Journal of Zhejiang University-SCIENCE A (Applied Physics & Engineering)
ISSN 1673-565X (Print); ISSN 1862-1775 (Online)
www.zju.edu.cn/jzus; www.springerlink.com
E-mail: jzus@zju.edu.cn

Comparison of various procedures for progressive collapse analysis of cable-stayed bridges*

Jian-guo CAI[†1], Yi-xiang XU[2], Li-ping ZHUANG[1], Jian FENG[†‡1], Jin ZHANG[1]
([1]Key Laboratory of C&PC Structures of Ministry of Education, Southeast University, Nanjing 210096, China)
([2]Department of Civil Engineering, Strathclyde University, Glasgow, UK)
[†]E-mail: caijg_ren@hotmail.com; fengjian@seu.edu.cn
Received Nov. 2, 2011; Revision accepted Jan. 4, 2012; Crosschecked Apr. 5, 2012

Abstract: Alternate path (AP) method is the most widely used method for the progressive collapse analysis, and its application in frame structures has been well proved. However, the application of AP method for other structures, especially for cable-stayed structures, should be further developed. The four analytical procedures, i.e., linear static, nonlinear static, linear dynamic, and nonlinear dynamic were firstly improved by taking into account the initial state. Then a cable-stayed structure was studied using the four improved methods. Furthermore, the losses of both one cable and two cables were discussed. The results show that for static and dynamic analyses of the cable-stayed bridges, there is large difference between the results obtained from simulations starting with either a deformed or a nondeformed configuration at the time of cable loss. The static results are conservative in the vicinity of the ruptured cable, but the dynamic effect of the cable loss in the area farther away from the loss-cable cannot be considered. Moreover, the dynamic amplification factor of 2.0 is found to be a good estimate for static analysis procedures, since linear static and linear dynamic procedures yield approximately the same maximum vertical deflection. The results of the comprehensive evaluation of the cable failure show that the tread of the progressive failure of the cable-stayed bridges decreases when the location of the failed cables is closer to the pylon.

Key words: Progressive failure, Structural failures, Collapse, Linear analysis, Nonlinear analysis, Dynamic analysis, Cable-stayed bridges
doi:10.1631/jzus.A1100296 **Document code:** A **CLC number:** TU312+.3

1 Introduction

It has recently become evident that abnormal loads need to be considered in the design of structures so that progressive collapse can be prevented. Building collapses such as Ronan Point, Alfred P. Murrah and the World Trade Center have shown the catastrophic nature of progressive failure, and with an increasing trend towards more terrorist actions in the future, it is clear that structural design must include progressive collapse mitigation (Georgakopoulos, 2005). 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. From an analytical viewpoint, progressive collapse is a dynamic event, and the motion is initiated by a release of internal energy due to the instantaneous loss of a structural member. 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 (Marjanishvili and Agnew, 2006).

Following the approaches proposed by Ellingwood and Leyendecker (1978), the design guidelines (ASCE-7, 2002; GSA, 2003; DOD, 2009) define the following methods for structural design of buildings

‡ Corresponding author
* Project supported by the National Natural Science Foundation of China (No. 50478075), the Jiangsu "Six Top Talent" Program of China (No. 07-F-008), the Priority Academic Program Development of Jiangsu Higher Education Institutions, and the Scientific Research Foundation of Graduate School of Southeast University (No. YBJJ0817), China

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). Different simplified procedures for simulating the effects of progressive collapse can now be found (Ellingwood and Leyendecker, 1978; Kaewkulchai and Williamson, 2004; Grierson *et al.*, 2005; Agarwal *et al.*, 2006; Val and Val, 2006; Bao *et al.*, 2008; Khandelwal *et al.*, 2008; Kim and An, 2009; Menchel *et al.*, 2009; Lee *et al.*, 2009; Fu, 2009; 2010; Alashker *et al.*, 2010; Liu, 2010; Kwasniewski, 2010; Scott and Fenves, 2010). However, the work in this field refers predominantly to buildings. Recent events, such as the collapse of the I-35W deck truss bridge (Astaneh-Asl, 2008; Hao, 2010) in Minneapolis, Minnesota, bring into sharp focus the need to incorporate progressive collapse into the design of major bridges. Jenkins (1997) pointed out that terrorist attacks on public transportation have increased over the past quarter of a century. Bridges are attractive terrorist targets because of their easy accessibility as well as the devastating consequence on the society when damaged (Yan and Chang, 2009).

Compared to buildings, bridges are primarily horizontally aligned structures with one main axis of extension. Thus, the possible mechanisms of collapse are different. Given structural efficiencies intrinsic to long-span bridges, designing against progressive collapse has not been a major consideration in the development of bridge form. Cable-stayed bridge is the only bridge form routinely designed for cable loss. The loss of cables must be considered as a possible local failure since the cross sections of cables are usually small, and therefore provide low resistances against accidental lateral loads stemming from vehicle impact or malicious action (Wolff and Starossek, 2008). The loss of cables can lead to overloading and rupture of adjacent cables. Furthermore, the stiffening girder is in compression and a cable loss reduces its bracing against buckling (Starossek, 2006). Zoli and Steinhouse (2007) gave the examples about the loss of cable in some situations and conducted a comprehensive evaluation of multi-cable loss to assess the structure's resistance to progressive collapse based on a new bridge in the northeast United States. Wolff and Starossek (2008) examined the structural response of a cable-stayed bridge to the loss of one cable by means of dynamic analyses including large displacements and studied the effects of cable sag, transverse cable vibrations and structural damping. Yan and Chang (2009) proposed a probabilistic assessment framework to quantitatively analyze the vulnerability of cable-stayed bridges under terrorist attack. Then they also developed a technique based on the plastic limit analysis for the vulnerability assessment of single-pylon cable-stayed bridges (Yan and Chang, 2010).

