Robustness assessment of suspension bridges

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ABSTRACT: Structural robustness – defined as the insensitivity of a structure to initial damage – has been recognized as a desirable characteristic of structural systems because it mitigates their susceptibility to progressive and disproportionate collapse. The robustness of suspension bridges is investigated using a generic suspension bridge design. Based hereupon, recommendations for a robust design of suspension bridges are developed.

1 INTRODUCTION

A disproportionate collapse can be prevented by ensuring collapse resistance, which is achieved by reducing the vulnerability of the structure or by increasing its robustness (Starossek and Haberland 2010). While vulnerability depends on the individual structural components, robustness depends on the whole structural system. Instead of by collapse resistance, a disproportionate collapse can also be prevented by reducing the exposure of the structure.

Five design methods are available to implement these strategies (Starossek and Haberland 2010). Event control reduces the exposure of the structure by reducing the probability of occurrence and/or the intensity of abnormal events through non-structural measures. The vulnerability of the structure can be reduced by protecting it through external structural measures thus mitigating the effect of abnormal events on the structure. It can also be reduced by locally increasing its structural resistance. This prevents or lessens an initial damage that could otherwise lead to disproportionate collapse. The robustness of the structure can be enhanced by alternative load paths that enable a redistribution of forces originally carried by failed components thus preventing a failure from spreading. It can also be enhanced through segmentation, that is, by dividing a structure into segments by dedicated segment borders; in this way, a failure is isolated within one segment (or, in particular cases, within two segments) thus preventing a failure from spreading disproportionately.

The two strategies ‘reducing exposure’ and ‘reducing vulnerability’ are only shortly discussed in the following. The strategy ‘enhancing robustness’ is the main topic of this paper. Robustness is defined here as the insensitivity of a structure to an initial damage (Starossek and Haberland 2010). It prevents a disproportionate progression of collapse.

The prevention of disproportionate collapse of long-span cable-supported bridges is of major concern as such structures are of high economic value and their collapse would be accompanied by a large loss of human life. Cable-supported bridges can be classified into cable-stayed bridges, suspension bridges, and combined systems. Suspension bridges can further be classified into earth-anchored and self-anchored suspension bridges, depending on the manner in which the main cables are anchored. There are similarities and differences between these types of cable-supported bridges that will be addressed here. However, the main topic of this paper are (earth-anchored) suspension bridges.

The main components of a suspension bridge are the pylons, the bridge girder, the main cables, and the suspension system composed of a multitude of hangers. Pylons, bridge girder, and main cables are primary load-bearing structural components. The suspension system as a whole, connecting the bridge girder to the main cables, is also a primary load-bearing component of the bridge. The individual hangers, however, can be considered secondary load-bearing components (Starossek 2009).

In the case of cable-stayed bridges, the main components are the pylons, the bridge girder and the stay-cable system composed of several stay cables.

2 COLLAPSE MECHANISM

According to Starossek (2007) different types of structures are susceptible to different mechanisms of collapse. This is also true for cable-supported bridges. For instance, a cable-stayed bridge is prone
to a mixed-type collapse composed of an initial zipper-like failure of some hangers and an instability-type collapse of the compressed bridge girder (buckling) (Wolff 2010; Wolff and Starossek 2008).

For an earth-anchored suspension bridge, the prevailing type of collapse is a zipper-type collapse, that is, a zipper-like failure of the hangers (Giuliani and Bontempi 2008). After the initial failure of one or a few hangers, the load originally carried by these hangers has to be redistributed to the adjacent hangers. If these hangers become overloaded, they fail in their function of alternative load paths and the failure progresses. This effect is intensified by the dynamic nature of the load impulse which results from an abrupt failure of a hanger. The failure of the Tacoma Narrows Bridge in 1940 can be seen as an example for such a collapse. After the first hangers snapped due to excessive wind-induced distortions of the bridge girder, the girder successively peeled off from the remaining hangers (Starossek 2009). The collapse stopped because the failed part of the girder broke away and disconnected from the still supported part of the girder thus preventing a further collapse progression.

