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THE EFFECTS OF GEOMETRICAL PROPERTIES ON PROGRESSIVE COLLAPSE IN CABLE-STAYED BRIDGES

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ABSTRACT: Cable-stayed bridges play a key role in the sustainable development of regions. In recent years, various kinds of these bridges, in terms of cable arrangement, have been built. Due to harsh conditions of their surroundings, several hazards always threat cable-stayed bridges one of which is the progressive collapse phenomenon which may give rise to disastrous events like disproportionate deformations or entire collapse and huge damages. This paper, consequently, aims to determine the effect of geometrical characteristics of this type of bridge on progressive collapse and introduce the best arrangement to deal with it. For an investigation of this phenomenon, assessment is carried out by non-linear time history analysis using SAP2000v17. The axial force of adjacent cables, therefore, will be evaluated within 0.1 second-step under specific load combination proposed by PTI recommendation in order to compare it with the ultimate limit. It can be concluded that the dimension of the deck and height of pylons do not have any significant impact on the behavior of the structure against progressive collapse. In addition, the evaluation of different arrangements of cables made it clear that Semi Harp and Fan arrangements can resist better against the subsequent failure of other cables compared with the Harp arrangement.

KEYWORDS: Cable-stayed bridge; Progressive collapse; Time history analysis; Plastic hinge; Redistribution of force.

1 INTRODUCTION

Due to numerous advances in construction technologies, including modified equipment and new techniques, man-made structures are now more complex than before and, consequently, many new issues in this study area are appearing. One of these issues, which is more or less likely, is a progressive collapse in structures. This kind of failure, due to the vast demolition, takes many lives and causes severe damages every year. It is a sad fact that even one element is capable of initiating the progressive collapse of the entire structure.

There have been several structures in which consequent collapse occurred just because of failure in one ordinary sub-element. For instance, Quebec Bridge in Canada was completely destroyed in 1907 owning to buckling of one truss element which carried the load of the deck. As a result, many codes have proposed some recommendations to reduce the risk of progressive collapse of structures. Compared to others, some codes such as ASCE-7(2006) and GSA (2003) and have paid more attention to progressive collapse, although their focus is mainly on the building scope [1]. According to ASCE definition, progressive collapse is the "spread of initial local failure from element to element, eventually resulting in the collapse of the entire structure or a disproportionately large part of it." Progressive collapse, in case of local damage, occurs due to lack of required capacity; it is initiated by several events such as corrosion, collision, construction errors and so on.

Cable-stayed bridges are one of the most prevalent types of bridges built throughout the world therefore, it is essential to fortify both new and existing bridges against any threat. Failure of several cables in Zarate-Bravo Largo Bridge in Argentina, failure of cable anchorage in Cycle Art Bridge in Glasgow and rupture of the main cable of Rion-Antirion are some of the important examples of progressive collapse in cable-stayed bridges. Accordingly, failure of cable is a common event in these types of bridges, implying the importance of cable failure analysis. Consequently, many investigations have been carried out on the methods for preventing this kind of failure. Their prescribed methods for reducing the risk of progressive collapse are generally classified into two main groups including (a) direct method consisting of specific local resistance (SLR) and alternative load path method (ALP) and (b) indirect method involving the tie method and compartmentalization. In the SLR method, designers fortify structural elements against local failure so that any local failure is avoided whereas in the ALP method the key element, which has the strongest impact on the structure, is removed and the response of the structure to this removal is analyzed. This study only considers the alternative load path method which is conducted either by static (linear and nonlinear) or dynamic (linear and non-linear) analysis. There are two reasons for doing this. Firstly, the behavior of structure obeys the structural system which is considered only in the ALP method and, secondly, unlike the SLR method, there is still a lack of knowledge about the ALP method in a way that the codes mostly contain merely discussion about SLR method.

2 STUDY BACKGROUND

Detailed studies on progressive collapse initiated after the demolition of a part of Ronan Point building in London in 1968 where a natural gas explosion on 18th floor caused subsequent damage in other stories.

The catastrophic event of September 11, 2001, and the destruction of the World Trade Center attracted universal attention to progressive collapse phenomenon and inspired many scientists and engineers to investigate this hazard. However, the roots for the study of progressive collapse date back to the 1970s, when Ellingwood and Leyendecker (1978), as pioneers, presented several methods in terms of local resistance and ALP method to mitigate the risk of spreading failure [2]. As basic research, in 2007, Starrossek introduced a typology of progressive collapse and classified it into six categories [3]. The type of progressive collapse in cable-stayed bridges in Starrossek's study was the Zipper type.

