Structural Performance of Steel-Concrete Composite Column Subjected to Axial and Flexural Loading

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Abstract

The steel-concrete composite column is a new composite member that can achieve constructability and economy by filling the empty space in the steel H-flange with concrete. Even before this study was conducted, it had been proven in another study on the axial capacity of a steel-concrete composite column that the steel-concrete composite column with a non-compact section has excellent structural strength. Further studies on flexural-compressive capacity are necessary, though, due to the structural characteristics of the column. As such, this study on the flexural strength of a steel-concrete composite column with a non-compact section that is under a constant axial force was conducted for the strong-axis and weak-axis directions, and the results were compared with the evaluations of various design code provisions. Based on the results of the experiment, the axial force-bending moment capacity of the column met the criteria of various countries for both axis directions. Specifically, the AISC-LRFD provisions evaluate the load-carrying capacity of the composite column too conservatively. Therefore, the AIJ and EC4 code provisions are considered desirable for use in evaluating the capacity of the axial force-bending moment of steel-concrete composite columns with non-compact steel section.

Keywords: composite column; beam-column test; non-compact section

1. Introduction

Common types of composite column design include concrete-filled tubes (CFT) and steel-reinforced concrete columns (SRC). However, these composite column concepts have limitations that may restrict their use in practice. For instance, the cross-section dimensions for small CFT’s are limited to available standard steel tube shapes, and larger columns must be custom made. SRC columns require extensive formwork, especially at the beam-column connections (Tremblay et al., 2000).

A number of studies on steel-concrete composite columns have recently been conducted by many researchers (Tremblay et al., 2000; Vincent, 2000; Chicoine et al., 2002, 2003; Oh et al., 2004) to identify a more effective column system that can make up for the weak points of conventional composite columns such as SRC and CFT while still possessing the latter’s strong points.

As illustrated in Fig.1., the steel-concrete composite column is a composite column whose plate (with a slender width-to-thickness ratio) has been welded to form an H-section (a non-compact section), and that has been filled with concrete in each side of its flange.

Fig.1. The Shape of Steel-Concrete Composite Column

This steel-concrete composite column features a non-compact section that has no longitudinal rebar and that has a reduced quantity of steel for economy, different from the partially encased composite column (PEC) that is widely used in Europe. As shown in
Recently, several experimental attempts were made to identify the compressive and bending capacity of the steel-concrete composite column, but most of these studies were conducted on the steel section composed of a compact section or on the smaller section of the test specimen. Also, most of experiments that have been conducted were on the steel-concrete composite column with a shear connection; no experimental study has so far been conducted on the steel-concrete composite column that is under both bending and compressive forces. In a study on the axial capacity of the steel-concrete composite column that was carried out, it was proven that the steel-concrete composite column with a non-compact section has excellent structural capacity (Tremblay et al., 2000; Chicoine et al., 2002; Oh et al., 2004).

Further studies are necessary, however, since the moment connections are widely used in Korea, and accordingly, both axial and bending forces are applied to such column. As such, this study on the bending capacity of the steel-concrete composite column that is under a constant axial force was conducted for the two axial force ratio and the direction of the load (strong-axis and weak-axis directions), and the results were compared with the predictions of various design code provisions to evaluate the axial force-bending moment strength of the steel-concrete composite column with non-compact steel section.

2. Beam-Column Strength of a Composite Column

The AISC-LRFD provisions assume that the moment capacity of a composite beam-column can be predicted using the following interaction formulae that were developed for standard steel cross sections (AISC, 1999):

\[
\frac{P_u}{P_n} \geq 0.2 ; \quad \frac{P_u}{P_n} + \frac{8M_u}{9M_n} = 1.0
\]  
Equation (1)

\[
\frac{P_u}{P_n} < 0.2 ; \quad \frac{P_u}{P_n} + \frac{M_u}{M_n} = 1.0
\]  
Equation (2)

In Equation (1) and (2), \(P_u\) and \(M_u\) are the axial load and moment capacities, respectively, defining the axial load-bending moment (P-M) interaction curve for the composite column. \(P_u\) is the axial load capacity (at zero bending moment) and \(M_u\) is the moment capacity (at zero axial load), and is determined from a plastic stress distribution on the composite cross section in accordance with several assumptions.