In the case of a self-anchored suspension bridge, where the bridge girder is in compression, any unzipping tendency is likely to be reinforced by the susceptibility of the bridge girder to buckling. A mixed-type collapse can thus occur, like in cable-stayed bridges, in which the features of zipper-type and instability-type collapses interact and reinforce each other (Starossek 2009).

3 PREVENTING DISPROPORTIONATE COLLAPSE

In a suspension bridge, the task of the pylons and main cables is to support the bridge girder. Thus, a failure of one of these two components usually leads to a collapse of the whole bridge. (In suspension bridges with multiple main cables, the failure of one main cable might not always lead to total collapse.) These components are key elements to the structure whose failure must be prevented by all means, whereas the (local) failure of the bridge girder does not necessarily imply total collapse. The bridge girder is connected with the main cables through the suspension system (i.e., the hangers). The suspension system can be designed as redundant thus preventing a progressive collapse of the suspension system and the bridge as a result of the failure of one or a few hangers. (The same holds for the stay cables of cable-stayed bridges.)

In contrast to pylons and main cables, which usually exhibit high local resistance, the hangers are more exposed and vulnerable and their individual failure, if no additional measures are taken, is more likely. Thus, it seems reasonable to assume the sudden failure of individual hangers in the design of the bridge or to take explicit additional measures to make hanger failure more unlikely. These two options are discussed in the following.

4 REDUCING EXPOSURE AND VULNERABILITY

The exposure and the vulnerability of the hangers of a suspension bridge can be reduced with the design methods event control and protection. The specific-local-resistance method seems inappropriate to reduce the vulnerability, at least for the currently preferred multi-suspension systems with narrow hanger spacing. This is due to the small cross-sectional area of the hangers, and thus their comparatively small resistance to accident-related or malicious actions in the lateral direction (Starossek 2009). Providing the hangers with increased local resistance would be uneconomical since the required material would be considerably more than statically required.

External protection measures are more meaningful. Such measures include barriers to fend off vehicles, like concrete barriers, strengthened guard rails, or crash cushions, possibly with a change of the primary design objective from passenger protection to structure protection. In view of the emergence of new types of threats, in particular aggressive and well-resourced malicious action, it is nevertheless advisable to take measures to protect hangers against such threats. For bridges of high significance and exposure, these measures should include fencing and electronic security systems to reliably deter trespassers from approaching the hangers (Starossek 2009). In particular, the hangers should be protected against explosions by appropriate physical shielding. This could be provided, for instance, by thick composite-material sheathing or energy-absorbing honeycomb structures that mitigate the effect of explosions and protect the hangers against directly mounted shaped charges or mechanically introduced damage. Such structures would have to be applied on the hangers up to a certain height (say, 5m) above deck level (Starossek 2009). Even the hanger connections must be designed for explosive action, that is, a detachment of a hanger from its anchorage must be prevented in the case of large dynamic deformation due to explosion pressure. The fire resistance of the hangers can be enhanced by coating with foaming material up to a certain height above deck level. This coating can be combined with the sheath that protects against explosive action and corrosion. To prevent hanger failure due to corrosion, efficient corrosion protection systems for hangers and anchors, and regular inspection, ideally complemented by constant monitoring, are needed (Starossek 2009). For particularly significant and exposed bridges, non-local event-control measures like aerial surveillance or anti-aircraft defence can be required.
5 INCREASING ROBUSTNESS

The critical components of a suspension bridge are the hangers. (The critical components of a cable-stayed bridge are the stay cables.) The failure of one (or a few) hangers (stay cables) can lead to a progressive collapse of the whole bridge. This collapse progression should be prevented by ensuring robustness. The robustness of a structure can be enhanced by the alternative load path method and the segmentation method (Starossek and Haberland 2010). In the case of cable-supported bridges, these options can be applied to the suspension system, the stay-cable system, or the bridge girder. Segmenting the bridge girder is not an option in case of cable-stayed bridges (and self-anchored suspension bridges) due to the compression forces present in the bridge girder. The alternative load path method is limited to the cable systems. Emphasise is put on suspension bridges in the following.

