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Deconstructable variable stiffness shear connectors in FRP deck resting on multigirder bridge system: Analytical model

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ABSTRACT

Deconstructable structures offer great flexibility and reduce material waste. Fiber Reinforced-Polymer (FRP) decks resting on steel girders combine high durability, lightweight, fast construction, and deconstructability. Composite action between the decks and girders is achieved using shear connectors. Until now, most shear connectors use adhesives or grouts, limiting their maintenance, reuse, or recycling. This paper presents a closed-form solution to analyze the behavior of an FRP deck connected to multi-girders using variable sequential stiffness shear connectors considering partial Degree of Composite Action (DCA). The analytical model, which is verified using Finite Element model and prior experimental studies, is further used to study stress and deflection for various multigirder configurations. Finally, the DCA considering variable stiffness shear connectors is presented and linked to effective width ratios, paving the way for adopting variable stiffness in bridge standards.

1. Introduction

1.1. Deck-on-girder bridge system

The deck-on-girder bridge system is typically composed of a deck panel resting on supporting girders, which is considered one of the rapid construction systems to execute bridges. Due to severe climate changes in many countries and fast population growth, the need for these rapid systems has been considered a challenge to: (1) reduce the economic and social impacts resulting from ordinary construction techniques; and (2) be easier in rehabilitation and repair [1,2]. Various materials have been used in the fabrication of deck-on-girder bridges, including concrete, steel, timber, Fiber Reinforced Polymers (FRP), and others [3–5]. Using FRP decks-on-steel girders is a promising system to get high-load capacity bridges with lighter weights [6–9].

Compared with conventional reinforced concrete (RC) material, FRP is considered a non-corrosive material with high durability and short curing duration [10–13]. However, the FRP deck's limited longitudinal stiffness compared with RC decks results in larger steel girder sections. Osei-Antwi et al. [14] compared FRP decks with balsa inserts (D-TI) to RC decks in terms of strength and deflection and found that RC decks

provide around 44% lower deflection and 66% higher strength than those of FRP decks. In addition, the bridge's load capacity is controlled by the orthotropic material properties of the FRP and the stiffness of the shear connectors between FRP deck and steel girders [15–23].

1.2. Shear connectors

Shear connectors are widely used in composite beam bridge systems. They connect the upper (decks) and lower parts (girders) by transferring shear forces between the two. The deck is connected to girders using different types of shear connectors, including steel clamps, bolts, shear studs, and others [24,25]. For FRP deck-on-girders, shear connectors can be adhesive bonds or mechanical connections [26]. In perfect conditions, adhesive bonds can be applied without experiencing stiffness degradation. However, they can degrade when subjected to moisture and extreme variations in temperature [27,28]. On the other side, mechanical or bolted connections can withstand extreme environmental conditions, but their installation process can be labor-intensive [15,16]. Some research was carried out to develop robust and easy mechanical shear connectors for FRP-steel deck-on-girder systems [29–31], which offer deconstructability of the system during the bridges' lifecycle.

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Fig. 1. Cross-section of deconstructable sleeve-type shear connector.

1.3. Design for deconstruction

Design for deconstruction (DfD) allows the elements to be easily maintained, reused, and/or remanufactured or recycled. This will reduce the amount of waste that might occur during demolition purposes [32]. Valipour et al. proposed a steel-concrete composite bridge deck girder system with deconstructable friction grip bolted shear connectors [33]. Nima et al. developed analytical and numerical models for deconstructable timber-concrete composite beams [34]. Injected steel-reinforced resin has been developed at Delft University to provide a demountable connection for the FRP-Steel bridge deck-on-girder system [31].

A non-grout sleeve-type shear connector, shown in Fig. 1, offers a controllable height depending on the deck's thickness, facilitating the erection process. Davalos et al. [18] carried out pushout tests to determine the stiffness of the shear connector. The results indicated that the stiffness (*K*) is divided into three stages, as shown in Fig. 2: at the first stage, the stiffness is 1.5 kN/mm, corresponding to shear stud stiffness; at the second stage, the stiffness increased to 7.9 kN/mm when the shear stud made contact with the bottom sleeve; and at the third stage, the stiffness reduced to 1.4 kN/mm, where the shear stud experienced yielding at its base until failure. A full-scale test was carried out using the sleeve connection, reporting a 25% Degree of Composite Action (DCA) based on strain measurements at the first stage of the inner T-section.

