# Design Method of Thin Steel-plate Composite (TSC) Beam Based on Eurocode

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## 1. Forward

As the number of new projects continuously decreases in the domestic construction market, construction companies in South Korea frequently enter overseas projects. Under this situation, domestic structural engineers are required to be able to design buildings in compliance with international standards. Structural engineers in South Korea are generally accustomed to the 2016 Korean Building Code (hereinafter referred to as "KBC 2016"[(1)]) enacted based on US structural design standards (ACI 318[(2)] and AISC 360[(3)]). However, they are insufficiently experienced with the other structural standards, including Eurocode[(4)-(6)], and thereby have difficulties executing overseas construction projects designed in accordance with those standards.

In attempts to maximize the characteristic advantages of steel structures and reinforced concrete structures while complementing their limitations, SENVEX CO., LTD. has developed various steel-concrete composite structures. Among the developed structures, the Thin Steel-plate Composite (TSC) beam has been verified its performances with regard to resistances of load[(7)-(22)] and fire[(23)-(26)] through experimental research, and multiple patents have been registered thereon (see Table 1). Furthermore, this TSC beam has been applied to various construction projects.

With its advantages mentioned above, TSC beam was recently proposed and applied for a large-scale warehouse construction project that was jointly ordered by the Jurong Town Corporation (JTC, Singapore's urban development corporation). Given that Singapore has designated Eurocode[(4)-(6)] as its national structural design standards, SENVEX has established the TSC beam design method based on

**Table 1. Patents related to TSC beams**

| Patent Title | Patent Registration Number | Registration Date |
|---|---|---|
| Formed Steel Plate Concrete Beam | No. 0617878 | 08.23.2006. |
| Reinforcing of TSC Beam with Wire Tension Method | No. 0777566 | 11.12.2007. |
| TSC Beam Formed with Folded in Top Flanges | No. 0872959 | 12.02.2008. |
| Composite Beam Putting the Truss Deck on the Flat Bar Shelf Welded to the Web Plate of TSC Beam | No. 1000269 | 12.03.2010. |
| Hybrid Beam with Separated Double Swellings and Assembling Method Thereof | No. 1404515 | 05.30.2014. |
| Composite Beam with Built-up Steel Plate | No. 1469798 | 12.01.2014. |

Eurocode[(4)-(6)].

This article provides a brief introduction of the TSC beam, discusses its structural and fire-resistance performances, and compares TSC design methods based on the different structural standards, KBC 2016[(1)] and Eurocode[(4)-(6)].

## 2. Thin Steel-plate Composite (TSC) Beam

### 2.1 Development Background and Major Advantages

A TSC beam forms a T-section with its concrete slab (see Fig. 1.). This beam is comprised of a U-type steel section and concrete filled in it. Its steel part has a lower plate, web, and upper plate, and is fabricated and assembled in factory. Studs welded at the upper flanges of this steel section serve as shear connectors transferring shear forces induced by the moment resisted by a beam. Through studs, a U-type steel plate, inner concrete, and concrete slab could be structurally integrated together as a T-type composite section which exhibits complete composite behavior. It is worth noting that a U-type steel plate of a TSC beam has such a high flexural rigidity that it can serve as a concrete formwork with a sufficient resistance of lateral pressure exerted during concrete placing. Therefore, TSC beam can achieve non-supporting construction where the conventional formworks and shoring systems are not needed, and bring significant enhancement of the workability and constructional efficiency.