Using the alternative load path method to enhance the robustness of the whole bridge requires an excess of hangers. That means the suspension system consists of more hangers than necessary for the load transfer in typical situations and that the utilisation of the hangers is not at their limit, so there is a reserve capacity. To successfully apply the alternative load path method, the suspension system (as well as the other main components) must be able to withstand the dynamic forces (and possibly load concentrations) caused by the rupture of the hanger. Factors affecting the activation of alternative load paths of a suspension bridge are treated in Section 6.

Instead of by alternative load paths, a suspension bridge may be made robust by applying the segmentation method. This method prevents or limits a failure spreading that might result from an initial damage by isolating the failing part of a structure from the remaining structure. Therefore, the structure must be divided into segments by dedicated segment borders. In general, segment borders are formed (1) by strong components designed to arrest an incipient collapse (high local resistance to accommodate large forces) or (2) by weak components (structural fuses) at which failing parts can safely disconnect from the structure (eliminating continuity or reducing stiffness to accommodate large deformations and displacements) or (3) provide the segment borders with high ductility and large energy dissipation capacity (to accommodate large forces and large displacements at the same time) (Starossek 2009).

In the case of the suspension system of a suspension bridge, strong hangers (zipper stopper) at chosen locations (with a multiple of the load-carrying capacity of normal hangers so that this hanger can carry approximately the dynamically amplified load of half of the segment) can be used as segment borders to arrest a zipper-type collapse of the suspension system—thus using the first version of segment borders—which are able to withstand the dynamic forces and load concentration they are exposed to during the progressive collapse. Therefore, the bridge girder must be provided with enough resistance and ductility to span the whole unsupported segment length (by bending or catenary action) if all hangers within this segment fail. This requires that the segments should be comparatively small.

Also the second version of segment borders, that is, weak components, could be used; not for the suspension system, but for the bridge girder. The bridge girder can be segmented by structural fuses, that is, in the simplest approximation, joints or break-away hinges that give way at the beginning of a collapse and thus allow for a safe separation of the falling part of the girder (Starossek 2009). If a hanger failure occurs within one segment of the bridge, the resulting zipper-type collapse should be limited to this segment (or at least one additional segment). It is possible to segment the bridge girder, because, typically, there is no tension or compression force in it in the case of earth anchored suspension bridges as mentioned earlier. The design of such structural fuses is a very complex task. On the one hand, they must facilitate the load transfer from one segment to another, in particular regarding to the torsional stiffness of the bridge for ordinary situations (like wind forces) which is governed by the bridge girder. Also, in the case of an earthquake, the segmented bridge girder should act as a homogeneous girder. Moreover, there should be no gaping joint or vertical/torsional offset between two segments endangering the vehicles using the bridge. On the other hand, the structural fuse must facilitate an easy disconnection of the failing segment from the remaining bridge. Therefore, the geometry and detailing of such fuses must be done very carefully (taking into account symmetrical and asymmetrical damage conditions). Additionally, the structure must be able to account for the dynamic forces and the vibration introduced into the system by the disconnecting segment. Segmenting by structural fuses is further discussed in Section 7.