For the design of cable-stayed bridges, the current recommendations state that the sudden rupture of one cable should not lead to structural instability (i.e., global structural failure) and specify a loss-of-cable load case (PTI, 2001; FIB, 2005). The PTI (2001) provides prescriptive guidance in the extreme event of cable loss, both in terms of load applications and resistance factors. Two load application methods are prescribed. The simplified static method is to investigate the structure with a missing cable under factored dead and live loads combined with the static application of the dynamic force imparted from the severed cable. Alternatively, the PTI (2001) permits the use of dynamic analysis to a more accurately compute structural response due to an abrupt cable failure. However, little guidance is provided on how to conduct such a dynamic analysis and the design of the global structural system. To further contribute to the elaboration of design codes for progressive collapse of cable-stayed bridges, studies on the progressive collapse simulation procedures are compared to each other.

The present work focuses on the alternate load path method for the progressive collapse of cable-stayed bridges. This study will not consider the collapse triggering event, but will provide useful information on how to prevent further damage from occurring in the structure. Four different procedures: static linear, static nonlinear, dynamic linear and dynamic nonlinear are performed to explain the details of progressive collapse phenomena of cable-stayed bridges. This is carried out using the commercially available computer program SAP2000, although the procedures outlined can be followed using any finite-element computer program capable of nonlinear dynamic analysis.

The aim of this paper is to provide clear step-by-step descriptions of four increasingly complex methods for progressive collapse analyses of cable-stayed bridges. Linear static and linear dynamic analyses are conducted with the dynamic amplification factor given by PTI (2001) recommendations to account for the dynamic effects. The static nonlinear and dynamic nonlinear analyses are undertaken by taking into account large displacements and nonlinear material behavior. The four different analysis procedures are revised to consider the initial conditions. This is due to the fact that for prestressed structures, such as cable-stayed bridges, the geometry and behavior of the structure will be changed once it is prestressed. The progressive collapse analysis results of the four procedures with and without considering the initial conditions are compared. Then the response of a hypothetical single-tower cable-stayed bridge to the loss of one and two cables is investigated using the revised methods. Finally, conclusions and recommendations for the preferred analysis procedure on its accuracy and complexity to perform are given.

2 Description of the analysis

2.1 Bridge system and modeling

The schematic diagram of the cable-stayed bridge is shown in Fig. 1. The bridge has one single tower of 45 m high and two equal side spans of 60 m. The girder is assumed to be hinged with the tower at a height of 15 m and roller supported at both ends. It is also supported by 14 stay cables, 7 on each side. The material and geometric properties of the girder are assumed 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 stay cables are made of steel strands with the following properties: 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 made of reinforced concrete. The material and geometrical properties of the pylon are assumed 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 investigation is conducted by SAP2000 using a 2D bridge model. The pylon and girders are modeled with frame elements. The dead and live loads considered here are 80 and 40 kN/m, respectively. The stay cables have been prestressed in such a way that under dead and live loads, the girder has no deflection at the anchorage points.

2.2 Nonlinearity

True structural response to arbitrary loading is nonlinear, including material and geometric nonlinearity, and thus the nonlinear behavior of cable-stayed bridges under cable-loss cases should be considered. The plastic hinge is used in SAP2000 Program to take into account the material nonlinearity. There are two potential progressive collapse scenarios, i.e. flexural/buckling failure of the girders and overloading of the adjacent cables (progressive failure of the adjacent cables). Therefore, the material nonlinearity is considered for the cables and girders. As the cable can only be in tension, an axial plastic hinge has been introduced in the middle of the element.

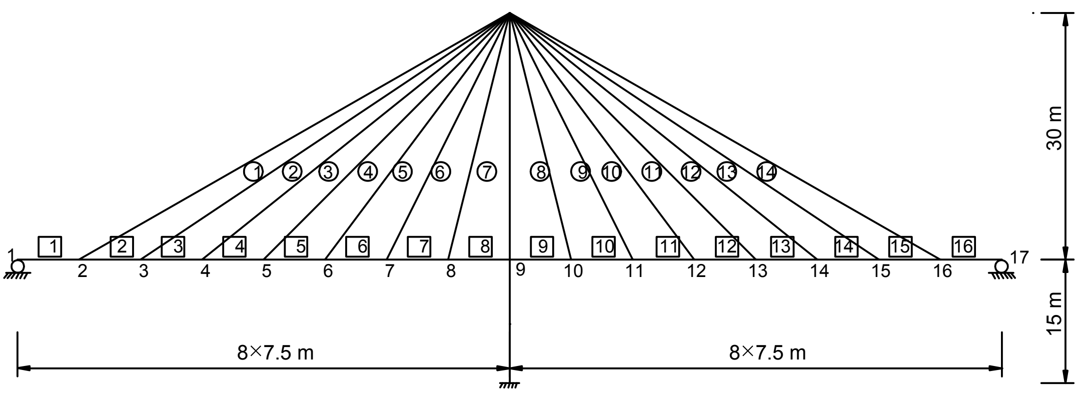

Fig. 1 A single-pylon cable-stayed bridge

The axial plastic hinge model for cables is given in Fig. 2. The yield axial force of a cable is 3312 kN, which is obtained from the material and cross-sectional properties. The elastic stretch rate of cables at the yield point is 0.63%; then the stretch rate at cable rupture is assumed as three times 0.63%, which is the value of Disp/SF at point *C* as shown in Fig. 2, where SF is the yield force or displacement of cables. The pylon and girders are subjected to bending moment and axial force. Thus, the P-M3 plastic hinges are assigned to both ends of elements. Hinge properties based on the Federal Emergency Management Agency-273 guidelines (FEMA, 1997) are adopted for the hinge model. Different performance levels are represented by circular symbols with different shadows, as shown in Figs. 2 and 3, where the immediate occupancy (IO), life safety (LS), and collapse prevention (CP) are three building performance levels on the FEMA-273 guidelines.

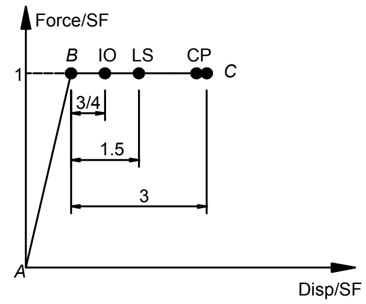

Fig. 2 Axial plastic hinge model for cables

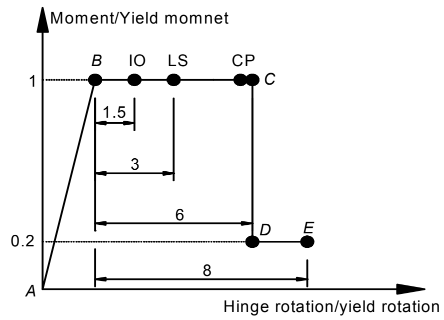

Fig. 3 P-M3 plastic hinge model for grids and pylon

Due to large deflections after the loss of cables, the effect of geometric nonlinearity is introduced in the nonlinear analysis procedures, i.e., the nonlinear static and dynamic methods. Then the catenary action in the girder with large deformation is included in the nonlinear analysis.