Although progressive collapse phenomenon in cable-stayed bridges because of a high degree of static indeterminacy is rare, it has been under study in recent decades owing to remarkable damages caused by those few collapses. As a basic study, Starrossek (1999) investigated the risk of progressive collapse in multi-span bridges and suggested some modifications for the placement of reinforcements and the depth of the deck to ensure the safety of structure against progressive failure from span to span [4]. Demolition of I-35WBridge in the US, which was related to progressive failure, intensified investigations in this study area and gave researchers a massage that this threat should be taken into consideration more seriously. After this event, Astaneh (2008) and Hao (2009) added some fundamental instructions to design criteria to reduce this kind of collapse to be included in design codes [5,6]. Regarding the methods for the analysis of subsequent failure, Jiang-gue et al. (2012) compared some various techniques and concluded that non-linear dynamic analysis is by far the most accurate method [7]. Fatollahzadeh, Naghipour & Hamidi (2016) studied progressive collapse phenomena in cable-stayed bridges during earthquakes through the finite element method [8]. Their study revealed that through base isolations the potential of progressive failure could be reduced. Kim, Seungjun & Kang (2016) suggested a rational cable failure analysis to trace the new equilibrium with structural configuration using static behavior of individual cable failure [9]. In practical projects, dynamic amplification factor (D.A.F), which is the ratio of maximum dynamic response to static response, plays a desirable role for structural engineers to skip tedious dynamic analysis in case of cable losses. This factor has been studied by lots of researchers including Ruiz-Teran & Aparicio (2007) who carried out an analysis of (D.A.F) due to accidental breakage of stay cable [10]. In the case of cable failure under blast loads, Aoki, Valipour, Samali & Saleh (2014), investigated cable failure due to different patterns of explosion and they took into account various influential parameters including strain rate, damage area, the volume of explosive material (TNT) [11]. The response of under-deck cable-stayed bridges to loss of stay cables was studied by Ruiz-Teran & Aparicio (2009) which demonstrated that even under 100% of traffic load, the case study bridges can sustain two out of five cable losses [12].

3 MATERIAL AND MODEL

3.1 Bridge model

The hypothetical bridge has 2 side spans with 100m length and a 200m main span. The deck is reinforced concrete box with f 'c = 34.32 MPa, Poison ratio=0.17 and Young,s modulus =30 MPa. The bridge has four 2m 2m rectangular pylons made of the same concrete. The steel used for cable elements in the model has f ' y = 1320 MPa, Poison ratio= 0.3 and Young,s modulus=180 GPa [7]. The cable elements are modeled by truss link which are tension elements with 7 cm diameter, whereas pylons and deck are modeled by frame elements. The damping value is considered 5%. Since differences in geometry properties (e.g. harp or fan) induce difference tension force in cables, therefore; the deck will be changed among various models in order to produce equal force to compare them in the same condition, because the initial force of cables plays an essential role in progressive collapse analysis. Different dimensions and cable arrangements have been presented in table 1.

Table 1. Different dimension and geometry properties

Geometric Parameter	Section	Model No.	Subset	Corresponding Deck	Diameter of Cables	Vertical Distance of Cable Connection
Dimension of the deckand height of the pylons	4.2	1	60 m height pylon	Fig.1.a	7cm	3m
		2	80 m height pylon	Fig.1.b	7cm	5m
		3	100 m height pylon	Fig.1.c	7cm	5m
Cable arrangement	4.3	4	Harp	Fig.1.b	7cm	5m
		5	Semi Harp	Fig.1.d	7cm	2m
		6	Fan	Fig.1.e	7cm	---

3.2 Loading regime

According to the PTI recommendation (2001) and GSA (2003), the following load combination is suggested in the case of evaluation progressive collapse:

$$1.0DL+0.75LL+1.0PS+1.0CL \qquad (1)$$

Where DL and LL are dead and live load in service condition, respectively [13]. PS is the prestress force of the cables and CL represents an equivalent force to simulate the sudden force of the cable or cables. Dead load is calculated automatically by sap2000 for gravity loads based on material density and element volume. Live load is considered 3809 kg/m (640lb/ft per lane) as a uniform load for four lanes and the vehicle type HS20-44 based on AASHTO specifications [14]. The prestress force is applied to cables in almost every cable-stayed bridges in order to limit deflection of the deck under service loads;

prestress value is specified by the desired displacement which is roughly 0.6fy in practical designs.