Also the third approach to segment borders could possibly be used for a suspension bridge. An early rupture of adjacent hangers could be avoided, and thus a failure progression, by providing the hanger-girder-connections with special devices that are high ductile, allow for energy dissipation, and that only transfer a specific maximum force to the connected hangers (Wolff 2010). Such devices used as segment border is a border between the girder and the suspension system. In this case, all hangers should be supplied with such devices. Restricting the force introduced in a hanger by permitting large displacements incorporate additional hangers in the load-transfer mechanism. Such a device should be replaceable after they have been active. In summary, this approach might be perceived as a combination of segmentation and alternative load paths.
6 ALTERNATIVE LOAD PATH METHOD

6.1 Modelling

The investigations presented in this section are based on Haß (2011). The robustness of a suspension bridge utilising the alternative load path method is investigated with a simplified bridge configuration. On the one hand, the influence of the bending stiffness of the girder on the robustness of the bridge concerning the hazard scenario hanger failure is investigated. On the other hand, the influence of the hanger spacing is investigated. The bending stiffness of the girder and the hanger spacing are interdependent in the context of an economical design. Additionally, it is investigated if the length of the hanger is of importance (to account for different locations at the girder).

The bridge configuration used for the investigations here is a long-span earth-anchored suspension bridge with a main span of approximately 2000m, which is comparable to the Akashi-Kaikyo-Bridge in Japan (1991m). The chosen sag to main span ratio is 1/10.

The roadway consists of two lanes (and a hard shoulder) in each direction with an overall width of 28.5m. The ratio of dead-load to live-load is 2:1. Only a preliminary design is performed. Since all stability problems are neglected here and thus only a bending design is performed, a global safety factor of 2.0 is used for the design of the bridge girder. The global safety factor of the hangers is set to 1.8.

The bridge girder is assumed as a simple double-symmetrical steel box girder with rectangular cross section and three webs (Figure 1). This unrealistic section is chosen to minimize the analysis demand for the preliminary design. The bridge girder used here has a higher bending stiffness than actually built bridge girders with the same cross-sectional height (Haß 2011).

As a simplification, the bridge is modelled as a long continuous beam, which represents a part of the main span (Figure 2). The modelled part of the girder consists of 20 hanger-to-hanger segments, each modelled with one element. As this is only a part of the girder, there are interferences in the border region. The border to account for the additional regions of the girder is on the one hand modelled by a fixed support and on the other hand by a hinged support to get two extreme values limiting the real behaviour. The hangers are represented by springs, whose spring stiffness is calculated from an intermediate hanger length of 84m and the axial stiffness of the hangers. Therefore, a modulus of elasticity of 165000N/mm² is assumed. The loads are converted and applied as a constant line load acting on the centre line of the girder, thus neglecting torsion.

Figure 1. Bridge girder

Figure 2. From bridge to model

6.2 Analysis

Subject of the investigation is the response of the bridge to the failure of one or some hangers. Only the direct effect of this initial damage is investigated without modelling a possible failure progression. Investigated responses are the load increase in the hangers, the deformations of the beam, and the bending moments in the girder. It is evaluated which number of hanger failures are bearable by the bridge. The decisive response is the force in the hanger adjacent to the failed ones, because an exceeding of its load carrying capacity triggers a failure progression.

The analyses are performed nonlinear dynamic according to the procedure outlined in UFC (2009), even if this procedure is not intended for the investigation of bridges. Damping is neglected and only the first vibration cycle is taken into account.

The investigated parameters are the bending stiffness of the beam (symbolised by the girder height), the hanger spacing, and the hanger length (to cover different locations of the girder). First, the parameters are varied individually and the remaining values
are taken from a reference model (with girder height $H_G=4.6m$, hanger spacing $d_H=15m$, and hanger length $l_H=84m$). In addition, combinations are considered to investigate the interactions between parameters.

First, the girder height ($H_G$) is varied in 1m steps in the span of 2m to 5m. Second, the hanger spacing ($d_H$) is varied in 7.5m steps in the range of 7.5m to 30m. These values are chosen to cover the bending stiffness range of real suspension bridge girders and the real built hanger spacing. Third, the hanger length ($l_H$) is varied to the values 42m, 84m and 126m to investigate the influence of the location along the bridge girder.