2.3 Loss scenario

For progressive collapse, the sudden loss of one single cable is required at a time given by the PTI (2001) recommendations. The cause of element failure is not considered in the current recommendations (PTI, 2001; FIB, 2005), i.e., the analysis being carried out as threat-independent. However, Starosssek (2006) suggested that the sudden loss of all cables in a 10-m range should be considered. In this study, the losses of one cable and two adjacent cables are investigated in the four analysis procedures.

2.4 Loading criteria for member loss

In accordance with the PTI (2001) recommendations and GSA (2003) progressive collapse guidelines, the following loading combination is used when evaluating the progressive collapse:

$$\text{Load}=1.0\text{DL}+0.75\text{LL}+1.0\text{PS}+1.0\text{CL}, \qquad (1)$$

where DL is the dead load, LL is the live load, PS is the prestress of cables, and CL is the equivalent force due to cable failure. The self-weight of elements is automatically generated by SAP2000 based on element volume and material density. For the dynamic analysis, the cable to be considered for failure is eliminated in the structural model and the corresponding cable force is applied to the anchorage nodes of the cable as shown in Fig. 4a. Then the dynamic analysis is carried out on the modified and loaded system. For the static analysis, the force from loss cable is applied to anchorage joints in the negative direction, as shown in Fig. 4b. Note that implicit to this assumption is an abrupt cable loss, then the results of the structure under dead and live loads could be superimposed with the load case of $-2F$. Thus, $-F$ effectively removes the cable by superposition, and the additional $-F$ represents the assumed equivalent dynamic impact factor of 100% (Zoli and Steinhouse, 2007).

3 Progressive collapse analysis procedures

In the current guidelines or research of progressive collapse, the effect of the initial state of structures

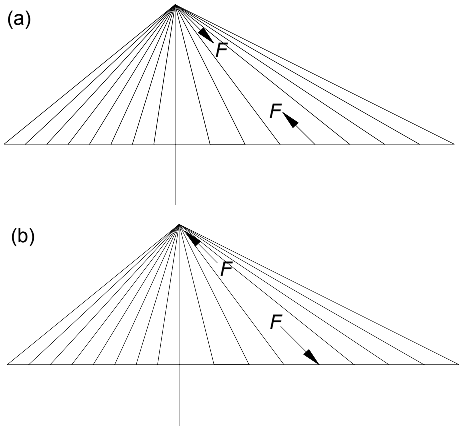

Fig. 4 Load direction for the dynamic analysis (a) and static analysis (b)

on the progressive collapse results is ignored. For most of the framed structures, the response is not strongly dependent upon whether the analysis starts from a deformed or a nondeformed configuration at the time of column failure (Kaewkulchai and Williamson, 2003). However, for a prestressed structure, the initial prestressed state is important. Hence, the influence of the initial state on the results of cable loss is considered.

The analysis for the cable loss is undertaken using four increasingly complex analysis procedures: linear static, nonlinear static, linear dynamic and nonlinear dynamic. The four analysis procedures, where the initial state is not taken into account, are the same as those given in Marjanishvili and Agnew (2006). Since the linear static analysis approach given by PTI (2001) recommendations is the simplest method that does not consider the nonlinear or dynamic effect, this linear static analysis procedure of cable-stayed bridges is the same as that of frame structures.

3.1 Nonlinear static procedure

For a nonlinear static analysis, step-by-step increased equivalent forces are applied due to cable loss until the maximum loads are reached or the structure collapses while allowing structural members to undergo nonlinear behavior. The nonlinear static analysis starting from the initial prestressed state is undertaken using the nonlinear staged construction in SAP2000. In the first step, the nonlinear static analysis of the complete structure is performed under the load combination given as

$$\text{Load}=1.0\text{DL}+0.75\text{LL}+1.0\text{PS}. \tag{2}$$

In the second step, the cable to be considered for failure is eliminated in the structural model and the corresponding cable forces are applied to the anchorage nodes of the cable as static loads. The nonlinear static analysis is undertaken on this modified structural system.

The comparison of results obtained from the nonlinear static analyses starting with deformed and nondeformed configurations are shown in Fig. 5. From the distribution of plastic hinges, it can be seen that the bridge within original state shows smaller forces of the remaining cables and girders when compared to the response obtained from the structure assuming that failure occurred while it was in its prestressed state. This is due to the fact that the pretension in the cables increases progressively after every step in the analyses starting with nondeformed configurations. It is obvious that this analysis does not coincide with the real situation.

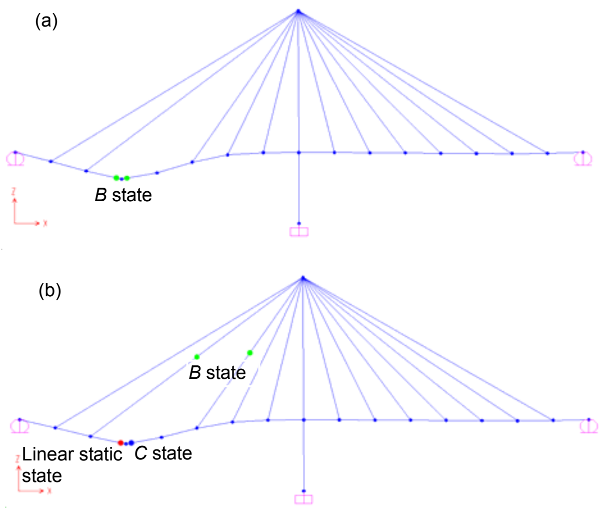

Fig. 5 Results of analyses starting with nondeformed (a) and deformed (b) configurations

3.2 Linear dynamic procedure

The real-time removal of cables is taken into account in the dynamic analysis for both linear and nonlinear methods. Thus, these two methods can also refer to the time-history analyses. These methods are more accurate than the static approach because they inherently incorporate dynamic amplification factors, inertia, and damping forces (Marjanishvili, 2004). Wolff and Starossek (2008) pointed out that damping has only marginal effects on the state variables of the bridge girder and the cables forces. Thus, damping is not considered in this study.

