Cable-Stay Bridges—Investigation of Cable Rupture

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Abstract: As the bridge engineering community sets sails to use longer and longer spans, more and more sophisticated analysis models have to be used in the design process. One of the significant problems represents cable rupture of cable stays. The problem is also addressed in guidelines for cable-stayed bridge design such as PTI (Post-tensioning Institute) Recommendations and EC3 by quasi-static analyses using DAF (dynamic amplification factors) to account dynamic effects, which can be conducted instead of using dynamic analysis. The results show that the value DAF depends on the cable rupture location and on the type and location of the examined state. Dynamic time history analysis is recommended. Some projects examples are highlighted in the paper, where the importance of above mentioned topic has been investigated, following different regulations and approaches. Professional bridge analysis and design software solution RM Bridge has been used for all investigations. The application can fulfill all requirements and deliver expected and accurate results. In addition, RM Bridge software also helps engineers as a tool to optimize structure design and increase resistance capacity for each element to ensure the structural safety in service stage.

Key words: Cable-stayed bridge, dynamic non-linear analysis, cable rupture.

1. Introduction

Failure of overloaded structural elements can lead to the collapse of the main bridge structure. Some examples of such events happened in last few years in Asia: Kutai Kartanegara suspension bridge in Indonesia, 2011 (Fig. 1); Chhinchu suspension bridge in China, 2007, due to overloading and broken cable; Krong Bong suspension bridge in Dak Lak, 2013.

In 2010, Binh Bridge has been seriously damaged by ship crash. In the same year, Vam Sat Bridge in Hochiminh city, Vietnam has been overloaded by live load, what resulted in stay-cable rupture.

Process and reasons of collapses have been often discussed and studied in recent years, although mainly buildings have been investigated. Few bridge investigations about increasing bridge collapses since year 2000 have been reported. Higher interest worldwide for that problem, especially in Asia and Vietnam, has been shown.

Investigation of cable rupture load has been

published in many research studies [1-4].

In most of the current commercial bridge software, the cable rupture load needs to be modelled manually. This results in slow, complicated calculation and comparison process. For our research, several software applications have been tested and finally RM Bridge [5] has been chosen. Very advanced and accurate solver allows investigation of any critical dynamic event for large span cable-stayed and suspension bridges. 4D application is based on full 3D model of the bridge. Non-linear dynamic analysis can consider any number and any location of stay-cable ruptures.

2. Basic Conditions

2.1 Modeling of Dynamic Cable Rupture

Cable rupture load is assumed to follow three models in Fig. 2. In Model (a), it is assumed that cable rupture happens at $t_p = 0$ (s) (sudden break), in Models (b) and (c), it is assumed, that cable rupture linear or non-linear depends on time t_p according to research results of Yukari Aoki [6], the value of t_p is in range between 1 ms to 10 ms. Research results of Wolff and

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Fig. 1 Collapse of Kutai Kartanegara Bridge.

Fig. 2 Calculation models of cable rupture dynamic load.

Fig. 3 Dynamic load in case of sudden cable rupture.

Fig. 4 Analytical one DOF (degree of freedom) system model of dynamic load for sudden cable rupture.

starossek [7] showed the worst situation, if sudden cable rupture is caused (Model (a)).

2.1.1 Model of One Mass System

In case of one mass system and sudden cable rupture, we prepare models (Figs. 3 and 4). Cable is connected with simple beam (length “ L ”, mass “ m ” and stiffness “ EI ”) at mid-span. Considering sudden

cable rupture, dynamic load F_{dyn} on beam at mid-span is applied. Analytical model of the problem is shown in Fig. 5:

Equation of equilibrium:

$$F_I + F_D + F_S + W = F_{dyn} + F_0 \quad (1)$$

where, F_I = inertia force, F_D = damping force, F_S = elastic force, W = self weight of structure, F_0 = static force of cable rupture, F_{dyn} = dynamic force of cable rupture.

In case of cable rupture, equation of equilibrium is as below:

$$F_S + F_I + F_S = F_{dyn}$$

$$\text{Or: } m\ddot{w}(x,t) + c\dot{w}(x,t) + k w(x,t) = p(x,t) \quad (2)$$

With zero damped system ($c = 0$):

$$m\ddot{w}(x,t) + k w(x,t) = p(x,t) \quad (3)$$

Applying Hamilton rules to establish equation of motion:

$$w(x,t) = \varphi(x)z(t) \quad (4)$$

where, $\varphi(x)$ represents shape fuction and $z(t)$ represents

Fig. 5 Analytical model of load, caused by sudden cable rupture.

Fig. 6 Analytical model of multi DOF structure, loaded by stay-cable rupture.

generalized coordinate system.

