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Research Article

Effects of CFRP Strengthening on Dynamic and Fatigue Responses of Composite Bridge

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This paper investigates the effect of CFRP strengthening on dynamic and fatigue responses of composite bridge using finite element program ABAQUS. Dynamic and fatigue responses of composite bridge due to truck load based on AASHTO standard are investigated. Two types of CFRP strengthening techniques, CFRP sheets and CFRP deck, are applied to both the damaged and undamaged bridges. For the case of damaged bridge, two through-thickness crack sizes, 3 mm and 6 mm in depth, are assumed at midspan of the steel girders. Furthermore, effects of the number of steel girders on the dynamic and fatigue responses are also considered. The results show that the maximum responses of composite bridges occur for dual lane cases. By using CFRP as a strengthening material, the maximum stress and deflection of the steel girders reduce and consequently increase the fatigue life of the girders. After introducing initial crack into the steel girders of the composite bridges, the fatigue life of the bridges is dramatically reduced. However, the overall performance of the damaged composite bridge can be improved by using CFRP, albeit with less effectiveness. Therefore, if cracks are found, steel welding must be performed before strengthening the composite bridge by CFRP.

1. Introduction

In the past 50 years, many types of bridges have been introduced in order to serve traffic growths. Due to advantage in construction time, as well as effectiveness, composite bridge appears to be one of the most popular bridge types which is widely adopted throughout the world. The main components of a composite bridge are decks constructed of reinforced concrete and supported by longitudinal steel girders. However, because of their normal deterioration, introduction of new safety standards, and increasing traffic volume and loads, a high percentage of older bridges require rehabilitation or reconstruction. Often, the choice between constructing a new bridge and rehabilitating an existing one must be made.

In most cases, rehabilitating existing bridges is more cost-effective than replacement and can be accomplished in a shorter period of time with less inconvenience to the traveling public. Various methods have been used in the past to strengthen steel bridges [1, 2]. One method used in the past on steel bridges is the bonding of steel plates to the tension flange of the girders. However, this method has several disadvantages, including difficulties in installing heavy steel plates at the bridge site, as well as the limited length of a plate that can be delivered to the site. In general, conventional strengthening techniques are labor intensive and disruptive to traffic flows.

Since 1990, application of carbon fiber reinforced polymer (CFRP) in strengthening civil engineering structures has become an attractive retrofit practice. The main advantages of CFRP in general for CFRP sheets are their high strength/weight and stiffness/weight ratios, high degree of chemical inertness in most civil engineering environments, and their nonmagnetic and nonconductive properties [3]. According to some previous research works on the use of CFRP sheets to repair and strengthen existing composite bridges, several researchers have conducted experimental tests on individual steel girders strengthened with CFRP under static loadings [4–6]. They have reported that CFRP sheets could significantly increase the ultimate load carrying capacity of intact girders and restore the ultimate load carrying capacity and stiffness of damaged composite girders. Tavakkolizadeh and Saadatmanesh [7] studied the fatigue behavior of both damaged and undamaged steel beams retrofitted with epoxy bonded CFRP. They have found that CFRP sheets not only tend to extend the fatigue life of a bridge more than three folds but also decreases the crack growth rate significantly. Miller et al. [8] demonstrated the use of CFRP in the field by strengthening a steel girder in a composite bridge. They found that the measured strain at the tension face of the steel girder reduced. For the case of replacing an old deteriorated concrete deck of a truss bridge with CFRP deck, an experimentally validated finite element (FE) model has been investigated [9, 10]. It was found that the fatigue life of the bridge after rehabilitation was doubled when compared to a prerehabilitated reinforced concrete deck system. From the above literatures, no study on the overall fatigue behavior of a composite bridge retrofitted with CFRP was noted. Hence, the effect of CFRP strengthening on the overall fatigue behavior of composite bridges is investigated in this paper.

The main objective of this paper is to investigate the effect of CFRP strengthening on the dynamic and fatigue responses of composite bridge using FE program ABAQUS. In the analysis process, dynamic behaviors and fatigue responses due to varied direction of moving truck load based on AASHTO standard are investigated. Two types of CFRP strengthening techniques, CFRP sheets and CFRP deck, are applied to both damaged and undamaged bridges. For the case of damaged bridges, two through-thickness crack sizes, 3 mm and 6 mm in depth, are assumed at midspan of the steel girders. Furthermore, the effect of the number of steel girders on the dynamic behavior and fatigue responses are also investigated.

2. Description of a Composite Bridge

The prototype bridge on which the analyses are based is a simple span composite bridge. AASHTO LRFD Bridge Design Specifications [11] were employed to design the prototype bridge in accordance with HS-20 notation live load. Material properties used for the design are shown in Table 1. The bridge is 9.93 m wide and has a length of 18 m, as shown in Figure 1.

The doubly reinforced RB9@0.10 m concrete deck is supported by three steel girders with a spacing of 4.63 m. Figure 2 shows the cross-section of the bridge, including the girders, diaphragms, and parapets.

The welded I-section girders have a depth of 1.2 m, a web thickness of 0.016 m, a flange thickness of 0.040 m, and a flange width of 0.5 m. The top flanges of the girders are attached to the reinforced concrete deck with shear studs. Diaphragms are joined at the centre and both ends of the girders. In order to study the effect of the number of steel girders on the dynamic behavior and fatigue strength, the number of steel girders of the above bridge is reassigned to be 5.

Table 1: Material properties of the prototype bridge.