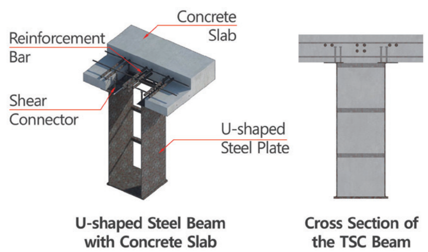


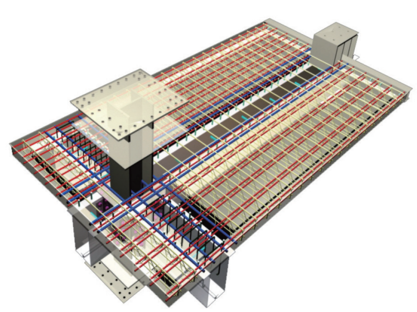

**Fig. 1 Shape and Components of the TSC Beam**

### 2.2 Verification of Structural Performance

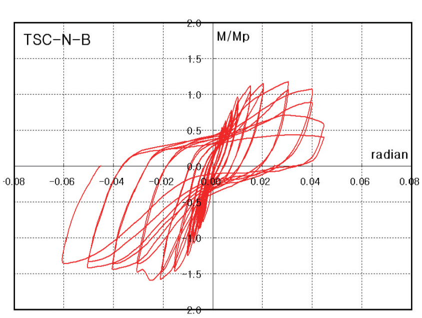
The structural experiments verifying the performances of the TSC beams have been reported as papers and published in numerous domestic journals. Kim, S. M. and Kim, G. S.[(7)], and Kim, S. S. et al.[(8)] demonstrated that it was valid to design a TSC beam according to the same existing design method of a composite beam with H-section steel. They also verified the flexural behavior of TSC beams with respect to the types of shear connectors, so that this result provides theoretical basis for the analysis of structural characteristics, structural design, and construction of TSC beams. Also, the structural performances of TSC beams have been verified through various studies such as the experimental study of TSC beams using the post-tensioning method[(9)], the flexural resistance testing of bottleneck-type low-depth TSC beams that use thin plates[(10)], and the evaluation of shear capacities for different forms of shear connectors in TSC beam[(11)].

Meanwhile, the seismic performance of the TSC beam connections also has been evaluated through the experimental researches on various beam-to-column connections with the combinations of TSC beams and different types of columns. The seismic performance of the TSC beam to-SRC column connections, as shown in Fig. 2, was verified through cyclic loading tests[(12)]. Kim S. J. et al.[(13)] experimentally evaluated the seismic performance of the beam-to-column connections comprised of the TSC beams and the columns with permanent steel-plate formwork. Hwang H. J. et al.[(14)] showed the seismic resistance of the TSC beam-to-concrete column connections, and Park H. G. et al.[(15)] developed a connection method that complies with the seismic regulations supplemented by the cyclic loading tests of the TSC beam-to-steel column composite connections. Park C. H. et al.[(16)] derived a design method which satisfies the seismic performance regulations of the composite connections and controls the limit states, from the complemented cyclic seismic testing of the TSC beam-to-H-section column connections in the strong-axis direction. Also, the seismic performance of the TSC beam-to-PRC (Pre-fabricated Reinforced Concrete) column connections was investigated by cyclic lateral loading tests[(17)]. In another study, researchers proposed the seismic resisting connection details and design methods through the cyclic loading tests of the TSC beam-to-PSRC (Prefabricated Steel Reinforced Concrete) column connections[18]. Also, the cyclic loading tests were conducted on the external connections composed of TSC beams and centrifugal reinforced concrete (CRC) columns[19].

(a) Typical setup of connection specimen



(b) TSC beam-to-SRC column connection specimen




(c) Experimental results: Moment-rotation angle hysteresis loop

**Fig. 2 Structural performance test setup and the result; load-displacement hysteresis curves[12] of SRC column-to-TSC beam connections**

These studies about the structural performance verifications of the connections that are composed of TSC beams and various columns also have been published in international journals, thereby demonstrating the validity of those structural features on the global stage. Park et al.[20] conducted cyclic loading tests of the TSC beam-to-RC column connections and TSC beam-to-PRC column connections. Lee et al.[21] performed cyclic loading tests of the connections that are composed of H-section steel columns, TSC beams, and corrugated deck-concrete slabs. Hwang et al.[22] reported cyclic loading test results of TSC beam-to-PSRC column connections.

### 2.3 Verification of Fire-resistance Performance

Given that their upper and lower flange parts and web steel plates are exposed to the external environment, TSC beams must provide sufficient fire-resistance performance. To this end, several studies have been performed concerning the verification of their structural and fire-resistance performances. For the fire-resistance performance evaluation of TSC beams, fire-resistance tests were conducted, considering the shape of members, exerted loads, retrofit methods of fire-resistant covering materials, and etc. as major experimental parameters. A numerical analysis was also followed to supplement the results of the experimental tests. In another study, fire-resistance tests were carried out to investigate the changes of plastic moment capacity of TSC beams at high temperatures. Furthermore, the efficiency of reinforcement in fire-resistive coatings and concrete is examined[24]. Additionally, the fire-resistance tests were conducted to experimentally determine whether TSC beams, sprayed with 25 mm thick fire-resistive coating could provide a three-hour fire-resistance under both loaded and unloaded conditions[25].

To apply the TSC beams to domestic buildings, the Certificates of Accreditation of Fire Resistant Construction issued by the Korea Institute of Civil Engineering and Building Technology have been renewed on a periodic basis since 2008[26]. Fig. 3 presents the examples of these certificates, which guarantee that the concerned TSC beam provides a two-hour fire resistance rating when sprayed with 16 mm thick fire resistive coating and a three-hour rating when sprayed with 21 mm coating. Also, it provides a one-hour, two-hour, and three-hour fire-resistance performance when coated with 0.65 mm, 1.30 mm, and 3.30 mm fire-proof paint, respectively.


