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TECHNICAL ARTICLE—PEER-REVIEWED

Progressive Collapse Analysis of Cable-Stayed Bridges

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Abstract Several codes have proposed guidelines to prevent progressive collapse. Although most of these standards are in progress, few recommendations for progressive collapse analysis and design of cable bridges or even bridges can be found. In this paper, progressive collapse analysis of a cable-stayed bridge is investigated. In this regard, the effects of changes in F_y, E and cross-section area of cables to progressive collapse resistance are studied. The evaluation is performed by alternate load path method and the nonlinear time history tool in SAP2000V17 software. The results of the analysis show that as the cross section and the modulus of elasticity of the cables increase, displacement of bridge decreases and the bridge's resistance increases against failures. Also, for the case where F_y of cables were increased, displacement of the bridge did not differ, and only the formation of the plastic hinges in the cables changed.

Keywords Cable-stayed bridge · Progressive collapse · Capacity curve

Introduction

To prevent progressive collapse, abnormal loads should be considered in the design of structures. Progressive collapse is a structural failure that is initiated by localized structural damage and subsequently develops, as a chain reaction, into a failure that involves a major portion of the structural system. As the action is initiated by a release of internal energy due to sudden member failure, progressive collapse is a dynamic phenomenon. This member loss disturbs the initial load equilibrium of external loads and internal forces, and the structure then vibrates until either a new equilibrium position is found or the structure collapses [1, 2]. Following the approaches proposed by Ellingwood and Leyendecker [3], the design guidelines [4–6] define the following methods for structural design of buildings to mitigate damage due to progressive collapse: the tie force method (indirect design), the specific local resistance method (direct design) and the alternative load path method (direct design). Although in the tie force method the ties between the structural members ensure structural integrity in a quantitative manner, Abruzzo et al. [7], Yi et al. [8] and Naji [9] demonstrated that the current TF method is inadequate for progressive collapse design.

Different simplified procedures for simulating the effects of progressive collapse can now be found [10–19]. However, the work in this area introduces primarily to buildings. Recent experiences, like the collapse of the I-35 W deck truss bridge [20, 21] in Minneapolis, Minnesota, acquire keen priority of the need to integrate progressive collapse into the design of bridge structures. Jenkins [22] identified that terrorist attacks on public transportation have swelled over the past quarter of a century. Bridges are stunning terrorist targets due to their facile availability as well as the destructive outcome on the society after demolition [23]. Compared to buildings, bridges are principally horizontally lined up structures with one major axis of continuation. Therefore, the viable collapse mechanisms are nonidentical. Designing against progressive collapse has not been a main issue in the progress of bridge regulations. However, cable-stayed

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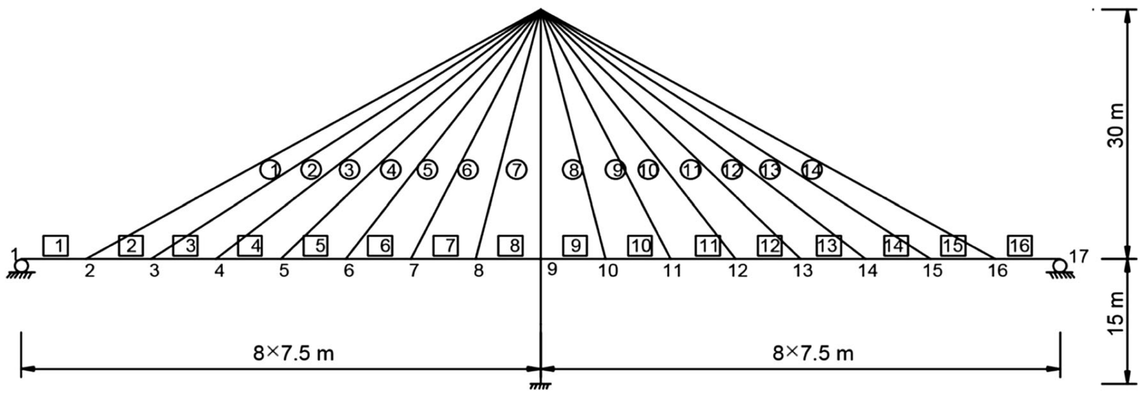

Fig. 1 A single-pylon cable-stayed bridge [39]

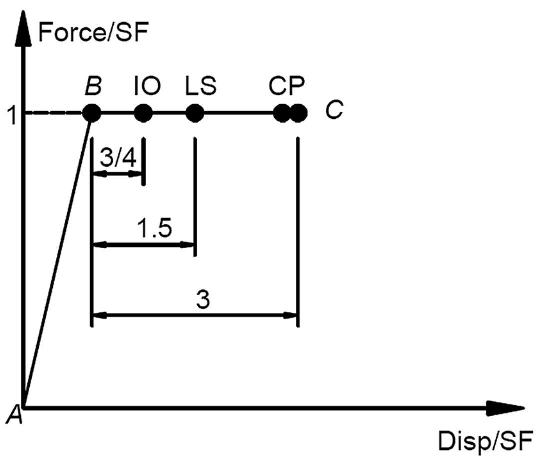

Fig. 2 Axial plastic hinge model for cables

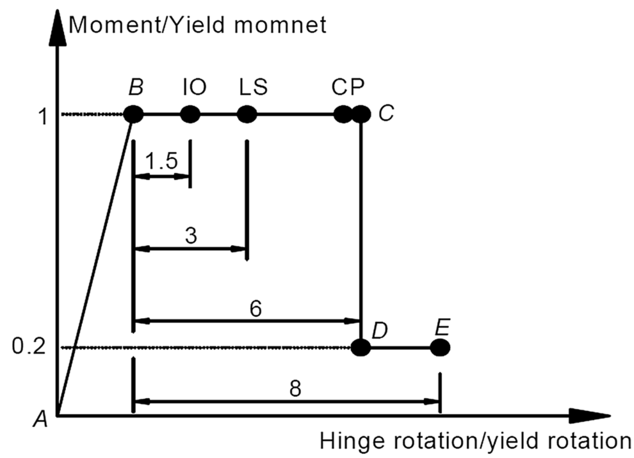

Fig. 3 PMM plastic hinge model for girder and pylon

bridge is the sole bridge system regularly designed for cable loss. The cable rupture event should be determined as a potential local failure because of small cable cross sections, which leads to low resistances against aleatory lateral loads originated from vehicle impact or harmful scenario [24]. The cable loss event can bring about overloading and rupture of neighboring cables. Moreover, the stiffening girder is in compression and a cable rupture decreases its buckling bracing [25].