1.4. Degree of composite action

The DCA can be defined as the ability of the shear connector to transfer the shear from the FRP deck slab to steel girders. Chen and Yossef [35] developed an analytical model for inner T-beam section considering partial degree of composite action, considering the differences of strains at the interface of the deck-beams. The analytical model was used to conduct a parametric study, based on which a design equation was presented for inner beam T-section with linear connector stiffness. Other studies reported partial DCA [36–39], but limited guidelines are available to calculate the effective flange width for decks mounted on multiple girders, especially when considering FRP deck systems with variable stiffness shear connectors [40,41].

The transfer of shear forces from the girder to the deck gradually develops compression force over the deck width along the span, known as the shear lag effect. The stress across the deck slab is assumed uniform in the design, which is not reasonable, as shown in Fig. 3. Added to this, variation in the spacing between the shear connectors may cause a decrease in shear strength, ductility, and a nonuniform distribution of shear force at the positions of sleeve shear connectors [42–47].

1.5. Objective

Lack of design formulas, including DCA, is a major challenge for FRP steel deck-on-girder composite bridge deck system. In this study, an analytical model is developed to study the effective flange width, which is verified based on a validated numerical model (FE model) using prior experimental investigation of FRP deck on steel girder system [48]. The analytical model accounts for the followings:

- 1) partial DCA,
- 2) shear-lag effect,
- 3) multi-girder configurations,



Fig. 3. Effective width [48].



Fig. 2. Deformation of sleeve-type shear connectors, (a) stage 1, K = 1.5 kN/mm, (b) stage 2, K = 7.9 kN/mm, (c) stage 3, K = 1.4 kN/mm.



Fig. 4. Multi-cell deck-bridge system showing distribution factors.



Fig. 5. Axial and bending moment for deck-on-girder system [35].







Fig. 7. Single-cell with two connectors.

4) variable stiffness for deconstructable sleeve-type shear connectors

2. Analytical model

An analytical model is developed for an orthotropic deck resting on multiple girders. Several assumptions are made:

(1) Linear elastic material; (2) symmetric cross-sections; (3) shear deformation is neglected, and therefore, the curvature is considered the same for the deck and girders; (4) torsion is not considered; and (5) all shear connectors have the same stiffness.

Axial force acting on the FRP deck can be described as [35]:

$$N_x\left(x,y\right) = \sum_{j=1}^{\infty} \left(C_{1j}\cosh\left(\xi_j y\right) + C_{2j}\sinh\left(\xi_j y\right)\right)\sin\left(\frac{j\pi x}{a}\right), \xi_j = \frac{j\pi}{a}\sqrt{\frac{\alpha_{66}}{\alpha_{11}}} \quad (1)$$

where C_{1j} and C_{2j} are constants that need to be determined. *x* is along the span direction, where the panel is simply supported at x = 0 and *a*. a_{11} and a_{66} can be determined based on constitutive relations of the orthotropic plate. Although C_{2j} was assumed to be zero in the previous model [35] based on symmetric boundary conditions, it cannot be neglected in this study for the general case of the multi-cell section.

The DCA depends on the shear force transferred through the connectors. Fig. 6 shows the strain profile for full-composite action when the shear force fully transfers from the upper deck to the lower girder where slip (X) is equal to zero. On the other side, slip is the maximum when there is no shear force transfer (i.e., non-composite action). Partial transfer of shear force leads to partial DCA.