Component	Properties	Value
Steel girder	Density	7850 kg/m^3
	Modulus of elasticity	200 GPa
	Poisson's ratio	0.3
RC steel	Density	7850 kg/m^3
	Modulus of elasticity	200 GPa
	Poisson's ratio	0.3
Diaphragm	Density	7850 kg/m^3
	Modulus of elasticity	200 GPa
	Poisson's ratio	0.3
Concrete	Density	2400 kg/m^3
	Modulus of elasticity	28.6 GPa
	Poisson's ratio	0.2

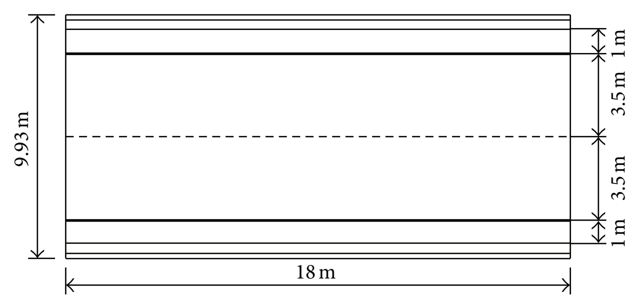


Figure 1: Plan view of the bridge.

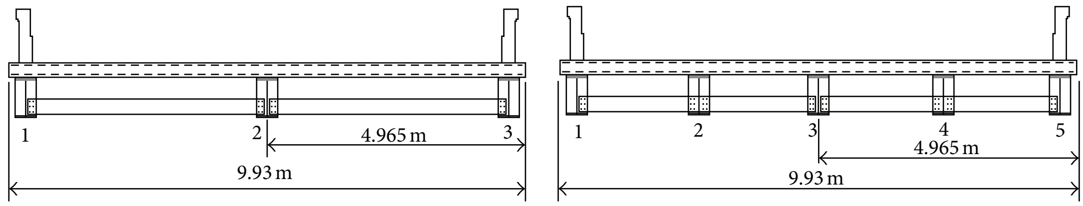
3. Finite Element Model and Verifications

A finite element model of the composite bridge was developed using ABAQUS finite element software [12].

3.1. Model Descriptions. The concrete deck, steel girders, and diaphragms are modeled with three-dimensional shell elements. Shell elements are used when the thickness dimension is significantly less than the other two dimensions. Conventional shell elements have six degrees of freedom, three for displacement and three for rotation. S4R elements are used to model the deck. This is a conventional four node, quadrilateral, stress/displacement shell element with reduced integration and a large strain formulation. It is a rectangular-shaped element with a node at each corner. The properties of the section are calculated by using the shell section option. This uses numerical integration through the thickness of the shell and is suited for solving nonlinear problems, in this case the analysis of a reinforced concrete deck. In this option the thickness of the shell and the number of integration points are defined.

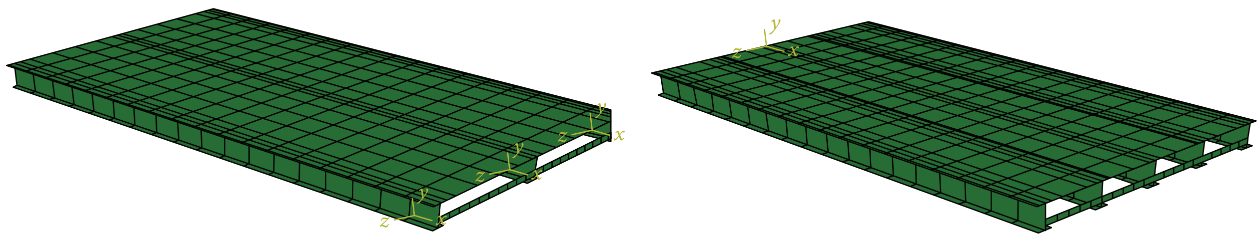
Once all the parts have been modeled and the section properties for each have been defined, the next step is to assemble these parts together using contact element. This allows forces and moments to transfer between parts. Figure 3 shows the FE model of the composite bridges.

For the case of composite bridge strengthened by CFRP sheets, CFRP sheets attaching to the bottom flanges of the



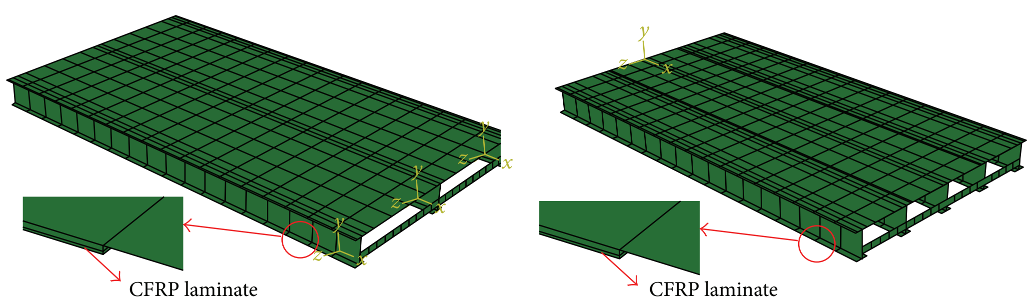
(a) 3 girders (b) 5 girders

Figure 2: Cross-sections of the composite bridges.



(a) 3 girders (b) 5 girders

Figure 3: Finite element model of the composite bridges.