Previous Works

Zoli and Steinhouse [26] provide the instances of some cable loss situations as well as a thorough assessment of progressive collapse resistance of bridges under multi-cable rupture. Wolff and Starossek [24] investigated a cable-stayed bridge response to the one cable rupture utilizing dynamic analyses involving large displacements and evaluating the outcomes of cable sag as well as transverse cable vibrations and structural damping. Yan and Chang [23] provided a probabilistic evaluation structure to quantitatively analyze the vulnerability of cable-stayed bridges. Moreover, a plastic limit analysis technique for the vulnerability assessment of single-pylon cable-stayed bridges is also proposed [27]. Das et al. [28] revealed modeling and analysis of a prototypical cable-stayed bridge utilizing a nonlinear dynamic method. Moreover, the response of the structural model is explored for various kinds of critical cable loss scenarios. Hashemi et al. [29] provided a comprehensive finite element analysis of a steel cable-stayed bridge by means of an explicit solver. Three different explosive magnitudes, i.e., small, medium and large, are considered at different positions above the deck level to examine the effect of the dimensions and place of the blast loads on the global and local response of the bridge components. Particularly, the outcomes of the computer simulations are utilized to identify the category and range of damage on the pylon and deck, and further to explore the potential cable loss cases related to an anchorage loss. In addition, the results of the finite element modeling are utilized to estimate progressive collapse response of a cable-stayed bridge under different detonation cases. Son and Lee [30] investigated the response of

