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Dynamic Analysis of Self-Anchored Suspension Bridge to Sudden Hanger-Breakage Event

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ABSTRACT

Served as important components of the suspension bridge, hangers generally bear large tension force and are designed with sufficient structural redundancy. However, it cannot overcome the major limitation that the mechanical properties of hangers are sensitive to corrosion and the corroded hangers may sudden break under certain circumstance. Although it is clear that a few researchers have been able to obtain higher dynamic amplification factors for cable supported bridges to the loss of one cable, there is little work with no clarity on the effect of the loss of cables on already corroded cables. This paper studies structural responses of a self-anchored suspension bridge to the abrupt hanger loss considering the influence of corroded hangers.

1. INTRODUCTION

Transferring the tension load from the supported stiffening girders, cables are essential load bearing members of cable supported bridges, and they are commonly composed of parallel galvanized wires with high strength. These cables are exposed to corrosion, fatigue and abrasion, leading to a reduction in their section and resistance capacity (Mozos 2010a,b). Although the cables are designed with special protection and sufficient redundancy, the breakage of corroded cables cannot be completely avoided to multiple hazards (e.g., car accident, overloading, fire damage, and wind induced vibration). Some unforeseen cable-breakage accidents of suspension bridges have occurred in the past decades, which led to mass casualties. For instance, caused by wind induced distortion of the bridge deck, the Tacoma Narrows Bridge collapsed after the sudden breakage of hangers in 1940. Triggered by one broken hanger (Kawai 2014, Qiu 2014), the Kutai Kartanegara Suspension Bridge (Fig.1) suffered catastrophic progressive collapse in 2011. Due the seriously corroded main cable broke, the Myaung Mya Bridge collapsed in April 2018.

From the viewpoint of static analysis, the breakage of a hanger leads to overloading of adjacent hangers and reduce the stiffening girders' bracing against buckling.

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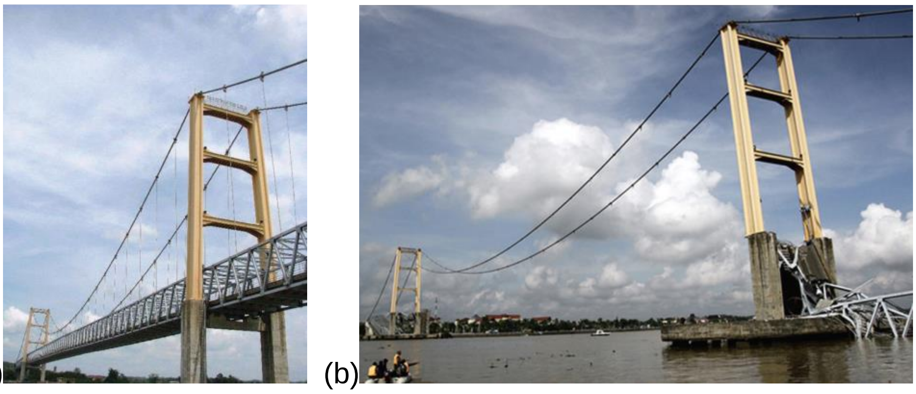


(a) (b)

Fig. 1 Collapse of the Kutai Kartanegara Bridge: (a) before collapse (b) after collapse

From the viewpoint of dynamic analysis, the hanger breakage-event is initiated from the instantaneous loss of structural member, which disturbs the initial equilibrium until the structure reaches a new equilibrium or progressive collapse (Marjanishvili 2006). As for the self-anchored suspension bridges, massive anchorage systems are no longer needed. Over the last 20 years, more than 60 self-anchored suspension bridges have been constructed in China (Li 2012, Xu 2016). The self-anchored suspension bridge represents an innovative bridge type in that the girders carry large horizontal compression (Ochsendorf 1999, Jung 2014). The girders of self-anchored suspension bridge are under axial compression and bending, any un-zipper tendency is likely to be enhanced by the susceptibility of the bridge girder to bucking. A mixture-type collapse can thus happen, like cable stayed bridges, in which the features of instability-type and zipper-type collapses interact and reinforce (Starossek 2010, Haberland 2012). However, the issues about progressive collapse caused by hanger-breakage events for bridge are briefly outlined compared with systematic research performed on building structures, (Shoghijavan 2017). Investigations on progressive collapse mechanism, collapse resistance assessment, and relevant codes are relative inadequate for bridges (Lu 2014, Das 2016,). Therefore, the focus on the hanger-breakage event deserves much attention.

