Nonlinear dynamic responses of suspension bridge to sudden breakage of hangers

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Abstract. It is found that the hangers in many suspension bridges in China faced serious corrosion. The resistance capacities of the hangers decrease for corrosion, and the corroded hangers may break suddenly under tension forces. The sudden breakage of a hanger can induce violent vibrations and large changes of internal forces of the whole bridge. The safety of suspension bridges is endangered by corrosion of hangers. Using nonlinear dynamic analysis method and adopting 3D finite element model, the nonlinear dynamic responses of an actual suspension bridge to sudden breakage of a hanger are studied in this paper. The influences of sudden breakage of hanger on the bridge are analyzed in detail. Based on the analysis results, some useful suggestions about design and maintain of hangers are proposed.

1. Introduction
In recent years, many accidents of breakage of cables, such stay cables in cable-stayed bridges and hangers in arch bridges, have happened in the world for the reasons of corrosion, fatigue or traffic accident. In China, serious corrosion and breakage of cables occurred in many bridges[1]. On September 7th in 2002, one stay cable of Guangzhou Haiyin Bridge that was only used for 7 years broke, and it hit an oil tank truck. On April 12th in 2011, two hangers of Xinjiang Kongque Bridge broke, and the deck supported by these hangers fell in the river. On November 11th in 2011, eight hangers of Yibin Southgate Bridge in China broke, and the deck fell in the river and there two men died in this accident.

Because the breakage of cable usually occurs suddenly, and it can cause strong vibration and large change of internal forces of the structure, the sudden breakage may endanger the safety of the bridge. Because one hanger is broken, Mahakam II suspension Bridge in Indonesia collapsed in 2011. It happened very fast, only about 30 seconds. This bridge was completed in 2002, and it was less than 10 years old. At least 11 people are dead and 30 people are missing in this accident.

The effects of breakage of stay cables on cable-stayed bridge were studied by many researchers. Considering the influences of different layout of stays, number of planes of stays, and stiffness of deck, Mozos and Aparicio studied the effects of breakage of stays on deck, tower and stays in detail[2][3]. The study showed that sudden breakage of hanger controls the design of deck and tower. Because the time that the breakage occurs (breakage time) has significant effects on the responses of structure, Mozos and Aparicio studied the breakage time through experiments and found that the breakage time was 0.00375s for damaged cables, and 0.0085s for undamaged cables[4].
Considering the geometric and material nonlinearity, Cai et al. studied the nonlinear responses and progressive collapses of cable-stayed bridge due to sudden breakage of stays[5]. When a single stay broke, the tension of other stays changed little. When two adjacent stays broke simultaneously, the bridge collapse did not occur, but the deck had large plastic deformation and the stays yielded.

Ruiz-Teran and Aparicio studied the effects of breakage of stays on under-deck cable-stayed bridge[6]. The study results showed that the responses produced by breakage of stays must be calculated using dynamic analysis method.

Using nonlinear dynamic analysis methods and considering the corrosion of hangers, the responses of a suspension bridge with main span of 200m due to the breakage of hangers are studied in this paper.

2. Structure of the suspension bridge and analysis method

2.1. Structure of the suspension bridge

A concrete self-anchored suspension bridge with main span of 200m and side span of 70m, as shown in figure 1, is used to study the structural responses caused by sudden breakage of hanger. Its stiffening girder is reinforced concrete box girder, as shown in figure 2. The tower is reinforced concrete with box section. The bridge has two main cables and each cable is made of 3937 paralleled, 5mm in diameter, high strength galvanized steel wires. There are 65 hangers on each side of the bridge, and the distance of the hangers along the girder is 5m. The hangers are made of 97 paralleled, 7mm in diameter, steel wires with strength of 1.67×10^6 kPa. The hangers are numbered from #1 to #65.