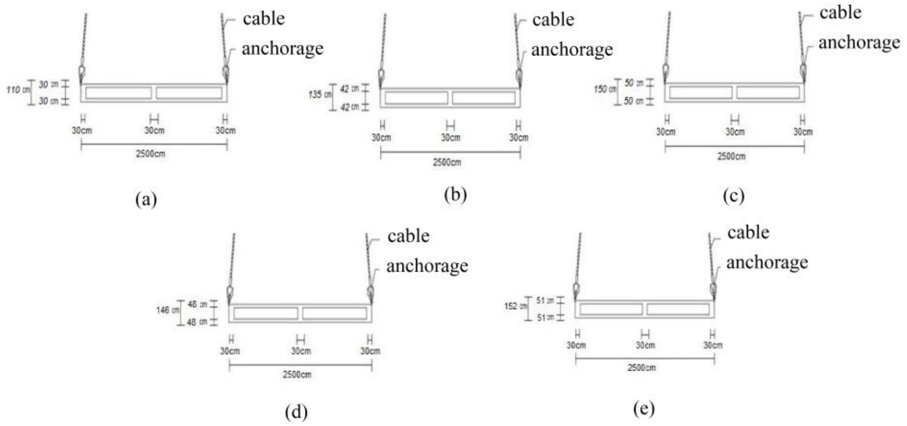


Figure 1. Dimension of the deck in each section

For modeling failure, an equivalent force in the opposite direction of the internal force of ruptured cable/cables will be applied to make the total force equal to zero. This force and its variation with respect to time are shown in Fig.2-a, and Fig.2-b, respectively. Within the first three seconds, the applied force has been considered zero in order to show the initial condition for a better understanding of this phenomenon. During this time all dead and live loads are applied gradually so that any vibration induced by these loads is avoided. Research shows that structural behavior is remarkably affected by the duration of cable removal. If time duration is large, the result will be close to the static behavior. On the other hand, if this time is less than the required value, the most important result will be cost increase and too long analysis time. In this study, this value, as well as time increments, are considered 0.1 seconds to provide the desired accuracy.

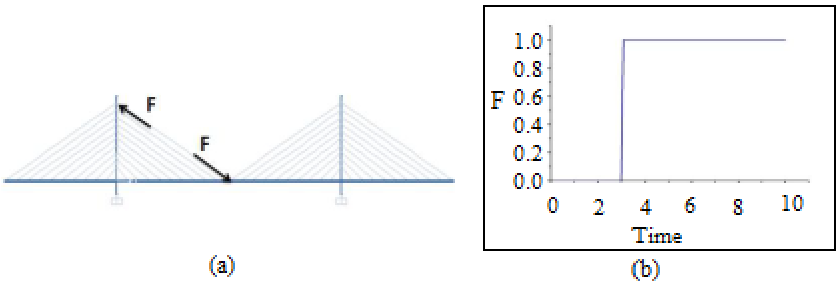


Figure 2. a) Equivalent force for modeling failure; b) Variation of equivalent force applied to ruptured cable with respect to time in sap2000

PTI recommendation suggests two general methods for the analysis of progressive collapse in structures. The first method is static analysis in which the structure is analyzed under dead and live load effects without ruptured cables. This process does not consider the effects of sudden removal therefore it is not accurate. The second method is the dynamic analysis of the structure and in which large inertia forces caused by sudden failure are taken into account, therefore it is much more accurate. PTI also suggests a dynamic amplification factor (DAF) in some cases, where the designers do not prefer to perform dynamic analysis. Although many studies have been carried out in this field, there is still a lack of knowledge and thus a consensus has not been achieved so far. For example, this factor is considered 2.0 in PTI, whereas SETRA (France recommendation) [15] place it between 1.5 to 2.0 [16]. In this study, dynamic analysis and the Newmark method are used to gain the best understanding of the behavior of a structure after the failure of some cables.

