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A Study on the Design of the Top Flange of a Modular T-Girder Bridge Using the Limit State Design Method

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Abstract: This study involved applying a design code change on a modular bridge design. The top flange of a modular T-girder bridge was examined by the Korean Highway Bridge Design Code Limit State Design (2015) and was compared with the Korean Highway Bridge Design Code (2010) in terms of the stability under the bending moment. In addition, the cross-sectional height and reinforcement amount were re-designed to obtain a safety factor similar to the original code. The reinforcement arrangement and development of the transverse joints were examined in the section considered. The result indicated that the application of the Korean Highway Bridge Design Code Limit State Design (2015) increased the bending moment safety factor and decreased the width of the transverse joints. The results of the re-design with respect to a safety factor similar to that in the Korean Highway Bridge Design Code (2010) indicated that it was possible to reduce the cross-sectional height and reinforcement amount. Furthermore, based on the obtained section, the results revealed that the width of the transverse joints could be reduced by changing the arrangement of lap splices from the straight bar to the loop.

Keywords: limit state design; modular bridge; top flange; joint design

1. Introduction

In recent years, a considerable number of bridges have required replacement, repair, and strengthening owing to their degradation in performance with aging induced by deterioration, accidents, and natural disasters [1]. Maintenance and replacement of an existing bridge or construction of a new bridge is a labor-intensive involving a series of on-site fabrication processes that require a long construction period. Other problems arise, including environmental impacts and passage restrictions around the site. Accordingly, many studies have been conducted on the prefabricated bridges that can cope with various demands such as the minimization of traffic congestion and environmental impacts, cost reduction, securing site safety, and the improvement of quality and constructability [2,3].

A precast modular bridge involves the application and implementation of a plug-in technique in which a bridge is constructed by combining prefabricated standard members. An additional function can be assigned, and performance can be upgraded by replacing the module. Accelerated construction is enabled in the case of a modular bridge, and uniform quality and economic feasibility improvements are expected through the minimization of cast-in-place construction based on the partial replacement of damaged members using precast members. However, the use of a number of precast members produces joints that are disadvantageous in terms of integrated behavior, safety, and serviceability such as in the case involving the deflection of a structure. Accordingly, several studies were conducted to improve the performance of such joints [4–6].

With respect to the prompt maintenance of a bridge, a study on the accelerated construction of a bridge was conducted by installing a precast deck that could promptly replace the deteriorated deck part of a bridge and by applying cast-in-place ultra-high-performance concrete (UHPC) to the joints [7]. The performances of the joint using high-performance concrete (HPC) and UHPC were verified by applying a prestress [8,9].

Three types of joint reinforcement shapes were suggested using the shape of a joint, the type and arrangement of reinforcements, and a lap splice as the variables, and a performance test was performed [10,11]. Moreover, the longitudinal and transverse performances of a U-bar precast joint were verified [12]. Additionally, a headed bar was used to enhance the construction efficiency of the modular bridge and transfer a force sufficiently to the adjacent module with 72% smaller lap length that corresponded to approximately 152 mm [5,11].

Various shapes of joint were designed and verified as mentioned above. In the case of a precast modular bridge developed in South Korea, the width and cross section of a joint were calculated based on the development and lap splice length of a reinforcement to design a transverse joint between members. The Korean Highway Bridge Design Code (2010) is based on the strength design method. When a joint was designed based on this code, an issue with respect to a limitation of the increased self-weight, caused by the height of the cross sections, to provide a sufficient strength and wide joint width based on reinforcement splice length of criteria was observed. Thus, the bridge design specifications of AASHTO (American Association of State Highway and Transportation Officials) were reflected in the design of the reinforcement development and splice length [13–15]. However, the current design method used both the strength design method as well as AASHTO design method; thus, the design standard is ambiguous. Therefore, the re-examination of a single design standard is required to optimize the design of a precast modular bridge and to efficiently design a joint. In South Korea, the Korean Highway Bridge Design Code Limit State Design (2015) based on a rational reliability analysis was recently established to secure a design standard in accordance with the international standard system [16]. Accordingly, the structural performance analysis process was changed, and a more efficient design was obtained owing to the change in standards for the development and lap splice length of a reinforcement that was necessary to design a joint, as well as the reinforcement amount and the height of a cross section.