In recent years, the hanger loss of self-anchored suspension bridges has been studied in Qiu (2014, 2015) and Shen (2014). To obtain a rational safety factor for the hangers, Shen (2014) conducted robustness analysis of other hangers with loss of a single hanger. The influence of different parameters on the dynamic responses of the self-anchored suspension bridge after the hanger-breakage event were studied (Qiu 2014). Existing studies suggest that the pattern of structural responses would be notably affected when the degradation on mechanical properties of hangers is considered (Pipinato 2012, Qiu 2015). In addition, dynamic analysis with consideration of both material nonlinearities and geometric nonlinearities are considerably important (Samali 2015, Shoghijavan 2017). However, little work is conducted to consider the effect of already corroded hangers on structural responses subjected to the hanger-breakage event, which highlight this research.

The objective of the paper is to assess nonlinear dynamic responses of a self-anchored suspension bridge after abrupt hanger loss, and the influence of corroded hanger is considered. The results can provide some suggestions in design of self-anchored suspension bridges.

2. CORROSION MODEL FOR HANGER

Modeling of corroded hangers is significant to the performances assessment of the self-anchored suspension bridge to the hanger-breakage event. To consider the mechanical behaviors of a corroded hanger, a modified series-parallel system is developed. The modified model is focused on mechanical properties of single corroded wire and corrosion distribution in a hanger.

2.1 Mechanical Properties of Corroded Wires

Fig. 2 shows the appearance of corroded cables including uniform corrosion and pitting corrosion. As shown in Fig. 2, the appearance is uniform corrosion when the corrosion is relative slight, and the appearance become pitting corrosion when the corrosion is serious.

(a)  (b) 

Fig. 2 Corrosive types of steel wire: (a) uniform corrosion (b) pitting corrosion

Fig. 3 shows force-strain curves of the typical corroded steel wires, while the diameter of the steel wires without corrosion is 5mm. As shown in Fig. 3, the constitutive laws of steel wires are affected by the corrosion extent. The basic mechanical parameters of the corroded wires can be obtained according to the fitting curves obtained from a great number of tension tests (Xu 2013). Associated with material nonlinearity, the constitutive laws for corroded wires can be expressed by Eq. (1).

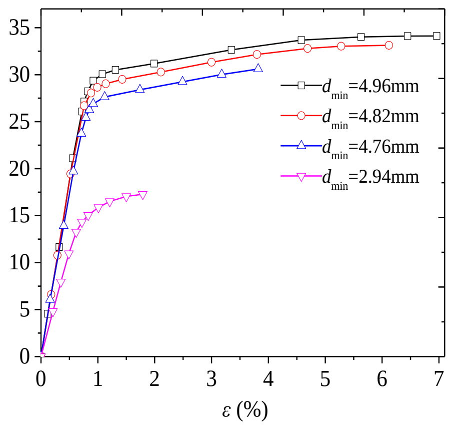


Fig. 3 Force-strain curves of one single wire

$$\begin{cases} F_w = F_y \dfrac{\varepsilon}{\varepsilon_y} & \text{if } \varepsilon \le \varepsilon_y \\ F_w = F_y + \dfrac{F_u - F_y}{\varepsilon_u - \varepsilon_y}(\varepsilon - \varepsilon_y) & \text{if } \varepsilon_y < \varepsilon \le \varepsilon_u \\ 0 & \text{if } \varepsilon > \varepsilon_u \end{cases}, \tag{1}$$

2.2 Corrosion Distribution in Hanger

Generally, the extent of corroded wires in a hanger varies from wire to wire along a cross section and in the longitudinal direction. Corrosion in a hanger is initiated by the penetration of corrosive media through the damaged sheath. For the free segments, determining the diffusion path of corrosive media is not simple because it could be transferred afterwards along the cross section to any place. To describe the corrosion distribution along the cross section, an assumption that steel wires in a cable from the non-corroded state to corroded state were in reverse proportion to the centroid of the cross section (Cremona 2003). Based on the inspection of the dismantled cables, (Xu 2013) proposed another assumption that steel wires started to corrode from the place where the sheath cracks until progressive dissolution of all wires. Fig. 4 shows the assumption, which is adopted in this study. To describe corrosion distribution along a cross section of a hanger, the corrosion diffusion ratio R_c is defined by Eq. (2)