Based on boundary conditions, the total shear force F(x) is

$$F(x) = \frac{-1}{\eta_i} \sum_{j=1}^{\infty} \frac{1}{\xi_j} \left(C_{1j} \sinh\left(\xi_j \cdot y\right) + C_{2j} \cosh\left(\xi_j \cdot y\right) \right) \sin\left(\frac{j\pi x}{a}\right)$$
(2)

where η is shear flow distribution factor depending on the shape of the cross-section, *i* can be replaced by *r* and *l*, which represents right and left sides of the cell shown in Fig. 4, *y* is the location of the girder. Single shear connector forces can be obtained as (Fig. 5)

$$F_{s}(x) = \int_{\frac{pitch}{2}}^{\frac{pitch}{2}} F(x)dx$$

$$\frac{-1}{\eta_{i}} \cdot \sum_{j=1}^{\infty} \frac{1}{\xi_{j}} \frac{j\pi}{a} \cdot \left(C_{1j} \sinh\left(\xi_{j} \cdot y\right) + C_{2j} \cosh\left(\xi_{j} \cdot y\right) \right) \cos\left(\frac{j\pi x}{a}\right) \cdot pitch$$
(3)

where $\ensuremath{\textit{pitch}}$ is the spacing between the connectors. The moment can be defined as

$$M(x) = \sum_{j=1}^{\infty} Q_j \sin\left(\frac{j\pi x}{a}\right)$$
(4)

where

$$Q_j = \frac{2}{a} \int_o^a M(x) \sin\left(\frac{j\pi x}{a}\right) dx$$
(5)

For a deck on two girders shown in Fig. 7, i.e., single-cell, the shear flow distribution factors are equal to one. Based on the equilibrium of the moment shown in Fig. 5, we have (Fig. 6)

$$M_{1}(x) + M_{2}(x) = \left(\frac{\eta_{r} + \eta_{l}}{n}\right)M(x) - (\eta_{r} + \eta_{l})\cdot F(x)\left(C'\right)$$
(6)

where *C*' is the distance between the neutral axis of deck and the neutral axis of the girders, and *n* is the number of shear connectors for the entire section, which can be defined for single cell as $n = \eta_r + \eta_l$. It should be noted that, due to symmetry in single cell section, η_r and η_l are equal to 1 and *n* is equal to 2. For more than one cell, the distribution factor can be obtained by solving the following two equations: one equation is obtained from the continuation of stress as:

$$\frac{N(x)_{(i-1),l}}{h_s} = \frac{N(x)_{(i),r}}{h_s}$$
(7)

where h_s is the slab thickness and *i* donates the cell number. The other



Fig. 8. Sequential load-slip curve for sleeve-type shear connector [18].



Fig. 9. Bridge model test [48].



Fig. 10. Loading conditions [48].

equation can be obtained from the summation of the shear distribution factors at the intersections of the cells as [49]:

$$\eta_{(i-1),l} + \eta_{(i),r} = 1 \tag{8}$$

Chen and Yossef [35] developed a closed-form solution based on the strain difference at the deck interface and acting forces and moments on the T-section. This model can utilize the same formula considering shear distribution factors discussed in Eq. (6) as



Fig. 11. Finite Element model.

Table 1

Equivalent properties of FRP honeycomb panel [48].

E_x (MPa)	E_y (MPa)	σ_{χ}	G_{xy} (MPa)
2560	2300	0.303	560







Fig. 13. Stress distribution at mid-span for 3-cell girder system.

$$\frac{d^2 F(x)}{K \cdot dx^2} = \frac{F(x)}{\omega \cdot E \cdot A} + \alpha_{11} \cdot N_x(x) - \frac{\left[\left(\frac{\eta_{i,r} + \eta_{i,l}}{n}\right)M(x) - \left(\eta_{i,r} + \eta_{i,l}\right)F(x)\left(C'\right)\right]\left(C'\right)}{b \cdot D_{11} + \omega \cdot E \cdot I}$$
(9)

Performing differentiation of F(x), then substituting F(x), N(x), M(x) using Eqs. (1), (2) and (4), the output equation can be solved at y = b and y = 0 shown in Fig. 7 to obtain C_{1j} and C_{2j} as follows:



Fig. 14. Stress distribution at mid-span for 4-cell girder system.



Fig. 15. Deflections at mid-span for 2-, 3- and 4-cell girder system.