Therefore, in the present study, the top flange of a precast modular bridge designed based on the strength design method was re-examined using the aforementioned newly established method, and the flexural stability was investigated. Furthermore, changes in the design of the joint were compared and analyzed based on a safety factor similar to the existing safety factor.

2. Design Progress

2.1. Current Modular Bridge

A modular bridge is a bridge system constructed by assembling prefabricated standard members such that it can cope with the diverse conditions at the site. The bridge can be extended with respect to the width and longitudinal directions, and the assembly is simple. When a combination design is completed through a standard module data base and simulation, it is purchased and distributed through the standard module supply chain, and the assembly is performed on-site. A standard module is fabricated as a ready-made product of a unit module that can be assembled; thus, the girder height is uniform and the change in the cross section is minimized [15].

The modular bridge type that was used in this study was developed in South Korea to cope with a bridge with a 20 and 40 m span length, and it is a PSC (Prestressed Concrete) Bulb Tee girder type with standard spans of 30, 35, and 45 m. As shown in Figure 1, the construction of a modular

T-girder involves the following: Standard module fabrication, module transport, longitudinal module connection with prestressing, module setting, transverse lap splice, and cast-in-place joint construction.

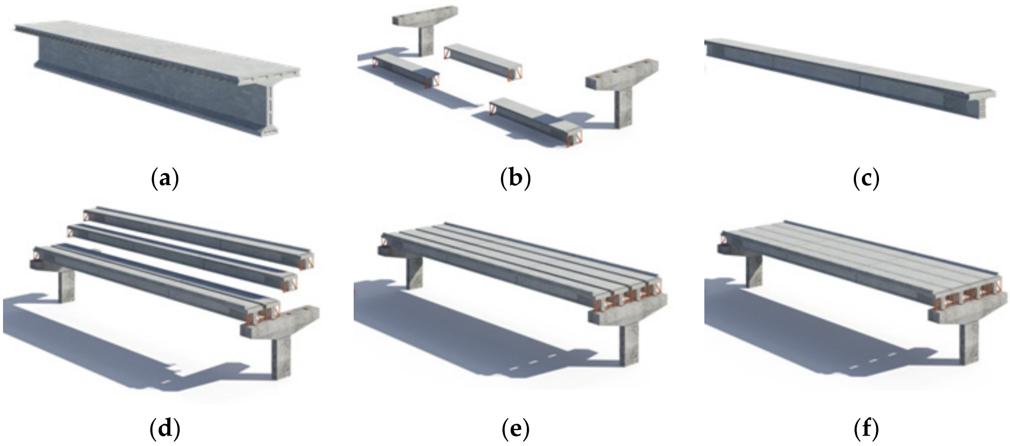


Figure 1. Concept of a modular bridge: (**a**) Standard module fabrication; (**b**) Module transport; (**c**) Longitudinal module connection with prestressing; (**d**) Module setting; (**e**) Transverse lap splice; (**f**) Cast-in-place joint.

The target transverse joint originally designed by the strength design method corresponds to the A-A section as shown in Figure 2. The beam that was considered for the design had a length of 2407 mm, a width of 1000 mm in the longitudinal direction of the bridge, and a thickness of 220 mm.

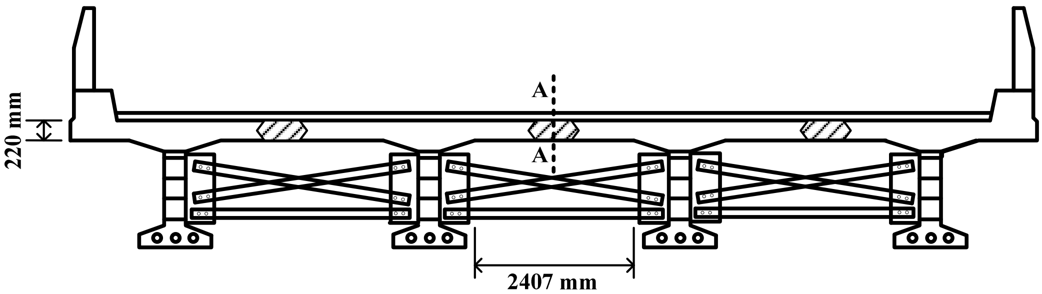


Figure 2. Schematic of the precast modular girder bridge.

