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# EVALUATION AND MONITORING OF CABLE LOSS IN CABLE STAYED BRIDGES

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## Abstract

Loss of cable constitutes an extreme load case for cable-stayed bridge. The rupture of cable induces an impulsive load to the deck and the pylon, which results in dynamic response in the overall bridge system. In spite of the influence of the cable loss verification in the design of cable-supported bridges, poor literature and research have been devoted to this topic, and design recommendations are providing simplified methods replacing the dynamic effect by factored static load. Accordingly, this paper investigates the dynamic response in case of cable loss with focus on dynamic amplification through case studies on an actual cable-stayed bridge. Time history analyses are performed for various cable loss scenarios considering two patterns: instantaneous loss of cable and progressive loss of cable. Progressive loss of cable which corresponds to a more realistic cable rupture pattern is simulated by decreasing the tension to zero in a short period. Dynamic deflections are compared to static deflections to quantify a dynamic amplification factor (DAF) for pylons, cables, and points on the deck. This paper intends to derive proper DAF for a cable-stayed bridge structure through adequate simulation of the cable loss that is sudden or a progressive rupture, as well as the use of appropriate models in the time of history analysis for the evaluation of the dynamic response of the cable-stayed bridge. Focus is done in providing features and guidelines that will help the designer to derive rational DAF during the assessment of the cable loss or rupture in cable-stayed bridges. In addition, monitoring of the state of the cables in cable-stayed bridge is addressed and solution to monitor optimally the whole set of cables by means of representative measurands is also discussed.

## INTRODUCTION

The consideration of the eventual occurrence of extreme events is critical during the design to achieve safer and reliable bridge in a lifetime perspective. Major codes are considering such extreme events, which include ship collision, car accidents, fire, loss or change of structural elements, and are prescribing evaluation of the corresponding structural behaviours. Among these events, the loss of cable constitutes a particular feature of cable-supported bridges. In spite of their structural importance, the cables of cable supported bridges may be subject to events that can cause their rupture such as a car accident, fire or seismic event like the Ji-lu Bridge in Taiwan in 1999 [1]. Several scenarios of cable rupture can be drawn that are sudden or a gradual rupture of cable, or local failure of the deck or progressive failure of one or multiple cables. Accordingly, major codes related to cable-supported bridges prescribe to assess the safety of the bridge fully loaded by the sum of the factored dead load, live load, and vehicular live load combined with the effect of the loss of cable multiplied by a dynamic amplification factor (DAF) under the assumption of quasi static behaviour of the bridge. Note that the PTI recommendations for cable-stayed bridges mentioned that the recommendations apply only for stay cables used in redundant cable-stayed bridges [2-4]. Despite the importance according to the loss of cable, the value for the DAF specified in the design guidelines seems to have been established without thorough research [5] or simply based on single-degree-of-freedom analysis. The application of such formerly proposed values of the DAF is likely to produce contradictory or undesirable results. Underestimated DAF may result in structural instability. On the other hand, cases may happen where the load combination for the cable loss governs the section of the deck [6]. This corresponds to an overestimated DAF that would result in an excessive number of cables and a deck section larger than required. Therefore, careful attention should be paid on the selection of the value of DAF, like what was done for Helgelend Bridge in Norway [6]. This paper intends to derive proper DAF for a cable-stayed bridge structure through adequate simulation of the cable loss that is sudden or a progressive rupture, as well as the use of appropriate models in the time history analysis for the evaluation of the dynamic response of the cable-stayed bridge. Focus is done in providing features and guidelines that will help the designer to derive rational DAF during the assessment of the cable loss or rupture in cable-stayed bridges. In addition, monitoring of the state of the cables in a cable-stayed bridge is addressed and solutions to optimally monitor the whole set of cables by means of representative measurands is also discussed.