The ACI provisions for a composite beam-column assume that the moment capacity of a composite cross section can be predicted based on the same assumptions that are used for ordinary reinforced concrete columns. Plane sections are assumed to remain plane and the moment capacity is assumed to be reached at an extreme concrete compression fiber strain (\(\varepsilon_c\)) of 0.003 (ACI, 2002).

The AIJ provisions for composite beam-columns assume that the steel section and the concrete develop their individual plastic strengths, and that the moment capacity of the composite column can be predicted by superposition of these strengths. The P-M interaction curve for the composite column is obtained as follows (AIJ, 1987).

\[
\text{When } 0 \leq P_u \leq uN_c \quad P_u = N_c \quad \text{and} \quad M_u = M_p + M_c
\]  
Equation (3)

\[
\text{When } P_u > uN_c \quad P_u = uN_c + N_s \quad \text{and} \quad M_u = M_s
\]  
Equation (4)

In Equation (3) and (4), \(N_c\) is the axial load capacity of the concrete core when it is subjected to any moment. \(M_p\) is the plastic moment capacity of the steel section. \(N_s\) and \(M_s\) are the axial load and moment, respectively, resisted by the concrete core. \(N_c\) and \(M_c\) are the axial load and moment, respectively, resisted by the steel section.

The EC4 provisions for composite beam-columns assume that the moment capacity of a composite section can be determined from a plastic stress distribution on the composite section while ensuring compatibility between the steel and concrete. The EC4 provisions assume rigid-plastic material properties for the steel and the concrete (CEN, 1994).

These all provisions can be applicable to compact sections that meet the width-to-thickness ratio. But because there isn’t related reference in case of SC composite column with non-compact steel section, the experimental test is necessary for the SC composite column.

3. Experiment on the Flexural-Axial Capacity

(1) Test specimens

The test specimens were listed in Table 1 and the corresponding details and specifications were illustrated in Fig.2. All the test specimens that were used were 350 mm x 350 mm in size and 4,440 mm in length, considering the capacity of the laboratory. The width-to-thickness ratio (b/t) of the beam-column test specimen was 25.0 and the thickness was 7 mm for both the flange and the web.

A plain bar 13 mm in diameter was used as a link. To ensure the application of uniform load, a 50-mm-thick end plate was attached at both ends of the column. The most critical parameter of the bending-compression experiment was the ratio of the applied axial force to the ultimate axial capacity \((P/P_n)\). The axial force ratios that were used were 30% and 40% of the maximum compressive strength, which is generally used to produce an effect similar to that of the actual
The equations suggested by Tremblay (Tremblay et al., 2000) were used to calculate the maximum compressive strength of the steel-concrete composite column with a non-compact section, in which the effect of the local buckling of the steel flange can be considered.

Where $A_c$ is the cross-section of the concrete; $A_{se}$ is the effective cross-section of the steel structure considering local buckling; $f_{ck}$ and $f_y$ are the specified strengths of concrete and steel, respectively.

\[
\lambda_p = \frac{b}{t_f} \sqrt{\frac{12(1-\nu^2)}{\pi^2 E_k} f_y} \quad (8)
\]

\[
k = -\frac{4}{(s/b)^2} + \frac{15}{\pi^4} \frac{(s/b)^2}{(2-3\nu)^2} \quad (9)
\]

* Specimens were named according to the following rules.
1) B1: Index of beam-column test specimen
2) S: Index of axis direction of steel cross section subjected flexural loading (S – Strong axis, W – Weak axis)
3) P30: Index of axial loading ratio to ultimate axial strength (30 – 30%, 40 – 40%)
compressive strength of the concrete and the yield strength of the steel, respectively; \(d\) is the depth of the column; \(t_f\) and \(t_w\) are the thickness of the steel flange and the steel web, respectively; \(b_e\) is the effective width of one steel flange; \(\lambda_p\) is the slenderness ratio of the flange; \(s\) is the interval of the link; \(b\) is \(1/2\) of the width of the steel column; and \(k\) is the buckling coefficient of the plate that varies depending on \(s/b\).