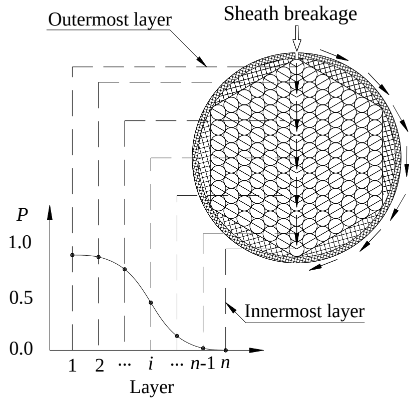


Fig. 4 Corrosion distribution in a cross section of a hanger

$$R_c = \frac{d_0 - d_{\min,i+1}}{d_0 - d_{\min,i}} = \frac{d_{e,i+1}}{d_{e,i}}, \tag{2}$$

where d_0 is the nominal diameter, $d_{\min,i}$ and $d_{e,i}$ are the minimum diameter and equivalent corrosion depth of the wire in ith layer. Investigations revealed that R_c varied from 0.39 to 0.89, and the average value was 0.48 (Xu 2013). It is feasible to obtain $d_{\min,i}$ and $d_{e,i}$ of all wires when the corrosion extent of steel wires within the same layer is further assumed to uniquely identify.

2.3 Modified Series-parallel System

Fig. 5 shows the modified model, where the independent wires in the model are divided into two parts. The length of the uncorroded part is L_u, and the length of corroded part is L_c with the same extent of corrosion. The extent of corrosion is not uniform either along the cross section of the hanger or in the longitudinal direction. As few data is available regarding corrosion distribution in the longitudinal direction, it is presumed that the total corrosion length L_c is α times of L, which is expressed by Eq. (3)

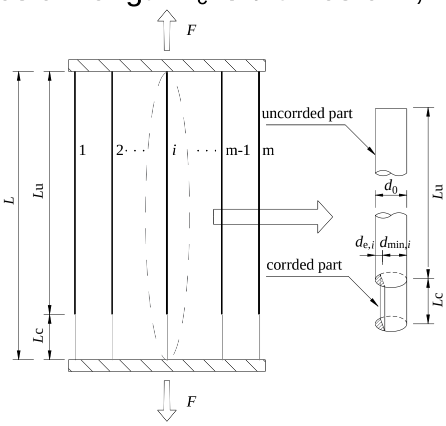


Fig. 5 Modified model of a hanger

$$L_c = \alpha L \,, \tag{3}$$

The ultimate bearing capacity of the wire is depended on the corroded part, which can be expressed by Eq. (4)

$$\max(F_{w,i}) = \max(F_{c,i}) \,, \tag{4}$$

where $F_{c,i}$ is the bearing capacity of the corroded part of the wire in the ith layer. When the bearing load of the wire is less than its ultimate capacity, the absolute elongation is expressed by Eq. (5)

$$\Delta L = L_{c,i}\varepsilon_{c,i}(F_{w,i}) + L_{u,i}\varepsilon_{u,i}(F_{w,i}) \,, \tag{5}$$

where $\varepsilon_{c,i}$ () and $\varepsilon_{u,i}$ () is the strain of the corroded and uncorroded segments of wire under the external load $F_{w,i}$, respectively.

The overall carrying capacity of the hanger gradually decreases when steel wires start to break, and finally the hanger breaks as the number of broken wires increases. The resultant bearing capacity of a hanger consisting of M layers can be obtained by Eq. (6)