3.3 Non-linearity

Due to the large displacement owning to large loads, both material and geometrical nonlinearity is probable. Material non-linearity is based on FEMA 356 [12] where for axial hinge, the force in the ultimate point for the 7cm-diameter cable is 6756.19 KN. The stretch rate of cables at the yield point is 0.73% and according to FEMA356, it is fourteen times as much at failure point [7]. Hinges for both pylons and decks are assumed to be a combination of axial force and bending moments (P-M2-M3), which are automatically calculated by sap2000. The axial hinge is illustrated in Fig.3. In addition, geometrical non-linearity caused by sagging of the cables and P-Delta effects is taken into account in an analytical model.

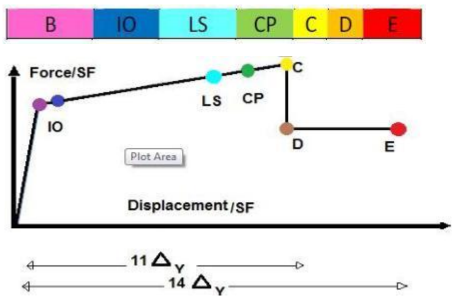


Figure 3. Plastic hinge for axial load

4 ANALYSIS

In this analysis, first, the key element whose removal has the largest impact on the bridge will be recognized first and then the effects of the dimension of the deck, the height of the pylons and finally, cable arrangements will be investigated.

4.1 Key element

According to the ALP method, the key element/elements should be removed and the consequent effects and secondary situation should be investigated. If other elements are able to sustain the secondary increased load, the structure can damp the huge amount of inertia force, an otherwise progressive collapse will occur.

One cable removal

Research indicates that in cases in which the initial forces are almost the same, the main cause of progressive collapse induced by cable failure is the distribution of the previous load of ruptured cable to adjacent cables and their disability to bear this increased load. Thus, to find the critical element, in the case of one cable removal, six scenarios of cable failure are considered and then the largest redistribution of the load to adjacent cables will be investigated. For this purpose, because of symmetry, a quarter of the bridge is selected and, in a separate process, 6 cables are removed. These cables and corresponding states have been illustrated in Fig.4.

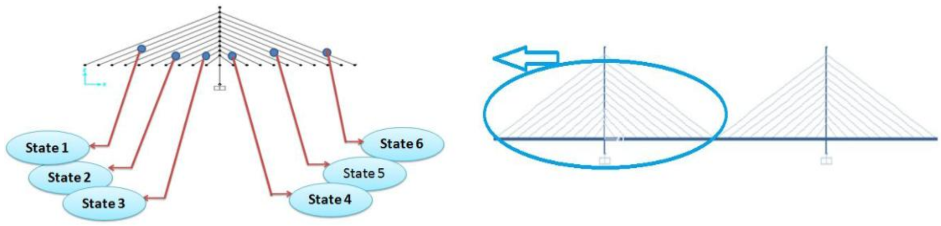


Figure 4. Different scenarios for failure of one cable

As the studies show that the nearest cable to ruptured cable takes the largest part of distributing force compared to other cables, with regard to the approximately same initial forces in all cables, it can be concluded that the risk of subsequent failure in this element is much higher than in others. Forces in closest cable in every six states have been shown in Fig.5.

The results show that in the sixth state, removal of the cable causes the largest redistribution of the tension force in the immediately adjacent cable; the amplitude of vibration, in this case, is also larger than other states. Accordingly, this cable (the mid cable of the middle span) is introduced as the critical cable and in the following sections, the effects of its removal with regard to some geometrical parameters will be studied.

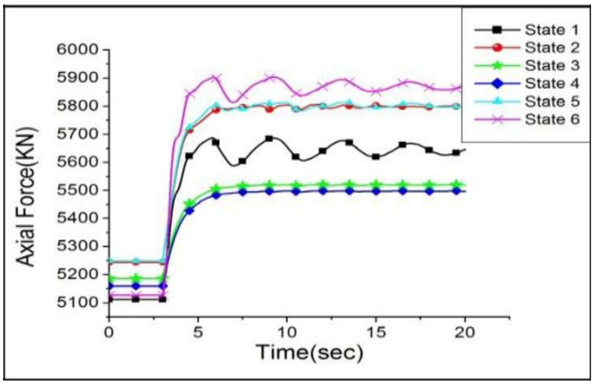
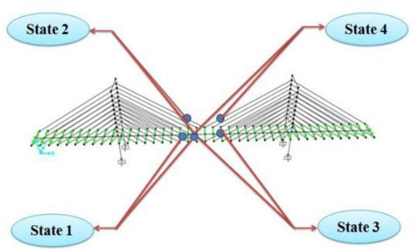


Figure 5. Maximum redistribution of tension force in adjacent cables in the case of one cable removal

There are other conclusions to be drawn from Fig.5 including (1) the contribution of vibration to subsequent failure is trivial as compared with that of redistribution of force and (2) to the extent that the ruptured cable is longer, the dynamic impact is stronger. For example, in the second and third states, since the ruptured cable is short, removal of the cable just causes redistribution and no dynamic effect can be observed.