**Fig. 3 Certificates of Accreditation of Fire Resistant Construction for TSC beams**

## 3. TSC Beam-related Regulations in KBC 2016 and Eurocode

KBC 2016[1] and Eurocode[4]-[6] differ in many aspects. For example, with regards to the classification system of steel elements that comprise steel sections and composite sections, the configuration of design equations, underlying theories, and design philosophies. Major regulations concerning the structural design of TSC beams are as follows.

### 3.1 Strength Reduction Factor

According to KBC 2016[1], the design strength of a member is calculated by multiplying the nominal strength by its appropriate strength reduction factor. In contrast, under Eurocode[4]-[6], the strength of a member is determined by its nominal strength divided by its partial factor, so that the design strength is reduced in terms of safety. Here, the partial safety factors are determined in accordance with the National Annex of the standards.

In KBC 2016[1], the strength reduction factors to determine the flexural and shear strengths of composite sections are given as follows.

- Flexural strength of composite section

$$\phi_b = 0.90 \tag{1}$$

- Shear strength of steel

$$\phi_v = 0.90 \tag{2a}$$

- Shear strength of concrete and reinforcing bar, or steel section and reinforcing bar

$$\phi_v = 0.75 \tag{2b}$$

According to Eurocode 4[6] (EN 1994-1-1), which concerns steel-concrete composite structures, the partial factors are defined differently for concrete, reinforcing bars, and steel members, as given below.

$$\gamma_c = 1.5 \tag{3a}$$

$$\gamma_s = 1.15 \tag{3b}$$

$$\gamma_M = 1.0 \tag{3c}$$

Here, $\gamma_c$, $\gamma_s$, and $\gamma_M$ refer to the partial factors for concrete, reinforcing bars, and steel members, respectively.

### 3.2 Steel Section Classification

Before describing how steel sections are classified, geometric shapes and variables for TSC beams need to be defined. The defined variables are shown in Fig. 4.

In Fig. 4, $b$ is the width of the section; $h$ is the height of the section; $t_w$ is the thickness of the web; $b_f$ is the width of the upper flange; $t_f$ is the thickness of the upper flange; $b_p$ is the width of the lower flange; and $t_p$ is the thickness of the lower flange.

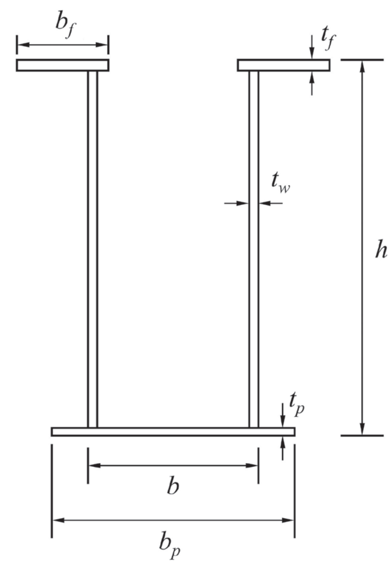



**Fig. 4 Definitions of geometric variables for TSC beams**

#### 3.2.1 Definition of Steel Class

Under KBC 2016[1], the elements that comprise the steel section are classified into three categories as follows: the compact element, non-compact element, and slender section element. In contrast, Eurocode 4[6] (EN 1994-1-1) classifies these elements into Class 1, Class 2, Class 3, and Class 4, according to Eurocode 3[5] (EN 1993-1-1), and their definitions are given as follows.

- Class 1 cross-sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.

- Class 2 cross-sections are those which can exhibit the plastic moment strength, but have a limitation in the rotation capacity due to local buckling.
- Class 3 cross-sections are those in which the stress in the extreme compression fiber of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance.
- Class 4 cross-sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

Simply, Class 1 and Class 2 correspond with the compact element, Class 3 corresponds with the non-compact element, and Class 4 corresponds with the slender section element.

### 3.2.2 Steel Section Classification under KBC: Limits of the Plate Width-Thickness Ratio

#### a. Steel Section before Concrete Placing

Steel sections before placing concrete in TSC beams are classified according to the limits of width-thickness ratios as provided in "0702.4 Section Classification by Local Buckling" of KBC 2016[1], as below.

- Width-thickness ratio of the upper flange element subject to positive bending moment ($b_f/t_f$)

$$\lambda_p = 0.38\sqrt{E/F_y} \tag{4a}$$

$$\lambda_r = 0.95\sqrt{k_c E/F_L} \tag{4b}$$

- Width-thickness ratio of the lower flange element subject to negative bending moment ($b_p/t_p$)

$$\lambda_p = 1.12\sqrt{E/F_y} \tag{5a}$$

$$\lambda_r = 1.40\sqrt{E/F_y} \tag{5b}$$

- Width-thickness ratio of the web element subject to positive or negative bending moment ($h_c/t_w$)