The initial conditions method (Buscemi and Marjanishvili, 2005) can be used to perform dynamic analysis starting from the deflected shape of the undamaged structure under normal service loads. Progressive collapse analysis using initial conditions is performed as follows: (1) perform static analysis to determine the deflected shape of the undamaged structure; (2) remove damaged bearing elements from the model; (3) perform dynamic analysis by assuming the initial conditions match the deflected shape of the undamaged structure (Buscemi and Marjanishvili, 2005). However, the initial state can only be set in the nonlinear analysis with SAP2000. Thus, the instantaneously unloading method is used in this study. Fig. 6a shows a schematic representation of the instantly unloaded time-history. The dynamic load, the normal service loads, and the prestress are applied to the damaged structure simultaneously. However, the initial state is not considered in the time-history given in Fig. 6a. The schematic representation of the cable force function considering the initial prestressed state is shown in Fig. 6b. Firstly, the load is applied to compensate for the force in the anchorage nodes of the lost cable, and the structure is vibrating until it reaches a steady state, settling into a configuration prior to the loss of the cable under combined dead and live loads. Once the model reaches the steady state, the cable loss event is simulated as an abrupt drop of $P(t)$ to 0. The unloading duration t_p is investigated by many studies (Zoli and Steinhouse, 2007), and it is assumed to be 10 ms in this study.

It could be interesting to know the minimum duration t_0 of the equivalent force to obtain the quasi-static response, i.e., the state prior to the loss of the cable under normal service loads. It is attempted to keep the dynamic response of the structure to a minimum while it reaches the steady state. The duration of the equivalent force is considered as a function of t_0/T, where T is the fundamental period of the

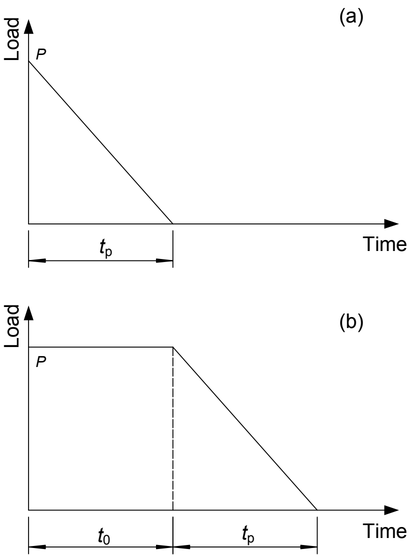

Fig. 6 Unloaded time-history for the equivalent force due to cable failure
(a) Load curve without the initial state; (b) Load curve with the initial state

undamaged cable-stayed bridges. The influence of different durations of the equivalent force is shown in Fig. 7. The function of t_0/T corresponds to 5, 10, and 15, respectively. It can be seen that the cable-stayed bridge achieves a nearly quasi-static response while the function of t_0/T is 10 and 15. Then the duration of the equivalent force due to cable loss is assumed $10T$ in this study.

The vertical displacement time-history of Node 5 after the loss of Cable 4 for linear dynamic analysis with or without the duration of equivalent force t_0 is shown in Fig. 8. It can be seen that the maximal vertical displacement is −257.2 mm using the load curve given in Fig. 6a. Using the load curve given in Fig. 6b, the vertical displacement is −52.3 mm when the structure reaches its steady state. This coincides well with the vertical displacement of undamaged structure under normal service loads, which is −56.2 mm. The maximal vertical displacement is −210.5 mm. The displacement response of the cable-stayed bridges without the steady state shows greater amplitude with the same unloading duration t_p compared to the results obtained from the analysis considering the steady state. Therefore, the load curve given in Fig. 6b is used in the following progressive collapse analyses.

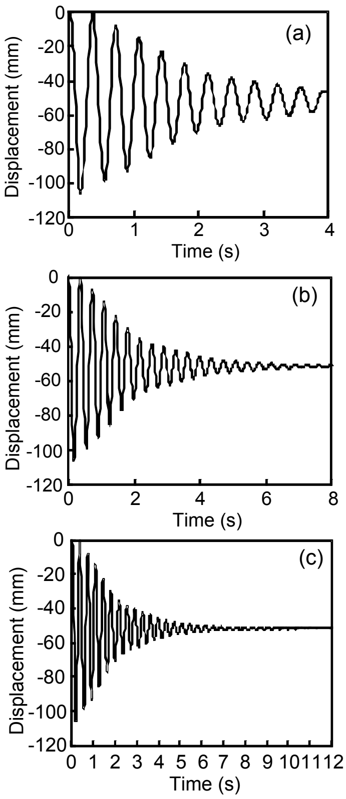

Fig. 7 Vertical displacement time-history of Node 5 for 5 (a), 10 (b), and 15 (c) times fundamental frequency of the equivalent force

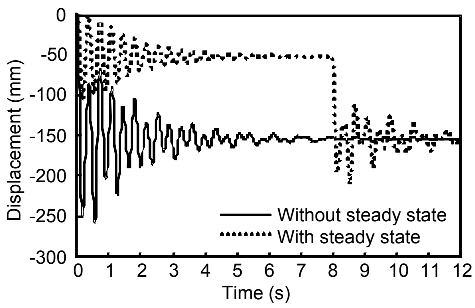

Fig. 8 Vertical displacement time-history of Node 5 for linear dynamic analysis

3.3 Nonlinear dynamic procedure

Nonlinear dynamic analysis is performed similarly to linear dynamic analysis with the exception that the structural elements are now allowed to enter their inelastic range. However, another methodology is used to perform the nonlinear dynamic analysis starting from the initial state. The main steps are as follows: (1) Find the configuration of undamaged structure (i.e., without the cable loss) under normal service loads. The combined loads defined by Eq. (1) with the direction of the equivalent force of cable failure given in Fig. 4a are applied. The nonlinear static analysis is used to determine the initial state. (2) Perform nonlinear time-history analysis with the initial state, where the equivalent force of the failure cable is applied in the opposite direction as shown in Fig. 4b.