As shown in Figure 3, the cross-sectional shape of the transverse joint was designed to be a rhombus such that it could resist a shear force irrespective of the direction. Seven of the D16 reinforcements were fabricated with a lap-splice length of 200 mm and a joint width was 400 mm.

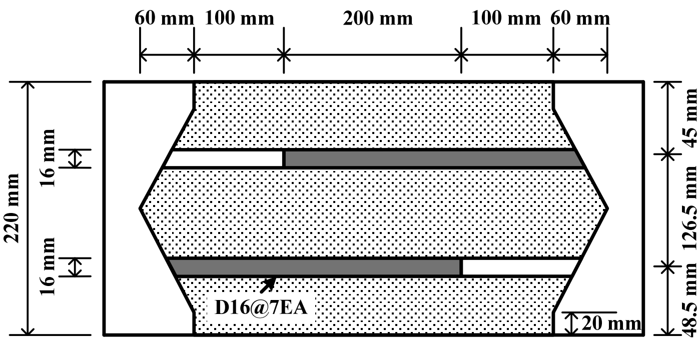
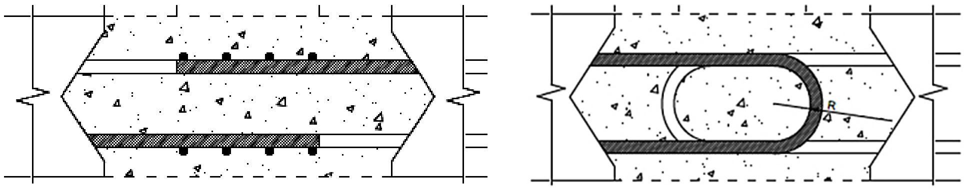


Figure 3. Detailed specification of the joint cross section.

With respect to the reinforcement arrangement of the transverse joint, a lap splice or loop (U-bar) was considered as shown in Figure 4. However, in the case of the loop, the splice length and the joint width could be decreased, but their application to a modular bridge was limited in terms of the required concrete thickness for securing a reinforcement bending radius and increased self-weight. Thus, it was finally designed as a general reinforcement lap splice. The bridge was then completed through cast-in-place construction with ultra-high-strength concrete (UHSC) (120 MPa), and a transverse prestress was not introduced.



(**a**) (**b**)

Figure 4. Reinforcement arrangement of the joint: (**a**) Lap splice; (**b**) Loop (U-bar).

Table 1 summarizes the respective design value of materials used for the strength design method.

Table 1. Design strengths of the material.

Classification	Value
Standard design strength of the concrete (MPa)	50
Average compressive strength of the concrete (MPa)	55
Average tensile strength of the concrete (MPa)	4.34
Standard tensile strength of the concrete (MPa)	3.04
Elastic modulus of the concrete (MPa)	32,902
Yield strength of the reinforcement (MPa)	400
Elastic modulus of the reinforcement (MPa)	200,000
Specific weight of the concrete (kN/m^3)	25.0
Specific weight of the pavement (kN/m^3)	23.0
Strength of the high-strength concrete of the joint (MPa)	120

2.2. Design

In this study, the top flange and transverse joint of a modular T-girder bridge designed using the Korean Highway Bridge Design Code (2010) (hereinafter, "Code 2010"), which was based on the strength design method, were re-examined using the Korean Highway Bridge Design Code Limit State Design (2015) (hereinafter, "Code 2015"). In order to re-design the height of the cross section and the reinforcement amount, the load and flexural strength were calculated. Reinforcement arrangement and the length of the development and lap-splice were also calculated to design a transverse joint, particularly in terms of joint width.

2.2.1. Load

In the case of the concrete of a joint, stress was not produced depending on the demolding after the completion of curing; thus, the self-weight could be ignored because the self-weight of the deck plate involved discontinuous surface before the transverse connection. Therefore, only the self-weight of the pavement was calculated as a fixed load. For both Code 2010 and Code 2015, the live load was calculated for the unit width of the deck plate using the following simple equation, in which the main reinforcement was perpendicular to the moving direction of a vehicle:

$$\frac{(L+0.6)\,P\,(1+I)}{9.6\times 0.8} \tag{1}$$

where L denotes the span of the beam (m). Additionally, in the case of P (kN), as shown in Figure 5, the load of the rear wheel of a truck was applied for the grade 1 standard. The constant 0.8, in the denominator, is intended to account for the bending moment of the continuous slab with more than three supporting points. The impact coefficient, I, varies based on the design standard.

