Assessing the Structural Behavior of a New UHPC-Infilled Top Chords Integrated Deck Plate System at Construction Stage

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Abstract: In this paper, a new deck plate system, in which the top chords are infilled with high-performance cement-based material such as high-strength mortar and UHPC (ultra-high performance concrete), was proposed. The bending capacity of the proposed deck plate system at the construction stage was investigated by performing a four-point bending test on seven specimens with a net span of 4.6 m. Test parameters included the type of top chords, type and amount of infilled material, and existence of inner shear connectors. The load versus displacement curves were plotted for all the specimens, and their failure modes were identified. In addition, a theoretical strength estimation process was developed for the proposed deck plate system. The strength values calculated by the estimation process were compared with the test results and analyzed. This comparison showed that the proposed system retained the peak strength and initial stiffness at a much higher level than the conventional system, and the theoretical strength estimation process can predict the failure modes and peak strength of the proposed system accurately.

Keywords: deck plate system; structural behavior; UHPC-infilled top chord; compressive strength; construction stage

1. Introduction

Recently, a variety of deck plate systems [1,2] have been developed by many construction companies and research institutions due to their advantages over the typical reinforced concrete floor system. The deck plate system can function as both the form for concrete placement and a flexural component to resist different types of structural load. The former allows a large amount of reduction in the construction period and expense as the cost related to concrete formwork takes approximately thirty percent of the total construction cost [3–5]. Due to the advantage of the latter, the deck plate can resist workloads during construction as well as the self-weight of concrete placed into the floor, and finally provide flexural resistance to the entire composite floor system after concrete curing [6–8].

The steel wire-integrated deck plate is one of the widely used deck plate systems in construction fields [9] and is generally manufactured by the factory-automated production process shown in Figure 1. The truss deck plate is largely composed of two parts, such as a truss girder and a bottom deck plate as illustrated in Figure 2. The truss girder provides flexural resistance to the entire deck plate system, while the bottom deck plate mainly plays a role in the formwork for concrete. The truss girder consists of three components, including top and bottom chords and lattice members connecting the two types of chords. All of these components are integrated into a single entity through spot welding as illustrated in Figure 1. Then, the truss girder can be attached to the galvanized bottom deck plate by electric resistance welding [5,10].
In order to implement a long-span deck plate system, its flexural capacity needs to be increased, which in turn requires the use of top and bottom chords of large diameters. As illustrated in Figure 1, its production requires mechanical equipment of increased capacity to process the steel wire of a large diameter wound on the coil. This increases the overall cost of the manufacturing process and limits the maximum available diameters of the top and bottom chords. The maximum flexural stiffness and strength of the deck plate system are also limited. Usually, the maximum diameter of the top and bottom chords used in the conventional steel wire-integrated deck plate system is not greater than 14 mm, and it can generally be applied to a relatively short span length of approximately 4 m unless temporary supports for fresh concrete and other temporary loads are provided at the construction stage. In recent years, the use of long-span floor systems has been popular to improve the efficiency of space usage [11], and this limitation can become a major disadvantage of the deck plate system.
In order to address this issue, a new deck plate system including the UHPC-infilled top chords shown in Figure 3 is proposed. In this system, the top chord mainly consists of a hollow shape steel member and infilled materials with high compression capacity such as high-strength mortar and UHPC (ultra-high-performance concrete) [12]. This combination can increase the buckling resistance of the truss girder when subjected to compression force induced by the bending moment. It is well known that the steel–concrete composite system has the advantages of increased structural strength and minimized self-loads [13,14]. In contrast to the conventional truss girder shown in Figure 3b, the flexural neutral axis of the proposed truss girder system can be located in the middle of the floor depth. Thus, balanced flexural failure can occur, and this can guarantee more optimized use of structural materials for the top and bottom chords. With this advantage, the proposed deck plate system can be applied to a span much larger than 4 m, which is the limit span length for most conventional steel wire-integrated deck plate systems, even if no temporary supports are provided at the construction stage.

Figure 2. Shape of the conventional steel wire-integrated deck plate.
Figure 3. Main idea of the UHPC-infilled top chord.

Figure 4a shows the isometric view of the proposed deck plate system, and its front view is given in Figure 4b. Among the possible shapes of top chords shown in Figure 3a, the lipped C-section is used in this study mainly because it is easy to fill compressive material into the open section, and the existence of the lip is helpful to integrate the infilled material into the C-section. Furthermore, a series of inner shear connectors are installed inside the steel top chord to prevent the separation of the infilled material from the C-section, similar to the shear connector developed at the interface between the concrete foundation and steel piles [15]. This device can guarantee safe load transfer from the upper structure to a group of foundation piles utilized for strengthening weak soil [16,17], and exhibited excellent structural performance under the application of lateral load [18], as well as axial load [19]. In addition, UHPC is used as the infilled material inside the top chord since it retains higher compressive strength and modulus of elasticity than most of normal concrete materials [20,21].

