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Finite Element Analysis of Cable-Stayed Bridges under the Effect of Accidental Loss of a Stay Cable

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Abstract: One of the main structural components of cable-stayed bridges is the post-tensioned stay cables, which stretch diagonally from the pylons to support the horizontal deck. These essential members may be lost due to vehicular collision, excessive corrosion of connections, or wind dynamic forces. Design code provisions generally require that the bridge is designed considering the likely event of accidental loss of any one cable. However, the effect of this loss has not been studied in great detail. The present paper is part of a series of extensive parametric static and dynamic studies aimed at developing efficient design rules. In the paper, the response of a cable-stayed bridge to accidental breakage of any one stay cable is investigated. A comprehensive 3-D finite element model of the bridge is developed to provide realistic member internal actions. A 3-D cable element is utilized to represent the stay cables, and geometric nonlinearities are accounted for in the study. The static effect of loss of a stay cable on the structural behaviour of the deck is examined. The study builds on previously developed in-house optimisation method for evaluating the optimum post-tensioning cable forces.

1. Introduction

Cable-stayed bridges have been known since the beginning of the 18th century, but great interest has been shown for long-span cable-stayed bridges only in the last four decades. Cable-stayed bridges provide an economical solution for the range of medium to long-span bridges, and offer a range of options to designers regarding not only the materials, but also the geometric arrangements (Hassan et al. 2012). Compared with suspension bridges, cable-stayed bridges are stiffer and require less material, especially for cables and abutments. Moreover, they are elegant, and much easier to erect (Troitsky, 1988).

Typically, stay cables of a cable stayed bridge are post-tensioned to off-set the effect of the bridge dead load (Gimsing, 1997). The post-tensioning cable forces directly influence the performance and the economics of the bridge, as they control the distribution and magnitude of the internal forces, adjust the bridge deck profile, and affect the overall design of the bridge (Hassan et al. 2012). The post-tensioning cable forces minimize the vertical deflection of the deck, and the lateral deflection of the pylon along the longitudinal direction of the bridge. Accordingly, the bending moment distribution along the deck becomes equivalent to that of a continuous beam resting on a series of rigid supports located at the cable-deck connections, and the pylon tends to behave as a pure axial member (Hassan 2013).

Stay cables in a cable stayed bridge are exposed to corrosion, abrasion and fatigue processes which may cause a reduction in their cross-section and a reduction in their resistance capacity (Mozos and Aparicio 2010). This reduction in their capacity to withstand the forces transferred by the deck and the pylon can cause the fracture of the cable, as it happened in the Zárate-Brazo Largo Bridge, Argentina, in 1996

((Andersen et al. 1999). Moreover, stay cables may break due to a collision resulting from a vehicular accident, lack of maintenance over a long period of time, or excessive corrosion of the connection. It is also possible that similar behavior may occur due to loosening of a cable before its replacement. Therefore, the accidental event of the failure of a stay has been incorporated into some design codes and recommendations, such as the Canadian Highway Bridge Design Code (CHBDC).

The objective of this study is to investigate effects of the loss of a stay cable on the structural behaviour of a particular cable-stayed bridge configuration. The response of the deck is examined. A comprehensive 3-D finite element model of a cable-stayed bridge is developed to provide realistic member internal actions using the finite element code SAP2000. A 3-D cable element is utilized to represent the stay cables, and the three sources of geometric nonlinearities, namely, large displacement, P-Δ, and cable sag effects are considered in the model. Design rules are developed for the static analysis of these components under sudden cable loss. The study contributes to the growing knowledge of the behavior of cable-stayed bridges and will eventually help improve future code provisions.

2. Description of the Bridges

The geometry of the selected bridge for this study is similar to the Quincy Bayview Bridge, located in Illinois, USA. The length of the main span (M) is 285.6 m, with two side spans (S) of 128.1 m. Therefore, the total length of the bridge (L) is 541.8 m, as shown in Figure 1(a).

The deck superstructure is supported by double planes of stay cables in a semi-harp type arrangement, where forty cables are anchored into each transverse H frame-shaped pylon. As such, eighty stay cables support the entire bridge deck, with forty supporting the main span and twenty cables supporting each side span.

The bridge has two lanes of traffic with a width of 12.2 m measured between centers of the cables. The typical cross-section of the bridge deck (Figure1(b)) consists of a precast concrete deck having a thickness of 0.23 m and a width of 14.20 m. Two steel main girders are located at the outer edge of the deck. These girders are interconnected by a set of equally spaced floor steel beams. The distance between each pair of floor steel beams is 9.0 meters

The pylons consist of two concrete legs, interconnected with a pair of struts. The upper strut cross beam connects the upper legs and the lower strut cross beam supports the deck. The lower legs of the pylon are connected by a 1.22 m thick wall, which is placed as a web between the two legs, as shown in Figure 1(c).

3. Finite Element Modelling of the Bridge

In general, the elastic stay cables are assumed to be perfectly flexible and to resist a tensile force only. The inclined stay cables of cable-stayed bridges will sag into a catenary shape due to their self-weight (Hassan et al. 2013a). The tension stiffness of a cable, which varies depending on the sag, is modeled by using the cable element provided by the SAP2000 analysis program.

On the other hand, three-dimensional frame elements are used to model the deck and the pylon. The deck is modeled using a single spine passing through its shear center. The axial stiffness of the deck and the moments of inertia about the vertical and transverse axes are obtained by converting the concrete slab to an equivalent steel section, using the ratio of the two elastic moduli. The finite element model of this bridge is shown in Figure 1(d).

Table 1 Material properties of the bridge.