2.2. Analysis model

To modulate the nonlinear vibration process caused by the sudden breakage of hanger, the software ABAQUS V.6.8 is used to establish a three dimension (3D) finite element (FE) model of the bridge, as shown in figure 3. In the FE model, the girder, towers and transverse beams are modelled using 3D beam elements, and their torsion stiffness and torsion mass are considered. The main cables are also modelled using beam elements with very small flexible stiffness, and contribution of the tension to the
flexible stiffness of the cables is considered. The hangers are modelled using truss elements. The main
cables between two hangers are modelled using 5 elements to consider geometry and vibration of the
main cables more precisely. The masses of clamps are concentrated on the nodes where the clamps are
located. Considering the piles of the bridge had little effects on the analysis of sudden breakage of
hanger, they are not modelled in the FE model.

\[ \text{Figure 3. FE model of the bridge.} \]

2.3. Nonlinear behavior of corroded hanger

To modulate the effects of corrosion of steel wires on strength of hanger, the model of corroded
hanger are established as shown in figure 4. The cross section area of the hanger is \( A \), and the length of
the hanger is \( L \). The steel wires of the hanger are divided into two parts named as part A and part B.
The wires in part A are not corroded, but the wires in part B are corroded. The cross section areas of
the two parts are \((1-\alpha)A\) and \(\alpha A\) respectively. To simplify the analysis, it is assumed that the corrosions
of every wire in part B are same. The length of corroded part (part B-2) of every wire is \( L_c \), and the
rest area of corroded part of every wire is \( \beta \) times of the original area. To consider the material
nonlinearity of hanger, the nonlinear relationship between stress and strain of steel wire is adopted in
the analysis, as shown in figure 5. The steel wires break when their strains reach 0.04.

\[ \text{Figure 4. Model of corroded hanger.} \]

\[ \text{Figure 5. Relationship between stress and strain of steel wire.} \]

2.4. Analysis method

Responses of the bridge due to sudden breakage of hangers on the bridge are analyzed by means of
nonlinear dynamic analysis using the finite element software ABAQUS V.6.8. In dynamic analysis,
the direct time integration method is used. First, the static state of the bridge under design load is
determined, then the broken hanger is removed suddenly and the tension of the broken hanger is
unloaded in breakage time \( \Delta t \). The breakage time \( \Delta t \) takes a value of 0.005s. A Rayleigh damping of
2% is used in the dynamic analysis.
3. Analysis results

In order to study the responses of the bridge caused by breakage of hanger, breakage of the left hanger #23 at the position of 1/4 of mid-span is taken as an example. The study includes nonlinear analysis and linear analysis. The nonlinear analysis considers both geometric nonlinearity and nonlinear behavior of corroded hanger, but the linear analysis considers only geometric nonlinearity. In the nonlinear analysis, for the hangers except the broken hanger, the values of $\alpha$, $\beta$ and $L_c$ are taken as 0.5, 0.7 and 0.1m. The stresses of the hangers are about $5.56 \times 10^5$ kPa under design loads before hanger #23 breaks in the two analysis cases. In the dynamic analysis, the hanger #23 broke at the time of 25s.

3.1. Responses of hangers

From the analysis results, it is can be seen that the breakage of a hanger has large effects on stresses of the hangers near the broken hanger, and has little effects on stresses of the hangers far away from the broken hanger. When the corrosion is considered, the stress of part B-2 in hanger #22 is $7.94 \times 10^5$ kPa before hanger #23 breaks, which is larger than the stresses of part B-1 and part A, as shown in figure 6. After hanger #23 breaks, the stress of part B-2 in hanger #22 reaches $1.55 \times 10^6$ kPa rapidly, and the steel wires in part B-2 break when their strains reach 0.04. The steel wires in part B withdraw from the structure, and the steel wires in part A bear the tension of hanger #22 alone. In the vibration, the stresses of steel wires in part A exceed the yield stress of $1.41 \times 10^6$ kPa, but they are less than the ultimate stress of $1.55 \times 10^6$ kPa.