Additionally, three combinations are compared to investigate the interaction of the parameters. These combinations are the reference model $S_0$ ($H_G=4.6m$, $d_H=15m$, $l_H=84m$), the flexible model $S_1$ with nearly the half of the girder height and the hanger spacing length (2.0m/7.5m/84m) and the very stiff model $S_2$ with double girder height and hanger spacing length (9m/30m/84m). The last combination is unrealistically stiff but is useful to discuss the different influencing factors.

### Table 1. Girder height

<table>
<thead>
<tr>
<th>$H_G$</th>
<th>maximum hanger force</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td>MN</td>
</tr>
<tr>
<td>2.0</td>
<td>7.3</td>
</tr>
<tr>
<td>3.0</td>
<td>7.3</td>
</tr>
<tr>
<td>4.0</td>
<td>7.3</td>
</tr>
<tr>
<td>4.6</td>
<td>7.3</td>
</tr>
<tr>
<td>5.0</td>
<td>7.3</td>
</tr>
</tbody>
</table>

Legend: + hanger survives, ~ hanger might fail, – hanger fails; percentage values mark the change to the pristine state; nom denotes the nominal capacity of the hangers; bold face indicates the values of the reference model $S_0$

### Table 2. Hanger spacing

<table>
<thead>
<tr>
<th>$d_H$</th>
<th>maximum hanger force</th>
</tr>
</thead>
<tbody>
<tr>
<td>m</td>
<td>MN</td>
</tr>
<tr>
<td>7.5</td>
<td>4.3</td>
</tr>
<tr>
<td>15.0</td>
<td>7.3</td>
</tr>
<tr>
<td>22.5</td>
<td>10.3</td>
</tr>
<tr>
<td>30.0</td>
<td>13.4</td>
</tr>
</tbody>
</table>

### Table 3. Hanger length

<table>
<thead>
<tr>
<th>$l_H$</th>
<th>maximum hanger force</th>
</tr>
</thead>
<tbody>
<tr>
<td>M</td>
<td>MN</td>
</tr>
<tr>
<td>42</td>
<td>7.3</td>
</tr>
<tr>
<td>84</td>
<td>7.3</td>
</tr>
<tr>
<td>126</td>
<td>7.3</td>
</tr>
</tbody>
</table>

### Table 4. Combinations

<table>
<thead>
<tr>
<th>Model</th>
<th>maximum hanger force</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nom</td>
<td>MN</td>
</tr>
<tr>
<td>S1</td>
<td>4.2</td>
</tr>
<tr>
<td>S0</td>
<td>7.3</td>
</tr>
<tr>
<td>S2</td>
<td>13.9</td>
</tr>
</tbody>
</table>

### 6.3 Results

The investigation of the effect of the beam height, and thus the bending stiffness of the bridge girder, demonstrated that in the event of hanger failure the resulting maximum hanger forces (Table 1) and vertical displacements decrease while the maximum bending moments and the affected girder range increase with increasing bending stiffness. A stiff girder is able to redistribute the additional load from hanger failure across a longer range to other hangers. Thus, the additional load to the adjacent hangers is comparatively small. On the other hand, a relatively flexible girder only affects a small number of hangers due to large local deflections. Because of the comparatively low resistance against deflection, girders with a low beam height lead to smaller maximum bending moments. As the maximum hanger force is the decisive factor for the robustness assessment of a suspension bridge, it can be reasoned that the robustness increases with increasing flexural stiffness of the girder.

Increasing the hanger spacing leads to increased maximum hanger force (Table 2), girder deflection, bending moment and affected girder range. The number of bearable hanger failures decreases with increased hanger spacing. Note that outlined behaviours of the bridge might not directly be comparable because the initial damage is not comparable. When applying the unsupported girder length as initial damage, it can be seen that, independent from the hanger spacing, the failure progression is triggered by an unsupported length of 60m. Thus, with comparable initial damage, the hanger spacing is not directly decisive for the robustness of the bridge (but in real suspension bridges, the hanger spacing influences the girders stiffness). On the other hand, a straying vehicle might destroy only one hanger, independent from their spacing. In this case, a small hanger spacing would be preferable.