Two cables removal

As established in the previous section, the mid cable of the middle span is the critical cable and the largest amount of redistributed force transfers to the closest cable. Therefore, in the case of two cables failure, it has proved reasonable to consider only three cables adjacent to the ruptured cable. These four states are illustrated in Fig.6.



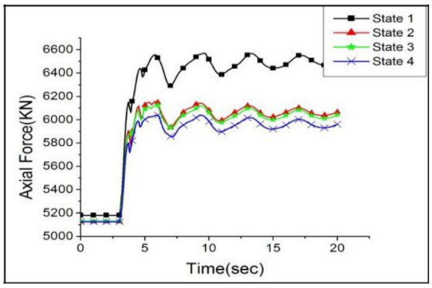


Figure 6. a) Different scenarios for failure of two cables; b) redistribution of tension forces in adjacent cables in the case of two cables removal

It can be observed from Fig.6.b that in state 1 where two adjacent cables are removed from the same stays plane, the structural elements and especially, the nearest cable sustain further secondary forces. Therefore, the risk of progressive collapse within the structure because of getting closer to the final force limit is much higher. This secondary force is 1386.6, 1029.9, 995.4 and 914.3KN in the states 1 to 4, respectively. This calculation can be performed for any number of cables and it can be proved that for three and more cables removal, the same results can be obtained.

4.2 The effect of dimension for structural elements

4.2.1 Dimension of the deck

Among different models described in Table 1, dimensions of the deck and height of the pylons are changed simultaneously. Therefore, if the geometry properties have an impact on the response of structure against progressive collapse, it can be observed in this section. The number of adjacent cables is illustrated in Fig.7. Likewise, Fig.8 and Fig.9 indicate tension force after the failure of two and three cables within these adjacent cables, respectively. One cable removal situation is eliminated because its impact is negligible.

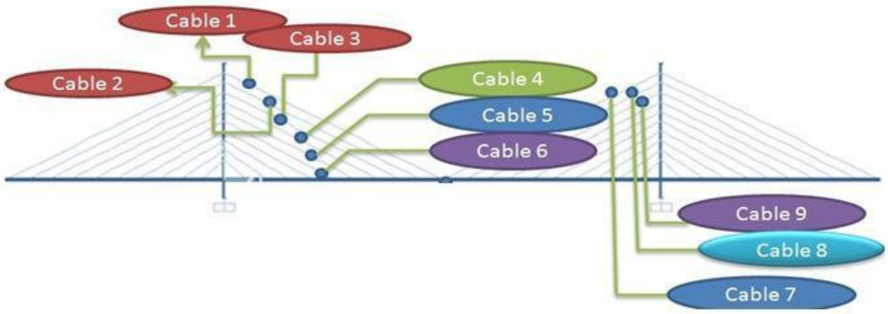
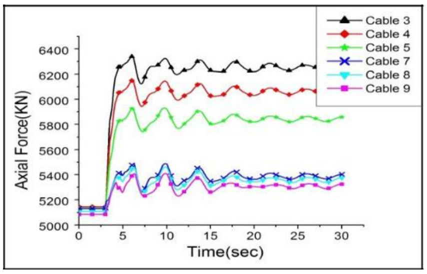


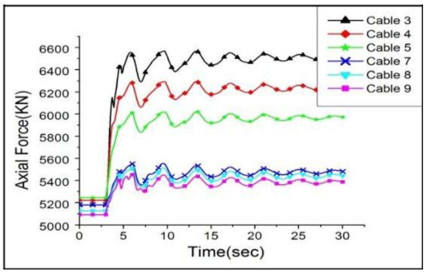
Figure 7. Number of cables adjacent to ruptured cable