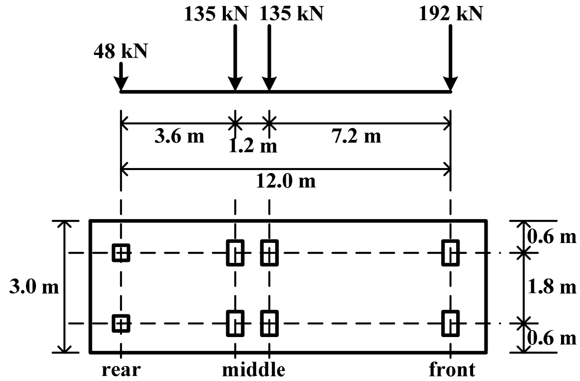


Figure 5. Characteristics of the design truck.

The impact coefficient of the upper structure for Code 2010 was calculated as shown in Equation (2), where L denote the length of the span in which a live load was loaded such that the peak stress could be applied to the design member.

$$I = \frac{15}{40 + L} \leq 0.3. \tag{2}$$

Conversely, in Code 2015, 0.15 was applied at the fatigue limit state, and 0.25 at all limit states, excluding the fatigue limit state, for all the members.

2.2.2. Flexural Strength

In Code 2010, the relationship between the compressive stress distribution of the concrete and the strain of the concrete was expressed as an equivalent rectangular stress block as shown in Figure 6a. The ultimate strain of the concrete was assumed to be 0.003. Figure 6b shows the stress and strain relationship for Code 2015.

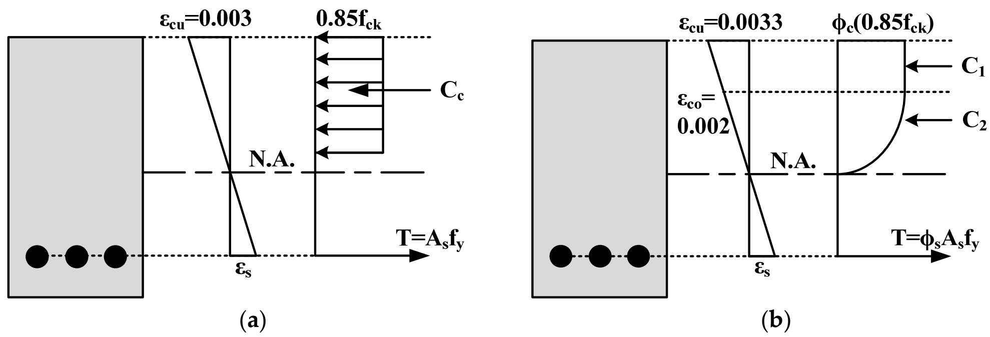


Figure 6. Stress and strain relationship between concrete and steel: (**a**) rectangular stress distribution of strength design method; (**b**) parabolic stress distribution of limit state design method.

A parabolic stress and strain relationship was used, and the strength was calculated using an equilibrium of forces and deformation compatibility. The limit strain of the concrete was limited at 0.0033. The stress of the concrete for each interval was obtained as shown in Equations (3a) and (3b).

$$0 \leq \varepsilon_c \leq \varepsilon_{co},\ f_c = \phi_c\,(0.85 f_{ck}) \left[1 - \left(1 - \frac{\varepsilon_c}{\varepsilon_{co}}\right)^n\right] \tag{3a}$$

$$\varepsilon_{co} \leq \varepsilon_c \leq \varepsilon_{cu},\ f_c = \phi_c\,(0.85 f_{ck}). \tag{3b}$$

In the above equations, ϕ_c denotes the material coefficient of concrete, n denote the shape factor of the ascending curve, ε_{co} denotes the strain when peak stress is first reached, and ε_{cu} denotes the ultimate strain. The material coefficient is changed by the limit state, and 0.65 was used in the ultimate limit state that corresponded to the basic load combination given normal vehicle traffic. The shape factor, peak stress, and ultimate strain corresponded to 2.0, 0.002, and 0.0033, respectively, when the strength of the concrete is less than or equal to 40 MPa. When the strength of the concrete exceeded 40 MPa, ε_{co} increased by 0.0001, and ε_{cu} decreased by 0.0001 for every 10 MPa increase in strength. Additionally, n is determined using Equation (4):

