Avoiding Disproportionate Collapse of Major Bridges

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Summary
Avoiding disproportionate collapse following some small triggering event is an important aspect in the design of major bridges. A general approach for designing structures against disproportionate collapse is outlined and applied to bridges. Compared with buildings, bridges are primarily horizontally aligned structures with one main axis of extension. The provision of alternative load paths, therefore, is often not only more difficult but also less important. It is found that continuous girder bridges are preferably made collapse resistant by segmentation—which can require the insertion of joints or hinges—or by reducing the probability of key element failure. In cable-stayed or suspension bridges, the stay cables or hangers are the key elements that are particularly vulnerable. A collapse triggered by the loss of a cable is prevented by designing the bridge for loss-of-cable load cases; corresponding nonlinear dynamic analyses are presented. This measure should be complemented by protecting the cables against vehicle impact and malicious action. The robustness of suspension bridges can be raised by the methods of segmentation and alternative paths. The protection of the components of the suspension cables against malicious action deserves particular attention. Arch bridges have similarities with suspension bridges and much of the respective statements apply.

Keywords: bridges; accidental circumstances; local failure; progressive collapse; design; concepts.

Introduction
Disproportionate collapse occurs when an initial local failure that is produced by some small triggering event (accidental circumstances) leads to widespread failure of other structural components such that a major part of the structure collapses. Such a phenomenon is also referred to as progressive collapse. Research on disproportionate or progressive collapse dates back to the 1970s; it has intensified since the 1995 bombing of the Alfred P. Murrah Federal Building in Oklahoma City and the events of 11 September 2001. These efforts have not yet led, though, to a consolidated and generally agreed set of nomenclature and procedures.

Guidance for the practicing design engineer is gradually evolving but is still limited to particular types of structures. Detailed design rules for buildings have been issued in the United States. These guidelines include both prescriptive and direct design approaches, the latter comprising the specific-local-resistance method and the alternative-path method. The Blue Ribbon Panel on Bridge and Tunnel Security authored recommendations to reduce the susceptibility of bridges and tunnels to malicious action, which are more directed toward policy makers, however, and less toward practitioners. For the design of cable-stayed bridges, the Post-Tensioning Institute (PTI) recommends that the sudden loss of one cable shall not lead to structural instability and specifies a loss-of-cable load case. Application of such a load case in design corresponds to the alternative-path method. The possibility of isolating an incipient collapse through segmentation—a direct design approach used for the Confederation Bridge, Canada—is not included in current guidelines.

A pragmatic and general approach for designing against disproportionate collapse is presented in recent publications by the author. Definitions are proposed, the shortcomings of current design methods are exposed, a framework of design criteria is presented, and various design methods are compared in them. These more general findings and concepts are briefly outlined here and then applied to bridges. The focus is on structures of large longitudinal extension, including but not limited to long-span bridges. The paper opens with the reminiscence of some bridge failures.

Failure Incidents
Descriptions and investigations of failures can be an important basis for research. In contrast to recent building collapses, which are well documented and analyzed, there are relatively few publications on the collapse of bridges. The Viadotto Cannavino, Italy, collapsed during construction in 1972. The triggering event was a formwork failure. The ensuing collapse was due to lack of robustness of the structural system (as defined in the following section). Another disproportionate collapse occurred during the construction of the Haeng-Ju Grand Bridge, Seoul, in 1992. After the failure of one span of that continuous prestressed concrete girder bridge, the collapse progressed through the adjacent ten spans, and an 800 m section of bridge was lost. In both cases, the continuous prestressing tendons in the superstructure of the bridge played a particular and disastrous role. When the Haeng-Ju Grand Bridge collapsed, most tendons resisted the enormous stresses caused by the rupture of the encasing concrete and the failure and fall of structural elements. The forces transmitted by the continuous prestressing tendons thus enabled the collapse to progress. Remarkably, it came to a halt at the expansion joints at both ends of the 800 m section.

A different course of events took place during the failure of the Tasman Bridge near Hobart, Australia, in 1975. Because of a ship impact, two piers of the bridge were destroyed and three spans of deck supported by them collapsed. The other 19 spans remained intact. The bridge deck was constructed of precast prestressed concrete beams. The nonoccurrence of collapse progression and, thus, the robustness of that bridge were related to the discontinuity of prestressing tendons between adjacent spans.
The collapse of the I-35W Mississippi River Bridge in Minneapolis, on the other hand, has partly been ascribed to the absence of alternative load paths within the structural system of that bridge.1 Because alternative paths require continuity, this case seems to indicate that the robustness of a system increases with its degree of continuity. On the basis of the previous examples, however, a contrary conclusion can be reached. The resolution of this apparent paradox requires a careful discussion of the term robustness.

Robustness and Collapse Resistance

The term robustness is defined as insensitivity to local failure, where insensitivity and local failure are quantified by the design objectives, which are part of the design criteria outlined below.12 Following this definition, robustness is a property of the structure alone and independent of the cause and probability of initial local failure. The term collapse resistance is defined as insensitivity to accidental circumstances. Again, the accidental circumstances, which comprise low-probability and unforeseeable events, are quantified by the design objectives. Collapse resistance is a property that is influenced by numerous factors including both structural features and the conditions for initial failure. One avenue for achieving collapse resistance—which is the actual design intent—is to make the structure robust.