Two cables removal

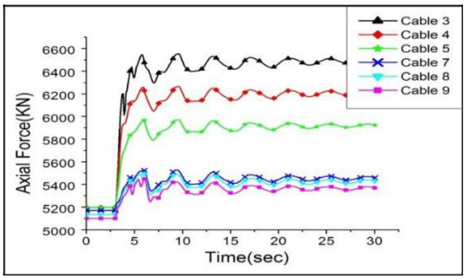
Based on the three different models described in Table 1, the critical cable/cables described in section 4.1 will be removed and the secondary state of equilibrium will be analyzed. Fig.8 indicates that removal of two cables gives rise to a large amount of vibration in and force distribution in cables nearby, but through comparison, it can be concluded that the effect of vibration on subsequent failure is negligible compared to force redistribution. Therefore, as mentioned before, by considering equal initial force, the value of redistribution force in the nearest cable represents the risk of subsequent failure. According to Fig.8, this value is 1210.4, 1386.6 and 1380.1 KN in models 1,2 and 3, respectively. In addition, the maximum vertical secondary displacement, which occurs in the middle of the mid-span and represents the dynamic force exerted on the structure due to sudden failure, is 44.7, 38.2 and 33.1 cm in models 1,2 and 3 respectively. Another result is that all three models can preclude progressive collapse and the large amounts of force exerted on the structure will be damped after a while.



(Model 1)



(Model 2)



(Model 3)

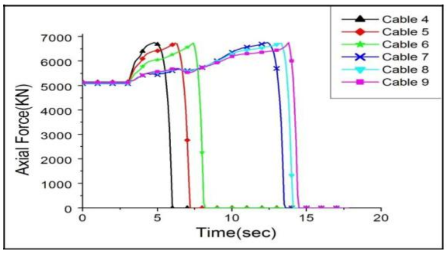
Figure 8. Cable tension forces after two cables removal

Three cables removal

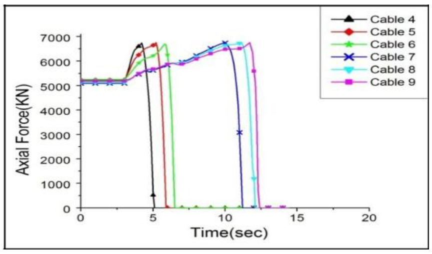
In this case, in all three models, the three most critical cables are removed and the force redistributed to nearby cables will be investigated. These values are illustrated in Fig.8.

In all three models, the axial force of the nearest cable reaches the failure value and then, subsequent failure will occur within the structure.

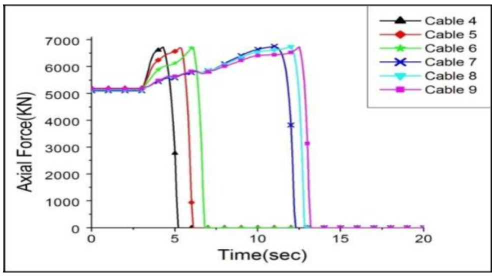
Based on the near values of redistributed force and further displacement in all three models (with different deck and pylon dimensions) in the case of two cables removal and also similar behavior of the bridge after three cables removal which lead to progressive collapse of entire structure, it can be concluded that dimension of the deck and height of the pylons do not have any significant impact on the behavior of structures against progressive collapse. In the following section, this result is used for making a comparison among different cable arrangements.



(Model 1)



(Model 2)



(Model 3)

Figure 9. Cable tension forces after three cables removal

4.3 The effect of cable arrangement on progressive collapse

The aim of this section is to investigate the effect of cable arrangement on the performance of the structure against progressive collapse. For this purpose, three different cable arrangements corresponding to the deck dimension described in Table-1 are presented. These arrangements are illustrated in Fig.10.

(Semi Harp)

(Harp)

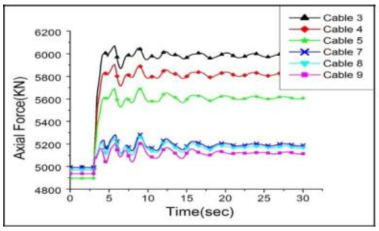
(Fan)

Figure 10. Different cable arrangement of the model

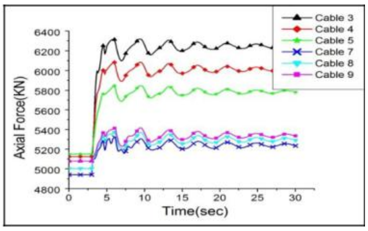
As described in the previous section, critical cables in each model are removed and the second axial force of nearest cables is compared. As in section 4.2, only the results of two and three cables removal are presented in Fig.11 and Fig.12.