This paper investigates the bending capacity of the proposed deck plate system at the construction stage by performing a four-point bending test on seven specimens with a net span of 4.6 m. Test parameters include the type of top chords, the type and amount of infilled materials, and the existence of inner shear connectors. The effects of these parameters on the failure modes and peak strengths of the test specimens are analyzed and discussed. In addition, a theoretical strength estimation process is developed for the proposed deck plate system. The strength values calculated by the estimation process were compared with the test results and analyzed.

The outline of this paper is as follows. Following the introduction, Section 2 provides the details of the experimental program such as the test specimens, testing equipment, and test procedure to investigate the structural performance of the proposed deck plate
system. In Section 3, the comparison between the results of the test and analysis is performed. In Section 4, the process of estimating theoretical strength for each failure type is suggested, and the estimated values by this process are compared with the test results. Finally, the summary and concluding remarks are provided in Section 5.

![Diagram of the proposed deck plate system](image)

**Figure 4.** Shape of the proposed deck plate system.

2. Experimental program

2.1. Test Specimens

In this study, a four-point bending test was performed on seven test specimens for the evaluation of the flexural strength of the proposed deck plate system at the construction stage. The shape and dimensions of the test specimen are illustrated in Figure 5. The length, height, and width of all the test specimens are 5000 mm, 170 mm, and 600 mm, respectively. The thickness of the floor deck plate is 0.5 mm, and D13 and D6 (deformed) steel bars were used for the bottom chord and steel lattice, respectively, in all specimens. As mentioned in the introduction, the net span length of the test specimens is almost the maximum that can be applied in construction.
Test parameters include the type of top chords, the type of infilled materials, existence of inner shear connectors, and amount of infilled materials. The cross-sectional shapes of the top chords used in the test specimens are illustrated in Figure 6. Three different types of the shapes were used such as the conventional type, non-infilled type, and infilled type, as shown in the figure. The conventional type (D13 steel bar) is used in the typical steel wire-integrated deck plate system, while the other two with the lipped C-section are used only in the proposed deck plate system. In the infilled-type top chord, two types of materials are infilled such as high-strength mortar and UHPC. The steel inner shear connectors shown in Figure 4 were installed in infilled-type top chord test specimens to guarantee the composite behavior between the lipped C-section and infilled material, and the distance between each inner shear connector was 500 mm. Four different amounts of the infilled material were used in the test specimens, which were 0%, 20%, 40%, and 100%. Among these, the last three cases are illustrated in Figure 7, and correspond to the infilled material length \( (l_0) \) of 1000 mm, 2000 mm, and 5000 mm, respectively. This parameter was introduced to investigate the effect of stiffness and strength increase by the use of infilled material, and the infilled material was located from the center of the span, in which a large bending moment is distributed. The details of the seven specimens are provided in Table 1, and the identification of each test specimen is given in Figure 8.
Figure 7. The amounts of infilled material used in the test specimens.

Figure 8. Specimen identification.

Table 1. Summary of test specimens.

| Specimen | Shape of top chord (unit: mm) | Type of infilled material | Existence of inner shear connector | Amount of infilled material |
|----------|------------------------------|--------------------------|------------------------------------|---------------------------|
| D13-X    | D13 rebar                    | N/A                      | N/A                                | N/A                       |
| CX-X0    |                             |                          |                                    | 0%                        |
| CM-D100  |                             | Mortar (GP400)           | Yes                                | 100%                      |
| CU-D20   | Lipped C-section (C-60x30x10x2) |                      | Yes                                | 20%                       |
| CU-D40   |                             | UHPC                     |                                    | 40%                       |
| CU-D100  |                             |                          | No                                 | 100%                      |
| CU-X100  |                             |                          |                                    |                           |

The yield strength ($F_y$) of the D13 steel bar used in the top and bottom chords is 525.4 MPa. The lipped C-section used in the top chord and lattice retain the yield strength values of 320.4 MPa and 500.0 MPa, respectively. These values were measured per ASTM A370 [22]. The high-strength mortar used as the infilled material is a factory produced product (GP400), and its measured compressive strength is 60.0 MPa. The mix proportion of the UHPC [23] is provided in Table 2, and its compressive stress–strain curve shown in Figure 9 was measured per ASTM C39 [24]. Figure 9 shows the compressive stress–strain curves of three UHPC cylindrical specimens, which increases almost linearly up to the peak value, unlike general concrete.

Table 2. Mix proportion of UHPC. (wt. % of cement).

| W/C   | Cement          | 1    |
|-------|-----------------|------|
| 0.23  | Slica sand      | 1.1  |
|       | Crushed quartz  | 0.35 |
|       | Steel fiber (2%) | 0.04 |
The compressive yield stress of UHPC used in this study was calculated based on recommendations by both AFGC-SETRA [25] and JSCE [26,27]. Figure 10 shows the compressive stress–strain material model suggested by the two standards. The compressive strength of UHPC is estimated based on the material model given in Figure 10. The average value of the maximum stresses of the three stress–strain curves shown in Figure 9 is 131.6 MPa. Based on this, the values of $f_{ck}$ estimated by AFGC-SETRA and JSCE are 109.7 MPa and 125 MPa, respectively. Similarly, the values of $\gamma_c$ estimated by the two standards are 1.5 and 1.3, respectively. Accordingly, the compressive strength values of the two standards can be computed as 62.2 MPa and 81.7 MPa, respectively. In this paper, the average of the two values, equal to 71.9 MPa, is used as the compressive strength of the UHPC material. Since UHPC is filled only inside the lipped C-section, its tensile behavior is not considered.