## DYNAMIC EFFECT IN CABLE-STAYED BRIDGE

### Dynamic Amplification Factor (DAF)

The DAF is defined as the ratio of the maximum response obtained through the so-performed dynamic analysis to the static analysis response (Equation 1).

$$DAF = \frac{\text{Maximum Dynamic Response}}{\text{Static Response}} \tag{1}$$

### Previous Research on Cable Loss in Cable-Supported Structures

Most research focused on the evaluation of DAFs in regard to traffic live loads on road and railway bridges. However, the flexibility and complex structural system exhibited by cable-stayed bridges make them more prone to be sensitive to dynamic effects than ordinary bridges like girder bridges. Particularly, considering that the dynamic effect produced by the loss of cable on the neighbouring structural members differs with the position of the lost cable, such effect should be evaluated for diversified scenarios by means of DAFs calculated through analyses implemented under live loads with a model representing the degraded bridge. Though, apart from a very few studies on DAFs related to the accidental breakage of stay cables in cable-stayed bridges, study or research expressly devoted to the evaluation of DAF in the cable-stayed bridge has nearly not been published to date.

Välimäki [7] was the first researcher who published results on the breaking of cables in a cable-stayed bridge by calculating these dynamic effects through the superposition of a number of normal modes. Accordingly, the author proposed an average DAF of 1.8 in the critical section. However, the results reported by Välimäki considered only the loss of the outermost cable at central span without accounting for the vibration modes including longitudinal, lateral, and torsional modes. Thereafter, Rui-Teran [5] performed numerical analysis on an under-deck cable-stayed bridge and found out that the DAF provoked by the sudden breakage of cables in cable-stayed structures may exceed a value of 2. The analysis applied a modal composition method and derived DAFs between 1.42 and 2.27 for the moment in the deck. The latter result is extremely important since the PTI and CIP recommendations as well as the Design Guidelines of KSCE [2-4] are prescribing a maximum value of 2 for the DAFs for cable-stayed bridges, which may sometimes lead to an unsafe design for some structural members and even for the bridge system itself. Another problem is the analysis method selected for the determination of the DAF and the definition applied for the DAF. Most of the designers assume that the value obtained after convergence of the dynamic analysis is identical to the value obtained by static analysis, which is erroneous. In regards to the analysis method, Zoli [8] advocated the need for dynamic as opposed to pseudo-static analysis prescribed in the PTI recommendations.

## PROPOSED APPROACH FOR CABLE LOSS

The approach proposed for the evaluation of the loss of cable in cable-stayed bridges can be divided into the simulation of cable loss and the evaluation of the DAFs in a cable-stayed bridge. Cable loss is generally simulated as impulsive load acting in a direction opposite to the original tension of the lost cable. Such load involves a wide range of frequencies affecting most of the vibrational modes of the cable-stayed bridge. Therefore, results obtained by modal composition should be compared to those of direct integration method to verify if the frequency components of the impulsive load have been correctly included. Overestimation of the DAF may be done due to the use of misleading static and dynamic responses. This means that the value obtained after convergence ($t \to \infty$) of the dynamic analysis should correspond to the value obtained by static analysis. Cable loss scenarios considering both progressive rupture and instantaneous rupture should be assumed in the dynamic analysis. Progressive rupture corresponds to the gradual loss of strands or wires in the cable, which concentrates the stress on the remaining strands or wires up to the yield strength. The instantaneous case will give mostly conservative results, while the progressive rupture bears more practical meaning.

DAFs should be evaluated for the loss of each individual cable in regards to the deck, pylon and cables, so as to derive the worst cable loss case and extract appropriate values of DAFs. In addition, DAF should be evaluated on the basis of not only the deflection but also the element forces. Theoretically, the DAF related to deflections are lower than those related to bending moments, themselves lower than those related to shear forces [5]. Accordingly, evaluation should firstly be done in regards to element forces. In the field, cable loss analysis is performed using the original bridge model corresponding to the non-deteriorated bridge. However, cable loss affects the geometry and stiffness of the structure. Therefore, there is a need to check the feasibility of such practice by evaluating the influence of the lost cable by means of live load envelope analysis.

### Simulation Model of Cable Rupture

Fig. 1 illustrates the previous approach adopted for the simulation of cable loss according to time. For the dynamic cable loss simulation, the relevant cable is inactivated, and a forcing function $P(t)$ is gradually applied to compensate for the force in the lost cable until steady-state, under dead and live loads. Then, this force is removed to simulate the cable loss and evaluate the dynamic response [8]. On the other hand, this study adopts the method illustrated in Fig. 2. Assuming that the response to the loss of cable can be linearly superposed, analysis is performed for the model in static equilibrium and from which the relevant cable has been removed. In this case, the forcing function $P(t)$ is applied with amplitude equal to the tension of the cable under live loads but in the opposite direction in order to simulate the rupture of the cable, which allows us to assume the corresponding response as resulting only from the rupture.