Steel	Modulus of elasticity, (Es)	= 200 GPa
	Unit weight, (γ_s)	= 77 kN/m^3
	Poisson's ratio, (v_s)	= 0.30
	Yield strength, (F_y)	= 350 MPa
	Unit price (C_{steel})	= 12,000 \$/ton
Concrete	Modulus of elasticity, (E_c)	= 24.87 GPa
	Unit weight, (γ_c)	= 24.0 kN/m^3
	Poisson's ratio, (v_c)	= 0.20
	Compressive strength, (f_c')	= 30 MPa
	Unit price ($C_{concrete}$)	= 4,218 \$/m^3
Cables	Modulus of elasticity, (E_{sc})	= 205 GPa
	Unit weight, (γ_{scable})	= 82.40 kN/m^3
	Ultimate tensile strength, (T_{cCable})	= 1.6 GPa
	Unit price (C_{cable})	= 60,000 \$/ton
Reinforcement steel	Yield strength, (f_y)	= 400 MPa
Asphalt	Unit weight, ($\gamma_{Asphalt}$)	= 23.5 kN/m^3

4. **Design Loads**

In this study, the effect of the cable loss on the structural behaviour of the bridge is evaluated under the combined effect of the following loads:

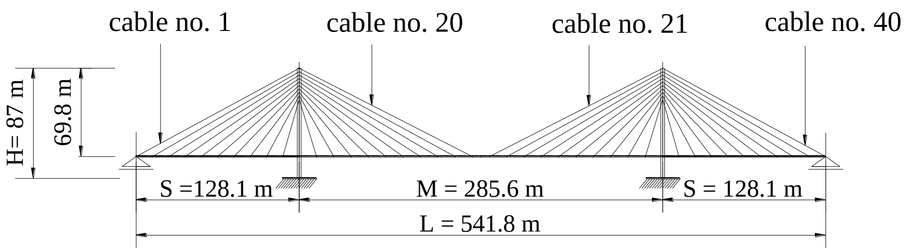
1. The self-weight of the bridge includes the weight of all structural components and appendages fixed to the structure, such as wearing surface, and traffic barriers.
2. The initial post-tensioning cable forces, which are calculated by an in-house developed optimization method (Hassan et al. 2013b)
3. All possible live load cases depicted in Figure 2, as provided by Walther et al. (1988).
4. In the current study, the magnitude of the design live load is assumed to be 9 kN/m/lane as specified by the CHBDC.

5. Numerical study

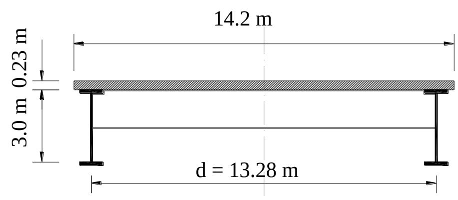
In order to illustrate the effect of the sudden accidental breakage of any one stay cable, all cables (cable 1 to cable 40 shown in Figure 1(a)) are considered and the effect of each cable loss on the behaviour of the different structural components is studied. Also, all live load scenarios shown in Figure 2 are applied to the bridge structure.. For the purposes of this paper, only six cable loses are presented (Cable 1, 5, 10, 11, 15 and 20), i.e. cables at the extreme end of the deck, between the extreme end and a pylon location, at close proximity to a pylon, at the mid-span of the deck, and between the mid-span of the deck and a pylon location. Also, only two load cases are presented in the present paper; load case 1 (Figure 2) and the bridge under its own weight and post-tensioned cable forces. Only the effect on the deck forces, moments and deflections are presented in the paper.

6. Results and Discussion

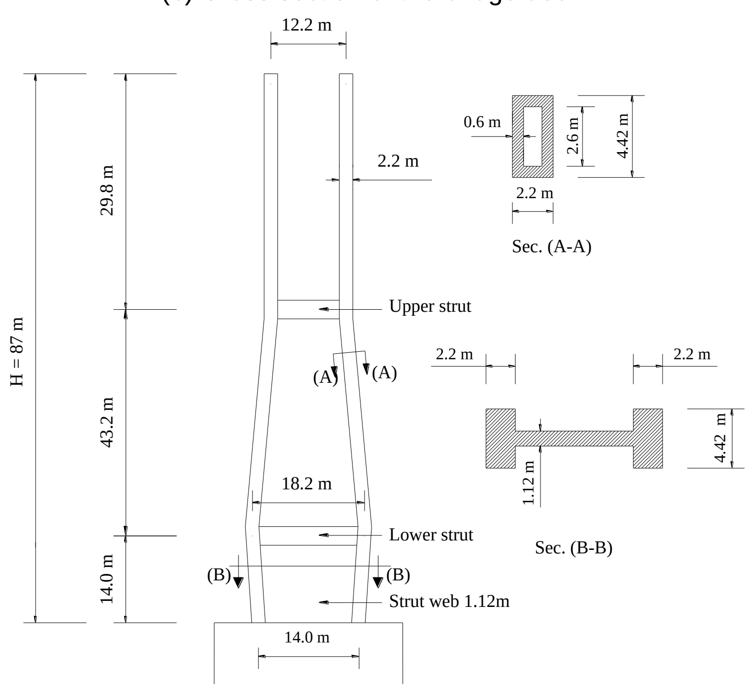
The detailed nature of the study provided extensive results which cannot all be reproduced in this paper. For the sake of brevity and clarity, only the results for the deck of the bridge under the load cases “Target” and Case (LL1) are presented in this section.The load case “Target” corresponds to when the bridge is not loaded, i.e. only the self-weight is acting on the bridge. Presenting these results enables a better assessment of the effects of the loads as well as the effects of the cable loss on the behaviour of the bridge.



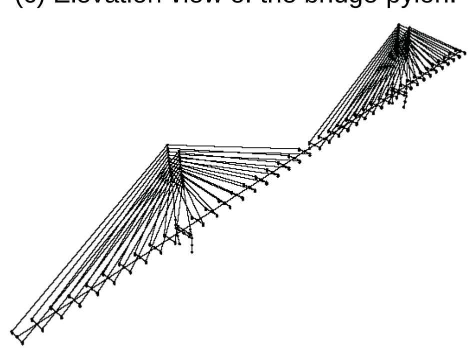
(a) Cables number and geometry of the bridge.



(b) Cross section of the bridge deck.



(c) Elevation view of the bridge pylon.



(d) Finite element model.

