INTRODUCTION

The failure of one structural element can lead to the failure of further structural elements and thus to the collapse of large structural sections or the entire structure. Such disproportionate collapses have been discussed and investigated for some years, but mainly for buildings. Generally, structures can be made collapse resistant by ensuring a high level of safety against local failure or by increasing their robustness. Robustness has been defined as the insensitivity to local failure. Increasing the robustness of cable-stayed bridges means to design for the loss of cables. The sudden loss of cables is a dynamic event and must be investigated in nonlinear dynamic analyses. For the design, only the maximum responses are of interest. For this, quasi-static analyses using dynamic amplification factors to account for the dynamic effects are advised. The size of these amplification factors is strongly discussed. In the first part of this paper, dynamic amplification factors are determined in nonlinear dynamic analyses for a typical cable-stayed bridge and the applicability of the quasi-static method is discussed. For creating robust structures, it is further necessary to know the collapse behavior of a structure after the loss of cables. With this, structural properties can be identified which are responsible for collapse propagation. In the second part of this paper, the collapse behavior of a cable stayed bridge after the loss of cables is investigated taking into account nonlinear material behavior and prevailing collapse types are identified.

1 INTRODUCTION

The failure of one structural element can lead to the failure of further structural elements and thus to the collapse of large structural sections or the entire structure. In many cases, the initial triggering event and the resulting damage are disproportionate. Such collapses have frequently been discussed and investigated in recent years and are generally summarized under the term progressive collapse. But work in this field refers predominantly to buildings.

With regards to the design of a structure against progressive collapse, the general aim is to ensure collapse resistance which means insensitivity to accidental circumstances. This can be achieved by ensuring a high level of safety against local failure or by using a design which allows for local failure. The structure's property of being insensitive to local failure is termed robustness (Starossek 2006b). The size of the local failure has to be defined by the design objectives. For cable-stayed bridges, collapse resistance is primarily achieved by increasing the robustness. The loss of cables must be considered as a possible local failure since the cross sections of cables are usually small and therefore possess a low resistance against accidental lateral loads stemming from vehicle impact or malicious action. Current recommendations for cable-stayed bridges constitute the sudden loss of one single cable (PTI 2007, fib 2005); other authors assume the sudden loss of all cables in a 10 m range (Starossek 2006a). The loss of cables can lead to overloading and rupture of adjacent cables. A collapse progressing in such a way is called a zipper-type collapse (Starossek 2007). Because the stiffening girder is in compression, the loss of cables, which leads to a reduction of bracing, increases the risk of buckling. For creating robust structures, it is further necessary to know the collapse behavior of a structure after the loss of cables. With this, structural properties can be identified which are responsible for collapse propagation. In this paper, the collapse behavior of a cable stayed bridge after the loss of cables is investigated taking into account nonlinear material behavior and prevailing collapse types are identified.

Structural robustness of a cable-stayed bridge

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ABSTRACT: The failure of one structural element can lead to the failure of further structural elements and thus to the collapse of large structural sections or the entire structure. Such disproportionate collapses have been discussed and investigated for some years, but mainly for buildings. Generally, structures can be made collapse resistant by ensuring a high level of safety against local failure or by increasing their robustness. Robustness has been defined as the insensitivity to local failure. Increasing the robustness of cable-stayed bridges means to design for the loss of cables. The sudden loss of cables is a dynamic event and must be investigated in nonlinear dynamic analyses. For the design, only the maximum responses are of interest. For this, quasi-static analyses using dynamic amplification factors to account for the dynamic effects are advised. The size of these amplification factors is strongly discussed. In the first part of this paper, dynamic amplification factors are determined in nonlinear dynamic analyses for a typical cable-stayed bridge and the applicability of the quasi-static method is discussed. For creating robust structures, it is further necessary to know the collapse behavior of a structure after the loss of cables. With this, structural properties can be identified which are responsible for collapse propagation. In the second part of this paper, the collapse behavior of a cable stayed bridge after the loss of cables is investigated taking into account nonlinear material behavior and prevailing collapse types are identified.
cation factors are determined and the limits of the quasi-static approach are outlined. In the second part of this paper, the collapse behavior of a cable-stayed bridge after the loss of cables is investigated. To trace the collapse progression following the rupture of one or more cables, geometric and material nonlinear dynamic analyses in the time domain are necessary. Hereby, critical elements are identified and the prevailing collapse type is described.