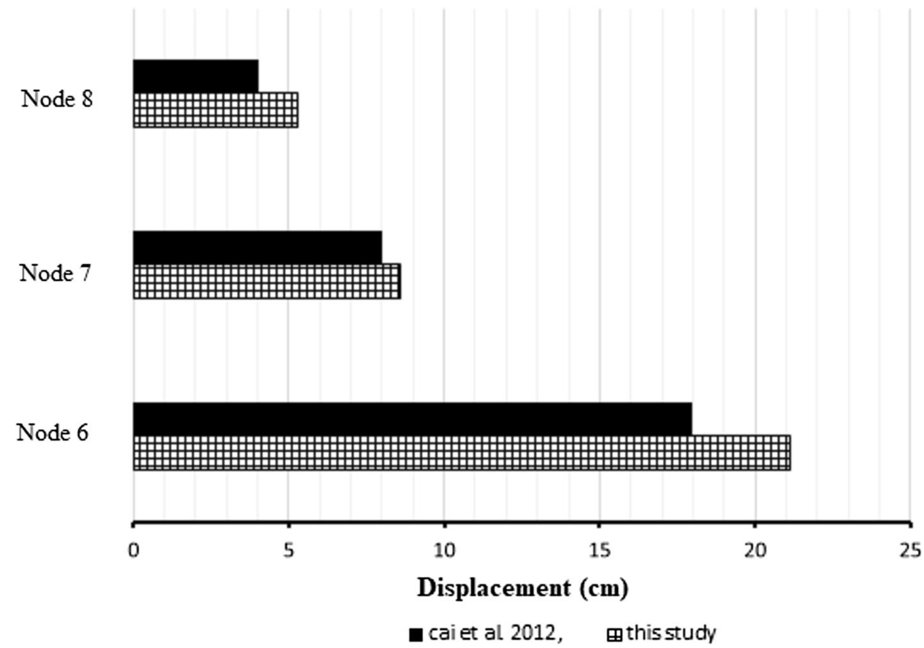

Fig. 4 Comparison of displacement of nodes 6, 7 and 8 after removing the cable 4

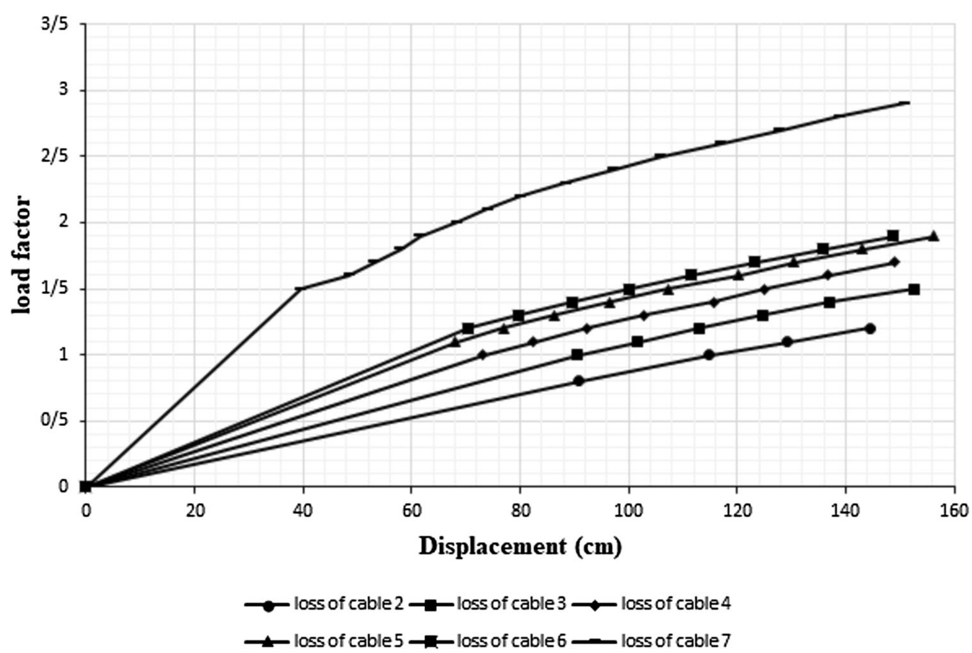

Fig. 5 Capacity curves for different cable removal scenarios

hollow steel box and concrete-filled composite pylons of a cable-stayed bridge treat with blast loads. A coupled numerical procedure with combined Lagrangian and Eulerian simulations was employed to determine the interaction of the deck and pylon with the air that transfers the explosion reaction to the bridge. Shoghijavan and Starossek [31] studied the structural performance of a long-span cable-stayed bridge after the accidental loss of one of its cables. The outcomes yielded that increasing the ratio of the bending stiffness of the girder to the axial stiffness of the cables leads to a larger bending moment on the girder. Samali et al. [32] analyzed detailed 3D finite element models of a typical cable-stayed bridge including material and geometrical nonlinearity. A parametric study is conducted and influence of symmetric and asymmetric cable loss actions, deck specifications and quantity of lost cables on the progressive collapse response of the bridge is explored.

Regarding the design of cable-stayed bridges, the current guidelines reveal that the accidental rupture of a single cable should not cause global structural failure. Hence, PTI [33] and FIB [34] identify a loss-of-cable load case. The

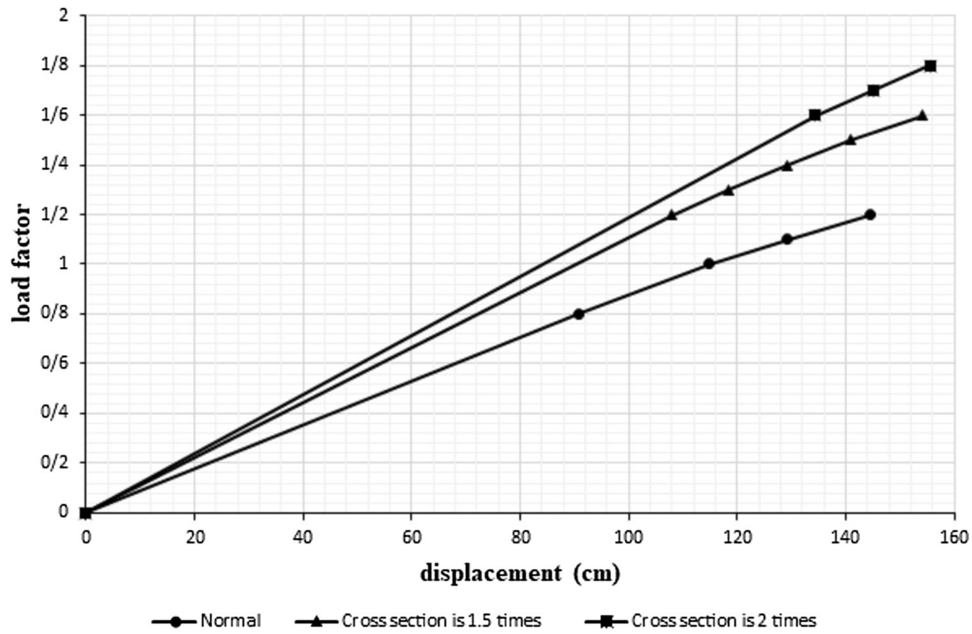

Fig. 6 Capacity curves after removal of cable 2 for three cable cross sections

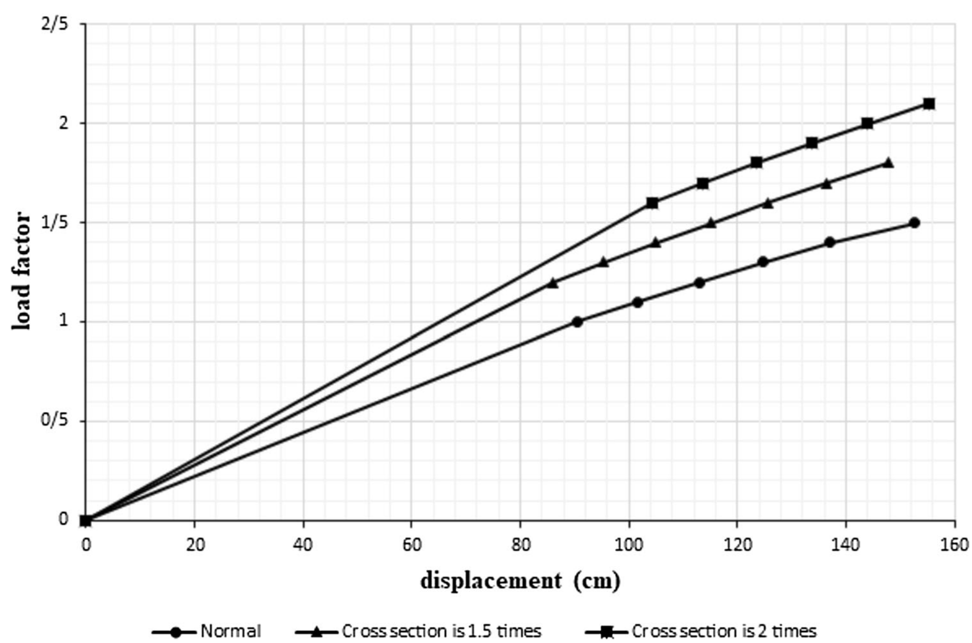

Fig. 7 Capacity curves after removal of cable 3 for three cable cross sections

PTI [33] dispenses instruction in the ultimate event of cable rupture, concerning load applications as well as resistance characteristics. Static and dynamic load application procedures are recommended. The simplified static technique is to carry out the structure with a removed cable under amplified dead and live loads joined with the static application of the dynamic force yielded from the critical cable. Alternatively, the PTI [33] allows utilizing dynamic analysis to compute sudden cable failure response. Nonetheless, little instruction is imparted on running such a dynamic analysis and design of the global structural model.