$$n = 1.2 + 1.5\left(\frac{100 - f_{ck}}{60}\right)^4 \leq 2.0. \tag{4}$$

2.2.3. Reinforcement Arrangement

The reinforcement bending should exceed the minimum inner radius to prevent damage of the reinforcement due to reinforcement bending and the damage of the concrete within the bent reinforcement. With respect to Code 2010, the standard hook of the main reinforcement is classified as 90° and 180°. In the case of the former, it needs to be extended by more than $12d_b$ from the bent end, and in the case of latter, it needs to be extended by more than $4d_b$ and 60 mm from the end of the half circle. The minimum inner radius is classified into three categories based on the diameter of the reinforcement, as summarized in Table 2.

Table 2. Minimum inner radius of reinforcement bending (Code 2010).

Reinforcement Type	Minimum Inner Radius (mm)
D10–D25	$3d_b$
D29–D35	$4d_b$
Over D38	$5d_b$

Conversely, in Code 2015, the bending angle of the standard hook is classified into three types: 90°–150°, +150°, and loop. When the bending angle corresponded to 90°–150° or +150°, it needed to be extended by more than $5d_b$ from the end of the bent half circle. The minimum inner radius of the reinforcement is classified into four types based on the shape and diameter of the reinforcement as summarized in Table 3. Therefore, bending reinforcement could be designed with a smaller radius as well as the height of the cross section.

Table 3. Minimum inner radius of reinforcement bending (Code 2015).

Reinforcement Type	Under D16	Over D19
Rounded bar	$1.25d_b$	$2.5d_b$
Deformed bar	$2d_b$	$3.5d_b$

2.2.4. Reinforcement Development and Splice Length

With respect to the design of the joint, it is necessary to calculate the lap splice length of the reinforcement. In Code 2010, the compressive strength of the concrete is considered to calculate the development length, and the lap splice length is obtained by adding 30% extra length to the development length. This is expressed in Equations (5) and (6) as follows:

$$l_d = \frac{0.90 d_b f_y}{\sqrt{f_{ck}}} \frac{\alpha\beta\gamma\lambda}{\left(\frac{c+K_{tr}}{d_b}\right)} \tag{5}$$

$$l_s = 1.3 l_d. \tag{6}$$

In the above equations, d_b denotes the diameter of the reinforcement, f_y denotes the design strength of the reinforcement, f_{ck} denotes the design strength of the concrete, α denotes the reinforcement arrangement position factor, β denotes the epoxy coating factor, γ denotes the reinforcement diameter factor, λ denotes the lightweight concrete factor, c denotes the thickness of the cover or the spacing of the reinforcement, and K_{tr} is the transverse reinforcement factor.

The tensile strength of the concrete and the shape of the reinforcement are considered in Code 2015, and the basic development length of the reinforcement with a diameter of d_b is calculated as shown in Equation (7) given below.

$$l_b = \left(\frac{d_b}{4}\right)\left(\frac{\sigma_{sd}}{f_{bd}}\right). \tag{7}$$

σ_{sd} denotes the design stress of the reinforcement, and f_{bd} denotes a design value for the bond strength of the deformed reinforcement, which is calculated as shown in Equation (8).

$$f_{bd} = 2.25 \eta_1 \eta_2 f_{ctk} / \phi_c. \tag{8}$$

η_1 denotes the factor relevant to the bond condition and the position of the reinforcement for the placement of the concrete, η_2 denotes the factor relevant to the diameter of the reinforcement, and f_{ctk} denotes the tensile strength of the concrete.

The design lap splice length is calculated as shown in Equation (9) given below:

$$l_0 = \alpha_1 \alpha_2 \alpha_3 \alpha_5 \alpha_6 l_b A_{s.req} / A_{s.prov}. \tag{9}$$

α_1 denotes the shape effect of the reinforcement with an appropriate cover thickness, α_2 denotes the effect of the cover thickness of the concrete, α_3 denotes the restraint effect of the transverse reinforcement, α_5 denotes the effect for the pressure that crosses the splitting surface formed along the design development length, α_6 denotes the factor determined by the ratio of the lap splice reinforcement to the total cross-sectional area, $A_{s.req}$ denotes the required reinforcement cross-sectional area, and $A_{s.prov}$ denotes the cross-sectional area of the used reinforcement.