Figure 1: Geometry and finite element model of the bridge.

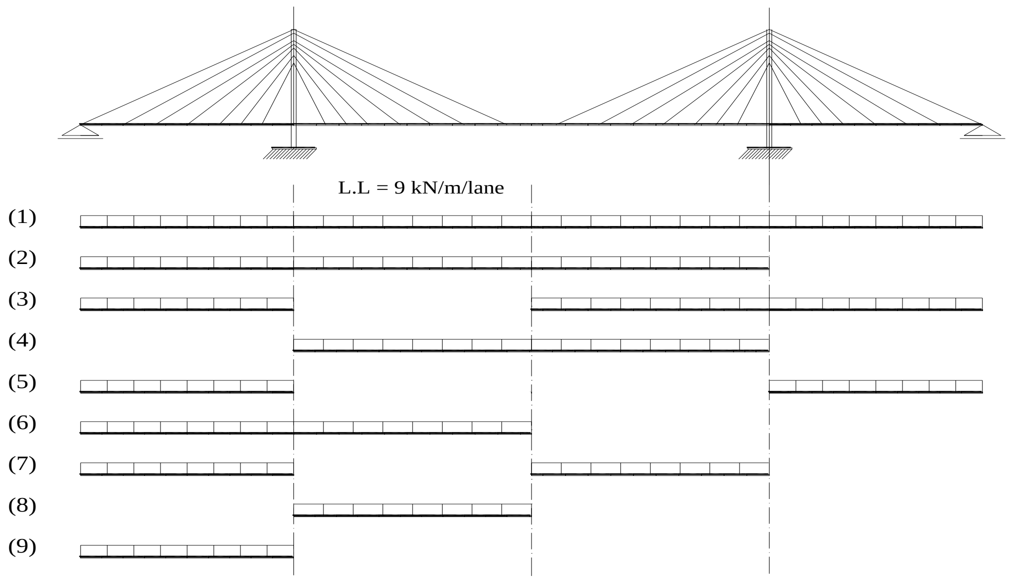


Figure 2: Live load cases used in the optimization design technique.

The next sections present cases when cables 1, 5, 10, 11, 15 and 20 are removed individually. Cable 0 refers to the case where all stay cables are present in the bridge and forms the basis for comparing the effect of losing a cable. It is also noted that the thick vertical grey lines in the figures below represent the locations of the pylons of the cable-stayed bridge.

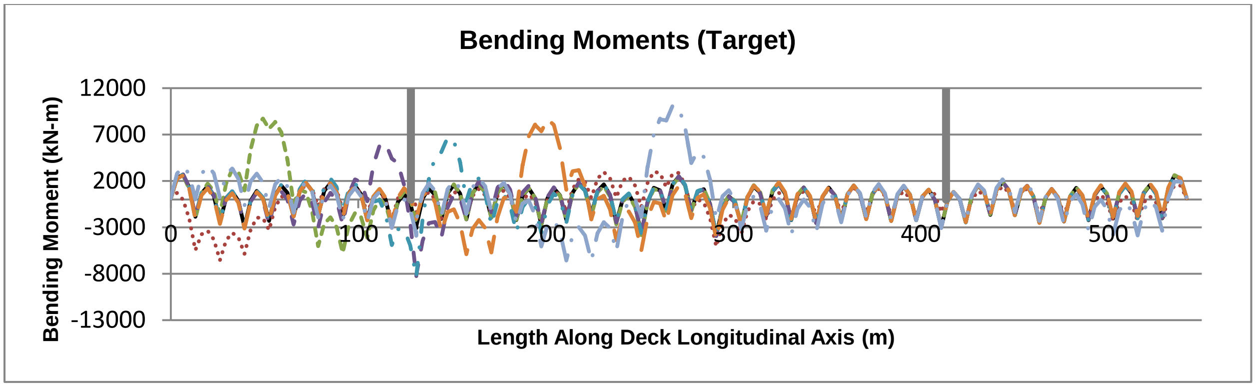
6.1 Bending Moments

Figure 3 shows the bending moment distribution along the length of the deck for the cases of Cable 0 and each of the six cables removed under the two load cases studied. As evident in Figures 3 (a) to (i) below, the effect of a cable loss on the bending moment of the deck is significant in the vicinity of the lost cable. The bending moment can increase significantly when compared to the cable 0 case.

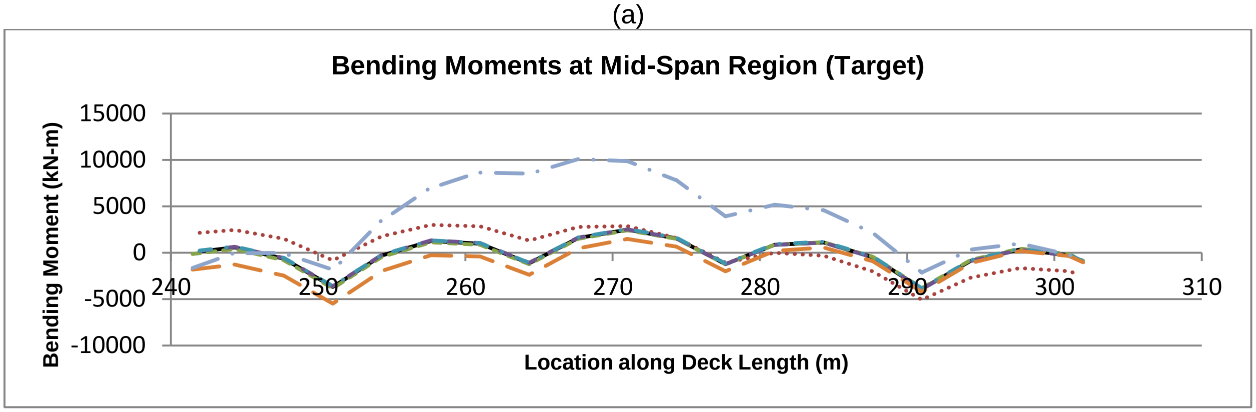
For the “Target” load case, the maximum effects of losing a cable are at locations where the bending moment is zero or close to zero. The maximum effect is caused by the loss of cable 20 at approximately mid-span of the entire deck. This can be explained by the fact that cable 20 is connected at mid-span, where the bending moment is greatest.