Two cables removal

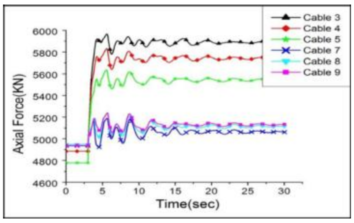
The critical cables obtained from section 4.1, which are the two adjacent cables in the middle of the mid-span, are removed and the secondary state of equilibrium is analyzed. Fig.11 reveals that although all three various cable arrangements can sustain two cables removal, the maximum force distribution which transferred to the nearest cable to ruptured cables has considerable differences. The related values are 1242.3, 1077.1 and 1027.1 KN in Harp, Semi Harp, and Fan arrangement, respectively. Maximum vertical secondary displacement in Harp, Semi Harp and Fan arrangement are 41.6, 29.6 and 25.9 cm respectively. With regard to these values, it can be concluded that in two cables removal state, the maximum redistribution of force and displacement occurs in Harp arrangement.



(Semi Harp)



(Harp)

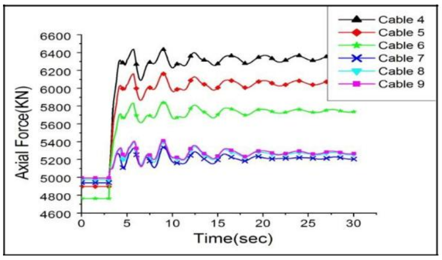


(Fan)

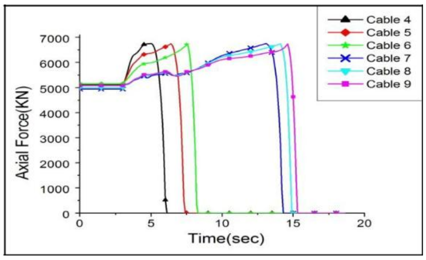
Figure 11. Cable tension forces after two cables removal

Three cables removal

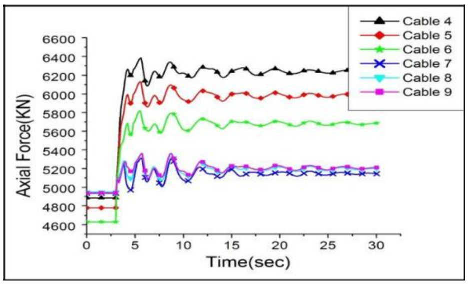
In three cables removal, differences are clear, where despite Semi Harp and Fan arrangement, Harp arrangement is extremely vulnerable to progressive collapse and subsequent failure is inevitable in other cables. Figure 12 shows the cable tension forces after three cables removal. By considering this process, it is predictable that in Semi Harp and Fan arrangement four cables removal cause subsequent failures in other elements. There is just one cable difference in the number of cables whose removal leads to a progressive collapse in the mentioned arrangements, but in case of collision (possible fully loaded truck with normal to high speed as the critical scenario) considering 10m horizontal distance between cable anchorage, failure of two or three cables instead of one cable is more likely. In addition, four cables removal state in cable-stayed bridges is highly unlikely and hardly reported.



(Semi Harp)



(Harp)



(Fan)

Figure 12. Cable tension forces after three cables removal

5 CONCLUSIONS

- The effects of geometry dimensions of the deck and pylons on a progressive collapse in the cable-stayed bridge are trivial. In all three models with different geometrical properties, subsequent collapse occurs by the failure of three cables. Moreover, the time of propagation of failure in all models is nearly the same
- When cable connections move to the upper part of pylons, the entire structure performs better against progressive collapse. Accordingly, Fan and, after that, the Semi Harp arrangement show much better responses against subsequent failure compared to the Harp arrangement. This difference in one and two cable removal states is not considerable, however, it can be clearly seen in three cables removal. In this situation, despite Harp arrangement, Fan and Semi Harp can sustain three cables failure and subsequent ruptures in other cables are prevented.
- The best performance of a cable-stayed bridge to deal with progressive collapse can be achieved by selecting upper-connected cable arrangements. Therefore, Fan and Semi Harp not only are economically better because of lower axial force in equal conditions, but they can also tolerate one further cable removal. In addition, as the reason for the next failure is redistribution of force to adjacent cables, the dimension of the deck and pylons does not have any impact on failure. As a result, design engineers had better avoid, if

possible, parallel cable arrangements and turn to Semi Harp or Fan arrangements.

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