Bending moment:

$$M_y(x, t) = M_{y,stat}(x)(1 - \cos \tilde{\omega}_n t) \quad (0 \leq x \leq L/2) \quad (5)$$

Shear force:

$$Q_z(x, t) = Q_{z,stat}(x)(1 - \cos \tilde{\omega}_n t) \quad (0 \leq x \leq L/2) \quad (6)$$

where, $M_y(x, t)$: dynamic bending moment, $M_{y,stat}(x)$: static moment, $Q_z(x, t)$: dynamic shear force, $Q_{z,stat}(x)$: static shear force.

2.1.2 Multi-degrees of Freedom System

Analytical model with multi DOF is shown in Fig. 6.

Equation of equilibrium at node “i”:

$$f_{li} + f_{Di} + f_{Si} = p_i(t), \quad i = 1, 2, 3, \dots, n \quad (7)$$

where, $p_i(t)$ = dynamic load; f_{li} = inertial force; f_{Di} = damping force; f_{Si} = elastic force.

In matrix form:

$$[f_I] + [f_D] + [f_S] = [p(t)] \quad (8)$$

In differential equation of motion:

$$[M][\ddot{\omega}] + [C][\dot{\omega}] + [K][\omega] = [p(t)] \quad (9)$$

where, $[M]$ = mass matrix; $[C]$ = damping matrix; $[K]$ = stiffness matrix.

2.2 Dynamic Amplification Factor DAF of Cable Rupture

Research goal was evaluation of difference between two options: cable rupture is applied as sudden load or

time dependent dynamic load. For this purpose, we decided to research DAF (dynamic amplification factor). DAF is defined as ratio of the most detrimental effect of the cable rupture load. DAF for one degree of freedom system can be mathematical defined as per Eq. (10):

$$DAF = \frac{\max z(x, t)}{z_{stat}(x)} = \frac{\max \omega(x, t)}{\omega_{stat}(x)} = \frac{\max M_y(x, t)}{M_{y,stat}(x)} = \frac{\max Q_z(x, t)}{Q_{z,stat}(x)} = \max |1 - \cos \tilde{\omega}_n t| \Rightarrow DAF_{max} = 2.0 \quad (10)$$

For one DOF model, loaded by cable rupture and calculated by RM Bridge DAF was evaluated as $1.9979 \approx 2.0$.

For multi DOF system, the dynamic factor of cable rupture load in the i th element (DAF_i) is calculated as follows:

$$DAF_i = \frac{z_{dyn,i}}{z_{stat,i}} = \frac{M_{yi,dyn}}{M_{yi,stat}} = \frac{Q_{zi,dyn}}{Q_{zi,stat}} \quad (11)$$

According to Eq. (2), the internal forces due to cable rupture load depend on the structure stiffness mass distribution, dynamic load effects (magnitude of dynamic loads) and structural damping coefficient (damping matrix).

2.3 Researched Parameters, Affecting DAF

2.3.1 Stay-Cable Angle

The analysis results showed that DAF does not

change significantly by increasing the cable-stay angle connection to main girder (Fig. 7).

2.3.2 Stay-Cable Tensioning Force

Relationship between initial stay-cable tensioning force and *DAF* is shown in Fig. 8.

2.3.3 Effect of Structure Damping

Damping force $[C]$ can be expressed as relationship between mass $[M]$ and stiffness of the structure $[K]$ (15):

$$[C] = \alpha [M] + \beta [K] \quad (15)$$

Considering structural damping variation, expected results are visible in Fig. 9.

Higher structural damping causes lower *DAF*. Damping parameters are visible in Table 1.

2.3.4 Stay-Cable Location

DAFs of main girder in dependency of cable position as well as static response dynamic cable rupture of any stay-cable have been investigated. Using 3D model in Fig. 10 and results of non-linear dynamic calculation due to cable rupture in Fig. 11,

Fig. 7 Relationship between the cable rupture force and *DAF*.

Fig. 8 Relationship between initial tensioning force and *DAF*.

Fig. 9 Relationship between structures' damping coefficient and *DAF*.

Table 1 Damping coefficient and damping parameters.

ζ (%)	ω_1	ω_2	α	β
0.20	10.716	18.522	0.0272	0.0001
0.40	10.716	18.522	0.0543	0.0003
0.60	10.716	18.522	0.0815	0.0004
0.80	10.716	18.522	0.1086	0.0005
1.00	10.716	18.522	0.1358	0.0007

dynamic amplification factor varies for different stay-cable locations as shown in Fig. 12.

3. Cable Rupture Influences Distribution of Internal Forces in Cable-Stayed Bridge

DAF depends on the cable rupture position and

structural damping coefficient. PTI (Post-tensioning Institute) Recommendations [8] and EC3 [6] have been checked by some case studies of cable-stayed bridges, calculated by RM Bridge. In the research process, some important basic conditions have been set:

- cable-stayed bridge has been considered as 3D model;
 - cable rupture has been applied as dynamic load;
 - non-linear dynamic analysis has been activated;
- Main girder, stay-cables and pylons have been investigated.

Three cases studies have been prepared:

Fig. 10 Cable-stayed bridge model with non-linear dynamic calculation. **Fig. 11 Non-linear dynamic calculation due to cable rupture.**

Fig. 12 *DAF* factor—any stay-cable rupture.