The values of the elastic modulus and compressive stress are summarized in Table 3 for the two types of infilled material such as high-strength mortar and UHPC. They are used when estimating the theoretical strengths of the test specimens, which are discussed in Section 4 in details. In the case of UHPC, these values are calculated based on the compressive stress–strain curve shown in Figure 9. In the case of mortar, the elastic modulus of the high-strength mortar is calculated by substituting the nominal compressive strength into the elastic modulus calculation formula by ACI 318-08 ($E_c = 4700 \sqrt{f'_{ck}}$) [28].
| Infilled material       | Elastic modulus (GPa) | Compressive strength (MPa) |
|------------------------|-----------------------|----------------------------|
| High-strength mortar   | 36.4                  | 60.0                       |
| UHPC                   | 40.0                  | 71.9                       |

2.2. Testing Equipment and Procedure

In this study, a four-point bending test was conducted to investigate the bending behavior of the proposed deck plate system at the construction stage as shown in Figure 11. This type of test is suitable for examining the peak strength and failure mode of the proposed system at ultimate stage. In some other research works such as Kim et al. [6], Shin et al. [7], Lee et al. [29], and Kim et al. [30], the distributed load is applied to deck plate test specimens by adding a bundle of steel plates at each loading step. This approach is preferable if the serviceability evaluation of the deck plate system is of main research interest. The support type of the test specimen is a roller, and this allows the displacement in the longitudinal direction, while constraining the one in the vertical direction.

In order to prevent localized failures from occurring before bending failure of the test specimen, two types of details were introduced to the test specimen and test setup. The first one is the end reinforcement details illustrated in Figure 11a, which consist of the N-shaped bar and end reinforcement plate. The former is a rebar of D10 bent several times and welded to the bottom of the lipped channel to enhance shear resistance at the support region. The latter was attached to the lip of the C-section to prevent its opening from being widened. The second is the details at load application to the test specimen, which can distribute the concentrated load over some area and prevent stress concentration at the bottom steel plate. They were carefully designed to induce bending failure of the specimens and evaluate their peak bending strengths. They consist of various components such as the steel plate, wood blocks, steel rectangular pipes, rebars, and epoxy resin, similar to those shown in the work of Lee et al. [31].

Figure 12 shows the location of the support, deflection measurement, and loading range on the bottom plate. In all of the test specimens, the length of the span is 4600 mm, and the distance from the loading point to the support is 1600 mm. Load was applied to each specimen at a rate of 15 kN/min using a hydraulic cylinder with a maximum capacity of 2000 kN. The force generated by the hydraulic cylinder was transmitted to the center of a steel frame, which was installed to apply two-point loading to the beam specimen. The distance between the two loading points is 1400 mm. The magnitude of the loading was measured by a load cell attached to the bottom of the cylinder, and the vertical displacement was monitored by a linear variable differential transducer (LVDT) installed at the midspan of the beam.
Figure 11. Test setup.

Figure 12. Locations of load application and LVDT measurement.
3. Test Results and Analysis

This section discusses the general behavior of the test specimens and investigates the effects of the test parameters such as the amount of infilled material, its type, and the existence of inner shear connectors on the peak strength and failure mode of the specimen.

Figure 13 shows the load–deflection curves of all seven test specimens, and Table 4 summarizes their test results, consisting of the peak load, initial stiffness, failure mode, and main failure mechanism. It can be observed from the table that most of the proposed deck plate specimens have values of peak strength and initial stiffness at least two-times greater than those of the conventional deck plate specimen, confirming the effectiveness of the proposed system. Furthermore, it can be seen from the figure that the load–deflection curves of the specimens with no or partially infilled material such as specimens D13-X, CX-X0, CU-D20, and CU-D40 are different from those with fully infilled material such as CM-D100, CU-D100, and CU-X100. In the case of the former, the load–deflection curve decreases rather gradually after reaching the peak strength, while the decrease is more dramatic in the case of the latter.

![Figure 13. Load–deflection curves of the test specimens.](image)

Table 4. Test results.

| Specimen | Peak load (kN) | Initial stiffness (kN/mm) | Failure mode | Main failure mechanism |
|----------|----------------|---------------------------|--------------|-----------------------|
| D13-X    | 16.35          | 0.319                     | Buckling of D13 rebar |
| CX-X0    | 36.12          | 0.641                     | Buckling of lipped C-section | Bending |
| CU-D20   | 33.85          | 0.601                     |              |                       |
| CU-D40   | 42.33          | 0.753                     |              |                       |
| CM-D100  | 55.74          | 0.724                     | Buckling of lattice member | Shear |
| CU-D100  | 63.42          | 0.861                     |              |                       |
| CU-X100  | 58.68          | 0.758                     |              |                       |

This characteristic can be identified as the deformed shapes of the two types of specimens, as shown in Figures 14 and 15, respectively. They indicate that the former failed by the buckling of top chord members such as the D13 rebar and lipped C-section, while
the latter by the buckling of D6 lattice members. If the lattice of the truss girder buckles, the effective depth of the truss girder is reduced significantly, resulting in a dramatic decrease in the bending capacity of the truss girder. As a result, the load–deflection curve can drop dramatically after reaching its peak value. The failure process of the two types of specimens can be explained by the bending and shear failure mechanisms, respectively. The theoretical strengths of the proposed deck plate system can also be derived based on the two failure mechanisms and are discussed in more detail in Section 4.