3. Results and Discussion

3.1. Load and Strength

With respect to the design of the transverse joint of the precast modular girder bridge, only the pavement was reflected in the fixed load, and the two codes utilized the same value. However, the live load and load factor were calculated based on each code. Table 4 summarizes the load and flexural strength for the cross section as shown in Figure 3. The values obtained based on the existing strength design method were re-examined based on the limit state design method. The result indicated that the factored load and the flexural strength decreased by 18.8% and 2.13%, respectively. Thus, the safety factors of Code 2010 and Code 2015 corresponded to 1.140 and 1.374, respectively.

Table 4. Load combination and flexural strength.

Load Type (kN·m)	Code 2010		Code 2015	
	Factor	Load (kN·m)	Factor	Load (kN·m)
Dead load	1.30	1.066	1.50	1.066
Live load	2.15	31.272	1.80	30.070
Factored load	68.621		55.725	
Flexural strength	78.207		76.543	
Safety factor	1.140		1.374	

It was expected that the height of the cross section and the reinforcement amount of the top flange would reduce when Code 2015 was applied because the safety factor increased from 1.140 in the case of Code 2010 to 1.374 in the case of Code 2015. Additionally, Code 2010 exhibited the disadvantage that the height of the flange and the weight increased due to the application of the loop splice. However, in the case of Code 2015, the design criterion with respect to the minimum inner radius of reinforcement bending was mitigated such that it was possible to change the method of splicing.

3.2. Height

Table 5 summarizes the changes in the flexural strength and safety factor based on the decrease in the height of the cross section. The height of the cross section was decreased at 5 mm intervals. Thus, the flexural strength also decreased constantly by 2.363 kN·m in the case of Code 2010 and by 2.502 kN·m in the case of Code 2015. The latter indicated a slightly larger decreasing ratio because of stress distribution is parabolic. With respect to the application of the limit state design method, in the flexural performance condition similar to the original one, it was possible to decrease the height of the cross section to 195 mm when the criteria of deflection was considered.

Table 5. Strength based on the change in the height of the cross section.

Height (mm)	Code 2010		Code 2015	
	Strength (kN·m)	Safety Factor	Strength (kN·m)	Safety Factor
220	78.207	1.140	76.543	1.374
215	75.843	1.105	74.041	1.329
210	73.480	1.071	71.538	1.284
205	71.117	1.036	69.036	1.239
200	68.753	1.002	66.534	1.194
195	66.390	-	64.031	1.149

3.3. Reinforcement

Figure 7 shows the change in the safety factor based on the change in the amount of reinforcement. Inappropriate results for the design standard (e.g., minimum spacing of reinforcement) were not considered. The original design involved a cross section with seven D16 reinforcements. The safety factor was less than 1 when the number of reinforcements decreased from seven to six; thus, it was inappropriate with respect to Code 2010. However, in the case of Code 2015, the safety factor corresponded to 1.187 even when the number of reinforcements decreased to six, and this indicated that it was safe. Additionally, when the type of reinforcement was changed to D13, Code 2010 was also inappropriate for the safety factor, but Code 2015 was appropriate, with safety factors that corresponded to 1.150 and 1.027 when nine and eight D13 reinforcements, respectively, were applied. Under the flexural performance condition (i.e., safety factor) similar to the original one, the number of reinforcements could be decreased by 14.3%, and the diameter of the reinforcement could be decreased by up to 18.0% and 27.0% for nine and eight D13 reinforcements, respectively.

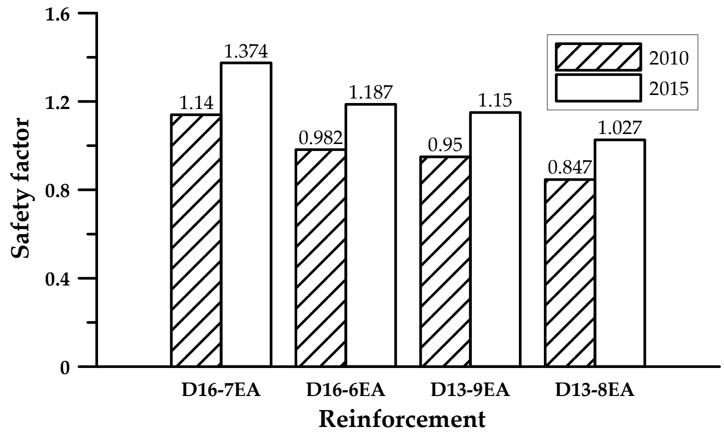


Figure 7. Changes in the safety factor based on the amount of reinforcement.

