Ultimate Strength Evaluation for Wide-Type Box Girders in Cable-Supported Bridges

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Summary

Wide-type steel box girders that have much larger size of width than depth are widely adopted in cable-supported bridges due to their superb rigidity and advantages in aerodynamics. In design such primary members it is inevitable to evaluate nominal strengths before applying various design parameters. As ultimate strength of wide-type steel box girders subjected to concurrent bending moment and axial force is governed by flanges in compression rather than in tension, separated deck panel system in compression was modeled and analyzed. General-purpose nonlinear finite element analysis program was used to perform evaluation of ultimate strengths of various hypothetical stiffened plate systems with U-ribs. The analyses results have been compared to expected strengths from available design specifications such as Eurocode 3, FHWA (Federal Highway Association), and JRA (Japanese Road Association). New design equations have been developed for ultimate strength as a function of plate and column slenderness parameters of the stiffened deck panel. It has been found that the proposed equations expect ultimate strengths reasonably permitting a way to more economic design than other design guides.

Keywords: wide-type steel box girders; ultimate strength; stiffened plate; U-rib; cable-supported bridges.

1. Introduction

Wide-type steel box girder has been widely used in cable-supported bridges due to their superb torsional rigidity, effectiveness in resisting lateral bending, advantages in aerodynamics, etc [9]. Especially, this kind of girders can offer super long span in cable-supported bridges by reducing the dead. In recent design practice, closed type stiffeners are more preferred than open type stiffeners as former have high performance on wheel load distribution and local rigidity. In general design procedure for box girder bridges, it is one of the important steps to evaluate nominal strengths of stiffened plate systems in compression. Objectives of present study are summarized as: (1) investigation of ultimate compressive strength for U-rib stiffened plate, and (2) proposition of strength curves based on the numerical methodology.

Major parameters effecting on ultimate strength of wide-type box girders subjected to bending and/or compression have been reviewed from existing codes. Hypothetical models for stiffened panel plate with U-rib were selected and analyzed utilizing a commercial package program ABAQUS [1]. Effects of initial imperfection, residual stresses, and material yielding were incorporated in nonlinear incremental analysis. Strength predictor curves were derived from numerical results using linear regression method.

2. A review on design codes

2.1 FHWA-TS-80-205

Although AASHTO LRFD bridge design specification [2] offers general provisions for box girder section, it recommends FHWA specification [12] for special purpose of design of long span box girder bridges. This specification was based on strut approach which utilizes the concept of stiffener strut consisting of one stiffener and the associated portion of flange plate with equally spaced stiffener spacing. The ultimate compressive strength of stiffened plate, P_u, is calculated as:

$$P_u = F_u A_f \quad (1)$$

$$\frac{P_u}{F_y} = f(\lambda_{pl}, \lambda_{col}) \quad (2)$$

Where A_f is the cross sectional area of flange and all longitudinal stiffeners, F_y the yield stress, F_u the ultimate compressive strength of a stiffener strut. This strength is an function of plate slenderness parameter, λ_{pl}, and column slenderness parameter, λ_{col}, as shown in Eq. (2) and given by interaction diagram shown in Fig. 1.7.206(A) in FHWA. Slenderness parameters are defined as:

$$\lambda_{pl} = \frac{w/t}{1.9}\sqrt{\frac{F_y}{E}} \quad (3)$$

$$\lambda_{col} = \frac{1}{\pi}\sqrt{\frac{F_y}{E}}\frac{L}{r} \quad (4)$$

where w is stiffener spacing, t thickness of flange plate, L transverse stiffener spacing, r radius of gyration of stiffener strut, E young's modulus. FHWA specification considers only two slenderness parameters presented in Fqs. (3) and (4). In order to prevent stiffener local buckling inherently, this specification provides a prerequisite requirement for stiffeners that the strength of stiffener must greater than that of stiffened panels.