Varying the hanger length indicates that the affected girder range increases with the hanger length, that is, with more flexible supports. This improves the load redistribution to the adjoining hangers. Accordingly, the maximum hanger force decreases with the hanger length (Table 3). However, the maximum values of the deflection and the bending moments increase with the hanger length. The robustness thus increases with the length of the hangers. But, even when applying extreme values for the hanger length, this effect seems to be comparatively small and is considerably depending on the flexural stiffness of the girder. The results of varying the hanger length imply that a suspension bridge is more susceptible to a hanger failure in the middle of the main span, where the hangers are short, than in the area around the pylons, where the hangers are long. However, in real suspension bridges, the main cables add additional flexibility to the short hangers at mid-span.
which is not captured by the investigations performed here due to the simplified model.

The results outlined in the previous paragraph confirm and complete the analyses of Giuliani (2009; also Giuliani and Bontempi 2008). There, a model of the Messina Strait suspension bridge is analysed for hanger failures at different locations of the girder. It is concluded, that in the middle of the main span, where the hangers are very short, the lowest number of hanger failures is bearable, whereas near the pylons, where the hangers are very long, two more hangers could fail before triggering a progressive collapse. Another interesting aspect shown by Giuliani (2009) is that the progression of the collapse accelerates in direction of short hangers due to the missing ductility of short hangers. Thus, in the case that the collapse is triggered near the pylons, the progression accelerates on its way to the middle of the main span.

To counteract the larger susceptible in the middle of the bridge, the shortest hangers should be kept as long as possible. This can either be done by reducing the sag of the main cable, which in consequence will affect the forces in the main cables and the pylons, or, to avoid this, by enhancing the pylon height at unchanged sag ratio of the main cable and thus increasing the length of the hangers by the additional pylon height.

According to the results presented above, a suspension bridge should generally be designed so that the bending stiffness of the girder is maximised and the hanger spacing is minimised. These general recommendations lead to uneconomically financial expenses. The parameters girder stiffness and hanger spacing have to be adapted to each other. The analysis of the three models (Table 4) showed that a by trend more flexible girder and a by trend closer hanger spacing compared to a very stiff girder in combination with a wide hanger spacing has to be preferred with regard to structural robustness. This effect should not hide the fact that a narrow hanger spacing may lead to a higher number of failing hangers as a result of the same cause due to the decreased distance and a smaller hanger diameter which leads to a decreased local resistance (Nair 2006).

6.4 Elastic length

Insight into the load redistribution within the bridge system is fundamental for the understanding of the static and dynamic load bearing behaviour after failure of one or more hangers (Wolff 2010). The redistribution of the force of the failed hanger to adjacent hangers occurs predominantly by deflection of the bridge girder. The bridge girder is elastically supported by the hangers. The spring stiffness of the support depends on the extensional stiffness and the length of the hangers. The girder can be modelled as continuously elastically supported beam (Wolff 2010) (see Figure 3). The bedding modulus results from smearing the discrete spring stiffness of the hangers. Due to the varying length of the hangers in longitudinal direction of the bridge, the bedding modulus would not be constant. However, when taking into account only a short section of the bridge girder the hanger length can be assumed as constant.

Figure 3. Continuously elastically supported beam

The main characteristic of a continuously elastically supported beam is the so-called elastic length (Wolff 2010, Chiaia and Masoero 2007):

\[ L = \sqrt{\frac{4Et l}{K_{h}}} \]  

(1)

The longer the elastic length is, the higher is the number of hangers that are involved in the bearing of a discrete load, and the failure of a hanger cable can actually be seen as such a discrete load. The bedding modulus \( K_{h} \) results from the smeared spring stiffness of the individual hangers \( k_{h} \):

\[ k_{h} = \frac{E_{G}A_{h}}{l_{h}} \] \quad and \quad \[ K_{h} = \frac{k_{h}}{d_{h}} \]  