(2) Material test
To figure out the mechanical characteristics of the steel members and steel bars, the tensile strength of the material was tested according to the KS (Korean Standard) B 0801 Standard for a Test Coupon for the Tensile Test of the Metal Materials. Three 7-mm thick steel plates and three steel bars were sampled for the tensile strength test of the steel members and steel bars. The steel member used in the test was SM 490, with nominal yield strength of 330 MPa, and the steel bar used was SR 30, with yield strength of 300 MPa. The results of the experiment confirmed that all of the specimens satisfy the requirements for each steel type. Table 2 shows the results of the tensile strength test of the members.

Table 2. Tensile Coupon Test Results

| Test Coupon | \(F_y\) (MPa) | \(F_u\) (MPa) | Elong. (%) | Yield Ratio(%) |
|-------------|---------------|---------------|------------|----------------|
| PL-7T (SM490) | 451 | 596 | 28 | 76 |
| \(\phi\)13 (SR30) | 307 | 435 | 32 | 72 |

The concrete that was used the ready-mix concrete of a specified compressive strength of 30 MPa, a coarse aggregate of 25 mm in maximum diameter, and a slump of 20 cm. A total of six specimens, three for each concrete batch, were manufactured according to the KSF 2040 standard for the 28-day compressive strength test. The results of the compressive strength test are shown in Table 3.

(3) Installation and loading program
As illustrated in Fig.3., the experiment was conducted by installing one 3,000-kN actuator and two 1,000-kN actuators on the strong floor and wall. The axial force was applied by loading control (=5 kN/sec) with the use of the 3,000-kN actuator, and monotonic loading was applied by displacement control (=0.1 mm/sec) with the use of the 1,000-kN actuator to create double curvature on the test specimens for flexural-compressive strength. The load condition and setup configuration of the test for flexural-compressive strength are shown in Fig.3.

(4) Measurement schedule
The bending moment \(M_1\) and \(M_2\) and the rotation angle \(\theta_1\) and \(\theta_2\) can be obtained from axial force \(N\), the load applied on the L-shaped moment arm \(P_1 \) and \(P_2\), the rotation angle of the 1,000-kN actuator \(\psi_1\) and \(\psi_2\), the rotation angle of the moment arm \(\phi_1\) and \(\phi_2\), the rotation angle of the beam-column test specimen, and the displacement of the end of the beam-column test specimens \(u_1\) and \(u_2\). These can be expressed quantitatively, as shown in Eq. (10) to (13), where \(Q\) is the shear strength applied on the beam-column test specimen, \(h\) is the length between the pins at both ends, and \(H\) is the length of the beam-column test specimen.

\[
M_1 = \frac{M_1 + N \cdot u_1 - Q \cdot \frac{H - h}{2}}{2} \quad (10)
\]

\[
M_2 = \frac{M_2 + N \cdot u_2 - Q \cdot \frac{H - h}{2}}{2} \quad (11)
\]

\[
\theta_1 = \phi_1 + R = \phi_1 + (u_1 + u_2) / h \quad (12)
\]

\[
\theta_2 = \phi_2 + R = \phi_2 + (u_1 + u_2) / h \quad (13)
\]
As shown in Fig. 5., a 300-mm LVDT displacement meter and a 100-mm strain gauge were installed to measure the rotation angles of the beam-column test specimens and in Eq. (12) and (13). The 300-mm displacement meter was installed parallel to the end plate of the beam-column test specimen to measure the displacement of the end of the test specimens \( u_1 \) and \( u_2 \), while the 100-mm displacement meter was installed in the longitudinal direction of the beam-column test specimen to measure the rotation angles of the moment arms \( \phi_1 \) and \( \phi_2 \). The plastic strain gauge, on the other hand, was installed on the top and bottom of the flange of the beam-column test specimen, 100 mm apart from the end, to measure the strain of the beam-column test specimen under the applied load.