1100

5100

6000

Table 2

Force (N)

Force-displacement data for the shear connector. -6000

-5100

-1100

0



Fig. 16. Force-displacement curve for analytical and FE results for shear connectors.

stiffness for each stage equals to the average stiffness along the beam at the specific loading step. This is due to the influence of different stiffness when the bridge is subjected to bending, as shown in Fig. 16.

$$\begin{bmatrix} C_{1j} \\ C_{2j} \end{bmatrix} = \begin{bmatrix} B_{11} \cdot \cosh\left(\xi_{j}b\right) - \frac{1}{\eta_{1}}\left(A_{11} + C_{11}\right) \cdot \sinh\left(\xi_{j}b\right) & B_{11} \cdot \sinh\left(\xi_{j}b\right) - \frac{1}{\eta_{1}}\left(A_{11} + C_{11}\right) \cdot \cosh\left(\xi_{j}b\right) \\ B_{11} \cdot \cosh\left(\xi_{j} \cdot 0\right) + \frac{1}{\eta_{2}}\left(A_{11} + C_{11}\right) \cdot \sinh\left(\xi_{j} \cdot 0\right) & B_{11} \cdot \sinh\left(\xi_{j} \cdot 0\right) + \frac{1}{\eta_{2}}\left(A_{11} + C_{11}\right) \cdot \cosh\left(\xi_{j} \cdot 0\right) \end{bmatrix}^{-1} \cdot \begin{bmatrix} M_{11} \\ M_{11} \end{bmatrix}$$
(10)

where

$$A_{11} = \frac{pitch}{K\xi_j} \left(\frac{j\pi}{a}\right)^2, B_{11} = \alpha_{11}, C_{11} = \frac{(\eta_r + \eta_l) \cdot (C')^2}{\left[b \cdot D_{11} + \omega \cdot E \cdot I\right]\xi_j} + \frac{1}{\left[\omega \cdot E \cdot A\right]\xi_j}, M_{11}$$
$$= \left(\frac{\eta_r + \eta_l}{n}\right) \frac{Q_j \cdot C'}{b \cdot D_{11} + \omega \cdot E \cdot I}$$
(11)

b is the width of the deck between each consecutive girder; and E, A and I are Young's modulus, cross-sectional area, and moment of inertia of the steel girder, respectively. ω is beam distribution factor, which is introduced to account for external girders. It is equal to 1.5 and 1 for exterior cells and interior cells, respectively, as shown in Fig. 4. K is the shear connector stiffness, which can be calculated based on the loaddisplacement curve, as shown in Fig. 8. The figure shows three different stages for shear connector stiffness: in the first stage, the stiffness is 1.5 kN/mm; in the second stage, the stiffness increases to 7.9 kN/mm; and in the third stage, the stiffness reduces to 1.4 kN/mm.

Eq. (2) can be generalized to calculate the total shear force at any stage as follows:

$$F_{total} = F_1(x) + F_2(x) + F_3(x) + \dots + F_m(x)$$
(12)

where m is the number of stages. It should be noted that the addition of forces in Eq. (12) is cumulative depending on each stage, while the

Utilizing the generalized form shown in Eq. (12), the DCA can be expressed as:

$$DCA = \frac{\left(\alpha_{11} \cdot N_{total}(x) - F_{total}(x) \left(\frac{(\eta_{t} + \eta_{t}) \cdot (C)^{2}}{\omega \cdot E \cdot I + b \cdot D_{11}} + \frac{1}{\omega \cdot E \cdot A}\right)\right)}{M_{total}(x) \cdot \left(\frac{\left(\frac{\eta_{t} + \eta_{t}}{n}\right) \cdot C'}{\omega \cdot E \cdot I + b \cdot D_{11}}\right)}$$
(13)

DCA calculation adopted in this study takes into consideration variable stiffness along the span through the summation of all the forces. Therefore, an average value of DCA is expected, which may vary from conventional DCA calculation based on localized forces.



Fig. 17. Stress variation along panel width shown at stage 3.



Fig. 18. Strain profile along the bridge height at stage 1 at (a) corresponding location of different sections, at (b) end-span, (c) quarter-span, and (d) mid-span.