2.2 EN 1993-1-5

For stiffened plates, Eurocode 3 recommends 'effective section method' for calculating the ultimate compressive strength in EN 1993-1-5 [5] and gives a strength equation as:

$$P_u = A_{c,eff} F_y \quad (5)$$

where $A_{c,eff}$ is effective section for overall reduction from sub-panel buckling and global buckling of stiffened plates, and calculated as:

$$A_{c,eff} = \rho_c A_{c,eff,loc} + \sum b_{edge,eff} \cdot t \quad (6)$$

$$A_{c,eff,loc} = A_{sl,eff} + \sum_c \rho_{loc} b_{c,loc} t \quad (7)$$

where $A_{sl,eff}$ is sum of the effective cross-sectional area of all longitudinal stiffeners, $\sum_c \rho_{loc} b_{c,loc} t$ the effective cross sectional area of all the sub-panels reduced for local plate buckling except for effective part of sub-panels which are supported by a web or a flange plate($\sum b_{edge,eff} \cdot t$), ρ_c the reduction factor for global buckling of the stiffened panel, ignoring local buckling of sub-panels. The reduction factor for global buckling, ρ_c, is determined from an empirical interpolation between the reduction factors for column-like buckling and for overall stiffened plate buckling and is defined as:

$$\rho_c = (\rho - \chi_c)\xi(2-\xi) + \chi_c \quad (8)$$

As shown in Eq. (6) ~ Eq. (8) the effective area is obtained mainly by the process of reducing the gross area in two steps, namely, the first step for sub-panel local buckling and the second step for global buckling. Concerning the first step, EN 1993-1-5 considers the effect of stiffener local buckling whereas FHWA prevents it inherently by the stiffener strength requirement. In the viewpoint of the second step, both codes consider both the plate behaviour and the column behaviour.

2.3 JRA & JSCE guideline

In JRA(Japanese Road Association) [7], ultimate compressive strength have been suggested as functions of the plate slenderness parameter only from the general plate theory and the functions are given as:

$f_{cr} / f_y = 1.0 \quad (R_R \le 0.5)$ (9a)

$f_{cr} / f_y = 1.0 - R_R \quad (0.5 < R_R \le 1.0)$ (9b)

$f_{cr} / f_y = 0.5 / R_R^2 \quad (f_{cr} / f_y = 0.5 / R_R^2)$ (9c)

where $R_R = \frac{b}{t}\sqrt{\frac{f_y}{E}\frac{12(1-\mu^2)}{\pi^2 k_R}}$, $k_R = 4n^2$ in which b, t, k_R and n are total plate width, thickness of plate panel, buckling coefficient and number of panels divided by stiffener. Contrasted with FHWA and EN 1993-1-5, JRA consider only plate local buckling effect using one parameters, R_R.

In case of JSCE guideline (for buckling design)[8], modified column approach is used to calculate ultimate compressive strength of orthogonally stiffened plate , $\overline{F}_u$, and the strength given as

$$\overline{F}_u = \overline{F}_{esm} \frac{b_{eff} \cdot t + A_l}{b_l \cdot t + A_l} \quad (10)$$

where $\overline{F}_{esm}$ is the ultimate stress of effective stiffener, A_l cross-sectional area of longitudinal stiffener, b_l and b_{eff} are full width and effective width of plate between stiffeners. For determining $\overline{F}_{esm}$ and b_{eff}, the following equations are required iteratively.

$$R_e = 0.562\sqrt{\frac{\overline{F}_{esm}}{E}} \cdot \frac{b_l}{t} \quad (11)$$

$$\frac{b_{eff}}{b_l} = 0.702R_e^3 - 1.640R_e^2 + 0.654R_e + 0.926 \quad (12)$$

$$\lambda_{eff} = \frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}}\frac{L_{eff}}{r_{eff}} \quad (13)$$

$$\frac{\overline{F}_{esm}}{F_y} = 1.0, \ (\lambda_{eff} \le 0.2) \quad (14a)$$

$$\frac{\overline{F}_{esm}}{F_y} = -\alpha\lambda_{eff}^3 + \beta\lambda_{eff}^2 - \gamma\lambda_{eff} + \delta, \ (0.2 < \lambda_{eff} \le 1.2) \quad (14b)$$