Figure 14. Deformed shapes of the specimens that failed by bending failure mechanism.
Figure 15. Deformed shapes of the specimens that failed by shear failure mechanism.

Table 5 lists the values of the peak load and initial stiffness for the four specimens of which the infilled material amount increases from 0 % to 100 %. The relative ratios of the peak loads and initial stiffness values with respect to the specimen with no infilled material (CX-X0) are also given for each specimen in the table. It can be noted from the table that, with an increasing amount of infilled material, the relative ratios of the peak load and initial stiffness increase up to 76 % and 34 %, respectively. This indicates that the increase in the amount of infilled material is helpful to improve the strength and stiffness of the proposed deck plate system.

Interestingly, the specimen with an infilled material amount of 20 % (CU-D20) does not show much improvement in the strength and initial stiffness, if compared to specimen CX-X0. This seems to happen because the range of infilled material is smaller than the distance of the two point loadings as can be seen from Figures 7a and 12, and thus some portion of the lipped C-section without the infilled material is subjected to the maximum bending moment between the two loading points. Consequently, it is recommended that the range of infilled material should be greater than the two loading point distances.

Table 5. Values of the peak load and initial stiffness of four specimens to clarify the effect of the amount of infilled material.

| Specimen | Peak load (kN) | Ratio of peak loads | Initial stiffness (kN/mm) | Ratio of initial stiffness values |
|----------|----------------|---------------------|--------------------------|----------------------------------|
| CX-X0    | 36.12          | 1.00                | 0.641                    | 1.00                             |
| CU-D20   | 33.85          | 0.94                | 0.601                    | 0.94                             |
| CU-D40   | 42.33          | 1.17                | 0.753                    | 1.17                             |
| CU-D100  | 63.42          | 1.76                | 0.861                    | 1.34                             |

Table 6 lists the values of the peak load and initial stiffness for the two specimens in which different types of infilled material such as high-strength mortar (specimen CM-D100) and UHPC (specimen CU-D100) were used. The relative ratios of the peak loads and initial stiffness values with respect to specimen CM-D100 are also given for each specimen in the table. It can be observed from the table that specimen CU-D100 has a peak load and initial stiffness 14 % and 19 % higher than specimen CM-D100, respectively. This indicates that the use of UHPC as the infilled material is more effective at increasing the strength and initial stiffness of the proposed deck plate system than the high-strength mortar.

Table 6. Values of the peak load and initial stiffness of two specimens to clarify the effect of infilled material type.

| Specimen | Peak load (kN) | Ratio of peak loads | Initial stiffness (kN/mm) | Ratio of initial stiffness values |
|----------|----------------|---------------------|--------------------------|----------------------------------|
| CM-D100  |                |                     |                          |                                  |
| CU-D100  |                |                     |                          |                                  |
Table 7 lists the values of the peak load and initial stiffness for the two specimens with (specimen CU-D100) and without (specimen CX-D100) inner shear connectors. The relative ratios of the peak loads and initial stiffness values with respect to specimen CX-D100 are also given for each specimen in the table. It can be observed from the table that specimen CU-D100 has a peak load and initial stiffness 8% and 14% higher than specimen CX-D100, respectively. This confirms that the use of the inner shear connectors is helpful to integrate the lipped-C section and infilled material, and thus can increase the strength and initial stiffness of the proposed deck plate system.

| Specimen | Peak load (kN) | Ratio of peak loads | Initial stiffness (kN/mm) | Ratio of initial stiffness values |
|----------|----------------|---------------------|----------------------------|---------------------------------|
| CU-D100  | 63.42          | 1.14                | 0.861                      | 1.19                            |
| CM-D100  | 55.74          | 1.00                | 0.724                      | 1.00                            |

4. Theoretical Strength Estimation Process of the Proposed Deck Plate System

In this section, theoretical strength estimation of the proposed deck plate system depending on its failure mode is discussed. As already discussed in Section 3, the specimen may fail either by bending or the shear failure mechanism. As a result, two theoretical strength estimations based on the two mechanisms are computed, and the smaller of the two strength values becomes the governing one. The strength values calculated by the estimation are compared with the test results and analyzed. This approach is largely based on the allowable stress design concept, and the proposed system fails when internal stress in the main components such as the lipped C-section, infilled material, bottom chord, and lattice reaches any critical value. Further details of this process are discussed in the following subsections.