Fig. 13 Model of Cao Lanh cable-stayed.

Fig. 14 Bending moment envelope diagram for main girder—cable rupture of any stay-cable—stayed.

(1) Case 1: Effect of cable rupture for Cao Lanh cable-stayed bridge—with two pylons, two stay-cable planes and spans 150 m + 350 m + 150 m, main girder width 24.5 m; 128 stay-cables; 120 m high H-shaped pylon (Figs. 13 and 14). *DAF* is lower than 2.0 for main girder bending moment (Fig. 15)

and higher than 2.0 for pylon bending moment (Fig. 16);

(2) Case 2: Effect of cable rupture for Bai Chay cable-stayed bridge—with two pylons, one stay-cable plane, span arrangement (35 + 86 + 129.5 + 435 + 129.5 + 86) m, main girder width 25.7 m, 112

stay-cables and 141 m high pylon. *DAF* of main girder elements is lower than 2.0 for negative moment and higher than 2.0 for positive moment (Fig. 17). *DAF* of pylon elements is higher than 2.0 for positive moment (Fig. 18);

(3) Case 3: Effect of cable rupture for Tran Thi Ly

cable-stayed bridge—with one pylon, three stay-cable planes, span arrangement $(4 \times 50 + 230 + 45 + 4 \times 50 + 30 + 26)$ m, main girder width 36.5 m; 63 stay-cables, 127 m high pylon.

DAF of main girder elements as well as of pylon elements is higher than 2.0 (Figs. 19 and 20).

Fig. 15 *DAF*—main girder M_z (Case 1).

Fig. 16 *DAF*—pylon M_z (Case 1).

Fig. 17 *DAF*—main girder M_z (Case 2).

Fig. 18 *DAF*—pylon M_z (Case 2).

Fig. 19 *DAF*—main girder M_z (Case 3).

Fig. 20 *DAF*—pylon M_z (Case 3).

4. Prevent Local Failure of Critical Elements in Design Stage

There exist various strategies to help preventing disproportionate failure. In comparison to building structures, bridges are primarily horizontally aligned structures with one main axis of extension. Current

strategies towards increasing the robustness of bridge structures can be divided into the following categories [6]:

- Prevent local failure of critical elements: control local resistance and protective measures;
- Assume local failure: alternative load paths and isolation by segmentation;

Fig. 21 Nga ba Hue Overpass Bridge in April, 2015.

Fig. 22 Model of Nga ba Hue Overpass Bridge structure—modelled/analyzed/optimized by RM Bridge software.

Fig. 23 Bending moment capacity of main girder.

Note: Case 1: bending moment capacity of main girder in initial design stage;

Case 2: bending moment capacity of main girder after adjusted tendon;

Cable loss: dynamic cable loss combination.

- Descriptive design rules.

Increasing the local resistance of critical elements within the structural bridge system is quite

straightforward and can be described directly in design requirements. If increased resistance is uneconomical or not possible, non-structural

protective measures can be used with physical barriers, surveillance systems, etc.

In RM Bridge, engineers can optimize design of each element by using reinforcement, tendons and initial cable force effectively. This method has been applied at Nga ba Hue Overpass Bridge in Danang city, Vietnam—with one pylon, two stay-cable planes, span arrangement (90 + 90) m (Figs. 21 and 22). According to the initial design, the positive moment capacity due to dynamic cable loss combination was not sufficient (Fig. 23, Case 1). Design is adjusted by increasing the number of tendons and on this way satisfying design requirements (Fig. 23, Case 2).

5. Conclusions

Bridge engineers are mostly interested in maximum impact of cable rupture to the bridge structure. Cable-stayed bridge design guidelines suggest a semi-static approach for consideration of dynamic impact. PTI (2007) suggestion for *DAF* is 2.0. EC3 (2006) suggestion for *DAF* is 1.5. In addition, PTI (2007) allows identification of additional factors in non-linear dynamic analysis to ensure economical-technical design.

Using RM Bridge software has shown that *DAF* coefficients for all elements can be evaluated, using non-linear dynamic analysis with time history functions. Results show that *DAF* coefficients for cable-stayed bridges cannot be specified without accurate calculation. Their value depends on location of stay-cable rupture, structural damping, stiffness of structural elements and type of cable-stayed bridge with one, two or more pylons. *DAF* coefficient can overstep value 2.0, especially in pylon elements. All *DAF* values should be based on non-linear dynamic calculation, where cable rupture is treated as time dependent load, using time history function. For

evaluation of those factors, professional bridge software should be used.

Additionally, RM Bridge solver enables optimization of the bridge structural elements in order to increase the bearing capacity of those and help engineers for better visual perspective on the design structural safety.

Acknowledgments

The authors thank the Ministry of Transport (MOT) Vietnam for being given the possibility to present some cable-stayed bridges in Vietnam within this paper.

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