![Figure 6. Stresses of the 3 parts in hanger #22.](image)

![Figure 7. Stresses of part A in hanger #22.](image)

![Figure 8. Stresses of part A in hanger #21](image)

![Figure 9. Torsion moments of main girder near tower T1.](image)
Figure 7 and figure 8 show the time histories of stresses of part A in hangers #21 and #22. It can be seen from the figures that stresses of hangers #21 and #22 change very largely after breakage of hanger #23, and the stresses of hanger #22 vary more markedly than those of hanger #21. When the corrosion is considered, the maximum stresses of hanger #22 and #21 are $1.42 \times 10^6$ kPa and $9.48 \times 10^5$ kPa respectively. When the corrosion is not considered, the maximum stresses of hanger #22 and #21 are $1.11 \times 10^6$ kPa and $7.27 \times 10^5$ kPa respectively. So, the corrosion has very large effects on the stresses of the hangers.

3.2. Responses of main girder
Figure 9, figure 10 and figure 11 show the torsion moment of main girder near tower T1, vertical bending moments of main girder near hanger #23 and vertical displacement of main girder near hanger #23 caused by breakage of hanger #23. When the corrosion is considered, the maximum torsion moment of main girder is $21014$ kN.m, the maximum vertical bending moment of main girder is $1365$ kN.m, and the maximum vertical displacement of main girder is $0.011$ m. When the corrosion is not considered, the maximum torsion moment, maximum vertical bending moment and the maximum vertical displacement of main girder are $15038$ kN.m, $2282$ kN.m and $0.006$ m respectively. So, the torsion moments of main girder caused breakage of a hanger are very large, and the vertical bending moment and displacement of main girder are small relatively. The maximum torsion moment and displacement become larger, and vertical bending moment become less when the material nonlinearity of hanger is considered in the dynamic analysis.

3.3. Responses of main cable
Figure 12 shows the time histories of stresses of main cable after breakage of hanger #23. The maximum stress of main cable is only larger than the initial value by $1.13 \times 10^4$ kPa, and the results obtained by the nonlinear and linear analysis are same nearly. Figure 14 and figure 15 show the time histories of displacement of main cable at the positions of the hangers #22 and #23. The displacements of main cable obtained by nonlinear analysis are very different from those obtained by linear analysis. Especially, when the vibration ceases, the displacement of main cable at the position hanger #22 obtained by nonlinear analysis is $0.072$ m, which is much larger than the value of $0.009$ m obtained by linear analysis. This is because hangers #22 and #24 yields in the nonlinear analysis. The maximum displacement of main cable at the position of hanger #23 is $0.167$ m, which shows that the amplitude of the vibration is very large. So, the breakage of a hanger causes little change of stress and large displacement of main cable.
3.4. Responses of tower

Figure 15 shows the time histories of moment of the bottom of tower T1 after breakage of hanger #23. The maximum moment caused by breakage of hanger is 4041kN.m. Figure 16 shows the time history
of displacement of the top of tower T1 after breakage of hanger #23. The maximum horizontal displacement caused by breakage of hanger is only 0.0019m. The moments and displacement obtained by the nonlinear are larger than those obtained by linear analysis, but the values are all very small.

3.5. Responses of bearing
Figure 17 shows the time histories of reaction forces of one bearing at tower T1 after breakage of hangers #23. The maximum compression force subjected by the bearings is 3875kN, which is 1.59 times of the initial value. The maximum compression forces obtained by the nonlinear and linear analysis are same nearly.

4. Conclusions
The following conclusions can be obtained through analyzing the responses of a suspension bridge with corroded hanger caused by sudden breakage of a hanger.

(1) The breakage of a hanger causes very large stresses of the hangers near to the broken hanger. If the adjacent hangers are corroded, the corroded part of steel wires in the hangers may break, and the part of steel wires without corrosion may yield during the vibration caused by the breakage of hanger.

(2) Through comparison of the results obtained by nonlinear analysis and linear analysis, when corrosion of hangers are considered, the changes of hanger stresses, torsion moments of main girder, displacement of main cable and displacement of tower top are much larger.

(3) The corrosion of hangers has marked effects on the responses of the bridge caused by breakage of hanger, so the study on the effects of hanger breakage should consider the corrosion of hanger. The corroded hanger should be replaced in time to avoid serious damage of the bridge.

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