(a) 3 girders (b) 5 girders

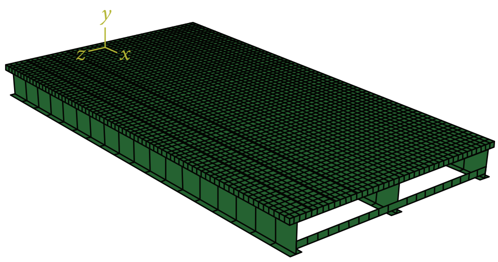
Figure 4: Finite element model of the composite bridges with CFRP laminate.

steel girders are modeled using 3-dimensional solid element, namely, C3D8R. The thickness and the width of CFRP sheets are 1.4 mm and 0.5 m, respectively. Special adhesive elements which are useful in modeling adhesives and bonded interfaces, namely, COH3D8 available in ABAQUS, are adopted into the FE model at the 3 mm thickness interfaces between the steel surface and CFRP sheets. For the case of composite bridge strengthened by changing concrete bridge deck to a CFRP composite deck, the CFRP deck panels are modeled as a sandwich-typed construction, using S4R elements, consisting of top and bottom face skins and a core component. Each cell of the cubic CFRP block has a dimension of 25 cm × 25 cm × 25 cm. The thickness of top and bottom face skins and a core component is 1 cm. Figures 4 and 5 show the FE model of the strengthened composite bridges.

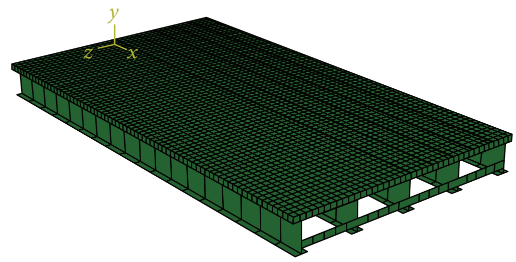
Material properties assigned to the FE model are summarized in Tables 1 and 2.

Table 2: Properties of special materials.

Component	Properties	Value
FRP composite deck	Density	1800 kg/m^3
	Modulus of elasticity	19.3 Gpa
	Poisson's ratio	0.33
	Max. tension stress	207 Mpa
	Max. comp. stress	207 Mpa
FRP laminate	Density	1500 kg/m^3
	Modulus of elasticity	165 Gpa
	Poisson's ratio	0.30
	Max. tension stress	2.80 Gpa
Adhesive	Density	1770 kg/m^3
	Modulus of elasticity	12.8 Gpa
	Poisson's ratio	—
	Max. tension stress	33 Mpa



(a) 3 girders



(b) 5 girders

FIGURE 5: Finite element model of the composite bridges with CFRP deck.

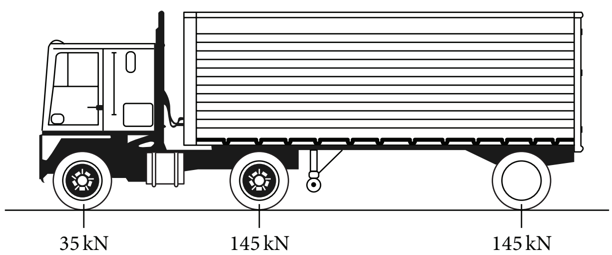


FIGURE 6: AASHTO-type HS20.

3.2. Loading Configurations. The applied loadings correspond to the tire loads of standard AASHTO-type multiaxle truck HS-20, as shown in Figure 6. Three directions of moving trucks are considered: single lane and dual lane with the same traffic direction and dual lane with opposite traffic direction.

The dynamic behaviors of the composite bridges are studied systematically by means of FE techniques performed in (semi)discretisation in spatial coordinates which is applicable to any type of structure, including the nonlinear type of behavior [13]. As a result, a discrete N-degree of freedom (N-DOF) system of equations is obtained:

$$M\ddot{d} + C\dot{d} + Kd = f(t), \tag{1}$$

where M, C, and K are the mass, damping, and stiffness matrices, respectively. $f(t)$ is the load vector (from the moving loads), d is the vector of nodal displacements, $\dot{d}$ is the vector of velocity, and $\ddot{d}$ is the vector of acceleration.

In order to integrate (1) with respect to time, generally a modal analysis and reduction leading to a reduced number of significant eigenmodes $n \ll N$ are performed, arriving at uncoupled equations which are integrated by standard time integration techniques such as by β-Newmark method. One simplest procedure to represent the load train is to apply load pulse time history for each node, depending on the time of arrival and discretisation. Therefore, in order to simulate the moving load, one may apply forces and moments as function of time to all nodes of the FE model of the whole structure. As shown in Figure 7, a concentrated force moves with velocity V from node 1 to node n of the beam which is composed of n nodes and $n-1$ elements.

When a beam is subjected to a concentrated force, P, the forces on all nodes of other beams are equal to zero. The value of the force at the node in the element that is subjected to the concentrated force is a function of time, as shown in Figure 8.

Ignoring moments at each end of each element (neglecting $f_2^{(s)}(t)$ and $f_4^{(s)}(t)$ in Figure 8), a simple linear interpolation for the forces would allow the whole procedure to be generalized much more easily, as shown in the following [14] (Figure 9):

$$f_1^{(s)}(t) = P\left(1 - \frac{x}{l}\right),$$
$$f_3^{(s)}(t) = P\left(\frac{x}{l}\right). \tag{2}$$

The time (t) during which the concentrated force moves with velocity V from node 1 to node i of the beam can be found from the following equation:

$$t_i = \frac{(i-1)\,\Delta x}{V}, \quad i = 1, 2, \ldots, n, \tag{3}$$

where Δx is the element length ($x_i - x_{i-1}$), and V is the train velocity.

3.3. Verification of the Finite Element Model. In order to verify the accuracy of the FE model of the composite bridges, equations for calculating the natural frequency of the bridge, introduced by Biggs and Suer [15], are employed. The equations are shown below:

$$f = \lambda^2 f_{sb}, \tag{4}$$

where λ = {1 for simple beam, 1.25 for pinned-clamped beam, and 1.5 for clamped-clamped beam}. Consider

$$f_{sb} = \frac{\pi}{2L^2}\sqrt{\frac{E_b I_b g}{w}}, \tag{5}$$

where L = the span length, g = acceleration due to gravity, $E_b I_b$ = flexural rigidity of the composite steel girder, and w = the weight per unit length of the composite steel girder.