3. Finite element model

A Finite Element (FE) model is created and validated using previous experimental results, which will be used to verify the analytical model derived in the previous section. The 1:3 scaled test consisted of three steel girders (W16×36, Gr50) with a span of 5500 mm and spaced 1200 mm on center [48]. A 130 mm thick FRP honeycomb deck was connected to steel girders using steel shear connectors at 600 mm on center. Steel bracing was added between the girders to provide lateral support for the flange section, as shown in Fig. 9. The panel was loaded to 50% service load at the intersection of the middle of the left deck and midspan, as shown in Fig. 10.

Linear elastic FE model using ABAQUS is created using four-node shell elements (S4R) with mesh size of 125×100 mm for deck and 88×85 mm for girder sections, as shown in Fig. 11. Young's modulus

and Poisson's ratio for steel are 200,000 MPa and 0.3, respectively. Equivalent properties of FRP honeycomb panel are shown in Table 1 [48]. The FRP deck is connected to the steel girders using CONN3D element where all stiffnesses are set to be rigid except for the longitudinal stiffness, which is set to 1.460 kN/mm [48]. All the forces between the two interfaces are connected through connector elements, without any additional features, such as friction. Boundary conditions are set to pin and roller at both ends of the girders. An 18,000 N load is applied on an area of $600 \times 250 \text{ mm}^2$ to simulate the same loading conditions in the test [48]. Deflection is recorded at 25%, 50%, 75% and 100% of the total load (P) and compared with experimental results, as shown in Fig. 12, where good correlation is achieved.



Fig. 19. DCA at different loading steps and locations along span based on FE results.



Fig. 20. DCA at different loading steps for FE results versus analytical results.



Fig. 21. Effective width ratio (b_{eff}/b) vs. DCAs for different cell configurations.

4. Verification of the analytical model

The FE model is further used to validate the analytical model, where the load is placed over the girders at the midspan to provide symmetric loading. The validation is divided into two sections: Section 4.1 discusses constant shear connector stiffness for multi-girder bridge, while Section 4.2 presents analyses for variable shear connector stiffness.

4.1. Constant stiffness

The shear connector stiffness (K) is chosen to generate 0, 25%, 50%, 75%, and 100% DCAs, respectively. Figs. 13 and 14 show the stress distribution at mid-span for 3-cell and 4-cell bridge girder systems, respectively, where good correlation between FE and analytical model results can be observed. Fig. 15 shows the deflection for double cell, 3-



Fig. 22. EWR versus DCAs for different beam spacings at exterior cells.

cell, and 4-cell bridge girder system, with the differences ranging within 2%, 3%, and 12%, respectively.

4.2. Variable sequential stiffness

The connector stiffness at each stage is assigned according to Table 2 following ABAQUS notation [50]. The double-cell bridge is loaded on three consecutive steps with three different loads (115.38 kN, 157 kN, and 120 kN) to study the effect of variable stiffness with load increase.

The shear connector forces along the span are calculated using Eq. (12) and compared with FE results. Fig. 16 shows the connectors forces at stage 1, which follow linear slope. However, at stages 2 and 3, the connectors forces are following different slopes, i.e., different stiffness, depending on the location of the shear connector. Mid-span connectors have the lowest forces and follow the lowest stiffness (K_1) during all loading steps. Connectors located further from mid-span are subjected to higher shear forces which exceed the maximum force in stage 1 (1100 N). Therefore, connectors start to follow (K_2) stiffness. Upon exceeding the second force limit (5100 N), the connector stiffness changes to follow the last stiffness (K_3) until failure for the third group of connectors, which are near the beam-end.

The analytical model is further verified by predicting the induced stress in the deck. Fig. 17 shows that the predicted analytical stress at stage 3 correlates well with the results from the FE results.

To calculate DCA for variable stiffness, the strain profiles along the height of the bridge at mid-, quarter- and end of span are compared to strain profiles for 0% and 100% DCA FE models, as shown in Fig. 18. DCA is calculated based on strain difference at the deck-girder interface, as shown in Fig. 6.

Fig. 19 reports localized DCA at the three locations along the span of the bridge. The results show that the DCA is consistent for the first stage when all the connectors are subjected to the same stiffness (K_1) . However, when the load increases, the DCA shows a non-consistent increase as a result of variable stiffness. DCA at quarter- and end span show higher percentage increase than that at mid-span, due to the increase of connectors stiffness at quarter- and end span. Similar behavior is noticed when stiffness decreases from K_2 to K_3 , where DCA at end-span decreases by 11% while DCAs at other locations remain constant.