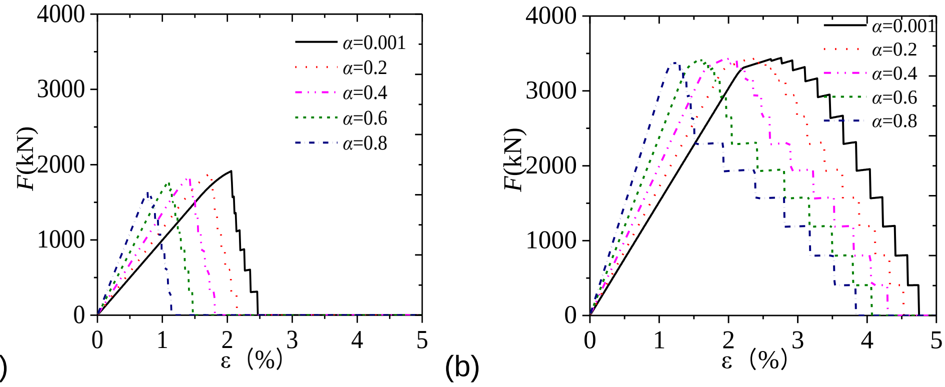
$$F = \sum_{i=1}^{M} N_i F_{w,i}(\Delta L) \,, \tag{6}$$

where N_i is the number of the wires in the ith layer, $F_{w,i}$ (ΔL) is the tensile force of wire under the elongation of ΔL. Once the extent of the worst corrosion in the outermost layer is provided, the mechanical parameters of corroded wires could be deduced. Thus, the relationship between the corrosion extent and the mechanical behavior of a hanger is established. It should note that, the modified model cannot reflect the uneven stress level of steel wires caused by friction and their roughness. Other respects, such as initial defects, stress concentration, residual strains and so on are not considered either in the model.

2.4 Mechanical Properties of Corroded Hanger

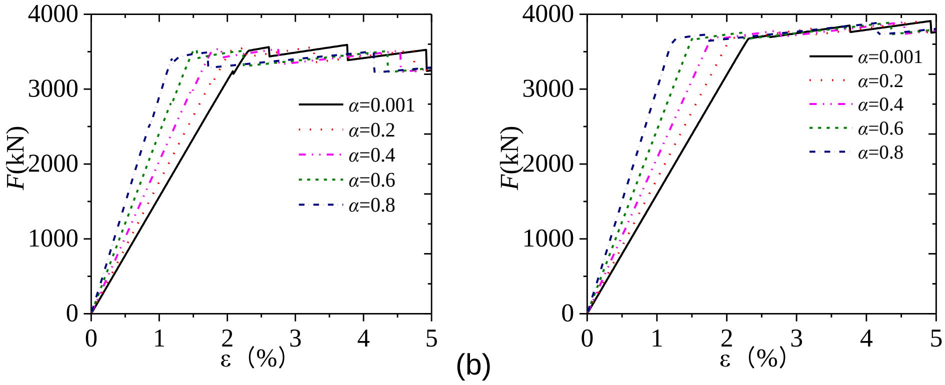
The proposed model of corroded hanger is concerned with three parameters: (1) the equivalent depth of the corroded wire $d_{e,1}$; (2) corrosion diffusion ratio along the cross section R_c; and (3) the corrosion length ration in the longitudinal direction α. Detailed discussions about these three parameters $d_{e,1}$, R_c and α are conducted with MATLAB programming language. A hanger consists of 127Φ5 mm wires is adopted to analyze the relation of corrosion extent and mechanical behaviors.

Fig. 6 to Fig. 7 show load-strain curves of the hanger under different conditions. The results suggest that force-strain curves of corroded hangers are different compared to force-strain curves of the corroded steel wires. When corrosion is slight (both $d_{e,1}$ and R_c are small), the yield strain of the hanger increases, and yield ration decreases with the growth of α as shown in Fig. 6(a). When corrosion is serious (both $d_{e,1}$ and R_c are large), the hanger lost its ductility, and tensile strain decreases with the growth of α as shown in Fig. 7(b).



(a) (b)

Fig. 6 Force-strain curves of the hanger when $d_{e,1}$ is 4mm (a) R_e=0.9 (c) R_e=0.5



(a) (b)

Fig. 7 Force-strain curves of the hanger when $d_{e,1}$ is 1mm (a) R_e=0.9 (b) R_e=0.5

3. Nonlinear Dynamic Simulation Methodology

3.1 Bridge System

The Zhuanghe Jiangshe Bridge is adopted as the prototype self-anchored bridge. Fig. 8 shows the general layout and details of the bridge. The bridge has a main span of 200 m and two side spans of 70 m. The stiffening girders have a concrete box section with transverse beams inside. There are 65 pairs of hangers, which are spaced with a span of 5 m, and the hanger is made of 127 parallel steel wires with a diameter of 5 mm.