$$\lambda_p = \frac{(h_c/h_p)\sqrt{E/F_y}}{(0.54M_p/M_y - 0.09)^2} \tag{6a}$$

$$\lambda_r = 5.70\sqrt{E/F_y} \tag{6b}$$

where $\lambda_p$ is the upper limit of the width-thickness ratio of the compact section; $\lambda_r$ is the maximum width-thickness ratio of the non-compact section; $E$ is the elastic modulus of the concerned steel; $F_y$ is the yield strength of the concerned steel; $k_c = 4/\sqrt{h/t_w}$ (Here, $0.35 \le k_c \le 0.76$); $h_c/2$ is the distance between the part compressed by the web and the neutral axis; $h_p/2$ is the distance between the part compressed by the web and the plastic neutral axis; $M_p$ is the plastic moment capacity; and $M_y$ is the yield moment capacity. In Eq. (4b), $F_L$ refers to the stress used in the determination of the nominal strength where the residual stress of the steel section is considered. It is determined as $S_{xt}/S_{xc}$ ratio shown below:

i. $S_{xt}/S_{xc} \ge 0.7$

$$F_L = 0.7F_y \tag{7a}$$

ii. $S_{xt}/S_{xc} < 0.7$

$$F_L = F_y S_{xt}/S_{xc} \ \ge\ 0.5F_y \tag{7b}$$

where $S_{xt}$ and $S_{xc}$ are the section factors for tensile flanges and compressive flanges, respectively.

#### b. Steel Section after Concrete Placing

The composite sections after placing concrete in TSC beams are classified according to the limits of width-thickness ratios of each element, as provided in "0709.1.4. Filled Composite Section Classification by Local Buckling" as follows.

- Width-thickness ratio of the flange element with respect to positive or negative bending moment ($b_f/t_f$, $b_p/t_p$)

$$\lambda_p = 2.26\sqrt{E/F_y} \tag{8a}$$

$$\lambda_r = 3.00\sqrt{E/F_y} \tag{8b}$$

$$\lambda_{\max} = 5.00\sqrt{E/F_y} \tag{8c}$$

- Width-thickness ratio of the web element with respect to positive or negative bending moment ($h_c/t_w$)

$$\lambda_p = 3.00\sqrt{E/F_y} \tag{9a}$$

$$\lambda_r = 5.70\sqrt{E/F_y} \tag{9b}$$

$$\lambda_{\max} = 5.70\sqrt{E/F_y} \tag{9c}$$

where $\lambda_{max}$ is the maximum allowable limit of the width-thickness ratio of composite members.

#### 3.2.3 Steel Section Classification under Eurocode: Limit of the Plate Width-Thickness Ratio

Eurocode[(4)-(6)] differs from KBC 2016[(1)] in defining the limits of the width-thickness ratio, i.e., element classification criteria. In Eurocode[(4)-(6)], the width-thickness ratio limits for each element of the U-type steel plates of TSC beams are shown in Tables 2 and 3.

The width-thickness ratio ($h_w/t_w$) limits for the web element defined in Eurocode[(4)-(6)] vary depending on $\alpha$, the ratio between the length of the area loaded in compressive stress and the total depth of the web element. In Class 3, the width-thickness ratio limits vary depending on the $\psi$ value. $\psi$ is the ratio between the yield stress and the stress at the edge of the zone in tension, determined by compatibility condition applied when the edge of the zone in compression is exerted to the yield stress.

### 3.3 Flexural Strength of TSC Beams

#### 3.3.1 Flexural Strength of the Steel Section before Concrete Placing

##### (1) Determination of the Flexural Strength of the Steel Section under KBC 2016[(1)]

According to the study of Kim S. M. and Kim G. S.[(7)], that a U-type steel section of the TSC beam is a single symmetric cross-section, where its strong axis is subject to bending, the nominal flexural strength of the steel section can be determined according to the regulations, as provided in "0706.4 H-section Steel Members that Contain the Compact Web or Non-compact Web Subject to Strong-axis Bending" of KBC 2016[(1)]. Thus, the nominal flexural strength $M_n$ is determined as the minimum value among the following strengths; the yield strength of the compression flange, the global lateral buckling strength, the local buckling strength of the flange, and the yield strength of the tensile flange. These strengths are shown as the following items.

a. Yield Strength of the Compression Flange

$$M_n = R_{pc}M_{yc} = R_{pc}F_yS_{xc} \qquad (10)$$

Here, $R_{pc}$ is the plastic factor of the web and determined using Eq. (11a) and (11b), as below:

**Table 2. Plate Width-Thickness Ratio Limits for Flange Elements of TSC Beams**

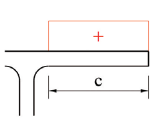
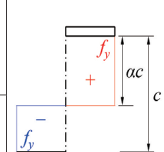
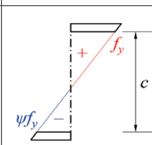
| Section Element Classification | Section and Stress Distribution | Ratio Limits for the Steel Section before Concrete Placing | Ratio Limits for the Steel Section after Concrete Placing |
|---|---|---|---|
| | | $(b_f/t_f$ or $b_p/t_p)$ | |
| Class 1 | | $\leq 9\sqrt{235/f_y}$ | |
| Class 2 |  | $\leq 10\sqrt{235/f_y}$ | $\leq 14\sqrt{235/f_y}$ |
| Class 3 | | $\leq 14\sqrt{235/f_y}$ | $\leq 20\sqrt{235/f_y}$ |

**Table 3. Plate Width-Thickness Ratio Limits for Web Elements of TSC Beams**

| Section Element Classification | Section and Stress Distribution | Ratio Limits for Steel Sections and Composite Sections ($h_w/t_w$) |
|---|---|---|
| Class 1 |  | (1) When $\alpha > 0.5$, $\leq 396\sqrt{235/f_y}/(13\alpha-1)$<br>(2) When $\alpha \leq 0.5$, $\leq 36\sqrt{235/f_y}/\alpha$ |
| Class 2 | | (1) When $\alpha > 0.5$, $\leq 456\sqrt{235/f_y}/(13\alpha-1)$<br>(2) When $\alpha \leq 0.5$, $\leq 41.5\sqrt{235/f_y}/\alpha$ |
| Class 3 |  | (1) When $\psi > -1$, $\leq 42\sqrt{235/f_y}/(0.67+0.33\psi)$<br>(2) When $\psi \leq -1$, $\leq 62\sqrt{235/f_y}(1-\psi)\sqrt{\psi}$ |

i. $h_c/t_w \leq \lambda_{pw}$

$$R_{pc} = M_p / M_{yc} \tag{11a}$$

ii. $h_c/t_w > \lambda_{pw}$

$$R_{pc} = \frac{M_p}{M_{yc}} - \left(\frac{M_P}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right) \leq \frac{M_p}{M_{yc}} \tag{11b}$$

where $\lambda_{pw}$ is the plate width-thickness ratio limit of the compact section for the web.

b. Lateral Buckling Strength

The lateral buckling strength is separately determined according to the unbraced length $L_b$ of the beam, as below:

i. When $L_b \leq L_p$, the lateral buckling strength does not have to be considered.

ii. When $L_p < L_b \leq L_r$,

$$M_n = C_b\left[R_{pc}M_{yc} - (R_{pc}M_{yc} - F_L S_{xc})\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq R_{pc}M_{yc} \tag{12a}$$

iii. When $L_p < L_b \leq L_r$,

$$M_n = F_{cr}S_{xc} \leq R_{pc}M_{yc} \tag{12b}$$

where $M_{yc} = F_y S_{xc}$

$$F_{cr} = \frac{C_b \pi^2 E}{(L_b/r_t)^2}\sqrt{1 + 0.078\frac{J}{S_{xc}h_o}\left(\frac{L_b}{r_t}\right)^2}$$

Here, $C_b$ is the correction factor (<1.5) for the lateral buckling moment, which is determined using Eq. (0706.1.1) in KBC 2016[1]; $J$ is the torsional constant; and $h_o$ is the center distance between the upper and lower flanges. When $I_{yc}/I_y \leq$ 0.23, $J$ is set to zero.

$L_p$, the limiting unbraced length for the yielding limit state, is determined using Eq. (13a), and $L_r$, the limiting unbraced length for the lateral buckling limit state, is determined using Eq. (13b).

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \tag{13a}$$

$$L_r = 1.95 r_t \frac{E}{F_L}\sqrt{\frac{J}{S_{xc}h_o}}\sqrt{1 + \sqrt{1 + 6.76\left(\frac{F_L}{E}\frac{S_{xc}h_o}{J}\right)^2}} \tag{13b}$$

Here, $r_t$ refers to the secondary radius of the effective section for lateral buckling and is determined as follows:

- For H-section steel members with rectangular-shaped compression flanges,

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\frac{h_o}{d} + \frac{1}{6}a_w \frac{h^2}{h_o d}\right)}} \tag{14}$$

Here, $a_w = \frac{h_c t_w}{b_{fc} t_{fc}}$ where $b_{fc}$ is the width of the compression flange; and $t_{fc}$ is the thickness of the tensile flange.