4. Test Results and Analysis

(1) Failure patterns

As illustrated in Figs. 6. (a) and (b), no crack were occurred on the test specimens B1-SP30 and B2-SP40 under the axial loading. When the bending moment was applied further, however, an initial crack was found at the location of the links at both ends of the test specimens, and more cracks developed and gradually increased in number on the concrete, particularly on the tensile side.

As the displacement increased gradually, the contact surface of the steel and concrete was separated and the concrete crushed on the compressive side, thus resulting in the maximum load. After the concrete was crushed, local buckling occurred at the steel flange, and the degree of buckling of the steel flange increased as the load increased. Such local buckling occurred between the links of both ends.

Similar to the test specimens for the strong axis, no crack was developed in the test specimens B3-WP30 and B4-WP40 during the axial loading test. The details were illustrated in Figs. 7. (a) and (b). However, tension cracks were developed and increased on the concrete surface, particularly on the tensile side, as the bending moment was further applied.

As the displacement increased, the tensile failure of the concrete occurred on the concrete surface, particularly on the tensile side, resulting in the maximum load. The further increase of the displacement resulted in the crushing of the concrete on the compressive side. Similar to the test specimens for the strong axis, the width of the concrete cracks on the tensile side increased gradually and the crushing of the concrete on the compressive side intensified with the increase of the initial axial force that was introduced.

(2) Bending moment-rotation angle \((M-\theta)\) relation

Under the initial axial forces 0.3 \( P_a (= 1,940 \text{ kN}) \) and 0.4 \( P_a (= 2,587 \text{ kN}) \), the bending moment \((M)\) - rotation angle \((\theta)\) curves for each test specimen are shown in Figs. 8. and 9. The vertical axis represents the load applied on the 1,000-kN actuator, which was calculated as the bending moment applied on the end of the beam-column test specimen, as shown in Fig. 4.

The horizontal axis, on the other hand, represents
the rotation angle of the beam-column test specimen. In the case of the test specimen for the strong axis, the elastic stiffness of B1-SP40 becomes larger than that of B2-SP30. It is due to the increased axial load. However, the maximum moment capacity of specimen B1-SP30 is 4.3kN-m greater than that of specimen B2-SP40. In the case of the test specimen B2-SP40, however, which had 10% larger initial axial force, the capacity rapidly dropped after the maximum capacity was reached. The moment capacity at 0.10 rad. was about 200kN-m which is lower than that of specimen B1-SP30. Therefore, it could be said that as the axial load increases, the column loaded in strong axis becomes stiffer while the maximum moment becomes smaller. In addition, the moment capacities of the column decrease more rapidly under the larger axial force.

In the case of the test specimens for the weak axis, the elastic stiffness of specimen B4-WP40 is greater than that of specimen B3-WP30. In addition, the maximum moment capacity of specimen B4-WP40 is also greater than that of the specimen B3-WP30 by 6.7kN-m. It is a different trend from the strong axis specimen. But its moment capacity dropped rapidly after the peak. Therefore, it could be said that as the axial load increases, the column loaded in weak-axis becomes stiffer and the maximum moment becomes larger. However, the moment capacities of the column decrease more rapidly under the larger axial force.

The difference of maximum moment capacities between strong- and weak-axis specimens could

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![Failure Pattern of Specimen B3-WP30](image-a)

(a) Failure Pattern of Specimen B3-WP30

![Failure Pattern of Specimen B4-WP40](image-b)

(b) Failure Pattern of Specimen B4-WP40

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![The Moment-Rotation Curve for Strong-axis Specimen](image-c)

Fig.8. The Moment-Rotation Curve for Strong-axis Specimen

![The Moment-Rotation Curve for Weak-axis Specimen](image-d)