Figures 3 (b) to (e) and (g) to (i) are close ups of the bending moment distribution presented in Figures 3 (a) and (f). The effects of cable loss on the bending moment can clearly be viewed, and the difference in magnitude in the bending moments at locations where stay cables were lost is readily assessed.

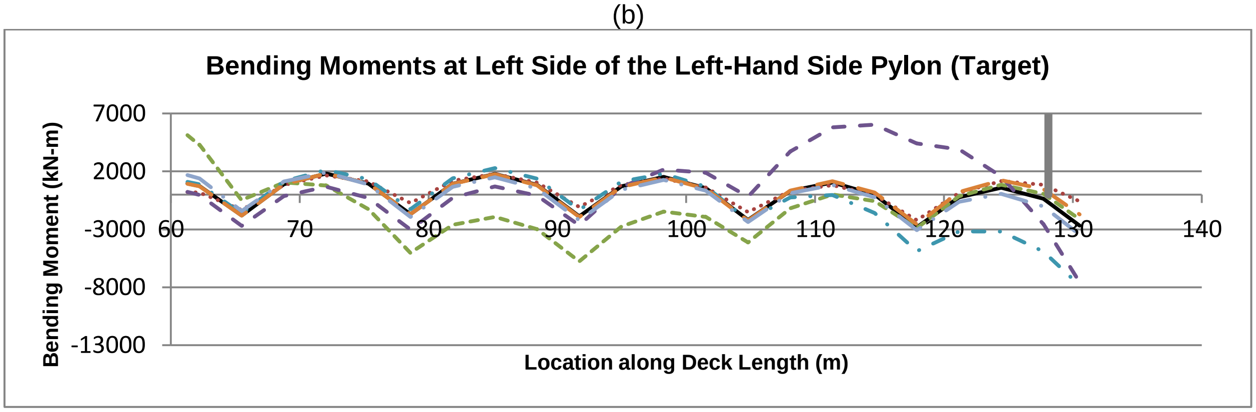
For load case (1), Figure 3 (f) shows that all the cable loss cases, except for the cable 1 case, caused an increase in the positive bending moment. Also, the increase (or peak) is followed by a peak in negative bending moment, and sometimes, preceded by a peak in negative bending moment of lesser intensity.
From the figures below, it is clear that the maximum response occurs at the location of the lost cable. Before and after these peaks in response, the behaviour of the bridge tends to be similar to the behaviour that can be observed for the Cable 0 case.


Bending Moments (Target)
12000
7000
2000
-3000
-8000
-13000
0
100
200
300
400
500
Bending Moment (kN-m)
Length Along Deck Longitudinal Axis (m)

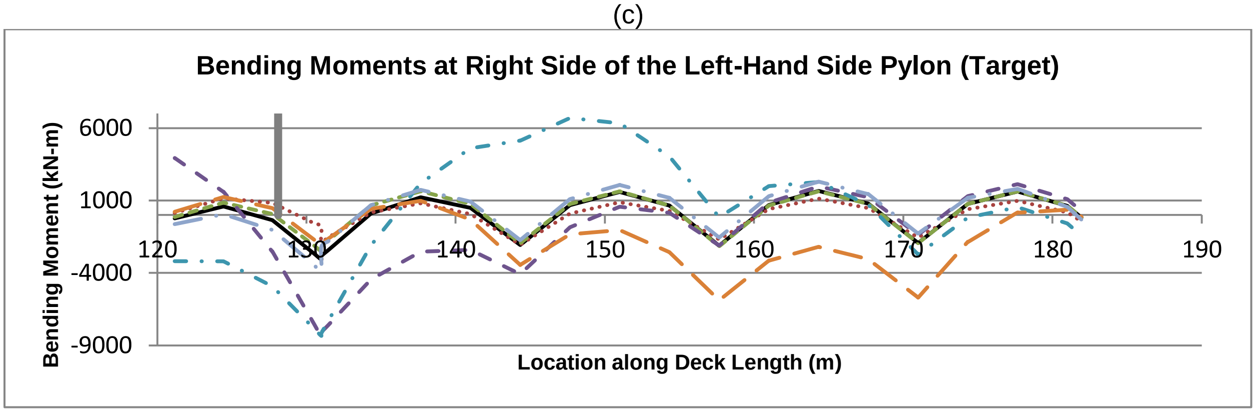
(a)


Bending Moments at Mid-Span Region (Target)
15000
10000
5000
0
-5000
-10000
240
250
260
270
280
290
300
310
Bending Moment (kN-m)
Location along Deck Length (m)

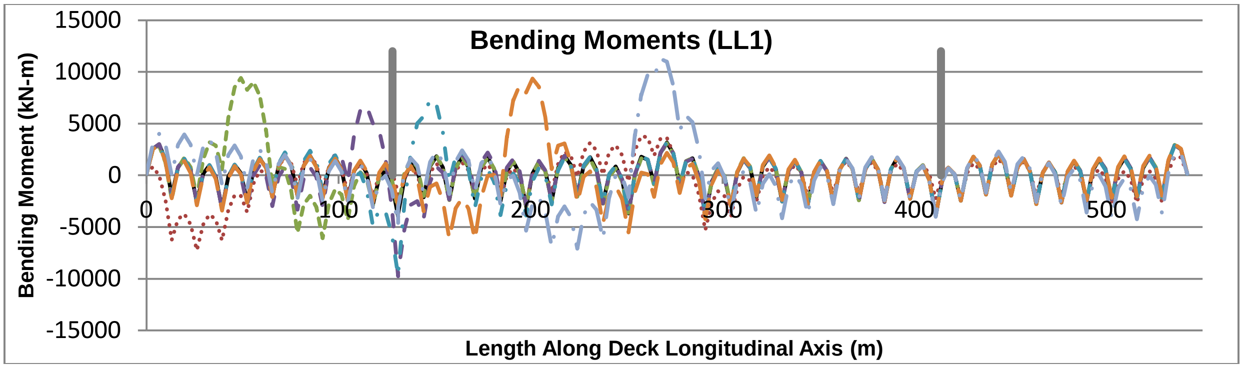
(b)