Research Significance

The current paper focuses on the event independent procedure for the progressive collapse analysis of cable-stayed bridges. This method will not take into account the collapse provoking scenario. Dynamic nonlinear analysis is conducted to describe the details of progressive collapse phenomena of cable-stayed bridges. In this regard, the effects of changes in F_y, E and cross-section area of cables to progressive collapse resistance is studied. The evaluation

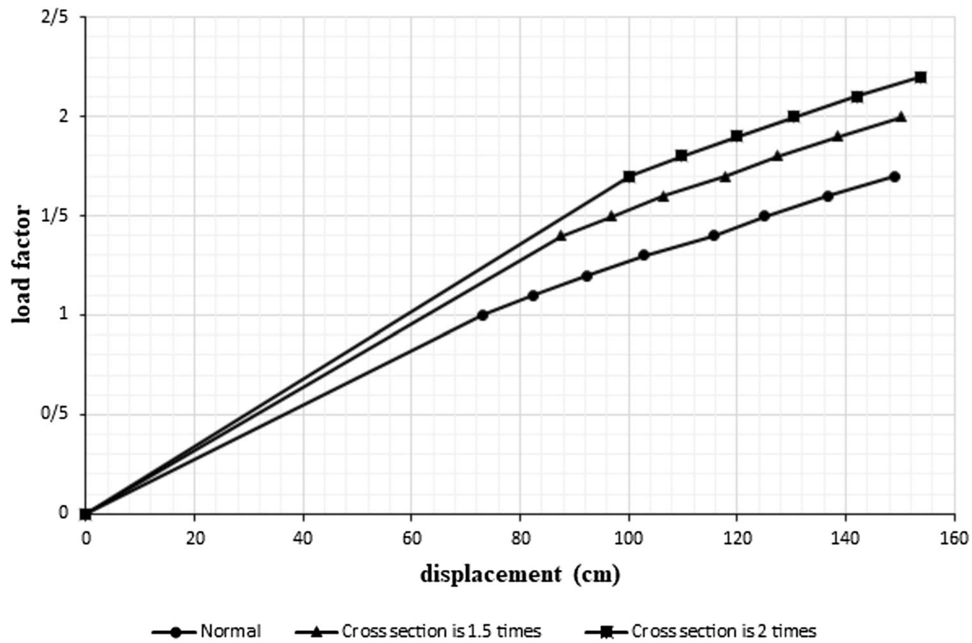

Fig. 8 Capacity curves after removal of cable 4 for three cable cross sections

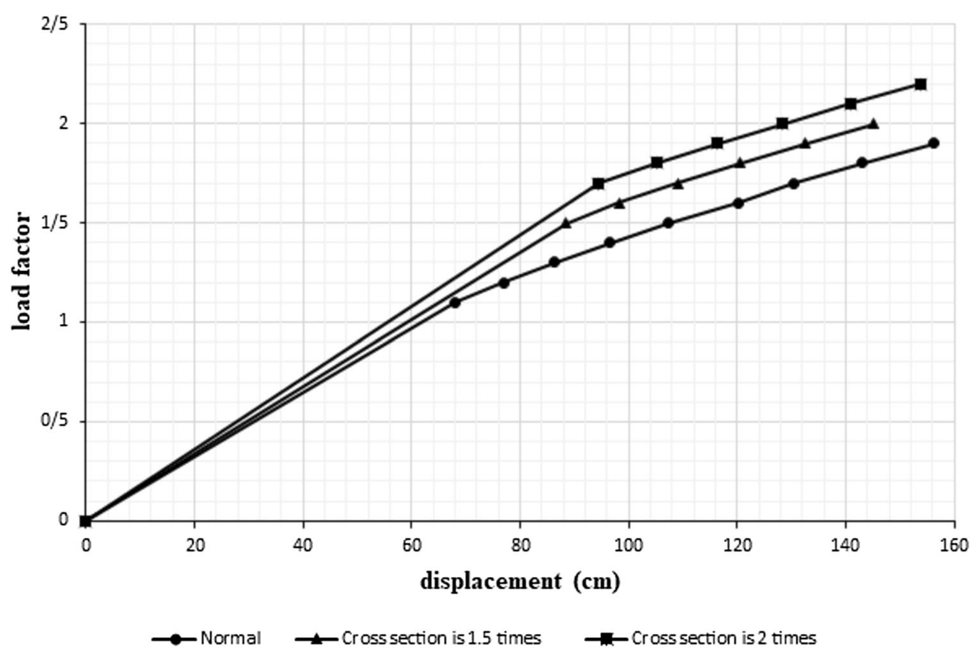

Fig. 9 Capacity curves after removal of cable 5 for three cable cross sections

is performed by alternate load path method and the non-linear time history tool in SAP2000V17 software.

Cable-Stayed Bridge Model

The diagrammatic illustration of the cable-supported bridge considered in this study is demonstrated in Fig. 1. The bridge contains a single tower of 45 m high and two identical side spans of 60 m. The girder is simulated to be hinged with the tower at a height of 15 m and roller supported at both ends. It is also stayed on 14 cables, 7 on each side. The material and geometric specifications of the girder are concluded as Young's modulus = 210 GPa, shear modulus = 84 GPa, Poisson's ratio = 0.3, cross-sectional area = 0.184 m^2, plastic modulus = 0.0414 m^3 and yield strength = 215 MPa. The supported cables are composed of steel strands with the bellow specifications: Young's modulus = 180 GPa, Poisson's ratio = 0.3, cross-sectional area = 25 cm^2 and yield strength = 1.32 GPa. The pylon is constructed of reinforced concrete. The material and geometrical specifications of the pylon are