Fig.9. The Moment-Rotation Curve for Weak-axis Specimen

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| Specimen ID. | B x D (mm x mm) | t (mm) | b/t | s (mm) | Yield strength of Steel (MPa) | Comp. strength of Conc. (MPa) | Loading Axial Force (kN) | Maximum Bending Moment (kN-m) |
|--------------|-----------------|-------|-----|--------|------------------------------|-----------------------------|--------------------------|-------------------------------|
| B1 – SP 30   | 350 x 350       | 7.0   | 25.0| 13 @ 175 | 451                          | 35.1                        | 1940                     | 723.2                         |
| B2 – SP 40   | 350 x 350       | 7.0   | 25.0| 13 @ 175 | 451                          | 35.1                        | 2587                     | 718.9                         |
| B3 – WP30    | 350 x 350       | 7.0   | 25.0| 13 @ 175 | 451                          | 35.1                        | 1940                     | 376.5                         |
| B4 – WP40    | 350 x 350       | 7.0   | 25.0| 13 @ 175 | 451                          | 35.1                        | 2587                     | 383.2                         |

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Table 4. Beam-Column Test Results
be explained in terms of failure mode previously mentioned. The failure mode of the test specimens for the weak axis was the tensile failure on the concrete surface. The flexural capacity for the weak-axis direction dropped comparatively lower than that for the strong-axis direction, to about 50% lower moment capacity than that for the strong-axis direction.

(3) Comparison with the evaluation of various design code provisions

Fig.10. shows the comparative results of the axial force (P)-bending moment capacity (M) curve of the composite column with a compact section, which satisfies the width-to-thickness ratio requirements specified in the design code (AISC, 1999; ACI, 2002; AIJ, 1987; and CEN, 1994).

It was found from the comparison that all the test specimens had greater axial force-bending moment capacity than those evaluated by the various design code provisions. In the evaluation of load-carrying capacity using the code provisions, the actual yield strength and compressive strength that were obtained through the material test were reflected and the influence of the strength reduction coefficient was not considered.

As shown in Fig.10. (a), it may be said that the AISC-LRFD code provisions evaluate the load-carrying capacity of the composite column too conservatively for the test specimens in the strong-axis direction. The experimental value was larger by 5% or more than the load-carrying capacity in the AIJ code provisions.

Fig.10. (b) shows that the maximum load-carrying capacity of the test specimens for the weak-axis direction is also larger than those in the AIJ and EC4 code provisions. Therefore, it may be said that no serious problems will be encountered in using the steel-concrete composite column with a non-compact section (b/t = 25.0) as a beam-column, and that its use would prevent a moment transfer to the weak-axis direction because the flexural strength for the weak-axis direction is lower than that for the strong-axis direction.

5. Conclusions

An experiment on the bending moment strength of the steel-concrete composite column with a non-compact section that is under a constant axial force was conducted for the strong-axis and weak-axis directions. The following results were obtained:

(1) The steel-concrete composite column showed the
failure pattern of the concrete crushing that occurred for the strong-axis direction, resulting in the reaching of the maximum load, and the tensile failure of the concrete that occurred for the weak-axis direction, resulting in the maximum load. Also, the flexural strength in the weak-axis direction was significantly lower by about 50% than that for the strong-axis direction.

(2) For strong- and weak-axis specimens, the elastic stiffness becomes greater as the axial force increases. The corresponding moments after the peak moments decrease more rapidly under larger axial forces. However, in the case of strong-axis specimen, the maximum moment becomes greater under the larger axial force while the corresponding moments become smaller in the case of weak-axis specimen.

(3) All the test specimens satisfied the axial force-bending moment capacity suggested in various design code provisions (AISC-LRFD, ACI, AIJ, and EC4). The AISC-LRFD code provisions, however, evaluate the load-carrying capacity of the composite column too conservatively. Therefore, it is considered desirable to use the EC4 and AIJ code provisions for the evaluation of the axial force-bending moment of the steel-concrete composite column.

(4) No serious problems are envisioned as regards the use of the steel-concrete composite column with a non-compact section (b/t = 25.0, s = D/2) as a beam-column. Its use could prevent a moment being transferred to the weak-axis direction because the flexural strength for the weak-axis direction is lower than that for the strong-axis direction.

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