The vertical displacement time-history of Node 5 obtained from nonlinear dynamic analyses starting with the deformed or nondeformed state are shown in Fig. 9 for the sudden removal of Cables 3 and 4. It can be seen that the maximal vertical displacement of cable-stayed bridges starting from the origin state is −978.2 mm, which shows greater amplitude when compared to the response (i.e., −811.7 mm) obtained from model with initial cable failure in its deformed configuration.

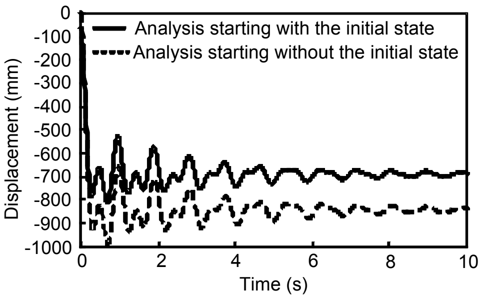

Fig. 9 Vertical displacement time-history of Node 5 for nonlinear dynamic analysis

As mentioned above, it is possible to conclude that for the static or dynamic response there are large differences between analyses starting with a deformed configuration and a nondeformed configuration at the time of cable loss. Then the analyses starting with the initial state (i.e., the prestressed state) will be used in the progressive collapse of cable-stayed bridges in the following sections.

4 Progressive collapse analysis results

The numerical calculation of cable loss is conducted using SAP2000 with the method given in Section 3. The losses of one cable and two cables are analyzed, and results, such as forces and stresses for main load-carrying members and vertical displacements for anchorage points of cables, are given.

4.1 Loss of one cable

The structural response of a cable-stayed bridge to the sudden loss of Cable 4 is shown in Figs. 10a–10c. Fig. 10a shows the cable force after the cable failure using the four methods. The cable forces of the undamaged structure are also given for comparison. It can be seen that considerable changes have been found in the cable force near the vicinity of the ruptured cable. However, the forces of cables at the locations further away from the ruptured cable remain (almost) unchanged by using the linear or nonlinear static method. It can be concluded that the influence of the sudden removal of a cable on the vicinity further away from the ruptured cable cannot be considered in the static analysis. Furthermore, the difference between the results given by linear and nonlinear analyses is minor. The reason is that there is no plastic hinge in the main load-carrying members after one cable loss, leaving the structure within the elastic range.

The vertical deflection of the nodes at the girder after the loss of Cable 4 is given in Fig. 10b. The end nodes of the adjacent cables experience relatively large displacements, since the sudden loss of a cable leads to the redistribution of the load to adjacent structural elements. It can also be found that there is little difference between the results given by the static and dynamic analyses (either linear or nonlinear). This is also because that the structure is within the elastic range after the cable failure. The vertical displacements of nodes near the vicinity of the ruptured cable given by the dynamic analysis are lower than those given by the static analysis. However, the vertical deflections further away from the ruptured cable given by the dynamic analysis are larger than those given by the static analysis. Thus, the static results are conservative near the vicinity of the ruptured cable but non-conservative at the locations further away from the ruptured cable.

Fig. 10c shows the stress ratio of the girders, which is automatically calculated by SAP2000 based on Eurocode 3 (EN 1993-1-1:2005). It can be seen from the curves that the stress ratios of Girders 4 and 5 are greater than 0.95 in the linear static analysis, and exceed 0.9 in the nonlinear static analysis. However, the stress ratios for the two dynamic analyses are less than 0.8. These results also show that the static results are conservative in the vicinity of the ruptured cable. It can also be found that the stress ratios of girders near the vicinity of the ruptured cable given by linear dynamic analysis are local minimal and the difference between these and the stress ratios given by nonlinear dynamic analysis is significant. This is due to the fact that the geometry nonlinear is not considered in the linear dynamic analysis. Thus, it can be concluded that the geometry nonlinearity is essential to the progressive collapse of cable-stayed bridges.

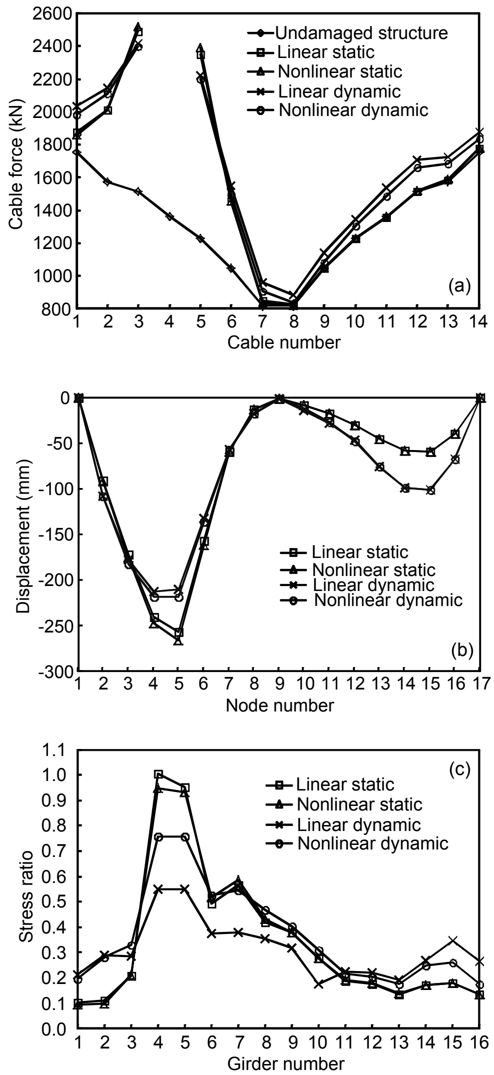

Fig. 10 Cable force (a), nodal vertical displacements (b), and stress ratios of girder elements (c) after the loss of Cable 4