where r_{eff} is radius of gyration of effective stiffener and L_{eff} is effective length of pseudo-stiffened plate. The parameters α, β, γ and δ for steel grades specified in JSHB[7] are show in Table 1. Similarly to FHWA, the JSCE guideline also adopted two parameters, R_e and λ_{eff}, in order to incorporate two different types of failures, namely, plate local failure and column-type failure of stiffened plate system. It is also noted that the JSCE guideline provide requirements for preventing premature local buckling in longitudinal stiffeners.

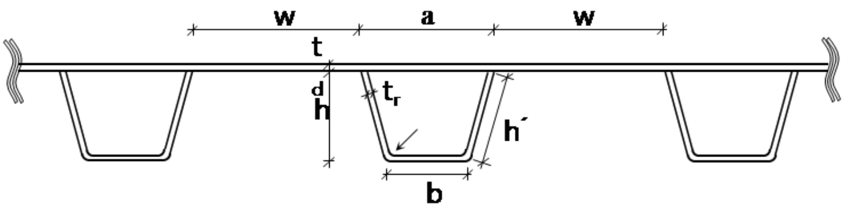
Table 1. Coefficient α, β, γ and δ in JSHB[7]

Steel grade	$\sigma_Y^*(MPa)$	σ_{rc}/σ_Y^*	α	β	γ	δ
SM53	353	0.23	0.299	0.618	0.852	1.135
SM58	451	0.20	0.425	0.820	0.903	1.142

3. Nonlinear FE analysis of stiffened plates

3.1 Geometry of model

After various preliminary numerical studies for U-rib stiffened deck panel systems, it has been identified that failure modes at ultimate stages can be categorized into two major modes; one is the plate local buckling in deck panels and global (overall) buckling of panel systems as in strut approach. These two modes of failures, than are governed by plate and column slenderness parameters shown in Eqs (3) and (4), respectively, were taken into account when hypothetical analysis models were set up. U-ribs considered in this study were also checked and set to prevent the stiffener local buckling and/or stiffener tripping. Each parameter was controlled by changing the deck panel plate thickness and deck panel spacing between transverse stiffeners. U-rib shape, thickness, and the spacing in lateral direction were unchanged. Details of hypothetical models are represented in Fig. 1, 2 and Table 2.



a	w	b	h	h′	R	t_r	L
400	500	250	240	521.4	40	8	3000~6000

Unit: mm

Fig. 1: Notation for U-rib stiffened plate

Table 2. Deck thickness of hypothetical model

Model ID	t_d (mm)
T08	8
T10	10
T12	12
T14	14
T16	16
T18	18
T20	20
T22	22
T24	24
T32	32
T40	40

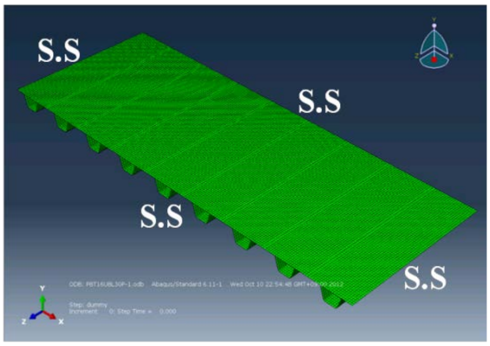


Fig. 2: Modelling by ABAQUS

3.2 Material properties

The most frequently adopted steel that is designated as SM490Y in Korean Specification for Roadway Bridges was assumed for material characteristics in present study. In particular, SM490Y has yield stress (355MPa) and ultimate stress (490MPa) that are similar to A 709M Grade 345W in ASTM Designation. The constitutional relationship between stress and strain was assumed to be elastic-plastic with yield plateau and strain-hardening rate as shown in Fig. 3. The mechanical properties of the steel are summarized in Table 3.