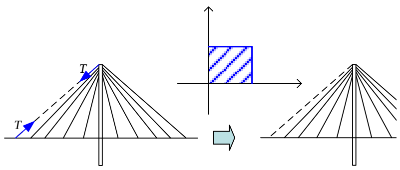


Figure 1. Previous cable loss simulation method.

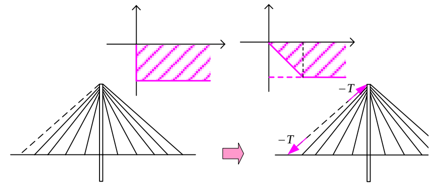


Figure 2. Cable loss simulation method adopted in this study

### Selection of Appropriate Dynamic Analysis Methods

The dynamic response due to external force oscillates around the value of the static deformation to converge gradually to this value according to the dissipation of energy induced by the damping of the structure. However, the impulsive force provoked by the rupture of a cable is acting instantaneously and exhibits white noise characteristics including a wide range of frequencies. Accordingly, the modal superposition method appears to underestimate the dynamic response and therefore may lose reliability for the computation of the dynamic response. This may explain the relatively small dynamic responses and the corresponding values of DAFs which are smaller than 1 suggested by Välimäki [7] and Zoli [8]. In addition, superposition method fails to provide uniqueness of the converged value even with larger numbers of modes. Therefore, direct integration is applied in this study with time step and total analysis time securing convergence of the solution.

## APPLICATION

### Seohae Cable-Stayed Bridge

Seohae Bridge (60+200+470+200+60m) is currently the longest cable-stayed bridge in Korea. Its five spans are constituted by 34 m wide stiffened steel girders with a precast slab. The superstructure is a prestressed, precast concrete deck supported by transverse steel floor beams and two longitudinal steel main girders along the sides of the bridge. More than 180 sensors of 10 types are actually installed in the major parts of the cable-stayed (Fig. 3), PSM (5,820m) and FCM (500 m) bridges. The H-shape concrete pylons are 182 meters high with a parallel strand cable (strand ø15.7mm, cable ø18~28 cm, HDPE tube) system that includes 144 cables spaced 12 meters apart and lengths from 54 m to 247 m. The 3-dimensional model used for the analysis has been calibrated with respect to the actual natural frequency measured in the bridge. The model is composed of 2118 frame elements and 1109 nodes. A damping value of 0.6% has been applied according to the Design Guidelines of Cable Supported Steel Bridges of KSCE [2].

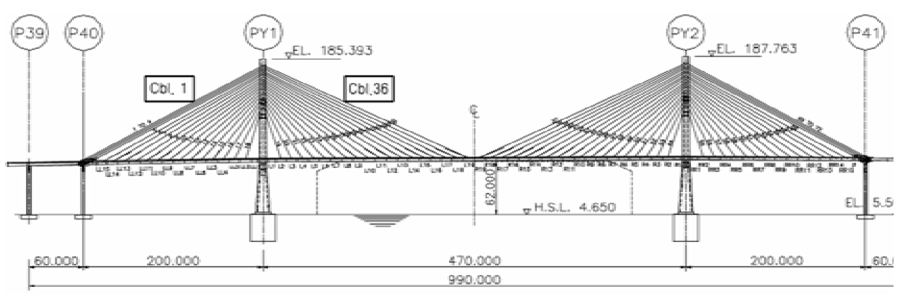


Figure 3. Structural details of the Seohae cable-stayed bridge.