From literature reviews, (4) and (5) are sufficiently accurate for calculating the natural frequency of a simply supported one-span bridge [16]. Therefore, by using (4) and (5),

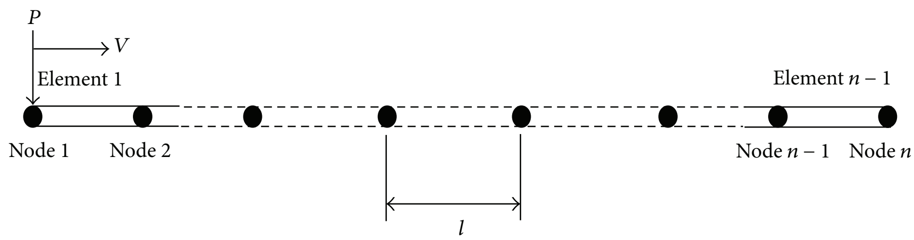


Figure 7: A beam subjected to a concentrated force, P, moving with velocity, V.

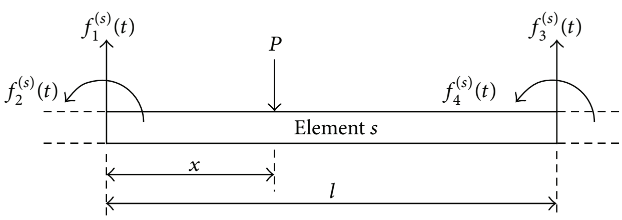


Figure 8: The equivalent forces of element s subjected to a concentrated force P.

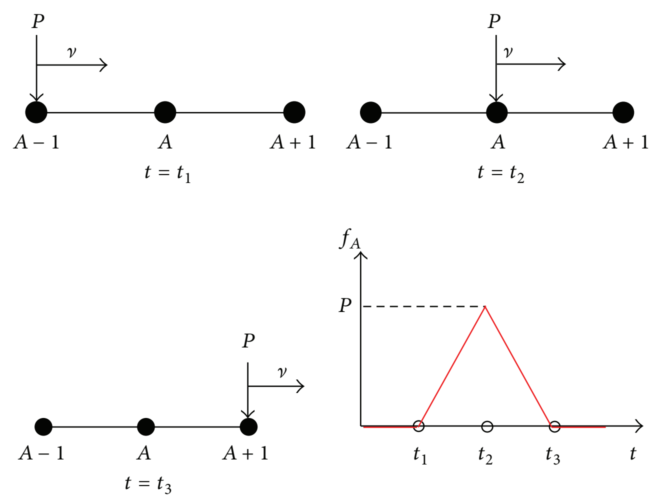


Figure 9: Definition of load at node A for moving load P in the finite element model.

the natural frequency of the bridge can be obtained. Thereafter, natural frequency analyses of the FE model of the composite bridge have been carried out. Comparison of natural frequencies of the bridges is shown in Table 3. From the results, it can be seen that results from the FE models show good agreement with Biggs's equations. This implies that the accuracy of the FE model of the composite bridges used in this study is acceptable.

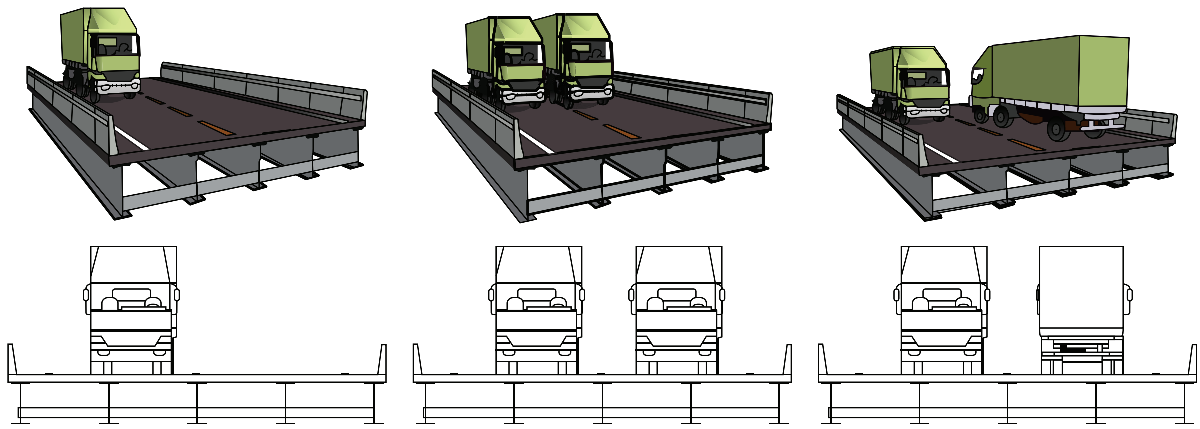
4. Case Studies and Analysis Results

4.1. Dynamic Analyses of Unstrengthened Composite Bridges under Moving Loads. In this part, dynamic analyses of unstrengthened composite bridges under various directions of moving trucks have been carried out in order to determine the direction of moving truck which gives the maximum response. The passing directions investigated in this study were single lane and dual lane with the same traffic direction and dual lane with opposite traffic direction, as shown in Figure 10.