4.2 Loss of two cables

The sudden removal of Cables 3 and 4 is investigated in this section. Fig. 11 shows the cable forces after the loss of Cables 3 and 4. It can be seen that the variations of the forces near the lost cables are significant. The axial forces of Cables 2 and 5 reach 3312 kN, which is the yield tension of cables, in the linear analyses (including linear static and linear dynamic). Also, the plastic hinges are found in Cables 2 and 5 using the nonlinear static and dynamic analyses. The demand to capacity ratio (DCR) is used in the GSA (2003) guidelines to evaluate the results of linear analysis, and the limit of DCR values is dependent on the cross-sectional dimensions and the construction materials. In all cases, these limit values are higher than 1.0, in order to account for the structure's ability to redistribute stresses. According to the GSA guideline, the DCR limit value for the cable should be 1.0. The DCR for Cable 2 using linear static analysis can be found by taking the ratio of the axial force in the cable to its ultimate capacity,

$$\mathrm{DCR}=\frac{F_{\mathrm{l}}}{F_{\mathrm{u}}}=\frac{3806\ \mathrm{kN}}{3312\ \mathrm{kN}}=1.15>1.0, \tag{3}$$

where F_{l} is the axial cable force given by linear analysis, and F_{u} is the ultimate tension capacity. The DCR of Cable 2 is 1.23 for the linear dynamic analysis. Using Eq. (3), it can be concluded that the cable-stayed bridge does not satisfy the GSA progressive collapse criterion with the results given by linear analysis, and Cable 2 should be removed from the structure. However, the results given by nonlinear dynamic analysis show that Cable 2 does not fail, although the plastic hinge starts to form in Cable 2. Therefore, the limit of DCR value, which is equal to 1.0, is conservative for cable-stayed bridges.

The vertical displacements of anchorage nodes after the loss of Cables 3 and 4 are shown in Fig. 12. Note that it was impossible to attain 100% of the equivalent force of the lost cable during nonlinear static analysis. Based on the result, it can be concluded that our proposed cable-stayed bridge appears to be susceptible to progressive failure. However, the nonlinear dynamic analysis shows that the structure would not actually be susceptible to progressive collapse. The maximum calculated defection of Node 4 at the failure (at 69.34% of the load) is −1064 mm. The vertical displacements of Node 4 calculated by linear static and linear dynamic analyses procedures are −737 mm and −690 mm, respectively. They are very close, which leads to the conclusion that the dynamic amplification factor of 2.0 used in the linear static analysis is a good estimate.

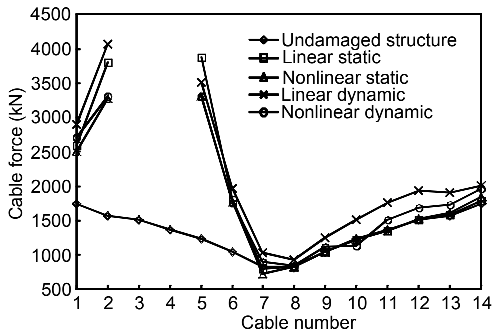

Fig. 11 Cable force after the loss of Cables 3 and 4

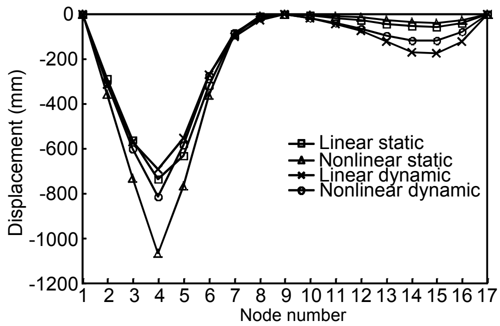

Fig. 12 Nodal vertical displacements after the loss of Cables 3 and 4

4.3 Analysis summary

In the proposed structure, the loss of a single cable does not lead to a progressive collapse. The cable tensions remain relatively small. Also, the plastic hinges are not formed in the girders and the deformations are not significant. For the simultaneous failure of two adjacent cables, the cable-stayed bridge remains intact, although plastic hinges are found in the bridge girder, and cables begin to yield and girder deformations increase sharply. If the live load can be increased by two factors or more until the ultimate state is reached (Yan and Chang, 2010), cables located adjacent to the lost cables will fail. Then damage will progress with more cables failing and collapse cannot be arrested. This is called a zipper-type collapse (Starossek, 2007).

If the DCR limit value is 1.0 for cables and girders, the evaluation criterion for linear analysis appears to be slightly more stringent in comparison to nonlinear analysis procedures, since in our analysis the bridge is kept intact in nonlinear dynamic analysis but fails in linear dynamic analysis. Since linear static and linear dynamic analyses yield approximately the same maximal vertical displacement, a dynamic amplification factor of 2.0 is necessary for the safe design of the cables, which is also shown by its wide acceptance in the literature (Wolff and Starossek, 2008). For the nonlinear static analysis, it is indicated that 69.34% of the load given by Eq. (1) is attained. The results are very conservative in comparison with the nonlinear dynamic analysis results. Therefore, to trace the collapse progression after an initial failure of one or more cables, geometrically and materially nonlinear dynamic analysis is the best candidate.

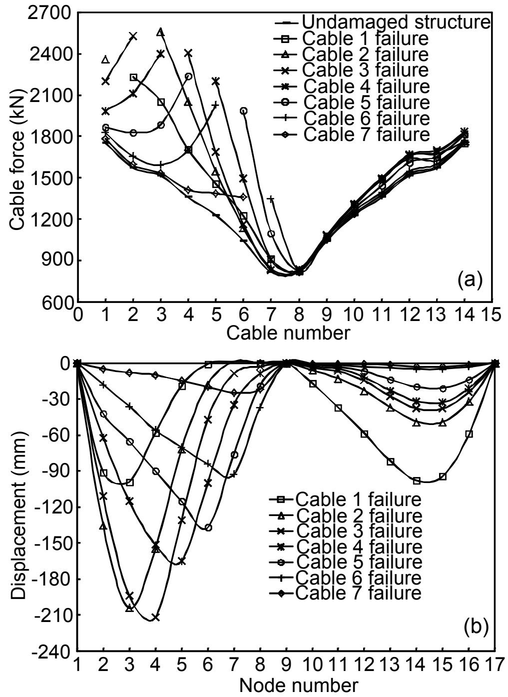

Fig. 13 Results after one-cable loss
(a) Cable force; (b) Nodal vertical displacement