**Dynamic Effects of the Loss of a Single Cable**

Dynamic analysis was performed selectively for cable loss cases expected to produce the largest effects on the structure in order to save computational time. The loss of a single cable affects not only neighbour elements but also the overall system including deck, pylon, and cables, so the response should be evaluated in global aspect. To allow comparison with previous studies [7], the loss of the outermost cable in the central span, Cbl. 36, is shown in Fig. 3 for the axial force in the edge girder of the deck. Fig. 4 compares the static axial force obtained by applying the force functions presented in (A) of Fig. 2 with the maximum and minimum dynamic responses according to the nodal points in the deck. Since Seohae Bridge is longitudinally and transversally symmetric, the results are arranged only for 1/4 of the bridge. It can be seen that the maximum responses occur at the connection of the deck and lost cable, leading to a DAF below 2.0 for the axial force of the deck. Besides, large dynamic effect occurred at proximity of the pylon PY1 (Node 1155) resulting in a DAF exceeding 5.0 for the axial force of the deck. Therefore, a unique value for the DAF appears to be inadequate to represent the difference in the dynamic effects according to the location. Table 2 summarizes the values of DAF under loss of the outermost cable of the central span with respect to the location and type of member force. Values between 1.1 and 1.65 are obtained for critical sections directly loaded by the impulsive load of rupture while larger values are observed for other sections due to small static responses. This trend appears clearly in the results relevant to PY1 where a maximum value of 75.0 was calculated. Consequently, the specification of a particular value for DAF can be seen to be meaningless without overall consideration of the whole set of cables. Need is, thus, to consider the loss of each individual cable so as to derive the worst case giving the most adverse effect on the structural stability of the bridge.

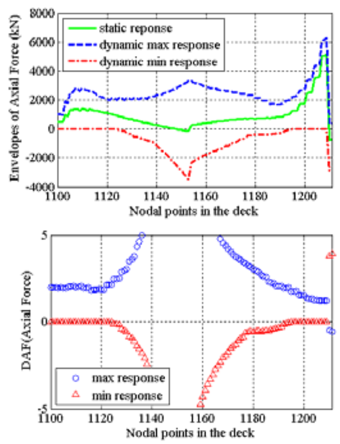


Figure 4. Envelopes of axial force in the deck for the loss of the outermost cable at central span.

Table 1: DAF under loss of the outermost cable of the central span w.r.t the location and type of member force.

| *DAF* / *Location* | *Axial Force* | *Shear* | *Moment* |
|---|---|---|---|
| Deck-cable connecting part | 1.3 | 1.3 | 1.3 |
| Other parts of the deck | 1.5~5 | 1.8~4.8 | 1.5~5 |
| Pylon – cable connecting part | 1.1 | 1.25 | 1.65 |
| Other parts of the pylon | 1.2~3.3 | 1.5~75 | 1.6~4.5 |
| Neihbouring cables | 1.45 | - | - |
| Other cables | 1.5~40 | - | - |

**Cable Loss Cases Producing the Largest Member Forces in the Deck**

Loss of individual cable has been analyzed for each of the 36 cables anchored to PY1. Fig. 5 plots the cases producing the largest and smallest member forces according to the location in the deck. Since the girder of cable-stayed bridge is mostly in the compression state, the axial force of the point in the girder located farthest from the pylon is seen to be affected at the most by the rupture of the nearest cable. This trend differs slightly at the proximity of the pylon, where the loss of the outermost cable in the side span (back-stays excluded) is seen to be decisive. For the vertical shear force and vertical moment, the effect of the loss of neighbouring cables appears to be determinant. Accordingly, the axial force can be seen to depend on the whole system while the vertical shear force and vertical moment are more sensitive to local forces. Local failure expected during the rupture of a cable is, thus, likely determined by the dynamic effects on the vertical shear force and moment, rather than those of axial force. Even if performing individual cable loss analysis for the whole set of cables is advisable for the verification of the effects of cable rupture on the girder, such approach is unavoidably time consuming. Therefore, it can be recommended to perform cable loss analysis at less for the outermost cable of the central span, the outermost cable in the side span (back-stays excluded), and cables anchored at vulnerable points of the girder.