Bending Moments at Left Side of the Left-Hand Side Pylon (Target)
7000
2000
-3000
-8000
-13000
60
70
80
90
100
110
120
130
140
Bending Moment (kN-m)
Location along Deck Length (m)

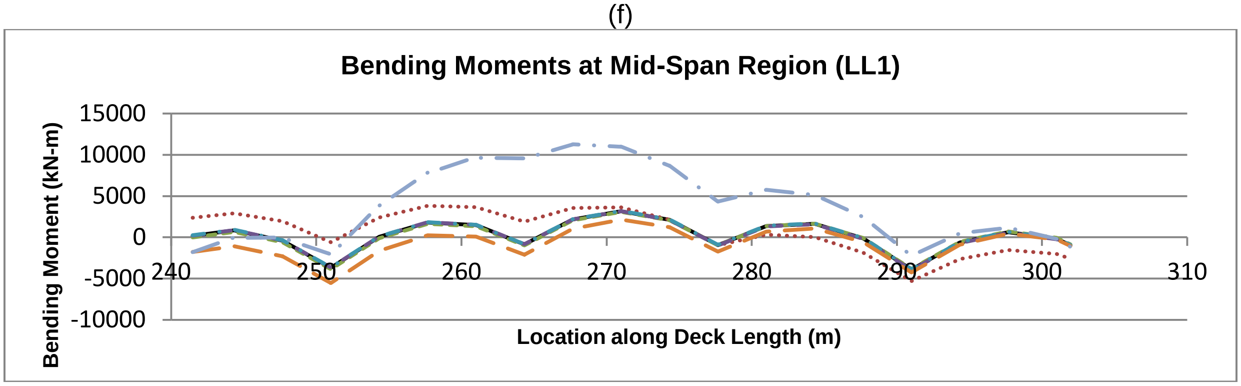
(c)


Bending Moments at Right Side of the Left-Hand Side Pylon (Target)
6000
1000
-4000
-9000
120
130
140
150
160
170
180
190
Bending Moment (kN-m)
Location along Deck Length (m)

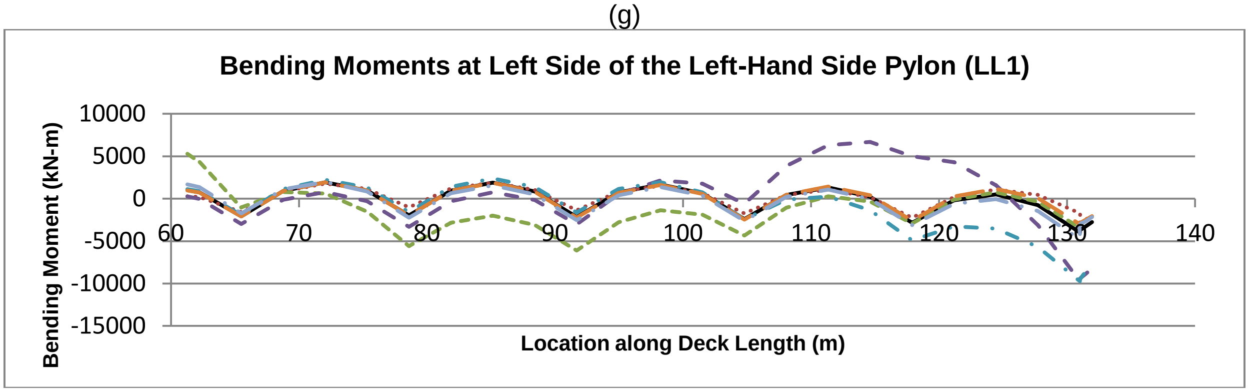
(e)



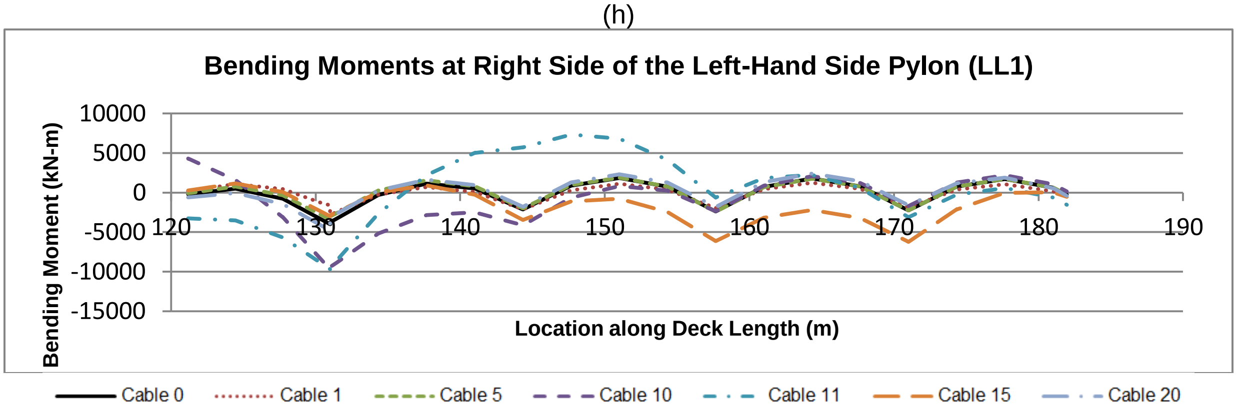
(f)



(g)



(h)



(i)

Figure 3: Bending Moments in the deck for Load Case “Target and Load Case (1).