c. Local Buckling Strength of Compression Flanges

i. For non-compact flanges

$$M_n = R_{pc}M_{yc} - (R_{pc}M_{yc} - F_L S_{xc})\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \tag{15a}$$

ii. For slender-section flanges

$$M_n = \frac{0.9 E k_c S_{xc}}{\lambda^2} \tag{15b}$$

d. Yield Strength of Tensile Flanges

$$M_n = R_{pt}M_{yt} = R_{pt}F_y S_{xt} \tag{16}$$

**(2) Determination of the Flexural Strength of the Steel Section under Eurocode[(4)-(6)]**

Eurocode[(4)-(6)] requires that the flexural strength of the steel section be separately determined for each steel section type using different design equations accordingly. The flexural strength of the U-type steel section of the TSC beam before concrete placing can be calculated using Eq. (6.13)-(6.15) of Eurocode 3[(5)], correspond to Eq.(17a)-(17c), as below:

a. For Class 1 or Class 2 sections

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}} \tag{17a}$$

b. For Class 3 section

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el,\min} f_y}{\gamma_{M0}} \tag{17b}$$

c. For Class 4 section

$$M_{c,Rd} = \frac{W_{eff,\min} f_y}{\gamma_{M0}} \tag{17c}$$

Here, $\gamma_{M0}$(= 1.0) is the partial factor considering material strength applied regardless of the class, $W_{el,min}$ is the minimum elastic section modulus, and $W_{eff,min}$ is the minimum effective section modulus. $W_{el,min}$ and $W_{eff,min}$ vary depending on the maximum elastic stress within the section.

### 3.3.2 Flexural Strength of the Composite Section after Concrete Placing

#### (1) Determination of the Flexural Strength of the Composite Section under KBC 2016[1]

Given that a TSC beam falls into the category of filled composite members, its flexural strength is determined according to “0709.3.4 Flexural Strength of Filled Composite Members” of KBC 2016[1]. The flexural strengths of compact-section TSC beams are calculated based on the plastic moment strength $M_p$ (Eq. 18) that can be obtained from the plastic stress distribution of the composite section.

$$M_n = M_p \tag{18}$$

The nominal flexural strength of the slender section is determined based on $M_y$, the moment that is induced when the tensile flange is yielded while the compression flange is first yielded. The flexural strength of non-compact section is calculated using Eq. (19), where the plastic moment and yield moment are linearly interpolated with respect to the width-thickness ratio.

$$M_n = M_p - (M_p - M_y)\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p} \tag{19}$$

A flexural strength of a TSC beam is determined based on the internal force of the T-shaped section while considering the contribution of the reinforced concrete slab thereto within the effective width. To this end, shear connectors equipped to its upper flange allow the reinforced concrete slab and the TSC beam to behave as one composite member. In KBC 2016[1], the flexural strength of the composite member that is subject to the positive moment is determined considering the plastic stress distribution. The compressive force of the concrete slab, $C$, which induces the moment resistance, can be determined based on the composite ratio between the steel beam and concrete slab. Thus, under Commentary Equation 0709.3.2.1 of KBC 2016[1], in estimating the plastic moment, the $C$ value is determined as the minimum value of the following items.

$$C = A_{sw}F_y + 2A_{sf}F_y \tag{20a}$$

$$C = 0.85 f_{ck} A_c \tag{20b}$$

$$C = \sum Q_n \tag{20c}$$

Here, $f_{ck}$ is the design compression strength of concrete, $A_c$ is the area of the concrete slab within the effective width, $A_{sw}$ is the cross-section area of the steel web, $A_{sf}$ is the cross-section area of the steel flange, and $F_y$ is the design yield strength of the steel member. $\sum Q_n$ is the sum of the nominal strengths of the stud anchors located between the positions where the positive moment is the maximum and the moment is zero.

In cases where the compressive force of slab $C$ is determined by the sum of the strengths of the shear connectors $\sum Q_n$(Eq. 20c) the beam is designed as a partial composite beam, and in the other cases, the beam is designed as a full composite beam.

The depth of the compression block $a$ is determined using Eq. (21), as below.

$$a = \frac{C}{0.85 f_{ck} b_e} \tag{21}$$

Here, $b_e$ is the effective width of the slab and determined as the minimum value of the following items: (1) one-eighth of the beam span (the distance between the centers of the support points); (2) one-half of the distance between the centerline of one beam and the centerline of the nearest beam; and (3) the minimum distance between the centerline of the beam and the edge of the slab.

#### (2) Determination of the Flexural Strength of the Composite Section under Eurocode[4]-[6]

Under Eurocode[4]-[6], the flexural strength of the composite section ($M_{Rd}$) is determined based on the class of the steel section and the composite ratio. In the case of Class 1 or Class 2 (compact sections), for which the section strength can be obtained based on plastic theories, the flexural strength can be determined using the full-composite or partial-composite concepts. Here, the determining factor is the composite ratio estimated based on the strength of the shear connectors placed between the concrete slab and the steel section.