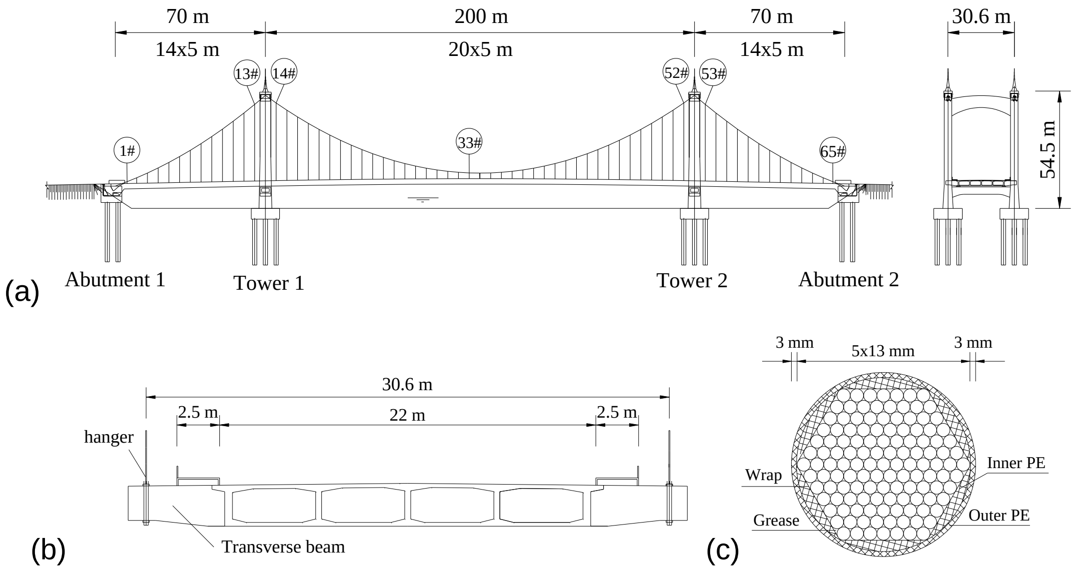


Fig. 8 Prototype self-anchored suspension bridge (a) general layout
(b) cross section of stiffening girder (c) cross section of hanger

3.2 FE model and framework of the hanger-breakage event

Fig. 9 shows the three-dimensional (3D) FE model, which is established using the software ABAQUS 6.14. In the FE model, the stiffening girders, transverse beams, and pylons are modelled by 3D beam elements. The main cables and hangers are modeled by the truss elements. To consider the vibration more precisely, the contribution of tension to their flexible stiffness is considered，the masses of clamps are concentrate to the nodes where they are located, and the main cable between two adjacent hangers is divided into 5 elements. The fundamental bending frequency of the self-anchored bridge is 0.41 Hz, and the first torsional frequency of the main girder coupled with the transversal beam is 1.00 Hz.

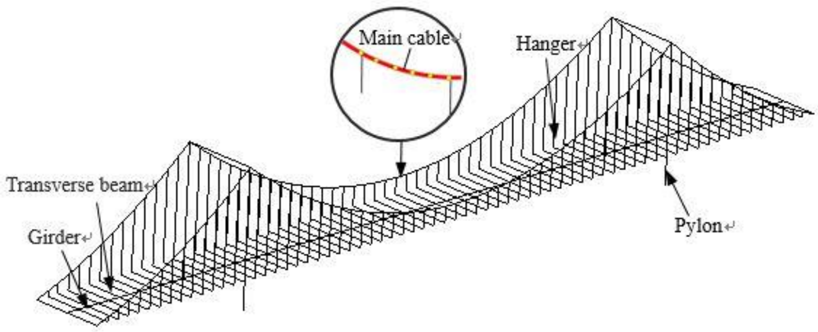


Fig. 9 FE model of the bridge

In the dynamic analysis of the hanger-breakage event, the key point of hanger-breakage event is to simulate hanger loss properly. Here, alternate load path (ALP) method is adopted to simulate the instantaneous hanger loss. Using the ALP method, the broken hanger is replaced by the equivalent initial tension force T_0, then the tension force T_0 is assumed to decrease to zero. Or keeping the broken hanger remaining on the bridge model, the hanger loss is realized through stiffness reduction. Fig. 10 shows the time history curve of the broken hanger, where the duration of the breakage process is Δt.