Furthermore, the effect of the number of steel girders on the dynamic response was also investigated in this study. A constant truck speed was assumed at 80 km/hr. By performing FE simulation, the dynamic response of the composite bridges was obtained in terms of displacement and stress. It was found that maximum responses obtained from all FE analyses occurred at midspan of the bridge. The patterns of responses, that is, stress and displacement, are similar for all cases of moving truck, as shown in Figure 11. The dynamic responses for all cases are summarized in Table 4.

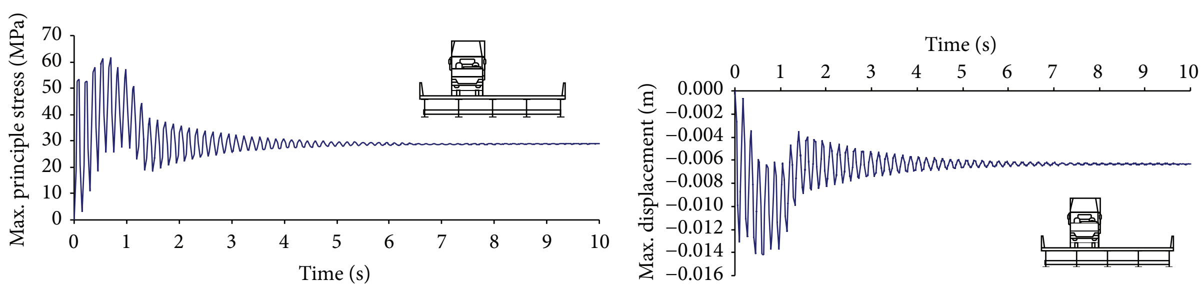
From Table 4, it can be seen that the maximum responses of composite bridges occur for the case of dual lane with the same traffic direction and dual lane with opposite traffic direction for both the 3-girder bridge and the 5-girder bridge. For the case of the 3-girder bridge, the deflection response and the stress response increase about 11% and 13%, respectively, compared to the single lane scenario due to the increase in truck weight on the bridge. For the case of the 5-girder bridge, the deflection response and stress response slightly increase, about 4.3% and 2.2%, respectively, compared to the single lane case. Figure 12 depicts the graphical comparison of the results. The small increment of dynamic responses for the case of the 5-girder bridge is due to the increase in the bridge stiffness. Therefore, based on the above studies, dual lane with opposite traffic direction of moving trucks is adopted for further investigation.

4.2. Effect of CFRP Strengthening on the Dynamic Response of Undamaged Composite Bridges. In this part of the study, CFRP has been applied to strengthen undamaged composite bridges. From the results of Section 4.1, the moving truck load in dual lane with opposite traffic direction was applied into the FE model in order to investigate the effect of CFRP strengthening on the dynamic and fatigue responses of the composite bridge. Dynamic responses were measured in terms of stress and displacement responses, whereas fatigue response was evaluated based on fatigue response formulae



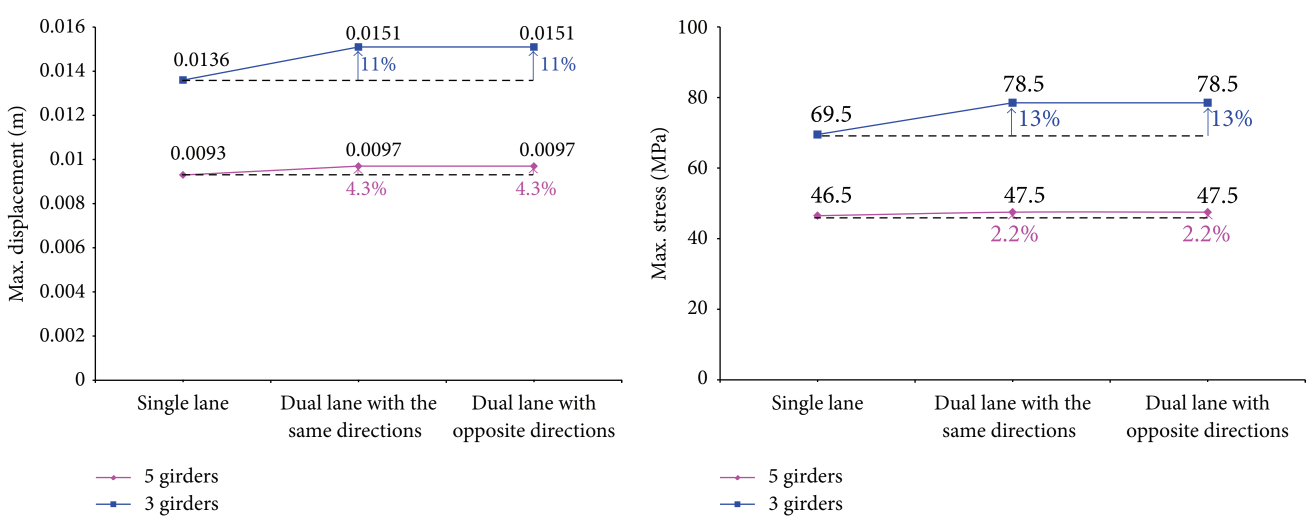
(a) Single lane (b) Dual lane with the same traffic direction (c) Dual lane with opposite traffic direction

FIGURE 10: Passing directions of moving trucks.



(a) Stress time history (b) Displacement time history

FIGURE 11: An example of dynamic responses extracted from the FE analysis.



(a) Maximum displacement versus traffic direction (b) Maximum stress versus traffic direction

FIGURE 12: Schematic of dynamic responses of unstrengthened composite bridges.

Table 3: A comparison of natural frequency of the bridges.

