maximum crack width was 0.3 mm.

2) The peak loads of the semi-rigid connection were determined with the tube tear out (i.e., the punching shear mechanism) in the vicinity of the weld joint. The tensile strengths were conservatively assessed by a modified yield line model. When tension bars are added, the strength can be increased by the bar yield strength.

3) The out-of-plane force-deformation relationships of the specimens were well-simulated using 3D finite element analysis. The behavior of the specimen T-350R was described by simple superposition of the calculated rebar force and numerical result without the reinforcements. The failure of concrete infill did not occur.

4) As intended based on the numerical model, the local connection failure was prevented in exterior joint specimens J-350R and J-350T. Particularly in J-350T with the thickened tube and the greater connection capacities, the concrete cracking was negligible throughout the test. An increased tube thickness was also beneficial in improving the panel shear resistance.

8.5 Connection Testing and Evaluation of CEFT Columns with Concrete Beams

In this study, potential connection details for PC beam-to-CEFT column
joints were studied. For penetration and anchorage of beam flexural rebars at the joint, connections details such as partial wall opening and coupler splice were devised, considering constructability and seismic reinforcement, respectively. To evaluate the seismic performance of the connections, cyclic load tests are conducted. Major findings from this research are summarized as follows:

1) Interior joint specimens with opening details attained the peak loads by the beam flexural yielding, and the load-carrying capacity was maintained up to 4% drift ratio. Despite the satisfactory strength and deformation capacity, considerable bond-slip failure of the beam flexural reinforcements occurred at the joint, causing significant pinching in hysteresis curves.

2) Among the seismic performance requirements of ACI 374.1-05, those for the strength and energy dissipation capacity (energy dissipation ratio 0.166 ~ 0.245) were satisfied by the interior joints (the stiffness requirement not met). In order to improve the seismic performance, it is required to increase the depth-to-diameter ratio ($h_c/d_b$) by increasing the column depth or reducing the rebar diameter, considering the fact that the effective joint depth is decreased due to seating length of the PC beam shells.

3) The shear failure of interior joints did not occur even when the shear demand was 89% ~ 123% of the concrete shear capacity due to additional contribution of the tube wall. If beam-column joints are confined by transverse beams, tie reinforcements may be omitted for better workability.

4) In the exterior joints, the couplers provided adequate tension anchorage for the beam longitudinal bars. In addition, headed studs protected the interface between the PC shell and the joint, pushing the beam plastic hinge away from the
connection. Consequently, the exterior joint specimens exhibited enhanced seismic performance.
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초 록

충전형 및 피복충전형 합성기둥은 콘크리트와 강관의 상호작용을 통한 구조적인 이점과 철골조 공사 및 충전부 타설의 급속시공으로 인하여, 국내외의 중대형 건설 프로젝트에 다수 적용되어 왔다. 최근에는 다변화하는 현장의 요구사항들을 충족시키기 위하여 고강도 강재 혹은 콘크리트의 적용이 확대되는 추세이다. 본 논문은 특히 고강도 강재를 적용한 충전형 및 피복충전형 합성기둥의 실용화를 목적으로, 기둥 및 보-기둥 접합부에 대한 다양한 실험과 해석연구를 다룬다.