4.1. Theoretical Strength Estimation Based on Bending Failure Mechanism

If a test specimen fails by the application of excessive bending moment, as observed in the specimens with relatively small or no amount of infilled material in the top chord such as D13-X, CX-X0, CU-D20, and CU-D40 in Section 3, its theoretical strength can be determined by the process shown in Figure 16. Its first step is to determine the neutral axis ($Y_{NA}$) of the deck plate cross-section by using Equation (1). This equation is derived based on the linear distribution of strain with respect to the neutral axis.
Figure 16. Theoretical strength estimation process based on bending failure mechanism.

\[
Y_{N.A} = \frac{A_t h_t + A_b h_b + n A_I h_I}{A_t + A_b + n A_I} (n = \frac{E_I}{E_s})
\]

(1)

where, \(A_t\), \(A_b\), and \(A_I\) are the cross-sectional areas of the lipped C-section, bottom chord, and infilled material, respectively. Similarly, \(h_t\), \(h_b\), and \(h_I\) are the distances from the bottom deck plate to the centroid of the lipped C-section, bottom chord, and infilled material, respectively. \(E_s\) and \(E_I\) are the elastic modulus of steel and infilled material, respectively, and \(n\) is the relative ratio of their elastic moduli. In the case without infilled material, both \(A_I\) and \(n\) become zero in the above equation.

In the next step, the normal stress distribution is determined by considering the critical strain for the given deck plate cross-section, as shown in the flow chart of Figure 16. This process is affected by the existence of infilled material, and thus, it is necessary to consider two cases without and with the infilled material designated as Cases A and B, respectively.

Figure 17 shows the normal strain distribution in the case with no infilled material inside the lipped C-section. In the figure, \(\varepsilon_t\) and \(\varepsilon_b\) indicate the normal strains at the centroids of the lipped C-section and bottom chord, respectively. As indicated in the figure, the bottom chords are subjected to tensile force while the lipped C-section is subjected to...
compressive force. Consequently, the two possible failure modes of the proposed system are the tensile yielding of the bottom chord and buckling of the lipped C-section. The former occurs if the tensile stress at the bottom chord reaches the yield strength (Case A-1) and the latter if the compressive stress at the lipped C-section reaches its critical buckling stress (Case A-2). The values of $\varepsilon_t$ and $\varepsilon_b$ and their corresponding failure modes are summarized in Table 8. In the table, $F_{y,b}$ and $f_{cr,t}$ are the yield strength and critical buckling stress of the bottom chord and lipped C-section, respectively. The critical buckling stress is expressed by Equations (2) and (3) and taken from the works of Abebe et al. [32], Narvydas [33], and Florescu et al. [34].

Table 8. Strain values for possible bending failure modes in case without infilled material.

| Case  | Failure mode                     | Strain value     |
|-------|----------------------------------|------------------|
| A-1   | Yielding of bottom chord         | In elastic range |
|       | $F_{y,b}/E$ ($=\varepsilon_{y,b}$) |                  |
| A-2   | Buckling of lipped C-section     | $f_{cr,t}/E$ ($=\varepsilon_{cr,t}$) | In elastic range |

![Strain distribution of the deck plate cross-section without infilled material.](image)

where $\lambda$ is the slenderness ratio of the compression member, $F_y$ is its yield strength, $K$ is its effective length factor, $L$ is the compression member length, $r$ is its radius of gyration of the area, $\lambda_p$ is the limiting slenderness ratio, $n$ is the safety factor, and $E$ is its elastic modulus. The determination of the effective length factor $K$ is discussed in detail in Section 4.3.

Once the values of $\varepsilon_t$ and $\varepsilon_b$ are determined, the axial resultant forces corresponding to them can be computed using Equation (4) below.

$$C_t = \varepsilon_t E_t A_t, \quad C_b = \varepsilon_b E_b A_b, \quad T = \varepsilon_b E_b A_b$$

where $C_t$ and $C_b$ are the resultant compressive force of the lipped C-section and infilled material, respectively, and $T$ is the resultant tensile force of the bottom chord. In the case without infilled material, $C_b$ becomes zero in the above equation. The procedure to deter-
mine the normal stress distribution of the deck plate cross-section without infilled material is illustrated in the form of a flow chart in Figure 18. It corresponds to the process from ₋ through ₋ shown in Figure 16. Finally, the bending moment strength of the deck plate cross-section ($M_n$) can be computed by using the following equation:

$$M_n = C_I (h_t - Y_{N,A}) + C_I (h_b - Y_{N,A}) + T(Y_{N,A} - h_b)$$  

(5)

Figure 18. Procedure to determine the strain distribution in case with no infilled material.

Figure 19 shows the normal strain distribution of the deck plate cross-section in the case with infilled material inside the lipped C-section. In the figure, $\varepsilon_t$, $\varepsilon_I$, and $\varepsilon_b$ indicate the normal strains at the centroids of the lipped C-section, infilled material, and bottom chord, respectively. As is similar to the case with no infilled material, the possible failure mode of the bottom chord is its tensile yielding (Case B-1). In contrast, the top chord consisting of the lipped C-section and infilled material does not easily fail by buckling, as it retains high composite bending stiffness compared to the case with no infilled material. As a result, its possible failure modes are the compressive yielding of the infilled material (Case B-2) and lipped C-section (Case B-3), and these failures occur if the normal stress reaches the compressive yield strength corresponding to each of the two cases, respectively. The values of $\varepsilon_t$, $\varepsilon_I$, and $\varepsilon_b$ and their corresponding failure modes are summarized in Table 9.
Figure 19. Strain distribution of the deck plate cross-section with infilled material.