5 Additional studies

A comprehensive study of one- and two-cable losses was undertaken using the nonlinear dynamic analysis procedure to assess the structure's resistance to progressive collapse. The cable forces and nodal vertical displacements after the loss of one cable are shown in Fig. 13. Due to the symmetry of the cable-stayed bridges, analyses are performed under the initial damage cable from Cables 1 and 7. It can be seen that increased cable forces are located adjacent to the lost cable, and the cable force almost remains constant on the other side of the pylon. The maximum cable force is 2559 kN, which is carried by Cable 3 under the loss of Cable 2, and it is lower than the yield tension of the cable, 3312 kN. Therefore, the cable-stayed bridge can sustain the failure of one cable without damage. The anchorage nodes of the lost cables experience relatively severe dynamic displacements. The maximum vertical displacement of Node 4 is −212 mm after the loss of Cable 3. Note that the vertical displacements of the nodes at the other side of the pylon cannot be considered as negligible under the loss of Cable 1. The maximum value reaches −98 mm. Moreover, the vertical deflection at the other side of the pylon decreases as the lost cable approaches the pylon.

Fig. 14 shows the cable forces and vertical nodal displacements after the loss of two adjacent cables. It can be seen that the cable adjacent to the lost cables reaches the yield force. The vertical deformation of the girder increases drastically, and the maximum displacement is −917 mm. Moreover, the resistance to the progressive collapse of cable-stayed bridges increases as the location of lost cables approaches the pylon, since the cables near the vicinity of the ruptured cable will not reach the tension yield when the lost cables are near the pylon and the maximum nodal vertical displacement decreases. Therefore, the trend of the progressive failure of the cable-stayed bridges decreases when the location of lost cables is closer to the pylon.

6 Conclusions

Four different analysis procedures (linear static, nonlinear static, linear dynamic and nonlinear dynamic) taking into account the initial condition of prestressed structure are developed to investigate the progressive failure of cable-stayed bridges.

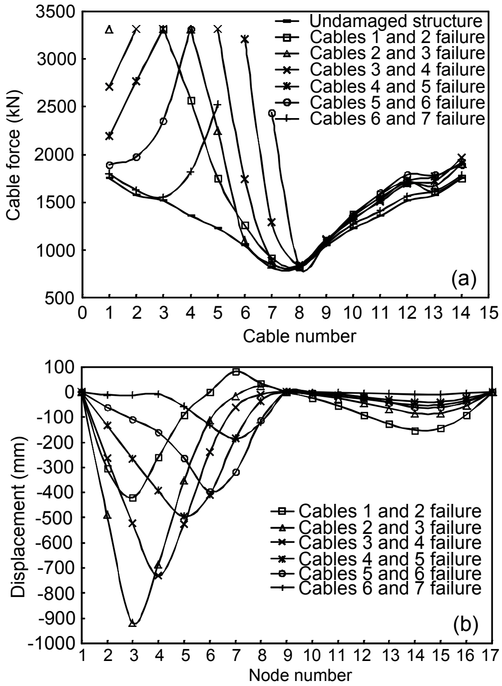

Fig. 14 Results after two-cable loss
(a) Cable force; (b) Nodal vertical displacement

Comparisons between the results obtained from analyses starting with a deformed and a nondeformed configuration at the time of cable loss show considerable differences in either the static or dynamic response of the cable-stayed bridge.

In the proposed structure, the loss of a single cable does not lead to a progressive collapse. The cable tensions remain relatively small. Also for the simultaneous failure of two adjacent cables, the cable-stayed bridge remains intact, although plastic hinges are formed in the bridge girder; cables begin to yield and girder deformations increase rapidly. The dynamic amplification factor of 2.0 is a good estimate for static analysis procedure, since linear static and linear dynamic procedures yield approximately the same maximum vertical deflection. DCRs for linear analysis procedures of cable-stayed bridges should be 1.0. Then we can conclude that the static analysis procedure is conservative, since in our analysis it is found that for the present case, the structure keeps intact in nonlinear dynamic analysis but fails in nonlinear static analysis. Moreover, the nonlinear static analysis procedure is not recommended, since it is too conservative in comparison with the nonlinear dynamic analysis results and it cannot provide complete information on structural response. Furthermore, to trace the collapse progression after an initial failure of one or more cables, the nonlinear dynamic analysis starting with the initial state given by a nonlinear static procedure is the best solution.

The results of the comprehensive evaluation of cable loss show that the cables near the vicinity of the ruptured cable do not reach the tension yield and the maximum nodal vertical displacement decreases when the lost cables are near the pylon. Thus, the resistance to the progressive failure of the cable-stayed bridges increases as the locations of lost cables near the pylon. Furthermore, the vertical displacements of the nodes at the other side of the pylon cannot be considered as negligible under the loss of outside cables. The vertical deflection at the other side of the pylon decreases as the location of the lost cable approaches the pylon.

References

Agarwal, J., England, J., Blockley, D., 2006. Vulnerability analysis of structures. *Structural Engineering International*, **16**(2):124-128.

Alashker, Y., El-Tawil, S., Sadek, F., 2010. Progressive collapse resistance of steel-concrete composite floors. *Journal of Structural Engineering*, **136**(10):187-196 [doi:10.1061/(ASCE)ST.1943-541X.0000230]

ASCE-7, 2002. Minimum Design Loads for Buildings and Other Structures. Reston, VA.

Astaneh-Asl, A., 2008. Progressive Collapse of Steel Truss Bridges, the Case of I-35W Collapse. Proceedings of 7th International Conference on Steel Bridges, Guimarães, Portugal.

Bao, Y., Kunnath, S.K., El-Tawil, S., Lew, H.S., 2008. Macromodel-based simulation of progressive collapse: RC frame structures. *Journal of Structural Engineering*, **134**(7):1079-1091.

Buscemi, N., Marjanishvili, S., 2005. SDOF Model for Progressive Collapse Analysis. Proceedings of the Structures Congress and the Forensic Engineering Symposium, New York.