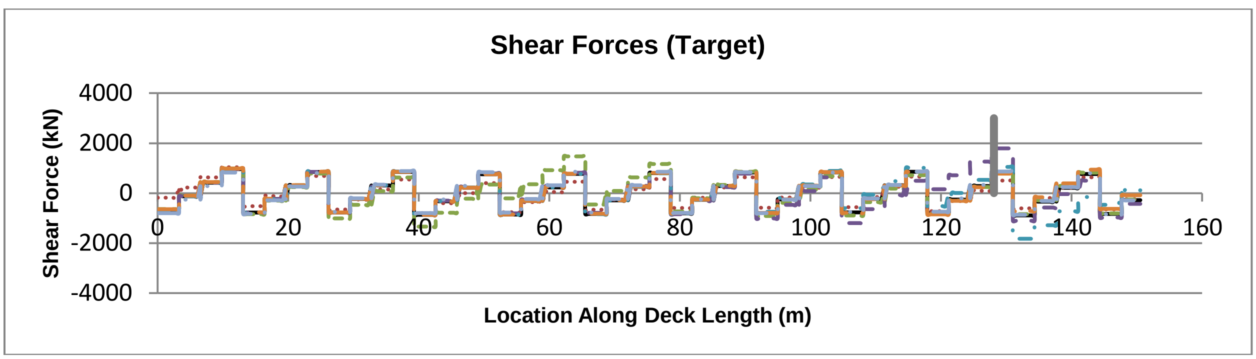
6.2 Shear Forces

Figure 4 (a) and (b) show the variation of vertical shear forces over the longitudinal axis of the bridge deck. It is noted that only the first 150 m of the left side of the bridge is presented here for clarity.

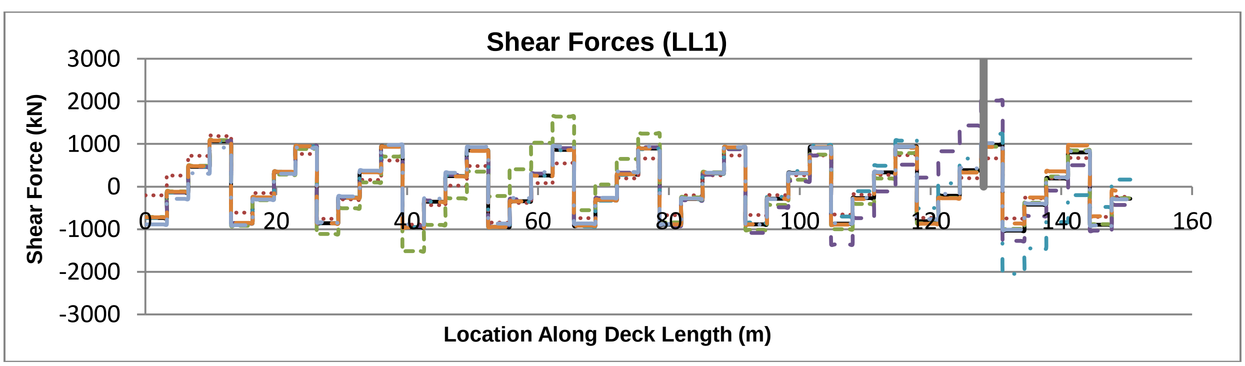
Clearly, the magnitude of the shear force increases in the vicinity of the lost cable over the case of Cable 0 where all the cables are in place. For load case “Target”, the greatest effect on the shear forces comes when cable 10 or 11 are lost, which can be explained by the fact that they are close to the pylon and therefore have an important role in supporting the shear developed at the pylon support.
For load case (LL1), it is seen that the loss of cable 11 produces an increase in shear force of magnitude comparable to the case when cable 10 is loss. Results presented in Figure 4 (b) also show that a peak in positive magnitude is always accompanied by a peak in the negative range. The location of the latter depends on the location of the lost cable on the bridge. As for the bending moments, the shear distribution tends to be similar to the one of the cable 0 case before and after the peaks in response.

6.3 Axial Forces

The axial force variation along the length of the deck is presented in Figure 5 (a) and (b). For load case “Target, it is seen that the loss of cable 1 greatly reduces the axial force along the deck. For the other cable loss cases, the effect is not as significant and tends to be similar to the cable 0 case. The same behaviour can be observed for load case (LL1).



(a)



(b)

Cable 0 Cable 1 Cable 5 Cable 10 Cable 11 Cable 15 Cable 20

Figure 4: Shear Forces in the Deck, (a) Load Case “Target”, (b) Load Case (1)