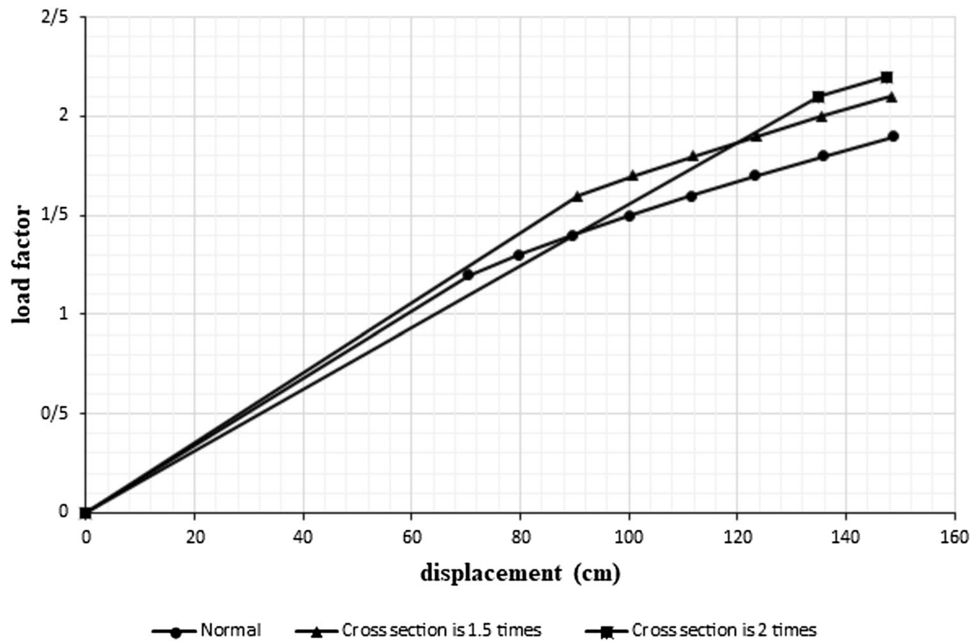

Fig. 10 Capacity curves after removal of cable 6 for three cable cross sections

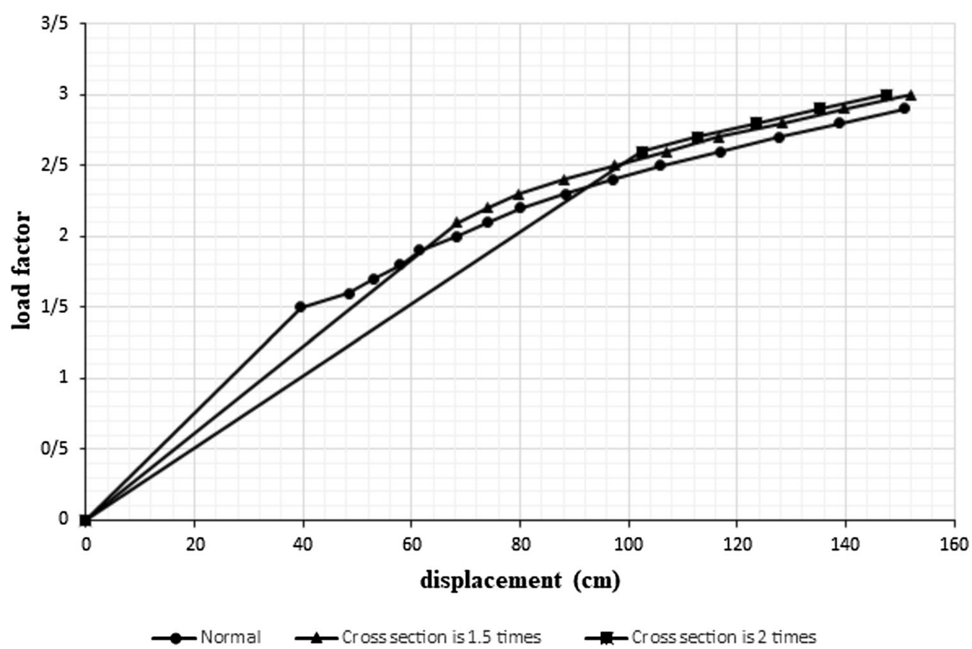

Fig. 11 Capacity curves after removal of cable 7 for three cable cross sections

considered as Young's modulus = 30 GPa, shear modulus = 12.8 GPa, Poisson's ratio = 0.17, cross-sectional areas = 7 m^2, plastic modulus = 8.75 m^3 and yield strength = 35 MPa.

The numerical analysis is performed by SAP2000 utilizing a 2D bridge representation. The pylon and girders are simulated with frame elements. The dead and live loads supposed here are 80 and 40 kN/m, respectively.

The supported cables have been prestressed to prevent deflection at the anchorage points under dead and live loads. The plastic hinge is modeled in SAP2000 to consider the material nonlinearity. The material nonlinearity is taken into account for the cables, pylon and girders. For cables and pylon, an axial plastic hinge has been modeled in the center of the element. For girders the plastic properties of the materials have been assigned by inserting a plastic hinge at 0.05 and 0.95 lengths. Axial plastic hinge model for cables is illustrated in Fig. 2 and plastic hinge model for girders and pylon is demonstrated in Fig. 3.