Table 3. *Mechanical properties of conventional steel*

Type	E (GPa)	F_y (MPa)	F_u (MPa)	ε_y	ε_{sh}	ε_u	E_{sh} (GPa)
SM490Y	200	355	490	0.001775	0.021	0.0585	3.6

3.3 Initial imperfection and residual stress

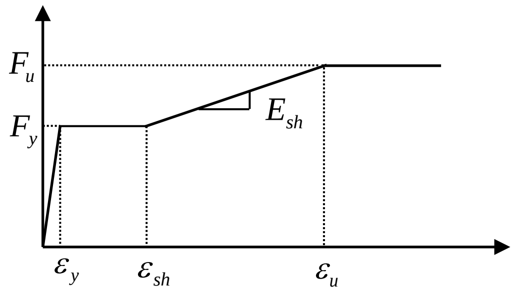


Fig. 3: Conventional steel model

As ultimate compressive strengths of stiffened plates are governed by plate local buckling or global buckling, the magnitude and types of *initial imperfection* (*I. I.*) may significantly influence on the strength. In order to evaluate effects of initial imperfections on in-plane compressive strengths, two different types of initial geometric imperfection were considered in this study. Fig. 4 shows the first eigenmode of stiffened deck panel system with relatively thick deck plates while Fig.5 shows that of the system with relatively thin deck plates. The thickness of deck panel was varied from 8 mm to 40 mm, respectively. According to Sheikh's study [11], the global buckling mode shown in Fig.4 is in accord with stiffener induced global buckling mode. Both the global buckling mode and plate local buckling mode were considered in the present analysis for initial imperfections. In ABAQUS, shapes of initial imperfections can be incorporated as independent analysis result files. For the reason that global buckling type is similar to column behaviour, the magnitude of maximum deflection (scaling factor) for initial imperfection, δ_{max}, for global buckling type was assumed to be $L/1000$ referring to the SSRC column curve [10]. It is noted that the assumed magnitude is a little bit in conservative side compared to $L/1500$ from AISC specification (2005) [3] and AASHTO LRFD bridge design specification (2007). For the plate local buckling type, the magnitude of maximum deflection was taken as $w/120$ based on the Article 3.5 of the Bridge welding Code [4].

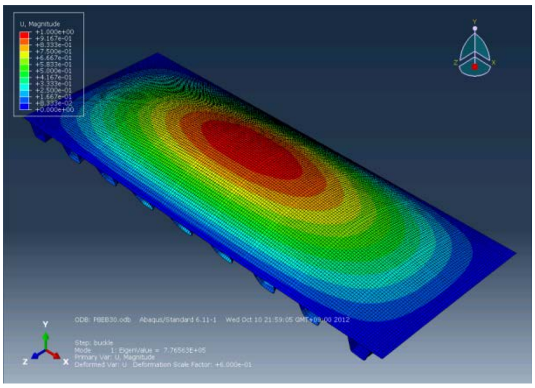


Fig. 4: Global buckling type I.I.

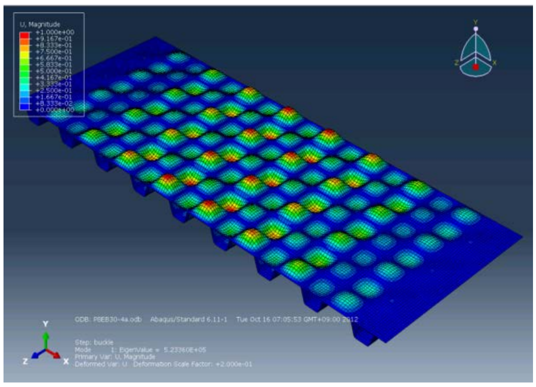


Fig. 5: Plate local buckling type I.I.

It is generally accepted in the analysis of steel structures that the residual stresses due to welding should be considered for possible effects on ultimate strength. Fig. 6 represents assumed residual stress distribution considered in this study. This model was known to be suggested by Fukumoto et al.[6] and Sheikh et al.[11].