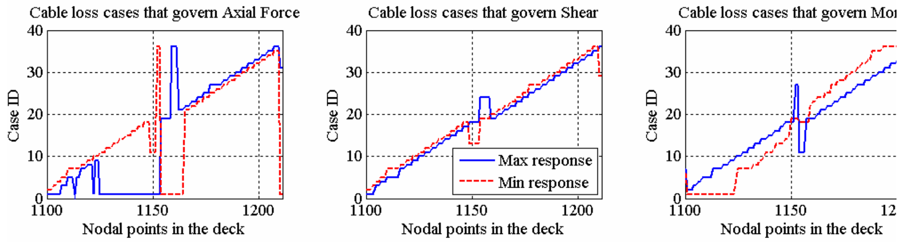


Figure 5. Cases producing the largest and smallest member forces according to the location in the deck.

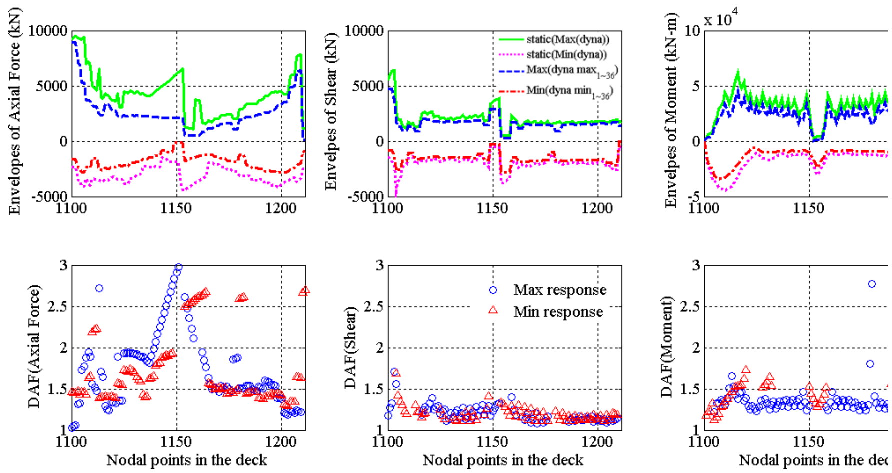


Figure 6. The largest and smallest member forces resulting from cable loss according to the location in the deck together with the corresponding DAFs.

Fig. 6 plots the largest and smallest member forces resulting from cable loss according to the location in the deck together with the corresponding DAFs. Values of DAF exceeding 5.0 occurred in the side span (Nodes 1100 to 1155) and around the pylon (Node 1155). Considering that the girder is almost exclusively in compression, values of DAF corresponding to the triangular dots (△) should be considered for the axial force. Hence, values of the DAFs to be considered reduce below 2.0 except for the neighbourhood of the pylon, which requires careful attention for the axial force. For the vertical shear force and vertical moment, DAFs are seen to remain between 1.5 and 2.0, which makes it possible to apply safely a value of 2.0.

**Contribution of Cable Loss Force to Resultant of Load Combination**

Equation (2) expresses the load combination for cable loss specified in the Design Guidelines of Cable Supported Steel Bridges of KSCE [2].

$$D + L(PS2) + I(PS2) + PS2 \tag{2}$$

where $D$ is the dead load, $L(PS2) = 0.5L$ is the live load during cable loss check, and $I(PS2) = 0.5I$ is the impact during cable loss check. Results reveal that the dead load appears to be dominant for the axial force, while the contribution of cable loss becomes determinant for the vertical shear force and vertical moment. Especially for the vertical moment, cable loss is likely to produce local failure like buckling.

### Effects of Live Loads According to the Model

The adequacy of using the original bridge model corresponding to the non-deteriorated bridge in cable loss analysis has been evaluated through the influence of the lost cable by means of live load envelope analysis for the loss of the outermost cable of the central span. Results reveal that the use of the original model introduced differences up to 30% for the axial force, 100% for the vertical shear, and 60% for the vertical moment. Accordingly, it is recommended to adopt a model from which the lost cable is removed.