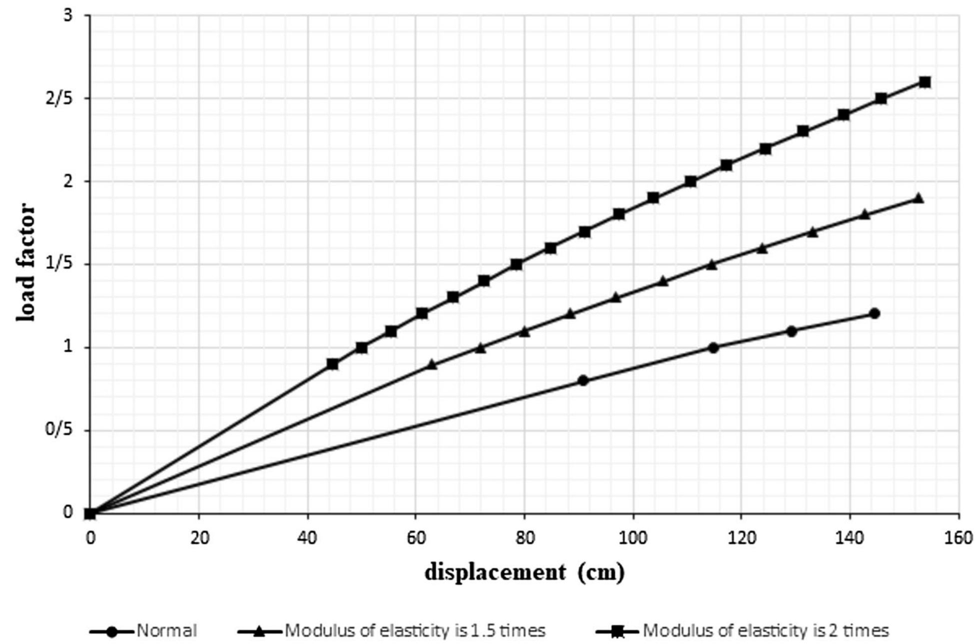

Fig. 12 Capacity curves after removal of cable 2 for three values of modulus of elasticity

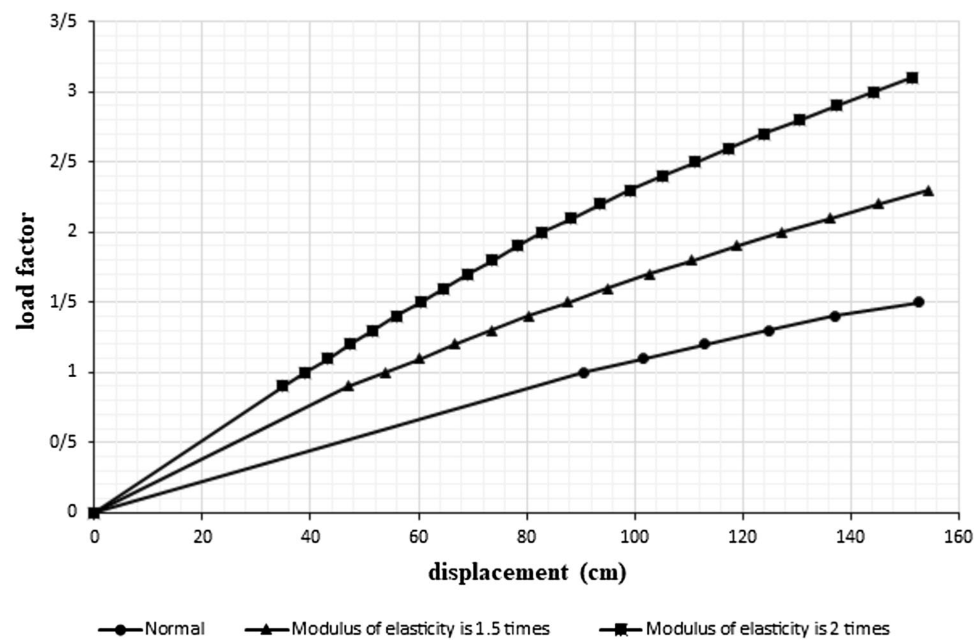

Fig. 13 Capacity curves after removal of cable 3 for three values of modulus of elasticity

Modeling Cable Removal

To consider dynamic effects of removing a column, at first, the undamaged structure is modeled and the axial force of the intended cable is calculated. Then, this internal force is replaced with the damaged cable. For modeling the cable removal action, two step forces are defined. Gravity load (W = DL + 0.25LL) and axial force of the intended cable (P) are applied simultaneously in 1 s. Load P still applied for 1 s more and then it is vanished while the gravity load is remained until the end of the analysis [35–38].

Validation

Cai et al. [39] analyzed the cable bridge in Fig. 1 by four analytical procedures, i.e., linear static, nonlinear static, linear dynamic and nonlinear dynamic. For validation, in this paper, after the removal of the cable 4, displacement of the nodes 6, 7 and 8 are compared with Cai et al. [39]. According to Fig. 4, the results obtained in this paper have good agreement with Cai et al. [39] results.

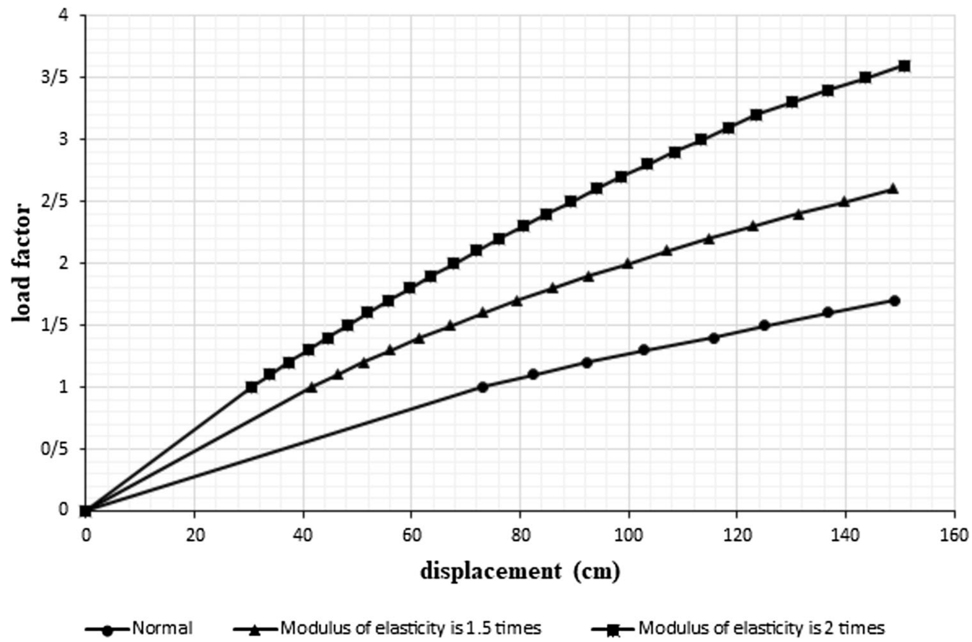

Fig. 14 Capacity curves after removal of cable 4 for three values of modulus of elasticity