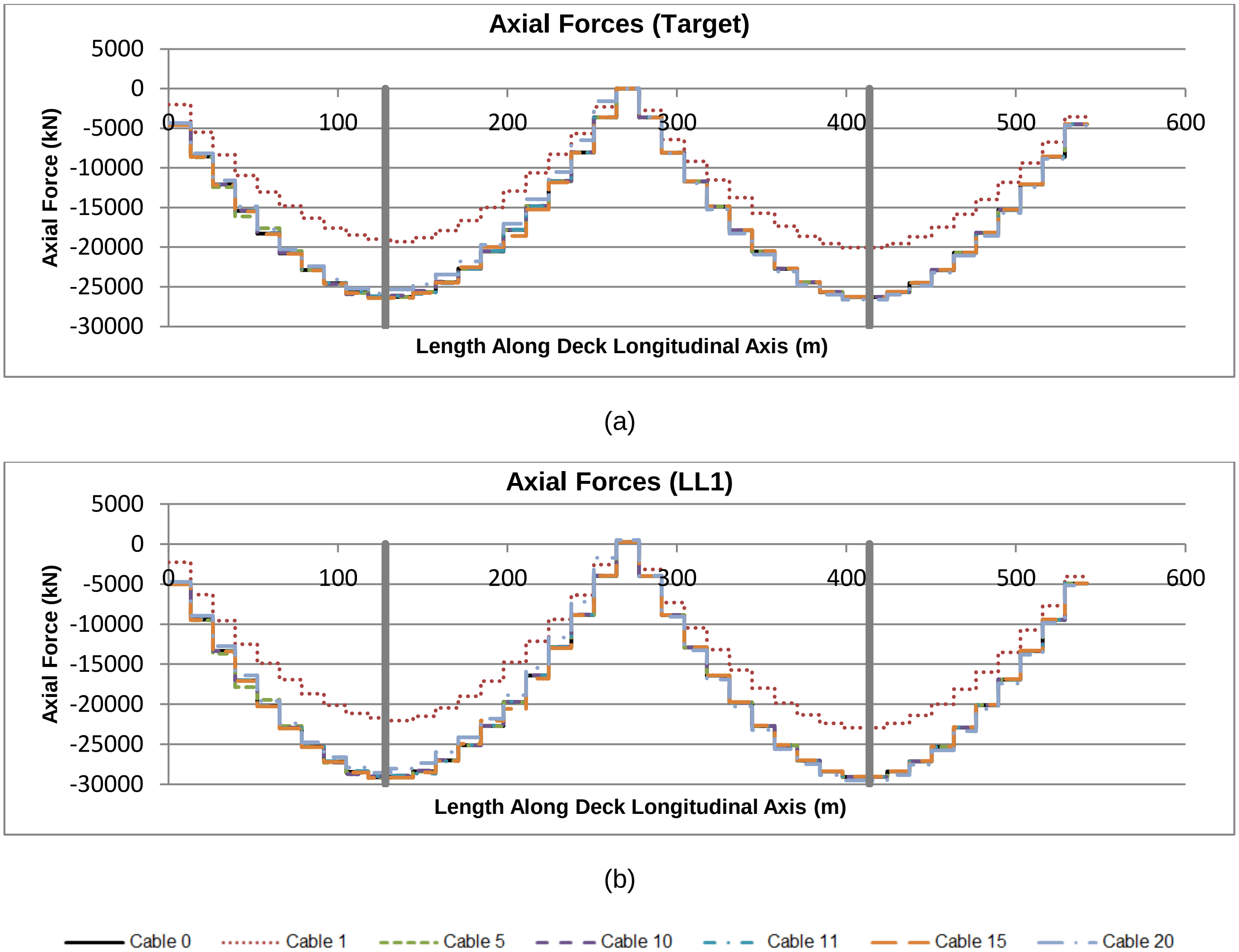
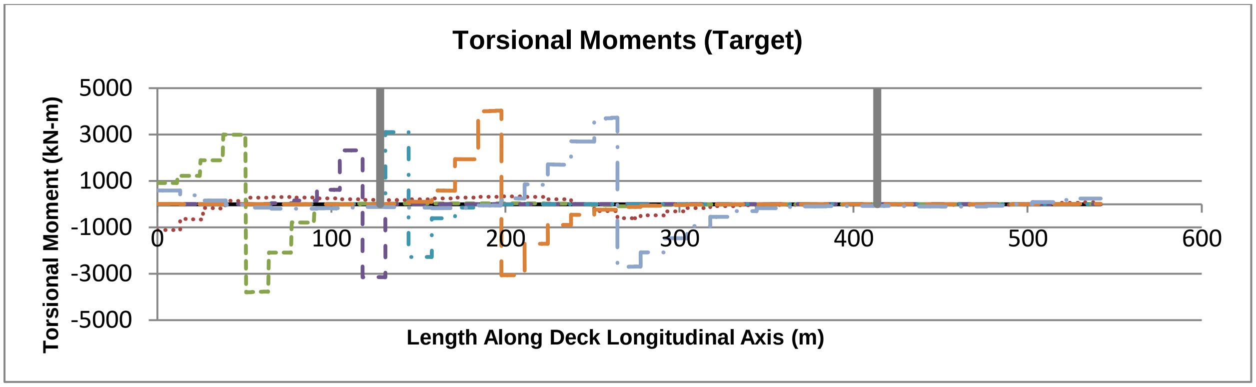


Figure 5: Axial Forces in the Deck, (a) Load Case “Target”, (b) Load Case (1)

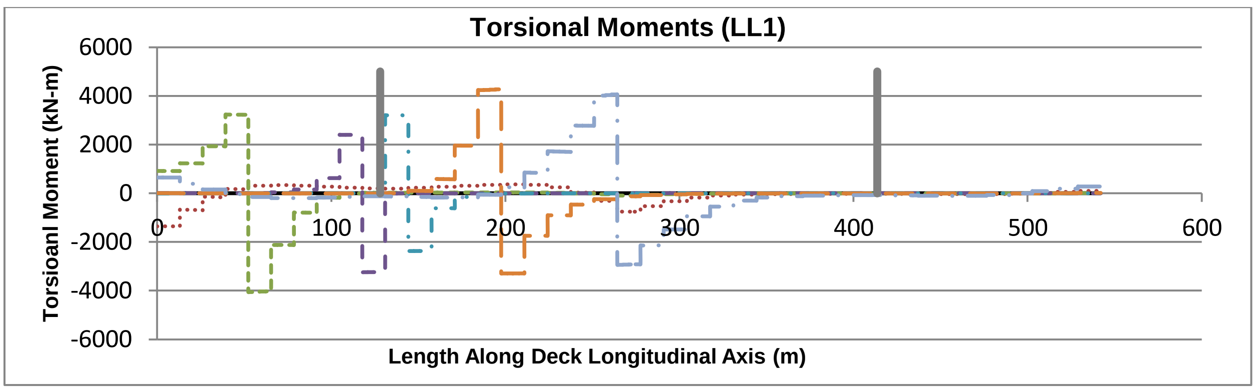
6.4 Torsional Moments

The torsional moments of the deck were studied to see the effect due to the loss of cables, and the results are presented in Figure 6 (a) and (b) below. The effects of cable loss on the torsional moments are negligible along the length of the bridge deck , except in the vicinity of the location of the lost cable, where significant increases in magnitude are observed. Indeed, for the case where all stay cables are present (cable 0), the torsional moments in the deck is zero. The greatest effect is caused when cable 15 (in the mid region between the pylon position and the bridge mid-section) is lost.