DOD (Department of Defense), 2009. Unified Facilities Criteria (UFC): Design of Structures to Resist Progressive Collapse. Washington, DC.

Ellingwood, B., Leyendecker, E., 1978. Approaches for design against progressive collapse. *Journal of the Structural Division*, **104**(3):413-423.

EN 1993-1-1:2005. Eurocode 3: Design of Steel Structures–Part 1-1: General Rules and Rules for Buildings. European Committee for Standardization, rue de Stassart, 36, B-1050 Brussels.

FEMA (Federal Emergency Management Agency), 1997. NEHRP Guidelines for the Seismic Rehabilitations of Buildings (FEMA-273). Building Seismic Safety Council, Washington, DC.

FIB (International Federation for Structural Concrete), 2005. Acceptance of Stay Cable Systems using Prestressing Steels. Lausanne.

Fu, F., 2009. Progressive collapse analysis of high-rise building with 3-D finite element modeling method. *Journal of Constructional Steel Research*, **65**(6):1269-1278. [doi:10.1016/j.jcsr.2009.02.001]

Fu, F., 2010. 3-D nonlinear dynamic progressive collapse analysis of multi-storey steel composite frame buildings —Parametric study. *Engineering Structures*, **32**(12): 3974-3980. [doi:10.1016/j.engstruct.2010.09.008]

Georgakopoulos, P.J., 2005. An Overview of Progressive Collapse in Structural Systems. Massachusetts Institute of Technology, UK.

Grierson, D.E., Xu, L., Liu, Y., 2005. Progressive-failure analysis of buildings subjected to abnormal loading. *Computer-Aided Civil and Infrastructure Engineering*, **20**:155-171.

GSA (United States General Services Administration), 2003. Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Project. Washington, DC.

Hao, S., 2010. A note of the I-35W bridge collapse. *Journal of Bridge Engineering*, **15**(5):608-614. [doi:10.1061/(ASCE)BE.1943-5592.0000090]

Jenkins, B.M., 1997. Protecting Surface Transportation Systems and Patrons from Terrorist Activities: Case Studies of Best Security Practices and a Chronology of Attacks. MTI Report No. 97-04, Mineta Transportation Institute, San Jose, California.

Kaewkulchai, G., Williamson, E., 2003. Dynamic Behavior of Planar Frames during Progressive Collapse. Proceedings of the 16th ASCE Engineering Mechanics Conference, Seattle, Washington University.

Kaewkulchai, G., Williamson, E., 2004. Beam element formulation and solution procedure for dynamic progressive collapse analysis. *Computers & Structures*, **82**(7-8): 639-651.

Khandelwal, K., El-Tawil, S., Kunnath, S.K., Lew, H.S., 2008. Macromodel-based simulation of progressive collapse: Steel frame structures. *Journal of Structural Engineering*, **134**(7):1070-1078.

Kim, J., An, D., 2009. Evaluation of progressive collapse potential of steel moment frames considering catenary action. *The Structural Design of Tall and Special Buildings*, **18**(4):455-465.

Kwasniewski, L., 2010. Nonlinear dynamic simulations of progressive collapse for a multistory building. *Engineering Structures*, **32**(5):1223-1235. [doi:10.1016/j.engstruct.2009.12.048]

Lee, C.H., Kim, S., Han, K.H., Lee, K., 2009. Simplified nonlinear progressive collapse analysis of welded steel moment frames. *Journal of Constructional Steel Research*, **65**(6):1130-1137. [doi:10.1016/j.jcsr.2008.10.008]

Liu, J.L., 2010. Preventing progressive collapse through strengthening beam-to-column connection, Part 2: Finite element analysis. *Journal of Constructional Steel Research*, **66**(2):238-247. [doi:10.1016/j.jcsr.2009.09.005]

Marjanishvili, S., 2004. Progressive analysis procedure for progressive collapse. *Journal of Performance of Constructed Facilities*, **18**(2):79-85. [doi:10.1061/(ASCE)0887-3828(2004)18:2(79)]

Marjanishvili, S., Agnew, E., 2006. Comparison of various procedures for progressive collapse analysis. *Journal of Performance of Constructed Facilities*, **20**(4):365-374. [doi:10.1061/(ASCE)0887-3828(2006)20:4(365)]

Menchel, K., Massart, T.J., Rammer, Y., Bouillard, P., 2009. Comparison and study of different progressive collapse simulation techniques for RC structures. *Journal of Structural Engineering*, **135**(6):685-697. [doi:10.1061/(ASCE)0733-9445(2009)135:6(685)]

PTI (Post Tensioning Institute), 2001. Recommendations for Stay Cable Design, Testing and Installation. Cable-Stayed Bridges Committee, Phoenix, USA.

Scott, M.H., Fenves, G.L., 2010. Krylov subspace accelerated Newton algorithm: application to dynamic progressive collapse simulation of frames. *Journal of Structural Engineering*, **136**(5):473-480.

Starossek, U., 2006. Progressive Collapse of Bridges—Aspects of Analysis and Design. Proceedings of the International Symposium on Sea-Crossing Long-Span Bridges, Mokpo, Korea.

Starossek, U., 2007. Typology of progressive collapse. *Engineering Structures*, **29**(9):2302-2307.

Val, D.V., Val, E.G., 2006. Robustness of frame structures. *Structural Engineering International*, **16**(2):108-112.

Wolff, M., Starossek, U., 2008. Robustness Assessment of a Cable-Stayed Bridge. Proceeding of the International Conference on Bridge Maintenance, Safety and Management, Seoul, Korea.

Yan, D., Chang, C.C., 2009. Vulnerability assessment of cable-stayed bridges in probabilistic domain. *Journal of Bridge Engineering*, **14**(4):270-278.

Yan, D., Chang, C.C., 2010. Vulnerability assessment of single-pylon cable-stayed bridges using plastic limit analysis. *Engineering Structures*, **32**(8):2049-2056 [doi:10.1016/j.engstruct.2010.03.005]

Zoli, T.P., Steinhouse, J., 2007. Some Considerations in the Design of Long Span Bridges against Progressive Collapse. Available from http://www.pwri.go.jp/eng/ujnr/tc/g/pdf/23/23-2-3zoli.pdf [Accessed on June 30, 2010].