Fig. 20 shows a comparison between average DCA values at each loading step from FE results and Eq. (13). The results show that the analytical model can capture the variation of DCA, with less than 10% difference compared with average FE results. Based on the DCA results, lower value can be further used to calculate the EWR following the DCA-EWR curves shown in Fig. 21.

5. Parametric study

A parametric study is carried out using full-scale dimensions of 21.33 m span, 2.44 m beam spacings and 1.8 m connector spacings following literature [51]. The deck material properties are the same as

those for the validated model. However, the FRP deck depth is set to 0.254 m and W40×199 steel girders are adopted with 0.983 m beam height. Fig. 16 shows the effective width ratio (beff/b) versus DCA for 3-cell and 4-cell girder system based on the validated dimensions. Due to the difference in the axial stress caused by the shear lag effect, the effective width ratio is divided into interior and exterior ratios to show the difference. Interior and exterior zones are shown in Fig. 14. The effective width ratio (EWR) can be calculated as follows:

$$EWR = \frac{\sum_{i} \sigma_{i} \cdot b_{i}}{\sigma_{\max} \cdot b}$$
(14)

where *i* is the number of segments along the width where stress σ_i is calculated, b_i is the width of the selected segment, which can be assumed as 10 mm; σ_{max} is the maximum stress per unit cell; and b is the width of the unit cell. The effective width ratio decreases with the increase of the DCA. Based on the bridge dimensions, DCA, and the system type (double cell, 3-cell, and 4-cell), the designer can obtain the EWR for each case. For example, for a 4-cell bridge girder system with dimensions listed in Section 3 and a desired 60% DCA, the EWR are 0.86 and 0.92 for interior and exterior zones, respectively. Based on the findings, the interior connectors lead to lower EWR than the exterior connectors. For conservative design, the EWR of interior connectors at 100% DCA can be used, while the analytical model can assist the designers in achieving a more economical design based on the location of the connectors and the dimensions of the bridge.

Fig. 22 reports EWR calculated for different beam spacings at different degree of composite action. The results show that EWR reduces with the increase of the DCA and the increase of the beam spacing.

6. Conclusions

This study expands the existing analytical model to evaluate EWR of beam-on-girder bridge system with multiple shear connectors considering partial DCA. A chart is presented to obtain the EWR for different DCA that can be used for design purposes. Moreover, the model incorporates variable sequential stiffness for deconstructable sleeve-type shear connector. The DCA is calculated at each stage and compared with FE results. It can be concluded that:

- The analytical model can predict the stress/strain and deflection considering partial DCA, and the results show good agreement with Finite Element and experimental results.
- (2) The analytical model can be used for multiple girder system with variable stiffness shear connectors, including double cell, 3-cell, and 4-cell girder systems.
- (3) DCA has different effect on EWR for interior and exterior zones for multiple cell systems.
- (4) A design guideline example is provided for designers by selecting the effective width for different degrees of composite action to facilitate a conservative and economic solution.
- (5) DCA changes with the stiffness of the connectors along the span.
- (6) EWR tends to be lower at interior connectors than exterior connectors. Similarly, EWR is lower at higher DCA and lower aspect ratio.
- (7) Generalized DCA can predict the variation of the stiffness along the span and provide conservative value compared to FE results. Therefore, it can be used for design applications with variable stiffness shear connectors.

CRediT authorship contribution statement

Mohamed Y. Abd El-Latif: Methodology, Writing – review & editing, Writing – original draft. Mohamed Elsayad: Methodology, Writing – review & editing. An Chen: Conceptualization, Methodology, Writing – review & editing. Mostafa Yossef: Conceptualization, Data curation, Investigation, Methodology, Project administration, Validation, Writing – original draft, Writing – review & editing.

Declaration of Generative AI and AI-assisted technologies in the writing process

No AI tool has been used in this manuscript.

Declaration of Competing Interest

We have no conflicts of interest to disclose.

Data availability

Data will be made available on request.

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