(2), (3)

This leads to:

\[ L = \sqrt{\frac{4E_{G}I_{G}}{E_{h}A_{h}}} \cdot d_{h} \cdot l_{h} \] \quad , with \( I_{G} = f(H_{G}) \)  

(4)

The elastic length depends on the ratio of the modulus of elasticity of the girder \( E_{G} \) and the hanger \( E_{h} \), the girder’s moment of inertia \( I_{G} \) (which actually depends on the girder’s height \( H_{G} \)), the cross-section of the hangers \( A_{h} \), the hanger spacing \( d_{h} \) and finally on the hanger length \( l_{h} \). (The parameters investigated in the study presented above are \( H_{G} \), \( d_{h} \), and \( l_{h} \).) With

\[ A_{h} = \gamma \cdot \frac{F_{h}}{\sigma_{h}} \] \quad and \quad \[ F_{h} = d_{h} \cdot (g + q) \] \quad and \quad \[ g = g_{G} + g_{2} = A_{G} \rho_{\text{steel}} + g_{2}, \] \quad with \( A_{G} = f(H_{G}) \)  

(5), (6), (7)

follows, that \( A_{h} \) depends on \( d_{h} \) (but is also dependent indirectly on \( H_{G} \) due to the dead load \( g \) of the girder). Hereby follows:
It can be seen from this equation that the hanger spacing \( d_H \) does not directly affect the elastic length but indirectly through \( I_G \) and \( g \) which are governed by \( H_G \) whereas \( H_G \) and \( d_H \) are interacting parameters in economic design. However, the hanger spacing influences the unsupported length in the case of a hanger failure. At comparable initial damages (constant unsupported length), the hanger spacing is of no direct concern. This perception can also be found in the performed parameter study. Also the other results of the parameter study can be confirmed by help of the elastic length: The higher the girder—and thus the bending stiffness—the longer is the elastic length. Thus, more hangers participate in the load bearing, and the load concentration in the most heavily loaded hangers (adjacent to the failed hanger) decreases. This can also be said for the hanger length, though in the parameter study, this effect was smaller than the effect from the girder height. This is because the girder height incurs decisively with a power of two in the calculation of the moment of inertia and thus the elastic length, whereas the hanger length incurs only with a power of one.

7 SEGMENTATION

The unsupported length of the girder increases continuously during the zipper-type failure of the hangers. This leads to an increasing force in the hangers particularly directly adjacent to the unsupported part. If this girder part does not separate from the remaining girder, the collapse will proceed over the whole bridge length. An uncontrolled separation can be observed in the case of the Tacoma Narrows Bridge. A controlled separation could be realised by introducing hinges in certain distances of the girder to ensure that the girder can separate at these points and thus the collapse stops (assuming that the remaining structure is able to withstand the occurring dynamic forces).

In the simplest way such hinges can be realised by break-away (rotation) hinges. However, such hinges might considerably reduce the capability of the bridge girder for load redistribution. The manner hinges influence the behaviour of the bridge after the failure of one hanger is studied with a girder of height \( H_G = 3 \) m and a hanger spacing \( d_H = 15 \) m. The investigated segment sizes and the relative locations of hanger failures are shown in Figure 4. The corresponding resulting maximum hanger forces are summarised in Table 5.

In the case of no hinges, the failure of one hanger is bearable with a utilisation of the adjacent hanger of 81\% (Table 5). The introduction of hinges always results in higher ratios of utilisation; a considerable dependency of the relative location of the initially failing hanger in the segment could be observed. Most disadvantageous is the failure of a hanger directly at a hinge (location A). In this case, depending on the construction of the hinge, it might also happen that two segments fall simultaneously. Even the failure of one hanger can become not bearable when introducing hinges. Furthermore, there is the tendency of increasing hanger utilization with smaller segments. This can be explained by the fact that the number of hangers participating in the load redistribution decreases due to more locally occurring deflections of the girder.