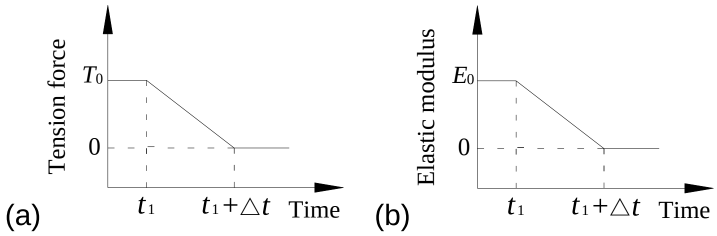


Fig. 10 Time history curve of the broken hanger

The procedures for the nonlinear dynamic simulation of hanger-breakage event are described by the following steps:

Step 1: Conduct nonlinear static analysis to obtain an equilibrium state. The analysis is performed on the self-anchored bridge with undamaged hangers.

Step 2: Conduct nonlinear static analysis to obtain the initial state. The analysis is performed considering the influence of corroded hangers.

Step 3: Conduct nonlinear dynamic analysis to simulate the hanger-breakage event through the elastic modulus reduction in Δt.

Step 4: Continue to perform nonlinear dynamic analysis after the hanger-breakage event to assess structural responses. until the bridge will reach the final equilibrium state.

3.3 Adopted assumption

To obtain the most representative scenarios of the hanger-breakage event, four specific aspects in the simulation are emphatically concerned including (1) the duration, (2) hanger loss scenarios, (3) loading scenarios and (4) nonlinearities.

Limited research about the exact value of Δt are available. It was believed that the maximum structural response can be obtained when Δt is less than 1/100th of structural fundamental period (Ruiz-Teran 2009). The value of Δt adopted 0.01 s in the dynamic response analysis of cable-stayed bridges (Zhou 2015). To obtain a relatively conservative assessment in current study, Δt takes a value of 0.005 s.

According to the recommendation (PTI 2012) and Chinese code, the abrupt loss of any one cable must not cause the collapse of the entire structure. Thus, a representative single hanger-breakage scenario is the considered. The loads considered for the potential progressive collapse assessment include dead loads and vehicular live load as specified in Chinese code. The dead load, as well as distributed vehicular live load is applied along the entire bridge, and the adopted load factors are as follows:

$$\text{Load} = 1.0\text{DC} + 1.0\text{DW} + 0.75\text{LL} + 1.0\text{CL}, \quad (7)$$

where DC is dead load of structural components, DW is dead load of surfacing asphalt and utilities, LL is full vehicular live load placed in stripped lanes, and CL is impact force caused by the sudden hanger loss.

In the linear elastic analysis, deformations are assumed to be small enough to ensure structural overall stiffness remains unchanged. Although actual responses should be acquired from the deformed structure, linear analysis is frequently used in bridge analysis. Nevertheless, in the analysis process of hanger-breakage event, nonlinear analysis is often inevitable for both static and dynamic analysis for the reasons of: (1) Large strains exceeding the elastic limit, and the material will be in the plastic range, (2) Geometric nonlinearity should not be neglected because cables are obviously affected by the cable sag effect, and (3) The interaction between the axial and flexural deformation in girders is influenced by large stress and deformation (Zhou 2015). The geometric nonlinearity, in terms of tension stiffening effects and large deformation effect, is inherently embedded of the FE formulation in the nonlinear time-domain analysis. The material nonlinearity is also inherently included in the nonlinear analysis in simulation by means of the designation of nonlinear material properties.

4. Results of structural responses

To study the responses of the bridge, one hanger numbered 23 is subjected to the hanger-breakage event. Considering corrosion distribution in a hanger is highly stochastic, Fig. 11 shows four typical load-strain curves of hangers. As shown in Fig. 11, case1 represents that the hanger is un-corroded, Case2 to case4 represents the extent of the hanger is slight, moderate and serious. In addition, the number of corroded hangers is divided into three conditions, condition 1 means two hangers adjacent to the broke hanger are corroded, condition 2 means four hangers adjacent to the broke hanger are corroded, condition 3 means all hangers in the broken hanger's side are corroded.