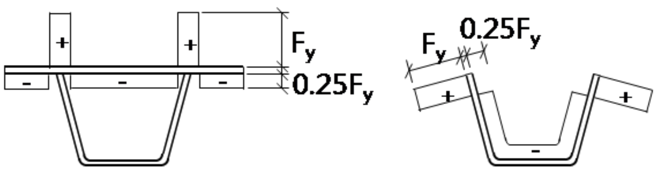


Fig. 6: Residual stress model.

4. Analysis results and proposed strength curve

4.1 Analysis results

Normalized ultimate strengths, F_u/F_y, as functions of the plate slenderness parameter w/t, are shown in Fig. 7 along with those from other available code predictions. In Fig. 7, FEA 1 and FEA 2 represent the strengths from analyses incorporating global buckling type and plate buckling type, respectively, as an initial imperfection mode shape. It is very interesting to note that ultimate compressive strengths of stiffened plate systems significantly depend on the shapes of initial imperfections as identified in Fig. 7. It is also noted that there is a certain inconsistency among design code predictions. The strength predictions by JRA monotonically decrease as the value of w/t increases. It may be reconfirmed from Fig. 7 that the strength predictions from Eurocode, FHWA, and JSCE guideline have considered interaction effect of column behaviour and plate behaviour showing the optimal point for the maximum.

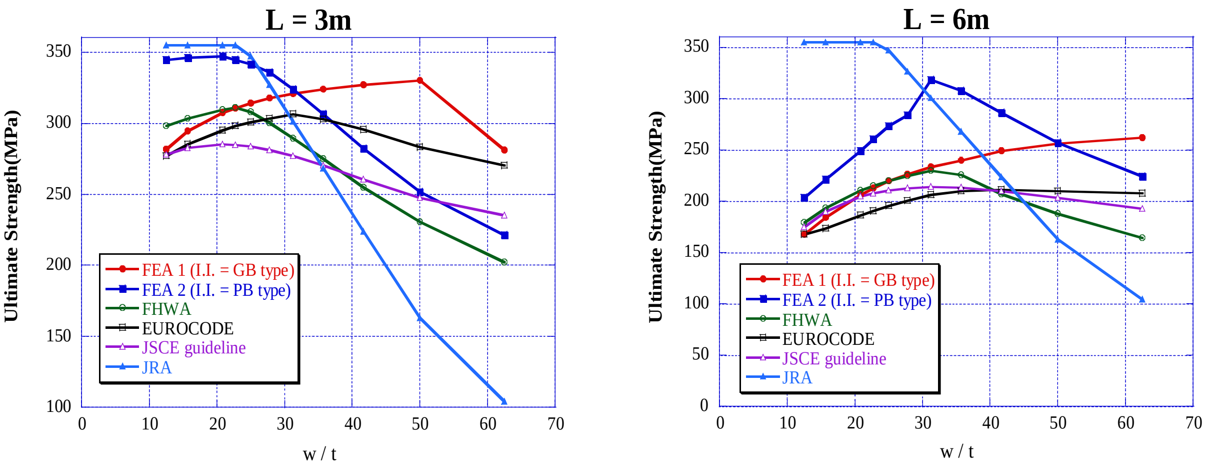


Fig. 7: Strength comparison with respect to plate slenderness parameter

4.2 Proposed strength curve

Stiffened deck panel systems have two strength curves due to two different mode shapes of initial imperfection. The strength F_{ugb} represent a strength curve from FEA 1 results that incorporated GB type initial imperfection while the strength F_{upb} represent FEA 2 results that incorporated PB type initial imperfection. The final strength curves in normalized values can be defined conservatively as minimum line as noted as low bound of strength curve in Fig. 8 and have the following equation form:

$$\frac{F_u}{F_y} = \min\left[\frac{F_{ugb}}{F_y}, \frac{F_{upb}}{F_y}\right] \quad (15)$$