The arrangement of hinges in the bridge girder thus increases the susceptibility to hanger failure. However, what does this imply for the robustness of the bridge?

On the one hand, the chance of a progressive hanger failure is higher in every model with hinges. On the other hand, the triggered progressive collapse stops at the segment boarders, limiting the resulting damage to that one (or maximum two) segment(s). If there are no hinges introduced, the chance of triggering a progressive collapse will be smaller due to the smaller degree of utilization of the hangers; however, in the worst case, the collapse will proceed until the ends of the bridge.

The introduction of hinges could increase the robustness of a suspension bridge because the resulting damage can be held comparably small and is concentrated more locally. On the other hand, it has to
be considered that the failure of some hangers can lead to the loss of one whole segment, whereas in the system without hinges such an initial damage would have been bearable. In this case, the resulting damage is increased by the hinges. But, beyond a certain extent of initial damage (i.e. in the analysed example the loss of two hangers), the hinges limit the resulting damage to one (or two) segment(s), whereas without them the whole bridge girder peels off the hangers (assuming that an uncontrolled separation does not occur).

It follows that the hinges ideally should not be there until the progressive collapse finally is triggered. It should start working as a hinge just before it is reached by the successive hanger failure to deter the collapse from progressing. Prior to that, the hinge has to react like the normal cross-section of the girder (nearly bending resistant connection of the two girder segments) to assure that there is no affection of the load redistribution after the failure of the first hangers (i.e. the number of failing hangers not triggering a progressive collapse in the system without hinges).

Such a hinge design could possibly be realised with a mechanic unlocking when the adjacent hanger fails and thus the hanger-girder-connection loses its tension force (mechanism of sliders and locking bars). A mechanic release can also be realised with a bar/bolt that shears off in the case that the bending moment in the hinge has reached a certain value. Utilising the plastic rotation of the girder for realising a break-away hinge is not a usable approach because the unsupported girder segment does not disconnect from the remaining girder until a certain rotation is reached; the overloading of the adjacent hanger is likely to occur before this rotation is reached. Additionally, such an approach might introduce tension forces in the girder due to catenary action in the unsupported segment.

8 CONCLUSIONS

The critical components of a suspension bridge are the hangers because they are vulnerable and exposed. The failure of one (or a few) hangers can lead to a progressive collapse of the whole bridge. On the one hand, explicit design measures could be taken to prevent a hanger failure. On the other hand, the collapse progression can be prevented by ensuring robustness.

The robustness of a suspension bridge can be enhanced by providing alternative load paths within the suspension system or by segmenting the bridge girder or the suspension system. These options were discussed.

The investigation concerning alternative load paths showed that the robustness of a suspension bridge increases when the flexural stiffness of the girder is increased or the hanger spacing is decreased (assuming unchanged girder stiffness). However, girder stiffness and hanger spacing are interdependent parameters. It has further to be investigated which of these parameters is more important. Additionally, it turned out that the failure of long hangers is less critical as the failure of short hangers.

The investigation of the segmentation method showed that introducing hinges in the girder reduces the robustness for small initial damage but increases the robustness for large initial damage. It turned out that the hinges should only develop in the case of imminent collapse progression. Until that point, the segment borders should behave like a monolithic connection.

Research on the robustness of suspension bridges is at its very beginning. Some issues have been addressed and clarified here, further questions are open for further research.

Concerning the investigation of the robustness with respect to the alternative load path method, at first, the design of the suspension system must be revised. The investigations should be repeated with an aerodynamic box girder to ensure the transferability to actually built suspension bridges concerning the range of the girder bending stiffness. Additionally, the modelling of the bridge must be refined to get more realistic results.

Concerning the segmentation method, the effect of zipper stoppers (strong hangers) and the hinge detailing (structural fuse) are topics of ongoing work.

REFERENCES