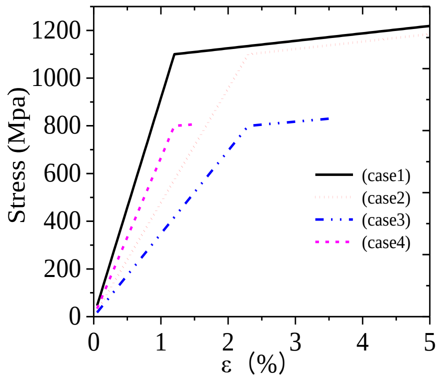


Fig. 11 Load-strain curves of the hanger

4.1 Responses of hangers

Fig. 12 shows the tensile stresses of hangers near the broken hanger. It can be seen from the results that the breakage of one hanger has an obvious effect on responses of the hangers adjacent to the broken hanger, and the event has little effects on tensions of the hangers far away from the broken hanger. The maximum tensile stress of the hangers is 853 Mpa for case1, which is 2.12 times of the initial value.

Fig. 13 shows the maximum tensile stress of the hanger under different situations. We can see that both corrosion extent and the number of corroded hangers has very large effects on the maximum response of adjacent hangers. When the effect of corrosion is considered, the maximum responses of the hanger dcrease, but the risk of progressive collapse of adjacent corroded hangers is increasing due to the variation of load-strain curves. The maximum responses of the hanger increase from case2 to case4, and the responses of the adjacented hanger are the largest under condition 2. Therefore, If the adjacent hangers are moderately corroded, the adjacent hanger may yield during the vibration. If the adjacent hangers are seriously corroded, the adjacent hanger may break following the broken hanger.

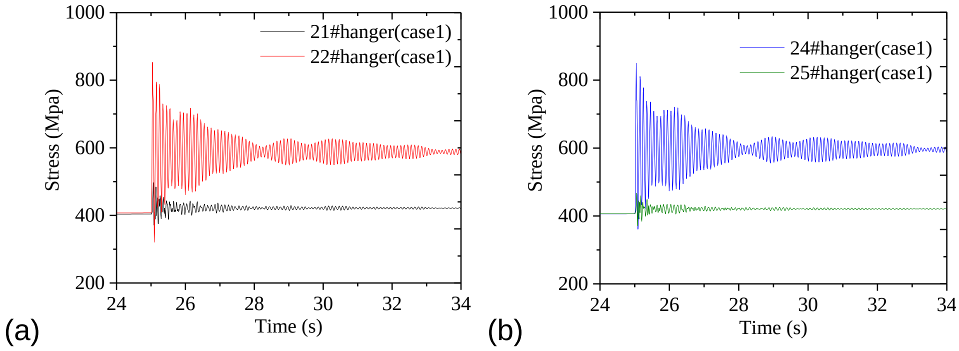


Fig. 12 hangers responses to the breakage of one hanger

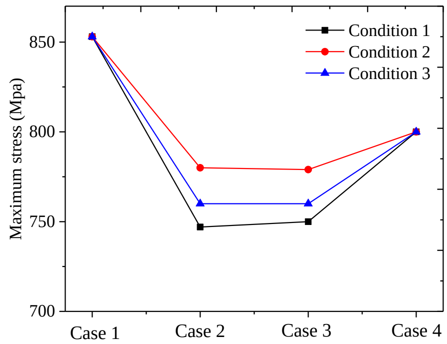


Fig. 13 Maximum tensile stress of the hanger numbered 22

4.2 Responses of the girders

Fig. 14 shows the dynamic responses of the girder in the section closed to the broken hanger and near the pylon. The results suggest that breakage of one hanger causes large responses of the girder, not only in the section near the broken hanger, but also in the section far away from the broken hanger. The maximum torsion moment of the girder in the section closed to the broken hanger is 19329 kN.m, and the maximum bending moment is 13070 kN.m, which is 2.06 times of the initial value. The maximum torsion moment of the girder in the section closed to the pylon is 15676 kN.m, and the maximum bending moment is 17851 kN.m.