2. INVESTIGATED BRIDGE SYSTEM AND ITS MODELING

The cable-stayed bridge being considered is shown in Figure 1. Two cable planes are placed in a modified fan arrangement with 80 cables in each vertical plane and a cable spacing of 15 m at deck level, apart from the closely spaced outermost back-stay cables. The cables have a total cross-sectional area of between 40.8 cm² and 117 cm². They are pretensioned in such a way that under permanent loads, the stiffening girder is not deflected at the cable anchorage points and the moment distribution corresponds to that of a continuous beam on rigid supports. Furthermore, the bending moments in the pylons are nearly zero. The pylons are made of reinforced concrete. The stiffening girder consists of a 21.60 m wide orthotropic steel deck, two 2.6 m deep longitudinal steel girders and cross girders spaced at 3.75 m. This corresponds to a slenderness ratio of 1/230. In longitudinal direction, the stiffening girder is only restrained by the cables.

The numeric investigation is conducted using a three-dimensional model of the bridge. The pylon and stiffening girder (longitudinal and cross girders) are modeled with beam elements. The cables are modeled with truss elements. In preliminary analyses, the influence of cable sag and the cable’s mass distribution on the dynamic response was investigated (Wolff & Starossek 2008). It pointed out that for the loss of one cable, a detailed modeling of cables leads to smaller dynamic responses. A detailed modeling thus seems necessary: The cables are modeled as a series of truss elements with distributed masses and are loaded by their self weight. This leads to a sag which is in equilibrium with the predetermined cable forces.

In addition to the permanent loads, two live load cases are here considered: live load on the main span and live load on the side spans. For the loss of a cable, in accordance with the PTI Recommendations (2007), the live load is reduced to 75%.

The loss of a cable is investigated by nonlinear dynamic analyses in the time domain, taking into account large deformations. Thus, direct time integration is used to solve the equation of motion. Firstly, the static initial state of the structure with the considered static load cases is calculated. The cable to be considered for failure is eliminated from the structural model and the corresponding cable forces are applied to the anchorage nodes of that cable as static loads. The time-history analysis is begun on this modified and loaded system at rest. To model the sudden loss of the cable, a step loading of the same size as the static cable force but acting in opposite directions is applied to both anchorage nodes. The influence of damping is investigated separately. When Rayleigh damping is used in numeric analyses, higher modes must be included for calculating the necessary damping parameters since the loss of short cables mainly excites high modes (Wolff & Starossek 2008). The numeric calculation is conducted using the finite element analysis program ANSYS.

3 DYNAMIC AMPLIFICATION FACTORS

The structural response of a cable-stayed bridge to the sudden loss of a cable can either be calculated by nonlinear dynamic analyses or by a quasi-static approach which accounts for the dynamic effects by a dynamic amplification factor (DAF). Therefore, the cable to be considered for failure is removed from the structural model, and the corresponding cable forces are applied to both anchorage nodes of that cable as static load. A static force which is the static cable force multiplied by a dynamic amplification factor is then applied to the anchorage nodes in the opposite direction to the initial cable force. For single-degree-of-freedom systems, this factor is 2.0.
This value is also advised by the PTI Recommendations (2007). But in complex systems such as cable-stayed bridges, other values are to be expected. The 5th edition of the PTI Recommendations allows the determination of a dynamic amplification factor in a nonlinear dynamic analysis, but stipulates a lower limit of 1.5. Investigations as to realistic ranges of dynamic amplification factors are rare. Single values are calculated for an arch bridge in Zoli & Woodward (2005), in a simplified manner in Hyttinen et al. (1994), and in Park et al. (2007).