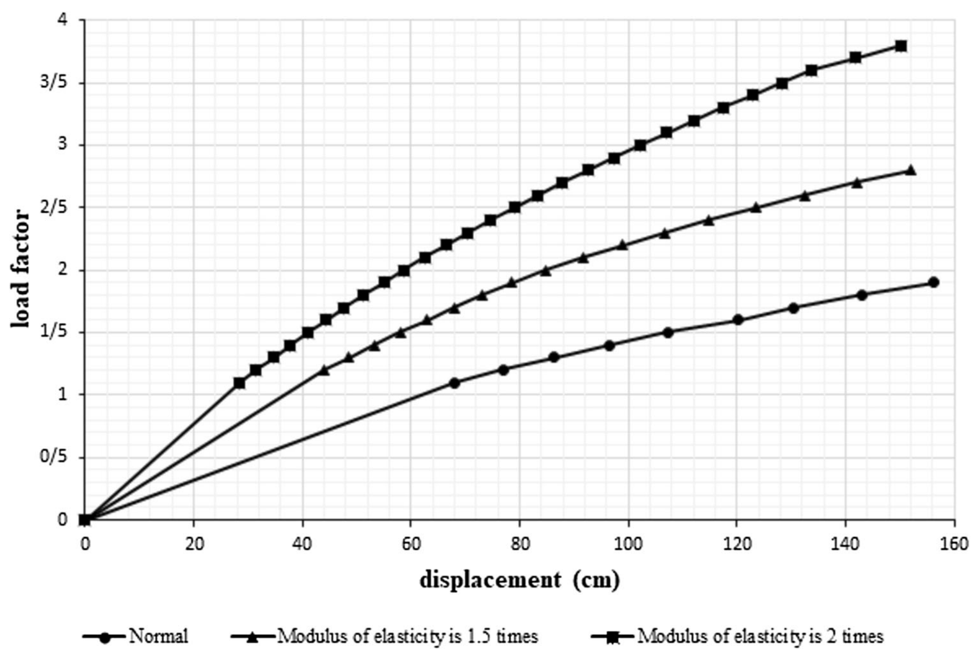

Fig. 15 Capacity curves after removal of cable 5 for three values of modulus of elasticity

Results

Capacity curves are plotted through incremental dynamic analysis (IDA) [40, 41]. For incremental dynamic analysis, for example, 5% of gravity loads (0.05(DL + 0.25LL)) is applied to the undamaged structure and axial forces of the intended cable are calculated. Then, this internal force is substituted for the intended cable and incremental dynamic analysis is performed and cable-removed point displacement is calculated. Then, 10% of gravity loads (0.1(DL + 0.25LL)) is applied and the displacement is calculated. Finally, after several increasing of loads, the resistance of the structure is obtained by plotting the load–displacement curve. This method is time-consuming in such a way that to investigate the effects of one column removal, several nonlinear analyses should be performed; however, it is the most accurate method. The alternative approach is the static pushdown analysis which does not consider the dynamic effects of column removal action and therefore, is not appropriate.

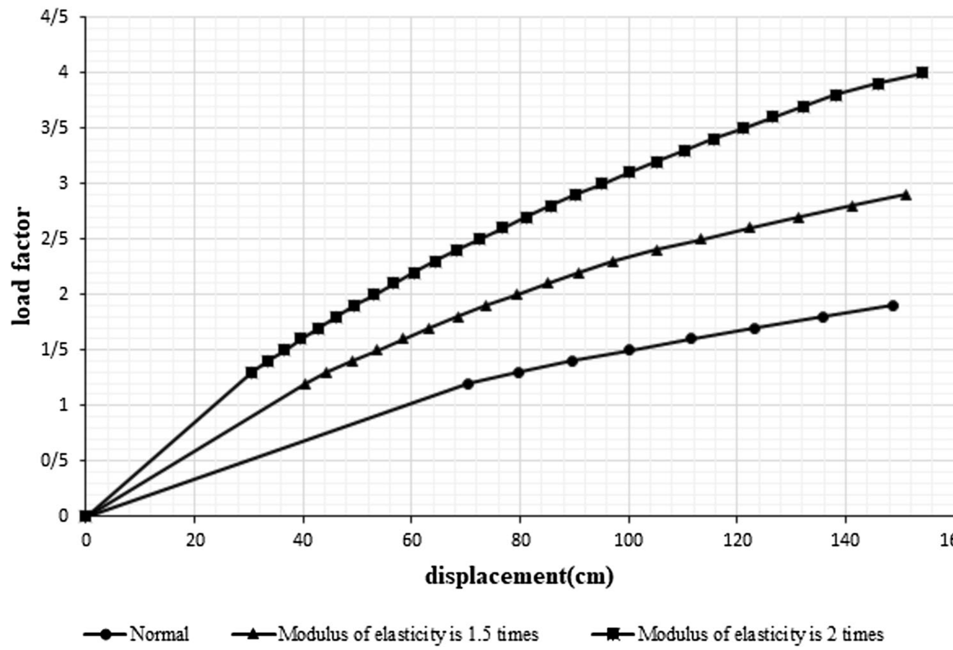

Fig. 16 Capacity curves after removal of cable 6 for three values of modulus of elasticity