### Progressive Rupture of Cable

The progressive rupture of cable is considered in order to simulate more realistically the problem of cable loss. Progressive rupture corresponds to the degradation of the cable due to gradual loss of strands or wires in the cable, which concentrates the stress on the remaining strands or wires up to the yield strength. The forcing function adopted to simulate progressive rupture is shown in (B) of Fig. 2. Fig. 7 compares the axial forces resulting from different duration (1, 2, 3, and 4 sec) of rupture. It is observed that the axial force reaches approximately 80% of that obtained with instantaneous cable loss leading to a DAF of 1.1. It should be noted that the dynamic response appears to reduce rapidly with a larger duration of rupture at the vicinity of the pylon (Node 1155). For duration of 1 sec, the DAF is seen to decrease down to 1.6. Consequently, former DAF with values exceeding 5.0 (Fig. 6) may be seen to be overestimated since the axial force is decreasing significantly with a larger duration of rupture. Considering that progressive rupture is more realistic, a value of 2.0 seems to be reasonable for the DAF.

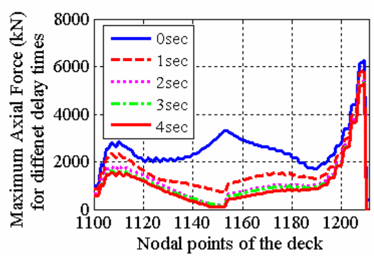


Figure 7. Axial forces of different duration of rupture

## CONCLUSIONS

This paper investigated the dynamic response in case of cable loss with focus on dynamic amplification through case studies on an actual cable-stayed bridge considering instantaneous loss of cable and progressive loss of cable. Cable loss in cable-stayed bridge should be evaluated through direct integration. Based on the investigation, it is recommended to adopt a model from which the lost cable is removed. The value obtained after convergence of the dynamic analysis should be verified to correspond to the value obtained by static analysis. DAFs should be evaluated for the loss of each individual cable in regards to the deck, pylon, and cables so as to derive the worst cable loss case and extract appropriate values of DAFs. The axial force was seen to depend on the whole system while the vertical shear force and vertical moment were more sensitive to local forces. Consequently, DAF below 2.0 can be applied for the vertical shear force and vertical moment, and particular DAF should be decided for the axial force. Cable loss scenarios considering both progressive rupture and instantaneous rupture should be assumed in the dynamic analysis. Results showed that the instantaneous case gave the most conservative result, while the progressive rupture bore more practical meaning with DAF reaching 80% of the instantaneous rupture. DAF’s exceeding 5.0 were seen to be overestimated since the axial force decreased significantly with a larger duration of rupture. Following, cable loss should be investigated with a wider set of cable-stayed bridges in order to gather a meaningful database. In addition, the consideration of progressive loss of cable seems to be advisable rather than instantaneous rupture in terms of practicality and economic efficiency. Considering monitoring issues, it has been seen that the outermost cables of the central and side spans were decisive in determining the level of the dynamic factors. Accordingly, sensors should be disposed imperatively in these cables or at their anchorage points to acquire measurands meaningful for the monitoring of the overall stability of the bridge under loss of cable. In case of cable loss, the longitudinal and transversal accelerations of the deck were seen to exhibit variations exceeding 600% between instantaneous and progressive rupture, which may help to determine an adequate event processing method as well as the type of rupture that occurred.

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## REFERENCES

1. Lee, G.C. and C.-H. Loh, *The Chi-Chi, Taiwan Earthquake of September21,1999: Reconnaissance Report*, in *Techncal Report MCEER-00-0003*, MCEER, 2000.
2. KSCE, *Design guidelines of cable-supported steel bridges.* 2006.
3. PTI, *Recommendations for Stay Cable Design, Testing And Installation*. 2000: Phoenix, AZ. p. 55-56.
4. SETRA, *CIP recommendations on cable stays*. 2002. p. 176-177.
5. Ruiz-Teran, A.M. and A.C. Aparicio, *Dynamic amplification factors in cable-stayed structures.* Journal of Sound and Vibration, 2007. 300(1-2): p. 197-216.
6. Svensson, H.S. and E. Jordet, *The concrete cable-stayed Helgeland Bridge in Norway*. Civil Engineering, 1996. 114.
7. Hyttinen, E., J. Välimäki, and E. Järvenpää. *Cable-stayed bridges, effect of breaking of a cable*, . in *Conference AFPC sur les ponts suspendus et à haubans*. 1994. Deauville, France.
8. Zoli, T. and R. Woodward, *Design of Long Span Bridges for Cable Loss*, in *IABSE SYMPOSIUM LISBON 2005*. 2005: LISBON.