Number of steel girders	Type of composite bridges	Natural frequency (Hz)		Percent difference (%)
		From (4) and (5)	From FE model	
3	Unstrengthened/strengthened with CFRP sheets	7.894	7.794	1.27
	Strengthened with CFRP deck	10.476	10.423	0.51
5	Unstrengthened/strengthened with CFRP sheets	9.398	9.324	0.79
	Strengthened with CFRP deck	11.106	10.921	1.67

Table 4: Dynamic responses of unstrengthened composite bridges.

Number of steel girders	Direction of moving trucks	Max. deflection (m)	Max. principal stress (MPa)
3	Single lane	0.0136	69.5
	Dual lane with the same directions	0.0151	78.5
	Dual lane with opposite directions	0.0151	78.5
5	Single lane	0.0093	46.5
	Dual lane with the same directions	0.0097	47.5
	Dual lane with opposite directions	0.0097	47.5

(6) to (8) specified in AASHTO-LRFD design specifications. Consider

$$y = \frac{A}{n(365)(\mathrm{ADTT})_{\mathrm{SL}}(\Delta F)_n^3}, \quad (6)$$

where A = a constant value (for category $C = 14.4 \times 10^{11}\,\mathrm{MPa}^3$), n = the number of stress range cycles per truck passage (obtained directly from the FE analysis), $(\Delta F)_n$ = the nominal fatigue resistance in which $(\Delta F)_n \geq (1/2)(\Delta F)_{\mathrm{TH}}$. Consider

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{1/3}, \quad (7)$$

$$N = n(365)(y)(\mathrm{ADTT})_{\mathrm{SL}}, \quad (8)$$

where N = the number of cycles, $(\Delta F)_{\mathrm{TH}}$ = the constant fatigue threshold (for category $C = 69$ MPa), and $(\mathrm{ADTT})_{\mathrm{SL}}$ = the average daily truck traffic in a single lane.

The process in obtaining the stress range and the number of stress range used in the fatigue response calculation begins with FE analysis of the studied bridge to get the time response. By using rainflow counting method, the stress range and the number of cycles are obtained from the time response and shown as a stress range histogram. After extracting both the stress range and the number of stress cycles using the above process, the fatigue life of the composite bridge can be evaluated using (8). Full results of the FE analyses are summarized in Table 5.

From Table 5, it can be seen that midspan deflections of both the undamaged 3-girder bridge and the 5-girder bridge have been reduced after being strengthened by CFRP. By using CFRP sheets, the midspan deflections reduced from 0.015 m to 0.0134 m (about 10.7%) and from 0.0097 m to 0.0082 m (about 15.5%) for the 3-girder and the 5-girder composite bridges, respectively. Moreover, if the concrete deck is replaced by a CFRP deck, the reductions of midspan deflections of both the composite bridges are about 34.7% and 20.6%, respectively. The reduction of midspan deflection is due to the increase in the bridge stiffness after attaching CFRP to the steel girders. Figure 13 illustrates graphical comparison of the results.

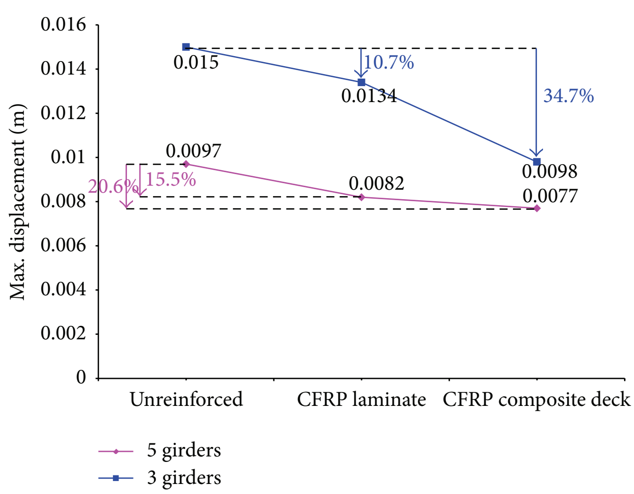
From the results of midspan stress, one can say that reduction of maximum stress occurring at midspan leads to an increase in the fatigue life of the composite bridge. The results in Table 5 show that, by using CFRP sheets, the fatigue lives increase from 25×10^6 cycles to 38.87×10^6 cycles (about 1.6 times) and from 112.24×10^6 cycles to 200.39×10^6 cycles (about 1.8 times) for the 3-girder and the 5-girder composite bridges, respectively. If CFRP deck is used, the fatigue lives of both the composite bridges increase about 11 times and 5.1 times, respectively. It can be seen that, by reducing the weight of the superstructures, the fatigue life of the composite bridges can be increased. Therefore, from the results of both midspan deflection and fatigue response of the composite bridges, it can be concluded that CFRP material can be used to improve the fatigue life of composite bridges.

4.3. Effect of CFRP Strengthening on Dynamic Response of Damaged Composite Bridge. For the case of damaged composite bridge, two through-thickness crack sizes, 3 mm and 6 mm in depth (about 7.5% and 15% of the total flange depth), were assumed at midspan of the steel girdersas shown in Figure 14. At a small region, approximately less than ten times the crack tip opening, a fine mesh was modeled to ensure accuracy of the results. In a large strain analysis, an absolute sharp crack tip should not be adopted. Therefore an initial small notch radius was defined at 0.03 mm.