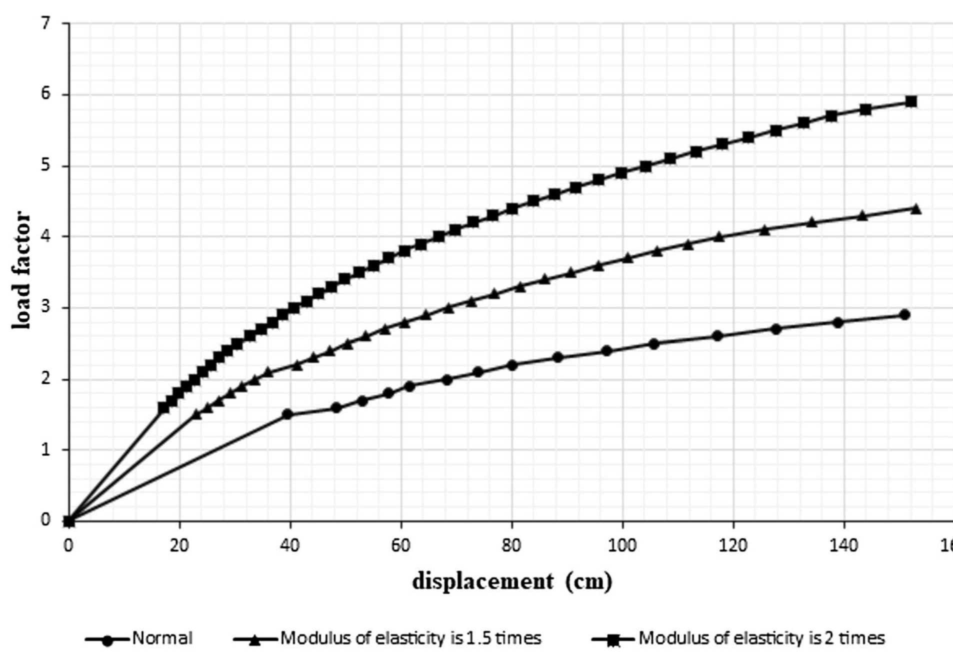

Fig. 17 Capacity curves after removal of cable 7 for three values of modulus of elasticity

Capacity Curve of the Cable-Stayed Bridge

Since the bridge is symmetric, only one half of it is analyzed. The capacity curves of the cables 2–7 removal are shown in Fig. 5. As it is seen, the bridge is more susceptible to progressive collapse when the cables that are farther from pylon are ruptured. That is, as the cable in the vicinity of pylon is removed (cable 7), the bridge can withstand about three times of the GSA specified load, while the load factor is about 1.2 for the farthest cable (cable 2). Moreover, the ductility as well as the stiffness of the structure is decreased by losing farther cables from the pylon.

The Effect of Cross Section of Cables

To investigate the effect of cross section of cables in progressive collapse resistance of a cable-stayed bridge, the cross-sectional area of all cables is increased 1.5 and 2 times. The results of the cables removal analysis are shown in Figs. 6, 7, 8, 9, 10 and 11.

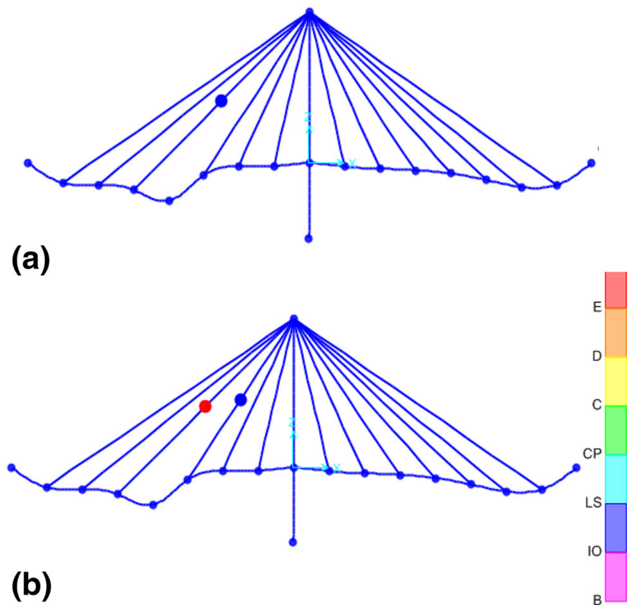

Fig. 18 Removal of cable four for load factor of 1.8: (**a**) 2 × F_y; (**b**) 1.5 × F_y

The results show that the effect of cable cross section in increasing the resistance and stiffness against progressive collapse is more for farther cables from pylon. However, for the cables in the vicinity of pylon, not only the resistance would not differ, but also the stiffness is even decreased.

The Effect of Modulus of Elasticity of Cables

To study the effect of modulus of elasticity of cables in progressive collapse resistance of a cable-stayed bridge, the modulus of elasticity of all cables is increased 1.5 and 2 times. The results of the cable removal analysis are shown in Figs. 12, 13, 14, 15, 16 and 17.

The results showed that increasing the modulus of elasticity of the cables makes the structure more robust against the progressive collapse. Unlike the cross section, the increase in structural resistance and stiffness due to increase in modulus of elasticity is uniform for different cables. Moreover, increasing the modulus of elasticity has a greater effect on the strength of the structure compared to cross section.

The Effect of Cable Yield Stress (F_y)

To study the effect of yield stress of cables in progressive collapse resistance of a cable bridge, the F_y of all cables are increased 1.5 and 2 times. For all cases, the capacity curves were similar to each other, i.e., increasing the yield stress of cables had no effect on progressive collapse resistance. The only effect was on the formation of plastic hinges, as shown in Fig. 18.

Conclusion

In this paper, progressive collapse analysis of a cable-stayed bridge is investigated. In this regard, the effects of changes in F_y, E and cross-section area of cables to progressive collapse resistance are studied. The evaluation is performed by alternate load path method and the nonlinear time history tool in SAP2000V17 software.

The results are as follows:

1. The bridge is more susceptible to progressive collapse when the cables that are farther from pylon are ruptured. Moreover, the ductility as well as the stiffness of the structure is decreased by losing farther cables from the pylon.
2. The effect of cable cross section in increasing the resistance and stiffness against progressive collapse is more for farther cables from pylon. However, for the cables in the vicinity of pylon, not only the resistance would not differ, but also the stiffness is even decreased.
3. Increasing the modulus of elasticity of the cables makes the structure more robust against the progressive collapse. Unlike the cross section, the increase in structural resistance and stiffness due to increase in modulus of elasticity is uniform for different cables. Moreover, increasing the modulus of elasticity has a greater effect on the strength of the structure compared to cross section.
4. Increasing the yield stress of cables had no effect on progressive collapse resistance. The only effect was on the formation of plastic hinges.

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