From Figures 6 (a) and (b), it is seen that the loss of a stay cable is translated by a peak in positive magnitude. As for other efforts, it is accompanied by a peak in negative magnitude of lesser intensity. Depending on the location of the lost stay cable along the bridge, the peak in negative magnitude comes before or after the positive peak.



(a)



(b)

Figure 6: Torsional Moments in the Deck, (a) Load Case “Target”, (b) Load Case (1)

6.5 Vertical Displacements

Finally, for the vertical deflection of the deck was assessed under the loass of stay cables. As for the other internal actions, amplifications in the displacements are observed in the vicinity of the lost cable, as shown in Figures 7 (a) and (b).

For load case “Target”, these peaks in displacements are usually located where the displacements of the deck for the cable 0 case are zero or close to that value. Except for the cable 1 case, where the peaks in displacements for load cases (1) are in the negative direction. As for other efforts, the peak is accompanied by a smaller peak of opposite magnitude. Except for cable 1 and cable 20 cases, the vertical displacement of the deck tend to be similar to the one for the cable 0 case before and after their peaks.

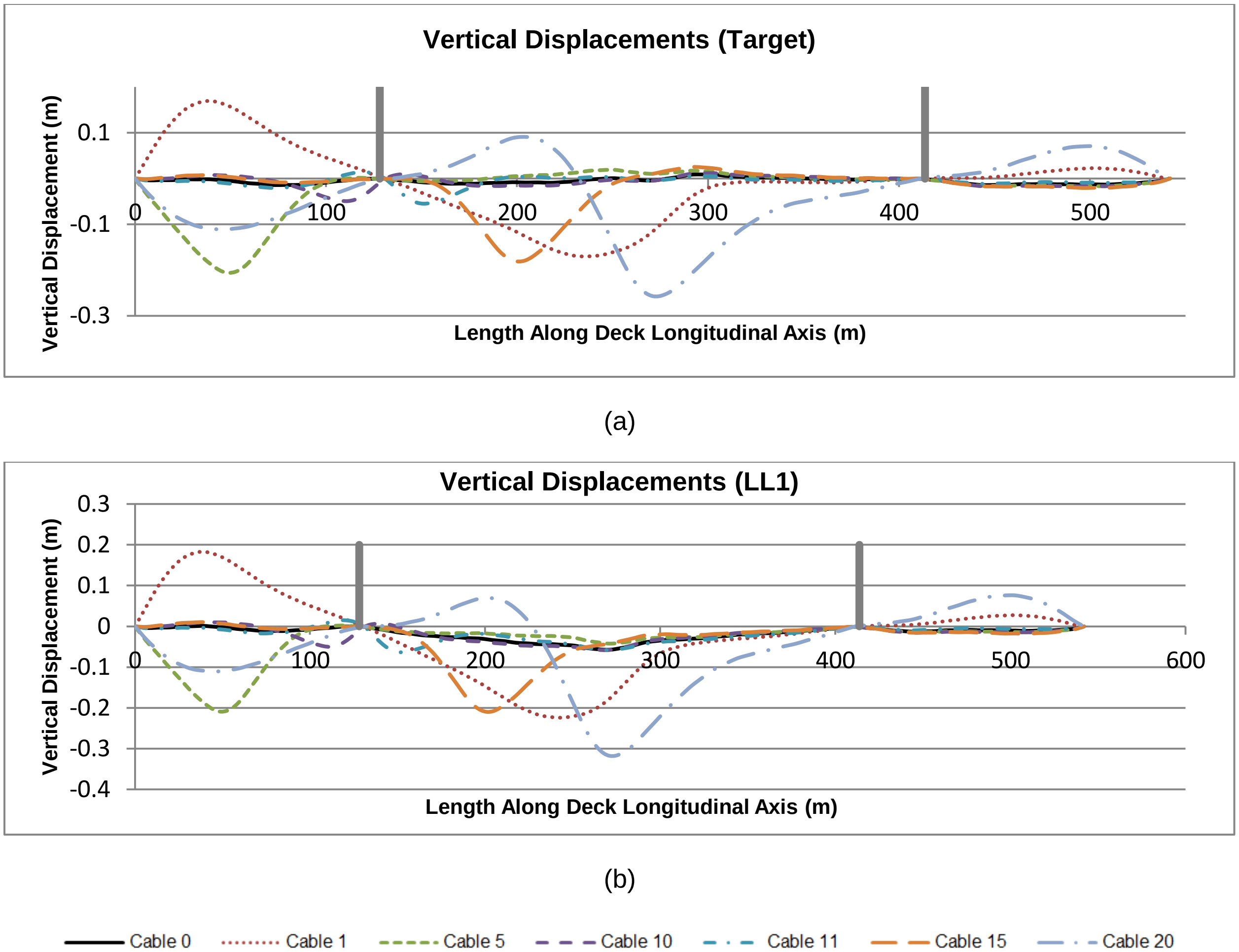


Figure 7: Vertical Displacements in the Deck, (a) Load Case “Target”, (b) Load Case (1)

7. Conclusions

From the observations presented above, it is seen that the loss of stay cables can have significant effect on the deck’s behaviour. Moreover, for load case “Target”, the maximum quantities observed for cable loss usually happens at locations where the quantity for the cable 0 case is zero or close to zero. For load case (LL1), the peaks were also preceded or followed by peaks in the opposite sign but of lesser magnitude. This indicates that the loss of a stay cable contributes to increasing internal actions in the deck at locations where it was not meant and designed for.

Each cable loss case presented showed that its maximal effect for one design quantity (bending moment, shear force, axial force, torsional moment, vertical displacement) is usually in the vicinity of the location of the lost cable. However, at other locations along the deck, the distribution of efforts and the vertical displacement tend to be similar to what is observed for the Cable 0 case.

From the results of this static analysis, it is evident that it is important to account for the loss of stay cables during the design of the bridge deck. The effect on the resistance capacity of the deck component is the next step of study.

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