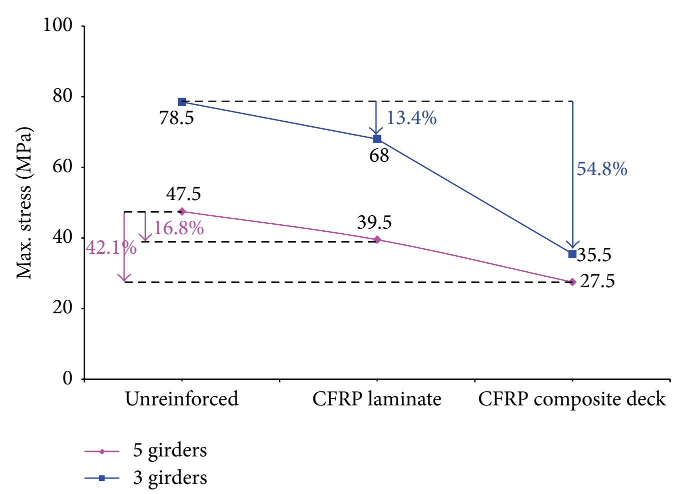
The studied bridges were then analyzed under dual lane configuration with opposite direction of moving trucks. Dynamic responses of the composite bridge were measured

Table 5: Full results of FE analyses of undamaged composite bridges.

Number of steel girders	Type of composite bridges	Midspan deflection (m)	Max. principal stress (MPa)	Fatigue life (N) ($\times 10^6$ cycles)
3	Unstrengthened	0.015	78.5	25.19
	Strengthened with CFRP sheets	0.0134	68	38.87
	Strengthened with CFRP deck	0.0098	35.5	278.13
5	Unstrengthened	0.0097	47.5	112.24
	Strengthened with CFRP sheets	0.0082	39.5	200.39
	Strengthened with CFRP deck	0.0077	27.5	572.69



(a) Maximum displacement versus traffic direction



(b) Maximum stress versus traffic direction

Figure 13: Schematic of dynamic responses of undamaged composite bridges.

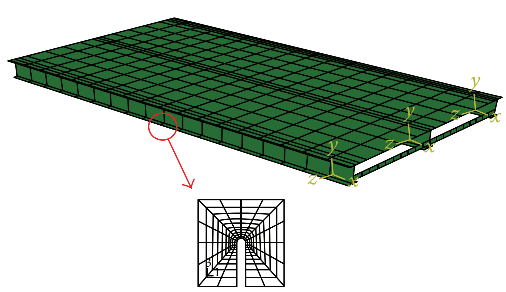


Figure 14: Finite element meshes in the crack tip region.

in terms of stress and displacement responses, whereas fatigue response was evaluated based on fatigue response formulae (8) specified in the AASHTO-LRFD design specifications. Full results of the FE analyses are summarized in Tables 6 and 7.

From Tables 6 and 7, strengthening the bridge by attaching CFRP sheets to the bottom flanges of the damaged steel girders can reduce midspan deflections from 0.0171 m to 0.0155 (about 9.4%) and from 0.0107 m to 0.0094 m (about 12.1%) for the 3-girder and the 5-girder composite bridges, respectively. However, when the concrete deck was replaced by a CFRP deck, the midspan deflection was even smaller than using CFRP sheets. The reductions of midspan deflections of both the composite bridges are about 28.7% and 18.7%, respectively.

After introducing initial cracks into the steel girders, the fatigue lives of the bridges were dramatically reduced from 25.19×10^6 cycles to 1.64×10^6 cycles (or about 15.31 times) and from 112.24×10^6 cycles to 7.23×10^6 cycles (or about 15.52 times) for the 3-girder and the 5-girder composite bridges, respectively. Strengthening the composite bridge not only reduced the midspan deflection but also increased the fatigue life of the bridge. The results from the FE analyses show that, for the case of a 3 mm crack in the 3-girder bridge (see Table 6), the fatigue lives increased from 1.64×10^6 cycles to 3.5×10^6 cycles (about 2.13 times) and from 1.64×10^6 cycles to 8.32×10^6 cycles (about 5.07 times) when strengthened by CFRP sheets and CFRP deck, respectively. The same trend on the results can also be observed for the remaining cases. The fatigue lives of the studied bridges were further reduced when the size of the initial crack was increased to 6 mm (see Table 7). A reduction in the fatigue lives by a factor as large as two was found in this study. Again, with the help of CFRP, both the fatigue life and the deflection response of the bridge can be improved.

Table 6: Full results of FE analyses of damaged (with 3 mm crack) composite bridges.

Number of steel girders	Type of composite bridges	Midspan deflection (m)	Max. principal stress (MPa)	Fatigue life (N) ($\times10^6$ cycles)
3	Damaged and unstrengthened	0.0171	92.8	1.64
	Damaged and strengthened with CFRP sheets	0.0155	81	3.5
	Damaged and strengthened with CFRP deck	0.0122	57.8	8.32
5	Damaged and unstrengthened	0.0107	56.1	7.23
	Damaged and strengthened with CFRP sheets	0.0094	51	12.15
	Damaged and strengthened with CFRP deck	0.0087	38.3	28.47

Table 7: Full results of FE analyses of damaged (with 6 mm crack) composite bridges.

Number of steel girders	Type of composite bridges	Midspan deflection (m)	Max. principal stress (MPa)	Fatigue life (N) ($\times10^6$ cycles)
3	Damaged and unstrengthened	0.0171	116.4	1.31
	Damaged and strengthened with CFRP sheets	0.0155	101.8	1.75
	Damaged and strengthened with CFRP deck	0.0122	63.2	5.91
5	Damaged and unstrengthened	0.0107	67.3	5.26
	Damaged and strengthened with CFRP sheets	0.0094	53.4	10.51
	Damaged and strengthened with CFRP deck	0.0087	41.9	20.59

5. Conclusions

A series of finite element analyses on dynamic and fatigue responses of composite bridges has been carried out in order to investigate the effect of CFRP strengthening on the overall behavior of composite bridges. Three-dimensional finite element models of composite bridges, that is, undamaged composite bridges with 3 and 5 girders, undamaged composite bridges with 3 and 5 girders strengthened by CRFP sheets and CFRP deck, and damaged composite bridges with 3 and 5 girders strengthened by CRFP sheets and CFRP deck, were modeled and analyzed under moving truck load. The dynamic behavior of undamaged bridges under moving trucks was first investigated. It was found that the maximum responses of composite bridges occur for the case of dual lane with the same traffic direction and dual lane with opposite traffic direction for both the 3-girder and the 5-girder bridges.