충전형 합성기둥에 고강도강재 적용 시, 높은 항복강도로 인하여 강관이 비조밀 혹은 세장단면으로 설계되는 일이 실무에서 빈번하게 발생한다. 그러나 관련 설계기준이나 연구결과는 아직 미흡한 상황이며, 이로 인해 고강도 강재의 적용이 제한되는 측면이 있다. 충전형 기둥에 관한 편심압축 실험연구에서는, 항복강도 746 MPa의 각형 세장단면과 항복강도 565 MPa의 원형 비조밀단면이 포함된다. 실험결과는 현행 설계기준에 따른 예측강도를 크게 상회하였으며, 소성내력에 거의 근접하였다. 한편, 고강도강재 세장단면의 소성강도를 발현하는 목적으로, 스티프너 보강상세가 제안되었으며, 강성 설계모델이 이론적으로 유도되었다.
최근 합성구조 기둥분야에서는 재료강도 제한의 완화가 설계기준 개정의 주요 안건 중 하나이다. 특히 충전형 합성기둥은 고강도 강재를 적용하기에 가장 이상적인 구조시스템으로서, 기존 완화를 위해서는 관련 설계법 개선이 필수적으로 요구된다. 충전형기둥에 관한 해석연구에서는, 다양한 재료강도와 단면 세장비를 고려한 합성단면의 휘압측강도 평가모델을 개발하는 것을 목표로 한다. 각형 충전형 기둥의 경우, 기존 실험 결과들을 기반으로 설계 변수들의 영향을 분석하고, 강관의 유효 최대응력을 정의하였다. 제안된 모델은 콘크리트 조기압괴의 영향을 고려함으로써 고강도 강재에도 적용이 가능하다. 추가적으로, 실험에 간편하게 활용될 수 있도록 휘압측 강도의 설계식을 제시하였다. 원형 충전형기둥에 관해서는, 축력비에 따른 콘크리트의 구속강도 저감모델이 개발되었다. 제안된 모델은 고강도 강재 적용 시 특히 중요하며, 실험결과를 보수적으로 예측하였다.

충전형 합성기둥의 구조물 저층부는 내력 증가 및 내화성능 개선을 목적으로 피복콘크리트가 추가될 수 있다. 이러한 피복충전형 합성기둥의 구조성능은 피복콘크리트의 건전성에 크게 의존하기 때문에 이에 대한 보강 상세의 역할 또한 중요하다. 피복충전형합성기둥에 관한 실험연구에서는, 다양한 피복부 상세와 강관형상 및 항복강도를 변수로 편심압축실험을 수행하였다. 실험결과, 얇은 피복콘크리트의 조기탈락은 발생하지 않았으며, 고강도 원형강관 적용 시 우수한 변형능력을 나타냈다. 실험체의 하중재하는
력과 힘강성은 현행 설계기준에 의한 예측치와 비교되었다.

충전형 합성기둥과 강재 보 연결 시, 경제성을 위해 가급적 무보강 상세가 권장되며, 이 경우 접합부의 해석모델은 연결부의 국부기동을 포함할 필요가 있다. 특히 피복충전형 합성기둥 적용 시, 고강도 강관의 면외변형은 피복콘크리트의 손상을 야기할 수 있으므로 연결부의 변형을 제어하는 것이 매우 중요하다. 강재 보-피복충전형 합성기둥 접합부에 관한 실험연구에서는, 무보강 상세를 기본으로 하여, 연결부의 면내 단조이장실험 및 외부접합부의 반복험실험을 수행하였다. 시험 결과를 토대로, 연결부의 강도평가법과 면외변형에 따른 피복콘크리트의 파괴모드를 분석하였으며, 연결부의 하중-변형 관계는 유한요소해석을 통해 구현하였다.

피복충전형 합성기둥은 충전형 합성기둥에 비하여 콘크리트보와의 접합이 상대적으로 용이하다. 그러나 강관 단면이 클 경우, 보와철근의 관통과 이음 및 정착에 대한 고려가 필요하다. 콘크리트 보-피복충전형 합성기둥 접합부에 관한 실험연구에서는, 현장 시공성과 구조성능을 고려한 접합부 상세를 고안하고, 보-기둥 외부 및 내부접합부의 반복가력실험을 통해 내진성능을 평가하였다. 접합부 상세로는, 강관 개구부 확장을 통한 보 관통형 상세와 커플러 이음 및 정착을 통한 보 비관통형 상세가 고려되었다. 실험결과를 토대로, 접합부 상세에 따른 강성과 하중재하능력, 에너지소산능력을 평가하였다.
주요어: 충전형 강관; 피복충전형 강관; 고강도 강재; 비조밀단면; 세장단면; 스티프너; 국부좌굴; 콘크리트 횡구속; 휘압축 강도; 합성 보-기둥 접합부; 콘크리트 보

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