In the following, dynamic amplification factors will be determined for the system described in Section 2. The dynamic amplification factor is calculated separately for all state variables $S$ in all structural elements. Therefore, the sudden rupture of a cable is modeled as described in Section 2 and the extreme dynamic responses in time history $S_{\text{dyn}}$ including the initial loading are determined. Additionally, the responses to the static removal of a cable including the initial loading $S_{\text{stat}}$ are calculated. The dynamic amplification factor is obtained by comparing these responses whereas the responses in the initial state $S_0$ are subtracted, respectively:

$$D_A F = \frac{S_{\text{dyn}} - S_0}{S_{\text{stat}} - S_0}. \quad (1)$$

The results show that a unique dynamic amplification factor cannot be specified. Instead, the value is dependent on the location of the ruptured cable as well as the type and location of the state variable being considered. For the deflections and the bending moments of the stiffening girder, very different dynamic amplification factors result if the rupture of one cable is considered. Amplification factors at locations further away from the ruptured cable are high, which is shown for the deflections in Figure 2. At these locations static responses are small. While the static removal of a cable mainly causes local deflections and bending moments, the sudden removal of a cable excites natural modes with deflections and moments over the whole girder length. Thus, the mainly excited natural modes are not affine to the static deflection curve. The dynamic responses at locations further away from the ruptured cable are, however, irrelevant when considering all cable loss load cases, because only responses in the vicinity of the ruptured cable are design governing. Concerning the positive vertical deflections, a dynamic amplification factor of between 1.5 and 1.8 results at these locations, depending on the ruptured cable being considered. The amplification factor for the positive bending moments in proximity to the ruptured cable lies between 1.3 and 1.6, while that for the negative bending moments between 1.4 and 2.7. In Figure 3, the envelopes of extreme bending moments from all load cases are shown, together with the corresponding amplification factors.

The dynamic amplification factor for the design governing dynamic axial forces in the stiffening girder at the anchorage points of the long cables is between 1.9 and 2.3. The dynamic amplification factor for the design governing dynamic axial forces

![Figure 2. Dynamic amplification factors (DAF) (a) of extreme bending moments in longitudinal girder in the plane of cable rupture due to permanent loads and loss of second longest cable at bridge center (b)](DOI: 10.3267/HE2008)
at the anchorage points of the short cables is several magnitudes higher than 2.0 because of very small static forces. But here, contrary to the deflections and bending moments of the stiffening girder at locations further away from the ruptured cable, the dynamic forces are not negligible.

The dynamic amplification factors for the cable forces in the cables adjacent to the lost cable are between 1.35 and 2.0, depending on the lost cable being considered. The higher values occur when a short cable close to the pylon ruptures. The dynamic forces in the cables adjacent to the lost cable are design - governing for all cable failures, as expected. A high amplification factor at locations further away is therefore irrelevant.

Special attention is necessary for the bending moments in the pylons. The dynamic amplification factor for the bending moments over the whole pylon height and for all cable losses is significantly higher than 2.0. At the pylon base values of about 20 for negative and about 8 for positive moments occur. The static moments are small. However, the dynamic bending moments in the pylon are significant (Fig. 4). Here too, higher modes which are not affine to the static deflection curve are excited. Furthermore, the pylon is not only excited by the step loading of the failing cable, but each of the cables, whose anchorage points are evenly distributed over a length of 26 m at the pylon head, also induces irregular forces which are composed of the redistributed loads from the failed cable plus the inertia forces from the bridge deck. These forces cause a complex structural response which cannot be simplified as described above.

The results show that in the present case, dynamic amplification factors can, at the most, be chosen smaller than 2.0 for the bending moments in the stiffening girder, since over wide parts, the values are smaller. For the safe design of the cables, a dynamic amplification factor of 2.0 is necessary. For the axial forces in the bridge deck as well as the bending moments in the pylons, high dynamic forces occur due to the sudden cable loss which cannot be safely accounted for by a quasi-static analysis using amplification factors. In particular, when the static re-
moval causes a decrease in responses while the dynamic removal causes an increase, quasi-static analyses cannot yield correct results. Dynamic analyses seem vital here.