In order to investigate the effect of CFRP strengthening on the dynamic and fatigue response of the composite bridge, two types of CFRP strengthening techniques, CFRP sheets and CFRP deck, are adopted into the FE model. The results from the FE study can be summarized as follows.

(i) There are significant reductions in both the midspan deflection and the maximum stress after using CFRP as strengthening material. Respective percentage reductions as large as 34.6% and 54.8% were found in the analyses.

(ii) Comparing performance between the two techniques, CFRP deck appears to be more superior to CFRP sheets. Nevertheless, replacing concrete deck with CFRP deck is rather more complicated. Therefore careful justification should be conducted.

For the case of damaged composite bridges, revealing results from the FE analyses are obtained and are summarized below.

(i) After introducing initial cracks into the steel girders of the composite bridges, the fatigue life of the bridges dramatically reduced about 15-folds.

(ii) Using CFRP to strengthen the damaged steel girders, the overall performance of the composite bridge can be improved. However, the level of improvement is not quite satisfactory since the fatigue life of the strengthened bridge is by far less than the undamaged/unstrengthened case. Therefore, if cracks are found during an inspection process, steel welding must be performed before strengthening the bridge by CFRP.

Conflict of Interests

The authors declare that there is no conflict of interests regarding the publication of this paper.

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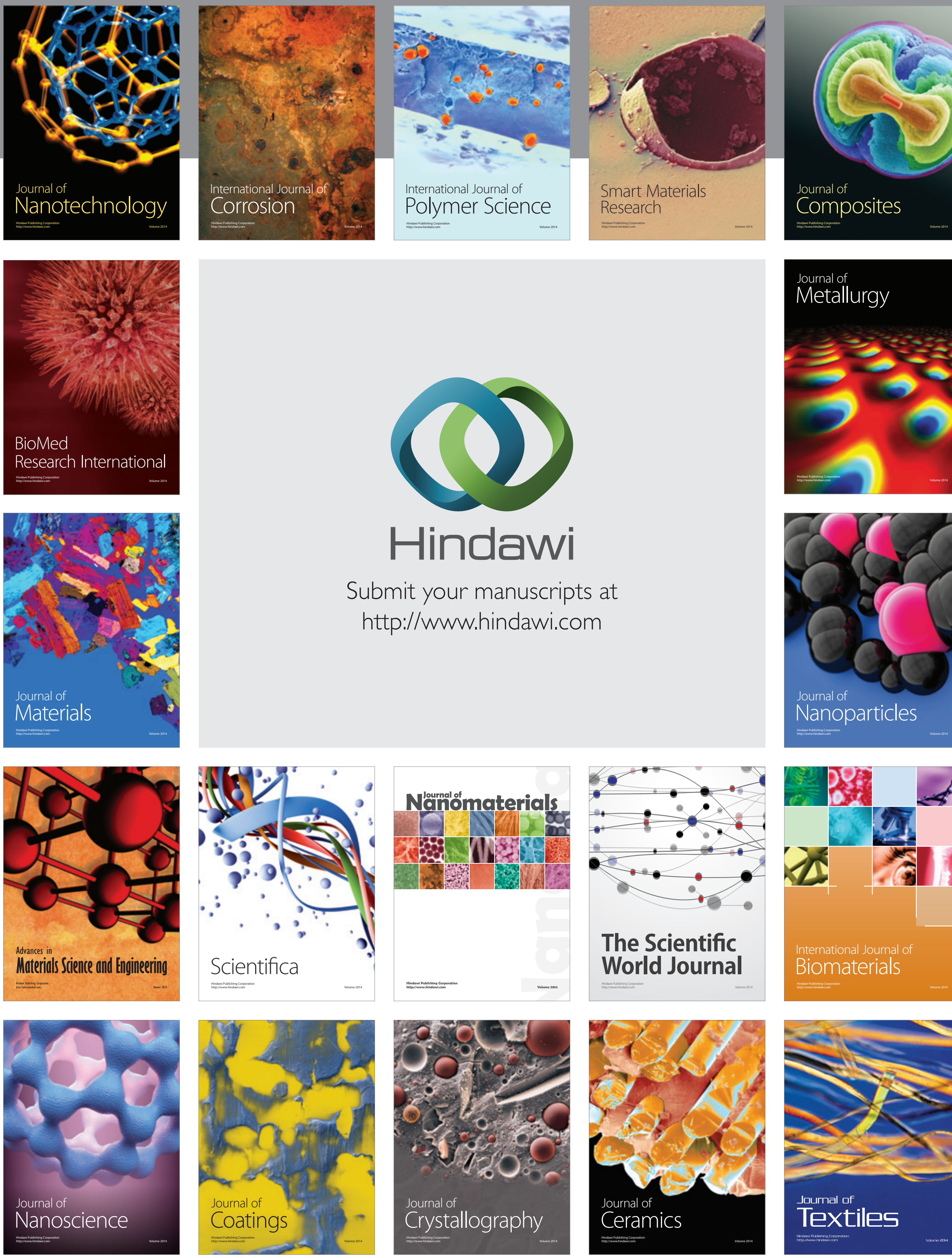
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