Table 9. Strain values for possible bending failure modes in case with infilled material.

| Case | Failure mode                      | Strain value                      |
|------|-----------------------------------|-----------------------------------|
|      |                                   | $\varepsilon_t$                   | $\varepsilon_I$                   | $\varepsilon_b$ |
| B-1  | Yielding of bottom chord          | In elastic range                  | $F_{y,b}/E_s (= \varepsilon_{y,b})$ |
| B-2  | Yielding of infilled material     | In elastic range                  | $F_{y,I}/E_I (= \varepsilon_{y,I})$ | In elastic range |
| B-3  | Yielding of lipped C-section      | $F_{y,t}/E_s (= \varepsilon_{y,t})$ | In elastic range                  |

Once the values of $\varepsilon_t$, $\varepsilon_I$, and $\varepsilon_b$ are determined, the axial resultant forces corresponding to them can be computed using Equation (4). The procedure to determine the normal stress distribution of the deck plate cross-section with infilled material is illustrated in detail in Figure 20. It corresponds to the process from Ⓑ through Ⓒ shown in Figure 16. Finally, the bending moment strength of the deck plate cross-section ($M_n$) can be computed using Equation (5), as in the case without infilled material.
4.2. Theoretical Strength Estimation Based on Shear Failure Mechanism

Test specimens may fail by the buckling of lattice members if they have a relatively large amount of infilled material in the top chord, as observed in specimens CM-D100, CU-D100, and CU-X100 in Section 3. In this case, the transmission of internal shear force plays an important role in determining the theoretical strength of the proposed system, and the process of its strength determination is illustrated in Figure 21a. Since the internal shear force is the largest at the support under the given loading condition, the lattice member nearest the support fails by buckling. The critical buckling stress of the lattice member \( f_{cr,L} \) can be computed using Equations (2) and (3) in Section 4.1. Then, its axial force strength \( P_L \) can be calculated using

\[
P_L = A_L \times f_{cr,L}
\]

where \( A_L \) is the cross-section area of the lattice member.

From the geometry shown in Figure 21b, the internal axial force of the lattice member can be decomposed into its vertical and horizontal components. Among them, only the vertical component is equilibrated by the support reaction force. Finally, the shear strength of the deck plate cross-section \( V_n \) can be computed using Equation (7).

\[
V_n = n_{trans} \cdot 2P_L \sin \theta = 2n_{trans} \frac{L_n}{(h_t - h_b)}
\]

where \( L_n \) is the length of the lattice member located between the top and bottom chords, \( \theta \) is the angle of the lattice member with respect to the horizontal line, and \( n_{trans} \) is the number of truss girders utilized in the test specimen. As already mentioned at the beginning of Section 4, among the two strength estimations given by Equations (6) and (8), the smaller one is the theoretical strength of the proposed deck plate system.
4.3. Comparison between the Results of the Test and Proposed Estimation Process

In this section, the test results are compared with the estimations by the theoretical strength equations discussed in Sections 4.1 and 4.2, and its accuracy is verified. In addition, appropriate values of the effective length factor $K$ are proposed, and they are included in the critical buckling stress equation stated by Equations (2) and (3).

Table 10 provides the theoretical strengths estimated by the process in Sections 4.1 and 4.2 ($P_e$), peak strengths by test ($P_t$), and their relative ratios ($P_e / P_t$) for each of the seven test specimens. The theoretical strength ($P_e$) listed in Table 8 is the smaller of the two load values ($P_{eb}$ and $P_{es}$) calculated using Equations (8) and (9), given below.

\[
P_{eb} = \frac{4M_n}{(l_n - a)}
\]
\[ P_{es} = 2V_n \]  

where \( l_o \) is the net span length of the specimen, \( l_i \) is the infilled material length, and \( a \) is the distance between the two load application points, as shown in Figure 22. Here, \( M_n \) and \( V_n \) are the theoretical strengths of the proposed deck plate system estimated by the process discussed in Sections 4.1 and 4.2, respectively. In the computation of \( M_n \) and \( V_n \), the effective length factor \( K \) is assumed to be 1.0, which is the theoretical value for hinge boundary conditions at both ends [35].

![Figure 22. Bending moment diagram of the test specimen.](image)

In the case of partially infilled specimens such as CU-D20 and CU-D40, both of the theoretical moment strengths with and without infilled material, denoted by \( M_{no} \) and \( M_{nx} \), respectively, need to be calculated. Then, the smaller of the two values \( (P_{bo} \) and \( P_{bx} \), computed by Equations (10) and (11), respectively, becomes the theoretical strength estimation of the proposed deck plate system \( (P_{es}) \).

\[ P_{bo} = \frac{4M_{no}}{l_n - a} \]  

\[ P_{bx} = \frac{4M_{nx}}{l_n - l_i} \]  

If \( l_o \) is less than \( a \) in Equation (11), the former is replaced by the latter, as in the case of specimen CU-D20. The smallest value among the three strength values \( P_{bo}, P_{bx} \) and \( P_{es} \) is represented by bold font in Table 10.