The results presented in this paragraph are for the undamped system under self-weight without live loads. However, the dynamic amplification factors for the design relevant state variables are nearly independent of the type of loading. Similar dynamic amplification factors are obtained for a system which is additionally loaded by live load on the main-span or back-spans. In (Wolff & Starossek 2008), the influence of damping on the structural response after cable loss is described: The effect of damping depends on the occurrence of the maximum responses in time-history. Since the design relevant maximum deflections and bending moments of the stiffening girder and the maximum cable forces develop early in time history, a damping has only a small effect here. Therefore, the impact on reducing the dynamic amplification factor is small. The bending moments in the pylons, however, are significantly reduced, even by a small damping ratio of \( \xi = 0.2 \% \). However, the dynamic amplification factors are still much higher than 2.0.

4 COLLAPSE BEHAVIOR

To trace the collapse progression after an initial failure of one or more cables, geometrically and materially nonlinear dynamic analyses are necessary. Alternatively, especially for buildings, a quasi-static nonlinear analysis has been proposed where the load is incrementally increased to follow the formation of inelastic regions. However, this kind of analysis is only valid if the dynamic response is dominated by a mode shape which is affine to the static deflection curve of the structure without the considered cable. This is, as stated in Section 3, not always the case. Thus, dynamic nonlinear analyses are conducted here. The procedure is described in Section 2.

The material behavior of the bridge girder and the cables is assumed to be elasto-plastic. In the collapse state, cable stresses can be significantly higher than in the initial state but also a slacking is possible. For these high stress variations, the cables’ force-deformation behavior is highly nonlinear and cannot be linearized. Taking into account cable sag and transverse cable vibration, therefore, becomes crucial. To account for the exact stress distribution in the bridge girder, a combination of shell and beam elements is used. The element size is adapted according to the strain gradients.

For the investigated bridge, the failure of one single cable does not lead to collapse progression. Only local plastifications develop at the anchorage point of the ruptured cable in the affected longitudinal girder, and deflections are not significant. Also, the cable tensions remain comparatively small. Live loads can be increased by a factor of three until ultimate state is reached.

Nevertheless it is important to know the failure of which cable leads to the highest responses since the structure then has only to be designed for these cable loss cases. The increase in cable stresses is highest when a short cable fails. Design controlling for the stresses in the bridge girder is the loss of a long cable near the center of the bridge. For the maximum bending moments in the pylons, the loss of a back-stay cable has to be analyzed. These elements are defined as the important elements of the structure. Additionally, elements whose failure leads to disproportionate collapse are defined as critical elements. These are for the investigated bridge, the loss of three adjacent short cables.

In Figure 5 the initial state (permanent and live loads) with the three cables considered for failure, the critical elements, is depicted. Only one half of the bridge is shown. After the sudden loss of the three cables, vertical deformations with plastic regions begin to develop firstly in the longitudinal girder of the damaged cable plane (in the front of the figure). Thereby, the normal forces acting in the whole section of the bridge girder are transferred to the longitudinal girder of the intact cable plane where vertical deflections are relatively small. Although this girder is continuously supported by the cables it cannot resist these high normal forces and begins to buckle in vertical direction. From this moment on, vertical deflections strongly grow and cannot be arrested since the bridge deck is not restrained by fix supports in longitudinal direction. Ultimate stresses in the bridge girder are exceeded. During this process, the upward deflections and the missing restraint in longitudinal direction cause a slacking of cables which leads to a disengaging from their anchors. The downward deflection of the longitudinal girder of the intact cable plane finally causes the rupture of the cable at this location. This final state of the bridge deck is shown in Figure 6. Due to the longitudinal motion of the bridge deck towards the damaged region, normal forces are transferred to the other, intact half of the bridge with the second pylon (not shown in the figures). This leads to high unbalanced cable forces in both bridge parts which in turn results in high bending moments in both pylons. The continuity of the bridge girder thus causes the failure of both pylons which is the total collapse of the bridge.