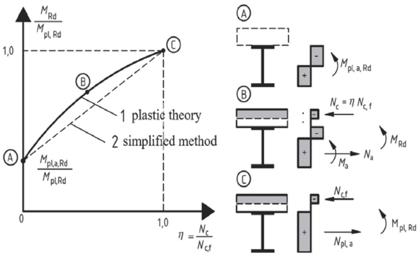



**Fig. 5 Change of plastic flexural moment strength depending upon composite ratio**

Fig. 5 illustrates the change in the plastic stress distribution and the plastic moment depending upon the composite ratio between the concrete slab and the steel section. For partial composite structures, the plastic moment strength $M_{Rd}$ is estimated by the compressive force exerted on the concrete slab, reduced by the composite ratio as demonstrated by Curve 1, which connects Point A, Point, B, and Point C in Fig. 5. Alternatively, it can be estimated in a simplified and conservative manner using Eq. (22), which connects Point A and Point C with the dotted line shown in Fig. 5.

$$M_{Rd} = M_{pl,a,Rd} + (M_{pl,Rd} - M_{pl,a,Rd})\frac{N_c}{N_{cf}} \tag{22}$$

Here, $M_{pl,a,Rd}$ is the plastic moment of the steel section, $M_{pl,Rd}$ is the plastic moment of the full-composite section, $N_c$ is the compressive force exerted on the concrete slab, and $N_{cf}$ is the compressive force exerted on the concrete slab when the full-composite plastic moment is induced.

The composite ratio $\eta$ of a TSC beam is determined based on the ratio of $N_c$ to $N_{cf}$. $N_c$ is determined as the minimum value of the following items: the design strength of the shear resistance of a single connector ($P_{Rd}$), plastic resistance within the effective width of the concrete slab ($N_{pl,c}$), and the plastic resistance of the steel member ($N_{pl,a}$). These three strengths are determined using the equations below.

$$P_{Rd} = \min\left(\frac{0.8f_u\pi d^2/4}{\gamma_V}, 0.29\alpha d^2\frac{\sqrt{f_{ck}E_{cm}}}{\gamma_V}\right) \tag{23}$$

Here, i. $3 \le h_{sc}/d \le 4 : \alpha = 0.2\left(\frac{h_{sc}}{d}+1\right)$

ii. $h_{sc}/d > 4 : \alpha = 1$

$$N_{pl,c} = 0.85 f_{cd} b_e D_s \tag{24}$$

$$N_{pl,a} = A_a f_{yd} \tag{25}$$

where, $f_u$ is the tensile strength of the stud ($\le$ 500 MPa); $\gamma_V$ is the partial factor for the shear resistance of the stud, $h_{sc}$ is the nominal height of the stud, $f_{cd}$ is the design value of the cylinder compressive strength of concrete ($=f_{ck}/\gamma_c$), $b_e$ is the effective width of the beam (determined according to "5.4.1.2 Effective Width of Flanges for Shear Lag" of Eurocode 4[6]), $D_s$ is the height of the slab, $f_{yd}$ is the design yield strength of the steel member, and $A_a$ is the cross-section area of the steel member.

### (3) Minimum composite ratio under KBC 2016[1] and Eurocode[4)-(6]

KBC 2016[1] does not provide any requirement regarding the number of shear connectors used; there is no standard to regulate the composite ratio. Upon this background, the commentary on "0709.3.2(3) Minimum Amounts of Shear Connectors" proposes to ensure that the composite ratio is at least 25% while mentioning possible problems that might occur when the ratio is lower than 50%. In this regard, when designing TSC beams in accordance with KBC 2016[1], it is proposed that the composite ratio be 50% or higher for the positive moment. Meanwhile, for the negative moment, the composite ratio is proposed to be 100% because, under this condition, the shear connectors can allow the slab reinforcing bars to fully exhibit their tensile force, as intended.

On the other hand, Eurocode 4[6] gives the minimum composite ratio in "6.6.1.2 Limitation on the Use of Partial Shear Connection in Beams for Buildings". The minimum composite ratio must be determined depending on the distance between the points at which the flexural moment is zero ($L_e$). This relationship is defined by Eq. (6.12)-(6.15) of Eurocode 4[6]. By applying 6.6.1.2(2), the minimum composite ratio can be linearly interpolated with respect to the area ratio of the upper flange to the lower flange, $a_f$ and expressed as Eq. (26).

i. When $L_e \le 27.5 - 2.5a_f$ :

$$\eta \geq 1-\left(\frac{355}{f_y}\right)\left[\left(0.975-0.225a_f\right)-\left(0.0375+0.0075a_f\right)L_e\right]$$
$$\text{and } \eta \geq 0.4 \qquad (26a)$$

ii. When $L_e > 27.5 - 2.5a_f$ :

$$\eta \geq 1 \qquad (26b)$$

Here, $a_f \equiv b_p t_p / 2b_f t_f \leq 3$.