From the results of Table 10, several interesting observations can be made as follows. First, the theoretical strength estimation indicates that the four specimens with no or partially infilled material, such as specimens D13-X, CX-X0, CU-D20, and CU-D40, fail by the buckling of top chords, which is the D13 rebar or lipped C-section in this case. This result coincides with the deformed shapes of these specimens at the final stage shown in Figure 14. Furthermore, in both CU-D20 and CU-D40 specimens, \( P_{bx} \) is smaller than \( P_{bo} \), indicating that the two specimens fail by buckling of the lipped C-section at the end location of infilled material, not at the section with infilled material. This prediction also agrees well with the test results discussed in Section 3. All of these results confirm the effectiveness of the estimation process in Sections 4.1 and 4.2.

Second, the theoretical strength estimation indicates that two specimens with infilled material such as CU-D100 and CU-X100 fail by the buckling of D6 lattice members, which again coincides with the test results shown in Figure 15. In contrast, specimen CM-D100 fails by yielding of the bottom chord, which is different from the test result. This seems related to the accurate estimation of the effective length factor included in the critical buckling stress equations (2) and (3) and will be discussed later in this section.
Third, the ratio of the theoretical strength and peak test strength ranges from 98.0 % to 106.8 % except in specimen D13-X. This is a very accurate result considering that the support conditions of the compression components such as the D13 rebar, lipped C-section, and D6 lattice member were assumed to be a hinge at both ends in the computation of the critical buckling stress. As can be seen from the buckling shapes of the compression members shown in Figures 14 and 15, their support conditions cannot be exactly a hinge, and thus more appropriate values need to be estimated.

The relative ratio of the theoretical and peak test strengths is plotted with respect to the effective length factor in Figure 23. These plots are created by categorizing the test specimens into three groups depending on the types of specimens, such as the conventional truss deck plate specimen, proposed deck plate specimens with no or partially infilled material, and proposed deck plate specimens with fully infilled material. The main compression members of the three groups, in which buckling failure occurred, are the D13 rebar, lipped C-section, and D6 lattice member, respectively.

![Figure 23](image_url)

Figure 23. Relative ratio plot of theoretical and test strengths with respect to the effective length factor.

It can be seen from the figure that the change in the relative ratio with respect to the effective length factor is more significant in Figure 23a,c than in Figure 23b. This is mainly because the slenderness ratios of the second group specimens are smaller than those of the first and third groups as can be observed from Figure 24, which shows the slenderness ratio ranges for the main compression components of the three groups in the critical buckling stress curve. This behavior is mainly attributed to the fact that the ratio of the moment of inertia to the cross-sectional area of the lipped C-section (= I/A) is much larger than those of the other two compression members, such as the D13 rebar and D6 lattice. As a result, the selection of an appropriate effective length factor can have a more significant
effect on the estimated theoretical strength in the case of the first and third groups than in the second group.

![Slenderness ratio ranges of the three main compression members in the critical buckling stress curve.](image)

**Figure 24.** Slenderness ratio ranges of the three main compression members in the critical buckling stress curve.

Table 11 lists the results of the theoretical strength estimation calculated by using the modified effective length factors, which are 1.359 and 1.018 for the D13 rebar and D6 lattice, respectively. They are calculated such that the average of the theoretical strengths is equal to that of the test strengths for each group of test specimens. The value of 1.0 is used for the effective length factor of the second group as the theoretical strength is not sensitive to its selection. The smaller of the two theoretical strength values $P_e$ and $P_s$ is denoted by bold font in the table. It can be noticed from the results of the table that the failure modes theoretically predicted coincide well with those of the test for all seven specimens. The estimation of appropriate effective length factors is based on a limited number of test data, and thus it may be improved by performing more tests on the proposed deck plate system.

Table 12 lists the estimated theoretical strengths and corresponding failure modes for specimens with different amounts of infilled material, in which the results of specimens CU-D50 and CU-D60 are added to those of Tables 10 and 11. It can be noted from the table that the test specimen fails by the buckling of lipped C-section up to the 50% of infilled material amount, but fails by the buckling of lattice member if the percentage is greater than or equal to the 60%. Consequently, the critical value is somewhere between 50 and 60%, and it is recommended to use a larger amount of infilled material than the critical value for economic feasibility of the proposed deck plate system.