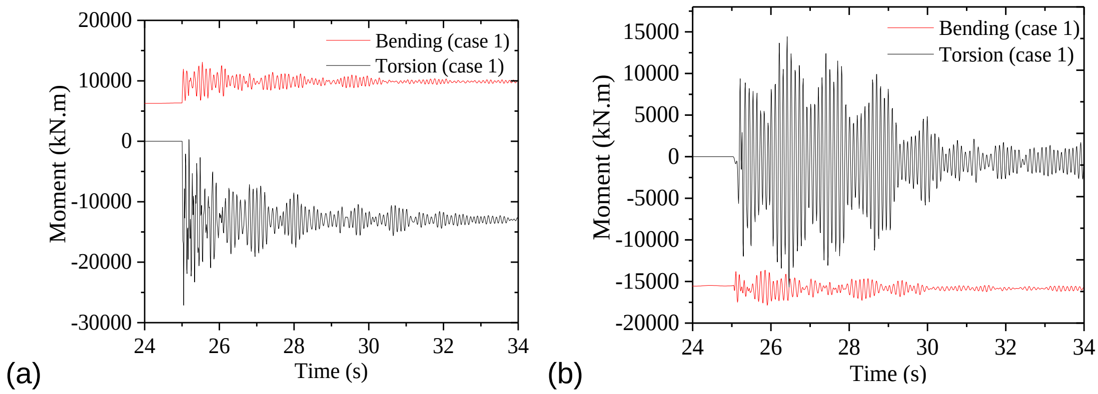


Fig. 14 Responses of the girder (a) near the broken hanger (b)near the tower

Fig. 15 shows the maximum responses of the girder under different situations. We can see that both the corrosion extent and the number of corroded hangers affect maximum responses of the girder. When four hangers adjacent to the broke hanger are moderately corroded under condition 2, the bending responses of the girder are the largest. When all hangers in the broken hanger's side are seriously corroded under condition 3, the bending responses of the girder are the largest.

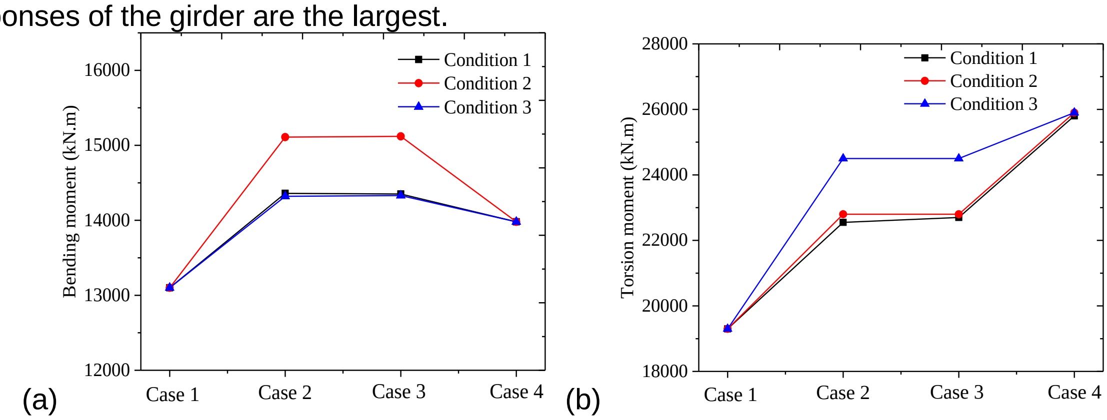


Fig. 15 Maximum responses of the girder (a) near the broken hanger (b) near the tower

5. Conclusion

Structural responses of a self-anchored suspension bridge with corroded hangers to sudden hanger loss are studied in this study, and the following conclusions are drawn:

(1) The force-strain curves of corroded hangers are more complicated and markedly different to single corroded steel wires. When corrosion is slight, the stiffness corroded hanger decreases. When corrosion is serious, the stiffness corroded hanger restored, but the hanger would lose its ductility and bearing capacity.

(2) The hanger-breakage event causes very large dynamic responses of the hangers adjacent to the broken hanger. If the adjacent hangers are corroded, they may break after the hanger-breakage event. If the adjacent hangers are not corroded, they may yield during the vibration. The event causes very large dynamic responses of the girder not only in the section near to the broken hanger, but also in the section near the tower.

(3) Both the corrosion extent and the number of corroded hangers affect structural responses. Structural responses would be exaggerated and accelerated due to the coupling effect of corrosion after a hanger-breakage event, so dynamic analysis of hanger- breakage event should consider the influence of corroded hangers.

(4) The corroded hanger should be replaced in time to improve structural robustness and avoid progressive collapse after the hanger-breakage event.

Acknowledgements

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