### 3.4 Deflection of Composite Member

"0709.3.2 Composite Beams with Steel Anchors (Shear Connectors)" of KBC 2016[1] does not provide a regulation concerning the estimation of the deflection of the composite section. The commentary thereon, however, proposes that the combination of working loads be adjusted to be within the elastic range, or that the amount of deflection be estimated considering the expansion effect caused by inelastic behavior in cases where the limit state of the composite section is predominantly determined by the deflection. Here, the amount of deflection is calculated based on the effective flexural rigidity, $EI_{eff}$. Under KBC 2016[1], the effective section secondary moment $I_{eff}$ is calculated by multiplying $I_{equiv}$, obtained based on linear elastic theories, by 0.75. In the meantime, KBC 2016[1] does not provide any regulation that directly concerns the long-term deflection of composite members due to concrete shrinkage and creep.

Similarly, Eurocode[4]-[6] does not explicitly provide any guideline or regulation concerning the long-term deflection. However, 5.4.2.2 and 5.4.2.3 of Eurocode 4[6] present elastic modulus ratios as well as matters related to the flexural rigidity when cracks occur on the concrete, respectively. In this study, the long-term deflection of TSC beam was examined and reviewed using the aforementioned regulations and Worked Examples of Eurocode 4[27], and the corresponding review result was finally approved by the inspection team of Singapore.

#### 3.4.1 Long-term Deflection of TSC Beam (Eurocode 4[6])

##### (1) Elastic Modulus Ratio for Long-term Loading

Given that the elastic modulus of concrete decreases over time, this change must be reflected in estimating the long-term deflection of concrete. The reduction in the elastic modulus of concrete varies depending on the type and the duration of the applied long-term loading. In Eurocode 4[6], the decrement in the elastic modulus of concrete is reflected in estimating the amount of long-term deflection of the concrete by using the elastic modulus ratio $n_L$. This relationship is expressed as Eq. (27) below.

$$n_L = n_0\left(1+\psi_L \phi_t\right) \qquad (27)$$

Here, $n_L$ is the elastic modulus ratio of the concrete to the steel member, and $\phi_t$ and $\psi_L$ are the creep constants that reflect the time and load characteristics, respectively.

##### (2) Deflection Considering Long-term Loading

According to the Worked Examples of Eurocode 4[27], the total amount of deflection $\delta$ caused by long-term loading can be estimated using the equation, as shown below, and each term of the equation can be more specifically expressed as shown in Fig. 6.

$$\delta = \delta_{perm} + \delta_{var} = \delta_{perm} + \left(\delta_{first} + \delta_{creep} + \delta_{shrinkage}\right) \qquad (28)$$

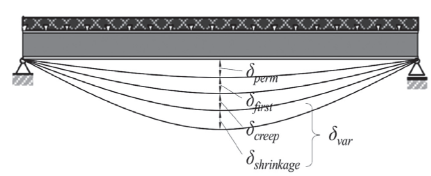



**Fig. 6 Total amount of long-term deflection δ**

Here, $\delta_{perm}$ is the deflection caused by permanent load, and $\delta_{var}$ is the deflection caused by various factors. $\delta_{var}$ is divided into $\delta_{first}$, $\delta_{creep}$, and $\delta_{shrinkage}$. $\delta_{first}$ is the deflection caused by the load exerted at 28-day age, and $\delta_{creep}$ is the deflection that starts to occur some time after the floor-finishing load and the initial live load are exerted. Finally, $\delta_{shrinkage}$ is estimated by adding up the total amount of deflection that occurs when cracks are generated at the edges of the concerned member by the finishing material load. When estimating $\delta_{first}$ and $\delta_{creep}$, the effective secondary moment $I_{eff}$, which reflects the long-term effects, is used.

## 4. Closing Remarks

In this article, a design method of the TSC beam based on Eurocode(4)-(6) was introduced, which had been established in an attempt to apply the TSC beam to Singapore. To begin with, a brief introduction to the TSC beam was presented along with its structural performance and fire-resistance performance verified by previous experimental researches. Also, major regulations concerning TSC beams from KBC 2016(1) and Eurocode(4)-(6) were summarized. In designing TSC beams, KBC 2016(1) and Eurocode(4)-(6) are similar in that a composite structure-based design is implemented but they differ in many aspects, for example, in terms of the strength reduction factors, the classification system of steel sections and composite sections, the equations for determining the limits of plate width-thickness ratio, and the equations for estimating the flexural strength of the composite section. Going forward, many domestic construction companies are expected to enter not only the Singapore construction market but also numerous construction markets across the globe. To respond to this growing trend, structural engineers in Korea need to equip themselves with the capabilities required to make use of various structural design standards. The authors hope this article will contribute to enhancing the competence of domestic structural design engineers.

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