At least three short cables have to fail initially to cause the total collapse of the bridge. However, the structure is more tolerant regarding the loss of long cables. This is due to the fact that at the anchorage points of the longer cables, the bridge girder exhibits smaller normal forces and therefore the increase in bending moments due to second order effects is small. Furthermore, the support of the bridge girder

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Figure 5: Von Mises stresses in the bridge girder due to permanent and live loads prior to loss of cables (one half of the bridge)

Figure 6: Von Mises stresses in the bridge girder at collapse state due to loss of three cables, permanent and live loads (for a better illustration, the pylon is omitted), -- slack cables, ■ ruptured cable

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in the bridge center is softer than at the pylons because of the smaller inclination and softer force-deformation behavior of the long stay-cables. The forces from the failing cables are distributed to a greater part of the bridge girder and to more cables. Details are given in (Wolff 2009).

According to Starossek (2007) six types of structural collapses can be distinguished: five pure collapse types and one mixed-type collapse. Each collapse type can be characterized by a propagating action which after the failure of one element leads to the failure of the next element. The study of propagating actions can give insights to the structural properties which promote collapse propagation.

The collapse propagation described above does exhibit features of at least three of the pure collapse types and must therefore be categorized as mixed-type collapse. At the beginning, there is the initial failure of elements which are responsible – although not mainly - for the stabilization of the bridge girder in compression. Lack of bracing leads to an increase in transverse deflections and high stresses due to second order effects in the longitudinal girder of the affected cable plane. These are features of the instability-type collapse although the failure is not pure buckling. The subsequent collapse propagation is, however, not characterized by the instability-type collapse since no other stabilizing elements are affected. Instead, it comes to a concentration of normal forces in the second longitudinal girder in the intact cable plane, and eventually its failure due to instability. Although the girder fails due to instability, this collapse stage is assigned to the zipper-type collapse which means, amongst other things, a failure due to a concentration of forces in structural elements which are similar in type and function, and directly adjacent to the initial (here afore) failed element. The compression force is thus responsible for the onset of collapse propagation and the failure of the bridge deck. If the failure of cables cannot be avoided, a prevention of collapse propagation is only possible by increasing the stiffness of the bridge girder.

A rupture of adjacent cables as a direct consequence of the initial failing cables - which is the main example for the zipper-type collapse and is often connected to the collapse of cable-stayed bridges - does not occur.

After the total failure of the bridge deck, compression forces can no longer be canceled out. Instead, tension forces develop in the bridge girder in the main span and are transferred to the pylons by the main cables. The pylons are pulled towards the main span of the bridge and fail in bending. This process exhibits features of the domino-type collapse: The two pylons are individual structures with mainly vertical load bearing capacity. Forces orthogonal to the main bearing direction cause an overturning and failure of these structures. The horizont-
due to the sudden cable loss, but also by irregular forces induced by each cable. This results in a dynamic response which cannot be identified in a factorized static analysis. Dynamic time-history analyses are recommended, at least for the critical cable loss cases which yield the highest responses.

In the second part of the paper, the collapse behavior of a cable-stayed bridge to the loss of cables was investigated. It was shown that the bridge cannot sustain the failure of more than two cables without serious damage. Its robustness is therefore limited to the failure of maximum two adjacent cables. The critical elements of the bridge were identified as three adjacent short cables. However, the cable-stayed bridge behaves more tolerant to the loss of long cables because of the absence of compression forces in the bridge girder. Failure of the critical elements increases the unbraced length of the bridge girder and thus the stresses due to second order effects. At the beginning of collapse propagation, the collapse can be categorized as an instability-type collapse. However, subsequent collapse stages are different, and thus the total collapse is classified as mixed-type collapse.

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