For normalized strength curve, basis function was set as second order polynomial function and given as

$$\frac{F_u}{F_y} = c_0 + c_1\lambda_{col} + c_2\lambda_{pl} + c_3\lambda_{col}^2 + c_4\lambda_{pl}^2 + c_5\lambda_{pl}\lambda_{col} \quad (16)$$

To fit the coefficients of Eq. (16), linear regression method was performed based on the least square

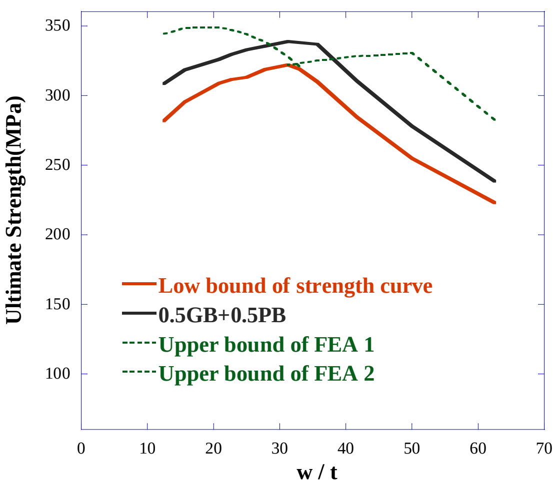


Fig. 8: Proposed strength curve

errors and the coefficients are summarized in Table 4. In Fig. 7, proposed strength curves are plotted as values of w/t varies. It seem that the proposed strength curve generally higher than existing code predictions due to more accurate evaluation of stiffened plate behaviours.

Table 4. Coefficients for proposed strength curve

c_0	c_1	c_2	c_3	c_4	c_5
1.26	-0.92	-0.005	-0.059	-0.313	0.62

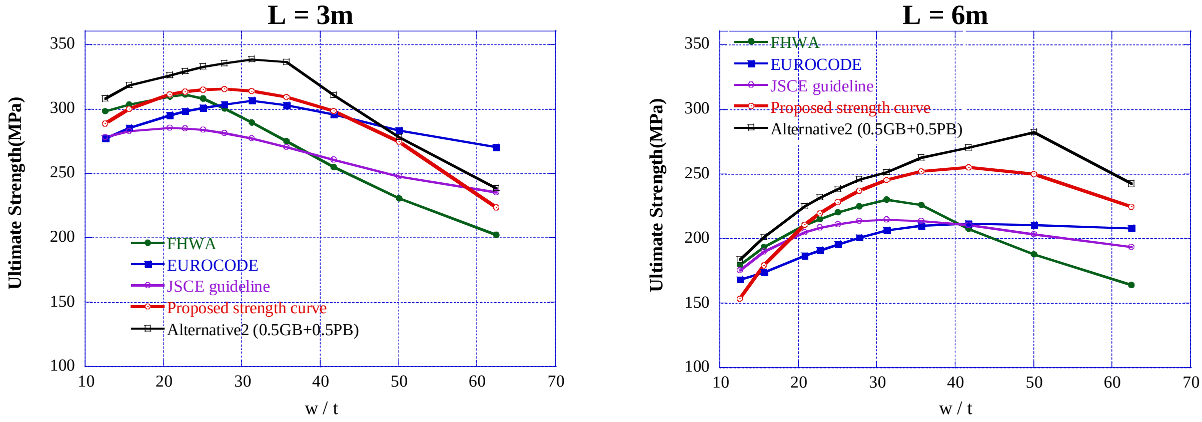


Fig. 7: Strength comparison with proposed method and existing codes predictions

5. Conclusion

Stiffened plates by U-rib were modelled and analyzed using finite element software, ABAQUS. Two types of initial imperfection were considered in the analyses for ultimate compressive strength, the global buckling mode and the plate local buckling mode, respectively. Based on numerical results, strength formulas were derived using linear regression method and the minimum envelopes were proposed for final ultimate compressive strength curves. The proposed method was compared with other existing code predictions. It has been found that the proposed method provides higher strengths than existing codes but in the conservative side for FE results.

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