### Table 11. Results of the theoretical strength estimation calculated by using the modified effective length factors.

| Specimen         | Theoretical results | Test results | Strength ratio ($P_e / P_l$) |
|------------------|---------------------|--------------|----------------------------|
|                  | Theoretical strength (kN, $P_e$) | Failure mode | Peak strength (kN, Failure mode $P_l$) |             |
|                  | Bending failure mechanism ($P_{eb}$) | Shear failure mechanism ($P_{es}$) | Buckling of D13 rebar | Buckling of D13 rebar |         |
| D13-X            | 16.35               | 59.89        | 16.35                      | 1.000        |
| CX-X0            | 36.14               |              | 36.12                      | 1.001        |
| CU-D20           | 36.14               |              | 33.85                      | 1.068        |
| CU-D40           | 44.48               |              | 42.33                      | 1.051        |
| CM-D100          | 62.03               |              | 58.51                      | 1.024        |
| CU-D100          | 62.15               |              | 63.42                      | 0.944        |
| CU-X100          | 62.15               |              | 58.68                      | 1.021        |
Table 12. Estimated theoretical strengths and corresponding failure modes for specimens with different amount of infilled material.

| Specimen | Theoretical strength (kN, \(P_e\)) | Failure mode       |
|----------|-------------------------------------|--------------------|
|          | Bending failure mechanism           | Shear failure      |
|          | w/o infilled material \((P_{bx})\) | mechanism \((P_{bo})\) |
| CX-X0    | 36.14 w/ infilled material \((P_{bs})\) | 59.89              |
| CU-D20   | 36.14                               | 59.89              |
| CU-D40   | 44.48                               | 59.89              |
| CU-D50   | 55.24                               | 62.15              |
| CU-D60   | 72.50                               | 59.89              |
| CU-D100  | N/A                                 | 59.89              |

5. Conclusions

In this study, a new deck plate system, of which top chords are infilled with high-performance cement-based material such as high-strength mortar and UHPC, was proposed. The bending capacity of the proposed deck plate system at the construction stage was investigated by performing a four-point bending test on seven specimens with a net span of 4.6 m. The test parameters included the type of top chords, type and amount of infilled material, and existence of inner shear connectors. The load versus displacement curves were plotted for all of the specimens, and their failure modes were identified. In addition, a theoretical strength estimation process was developed for the proposed deck plate system. The strength values calculated by the estimation process were compared with the test results and analyzed. The main conclusions of this paper are as follows:

1) The proposed deck plate system retains a much higher peak strength and initial stiffness than the conventional steel wire-integrated deck plate system. The test results indicate that the values of the peak strength and initial stiffness of the proposed deck plate specimens are at least two-times greater than those of the conventional deck plate specimen (D13-X), confirming the effectiveness of the proposed system.

2) The theoretical strength estimation process developed in this study can predict the failure modes and peak strength of the proposed deck plate system accurately. With the use of the effective length factor of 1.0, it was able to accurately predict the failure modes of all test specimens except specimen CM-D100, and the maximum relative error of peak strength estimation is only approximately 7%.

3) The use of calibrated effective length factors can improve the theoretical strength estimation of the proposed deck plate system. For example, it was able to reduce the relative error in the peak strength of D13-X specimen by approximately 43% and to predict the failure modes of all the test specimens accurately. However, this calibration was based on a limited number of test data, and thus, it may be improved by performing more tests on the proposed deck plate system.

4) The peak strength and initial stiffness of the proposed deck plate system increase with the increasing amount of infilled material. The comparison between the test results of the two specimens with no (CX-X0) and fully infilled material (CU-D100) shows that the peak strength and initial stiffness of the latter are 76% and 34% higher than those of the former, respectively.

5) The buckling of compression components is the main failure mode of the proposed deck plate system at construction stage. The test specimens with no or partially infilled material failed by the buckling of top chords, while those with fully infilled material by the buckling of lattice members. The transition from the former to the
latter occurs somewhere between the infilled amounts of 50% and 60%. It is recommended to use a larger amount of infilled material than the transition value for economic feasibility of the proposed deck plate system.

6) The use of UHPC as infilled material, instead of high-strength mortar, and inner shear connectors can increase the peak strength and initial stiffness of the proposed system. The test results show that that specimen infilled with UHPC (CU-D100) has a peak load and initial stiffness 14% and 19% higher than the one infilled with high-strength mortar (CM-D100), respectively. Similarly, the specimen with inner shear connectors (CU-D100) has a peak load and initial stiffness 8% and 14% higher than the one without them (CX-D100), respectively.

Currently, an efficient three-dimensional nonlinear finite element model is under development for the test specimens considered in this study, and the fully composite behavior of the proposed deck plate system after concrete curing is also being investigated.

Author Contributions: Conceptualization, D.-J.K. and Y.-H.K.; methodology, S.-G.H., H.-J.S. and D.-J.K.; validation, H.-J.S. and D.-J.K.; formal analysis, H.-J.S.; investigation, H.-J.S. and D.-J.K.; resources, S.-G.H. and Y.-H.K.; data curation, H.-J.S.; writing—original draft preparation, H.-J.S. and D.-J.K.; writing—review and editing, D.-J.K.; visualization, H.-J.S.; supervision, D.-J.K. and Y.-H.K.; project administration, D.-J.K.; funding acquisition, D.-J.K. All authors have read and agreed to the published version of the manuscript.

Funding: This work was supported by a National Research Foundation of Korea (NRF) grant funded by the Korean government (Ministry of Science, ICT & Future Planning) (No. 2020R1A2C1014806).

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The data presented in this study are available upon request.

Conflicts of Interest: The authors declare no conflict of interest.

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