3.4. New Design of Joint

In order to calculate the reinforcement splice length and width of the transverse joint of the precast modular bridge, an optimal cross section was first selected with respect to each variable, height, and reinforcement. For the purpose of comparison, the original cross section based on Code 2010 (the strength design method) and the cross section in which the number of reinforcements and diameter were varied based on the limit state design method were selected, respectively. The flexural performance (i.e., safety factor) of the original design that corresponded to 1.140 was used as the criterion. Table 6 summarizes the safety factor relative to variables. When six D16 reinforcements were used, the height of the cross section was decreased by 5 mm, and the amount of reinforcement was decreased by 14.3% while securing a flexural performance similar to the original one. In the case of nine D13 reinforcements, the amount of reinforcement was decreased by 18.0% without a change in height.

Table 6. Safety factor by height and reinforcement amount for the cross section.

Height (mm)	Code 2010	Code 2015	
	D16–7EA (1390.2 mm^2)	D16–6EA (1191.6 mm^2)	D13–9EA (1140.3 mm^2)
220	1.140	1.187	1.150
215	1.105	1.149	1.113
210	1.071	1.110	1.076
205	1.036	1.072	1.039
200	1.002	1.033	1.002
195	0.967	0.995	0.966

The differences in the joint width based on the change in the splice length of the reinforcement were compared with respect to the selected optimal cross section. For the ease of construction, the width of the transverse joint was set as twice the splice length considering the clearance. Table 7 summarizes the examined splice length and width of the transverse joint. In a manner similar to the original design, when a lap splice was applied, the width of the transverse joint decreased by 21.1% in the case of D16–6EA, and by 36.8% in the case of D13–9EA.

The application of a loop (U-bar) that was unavailable in the original design due to the limitation in the height of the cross section was enabled by Code 2015, and the minimum thicknesses corresponded to 196 and 178 mm when D16 reinforcement and D13 reinforcement, respectively, were used. Accordingly, the width of the joint decreased by 26.3% in the case of D16–6EA and by 47.4% in the case of D13–9EA.

Table 7. Optimal design of the joint.

Classification (mm)	2010	2015			
Height	220	215		220	
Reinforcement	D16–7EA (1390.2 mm^2)	D16–6EA (1191.6 mm^2)		D13–9EA (1140.3 mm^2)	
Basic development length	140.2	152.2		123.7	
Splice type	Lap splice	Lap splice	Loop	Lap splice	Loop
Splice length	182.3 ≈ 190	138.4 ≈ 150	135.8 ≈ 140	112.4 ≈ 120	98.14 ≈ 100
Joint width	380	300	280	240	200

The induced optimal width of the transverse joint was expected to reduce the volume of site work for concrete. This in turn was expected to be more conducive to the construction of the modular bridge and to increase the efficiency of cross-sectional performance in the top flange section prior to the attachment of the section to the bridge [15]. Additionally, the small optimal joint is potentially sufficient to transfer shear and moment between adjacent flange modules [5,11].

4. Conclusions

In this study, the top flange of a modular T-girder bridge was redesigned using the Korean Highway Bridge Design Code Limit State Design (2015). A new optimal cross section was obtained by performing a study with a safety factor that was similar to the designed flange using an existing strength design method (2010). The reinforcement arrangement and the development and splice length of the transverse joint were analyzed based on the obtained cross section. The result indicated that, in contrast to the strength design method, the use of the limit state design method decreased the height of the cross section from 220 to 195 mm based on a safety factor greater than 1 and decreased the amount of reinforcement by 27.0% with the use of a reinforcement of a smaller diameter.

In the case of a design to minimize the width of the transverse joint while maintaining the safety factor of the original design (1.140), the width was decreased by up to 47.4% with a height of 220 mm, a D13–9EA reinforcement, and a loop (U-bar) splice. The results revealed that the application of the limit state design method increased the flexural performance when compared to the application of the strength design method. It also improved the cross-section efficiency by maximizing the use of material properties in terms of the development and splice length of the reinforcement.

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