The Inadequacy of Current Design Methods

Modern design codes are based on reliability theory and are generally believed to rest on a sound mathematical foundation. Nevertheless, these methods fail with regard to the identification and proper treatment of a potential for disproportionate collapse. There are three reasons for this failure. First, the design equations are defined and applied on a local level only (check of cross-sectional forces or element stability). Structural safety, therefore, is likewise accounted for on a local level only. Second, low-probability and unforeseeable events are not taken into account. This simplification is inadmissible for nonrobust structures where low probabilities of local failure can combine into a much larger probability of global failure. Third, the underlying probabilistic concept requires specification of an admissible probability of failure. Considering the extreme losses that can result from disproportionate collapse, it is difficult to reach a consensus on the numerical value of the admissible probability of such an outcome.

Pragmatic Design Approach

The shortcomings of current design methods can arguably be only partly overcome within the framework of reliability theory. A general and pragmatic design approach is therefore suggested in which probability-based design, as described in the codes or in direct application of reliability theory, is complemented by additional assessment and measures with particular regard to disproportionate collapse. These additional considerations are governed by design criteria that satisfy applicable codes or are agreed upon by the parties involved in a particular project. The design criteria include the design objectives, which are, in particular, the allowable accidental circumstances, the allowable cases of initial local failure, and the acceptable extent of collapse and other damage. The currently available design strategies and methods to prevent disproportionate collapse are as follows:

1. Prevent local failure of key elements (direct design)
   (a) Specific local resistance
   (b) Non-structural protective measures
2. Assume local failure (direct design)
   (a) Alternative load paths
   (b) Isolation by segmentation
3. Prescriptive design rules (indirect design)

Some of these methods are well known;1–3; the segmentation approach has been introduced relatively recently.11,12 A structural element is identified as a key element if it is not larger than the structural part assumed to initially fail and its failure produces an extent of collapse larger than acceptable. The type of collapse can vary.17 The direct design methods are addressed in the following sections. A detailed presentation is given elsewhere.13 The prediction of the structural response to local failure—needed for identifying key elements and for applying the design strategy “assume local failure”—requires suitable verification procedures. It is suggested to use deterministic structural analyses, which should be as accurate as possible and consider all relevant scenarios. In general, nonlinear dynamic analyses are required. These may become extremely cumbersome though even intractable depending on the type of collapse. Cable-stayed bridges might fail in a zipper-like fashion, which is arguably the type of collapse most accessible to analysis. A corresponding computation was performed for a particular cable-stayed bridge; the results are presented later in this paper.

Design Strategies

Local Failure: Prevent or Assume?

Direct design can be based either on trying to prevent an initial local failure or on designing for such a case. If collapse resistance is to be achieved by preventing the local failure of key elements, the specific-local-resistance method and non-structural protective measures can be considered to provide high safety against local failure. These methods do not aim at enhancing robustness. On the other hand, if local failure is assumed, the alternative-path method or the segmentation method can be pursued to make the structure robust and to limit incipient collapse to an acceptable extent.

A Case Study: Isolation by Segmentation

The segmentation approach was used in the design of the Confederation Bridge, Canada, a prestressed concrete bridge that consists of 43 continuous main spans of 250 m each and shorter approach spans (Fig. 1).11,18 If this structure were to be made robust and collapse resistant by the alternative-path method, an initial local failure to be assumed would be the failure of a bridge pier (Fig. 2). This in turn would require designing a prestressed concrete frame with a double span length of 500 m—arguably a vain endeavour. The design method chosen was to limit the local failure by isolating the collapse. In consultation with the supervising authority, Public Works Canada, the acceptable extent of collapse was determined and, based thereupon, the location of collapse boundaries was derived (Fig. 2, pier D and hinge H1). The collapse must not transgress these boundaries; the collapsing part of the structure is thus isolated from the remaining structure. The structure is segmented by the collapse boundaries, which are chosen
on the basis of the design objectives. This choice of design objectives and collapse boundaries here corresponds to the minimum extent of collapse that could reasonably be achieved considering the available design options. In general terms, this means that the design objectives can often be specified on a project-by-project basis only.\textsuperscript{13}

The segmentation approach requires an investigation of the remaining structure for the loads resulting from partial collapse. Special attention has to be paid to the structural elements that form the segment borders (in the case considered, pier D and hinge H1); they isolate the collapse and their proper functioning is decisive for the efficiency of the approach. One design possibility is to provide these elements with high local resistance. Among other scenarios, the response of the structure to the left of hinge H2 after a sudden loss of support at that hinge was investigated. According to the preliminary design, the region between hinge H2 and pier D was intended to be composed of a cantilever beam and a precast drop-in girder monolithically connected to the cantilever tip by a cast-in-place joint and continuous prestressing tendons. The sequence of collapse is marked by several distinct events that were identified in static and dynamic analyses. With each new event, the analytical results increasingly depended on modelling assumptions. Particular uncertainty arose over the behaviour of the continuous prestressing tendons. Verification of adequate resistance of the segment border (pier D), and thus verification of collapse resistance, turned out to be impossible because of both the high dynamic loading and the analytical uncertainties.

Both problems can be mitigated by selectively eliminating continuity. By inserting joints, breakaway hinges or structural fuses, or by providing plastic hinges, the loading of the segment borders is reduced and the analysis is simplified. In the case of the Confederation Bridge, it proved particularly important to interrupt the continuity of the prestressing tendons to allow for an early separation of the falling drop-in girder from the remaining structure. Otherwise, the collapse could progress into the adjacent span (Fig. 2, left of pier D), and then further on. The possibility of such an outcome is documented by

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{Fig1.png}
\caption{Confederation Bridge, Canada (Photo: Roger Bragg, Auckland)}
\end{figure}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{Fig2.png}
\caption{Assumed cases of initial local failure and acceptable extent of collapse}
\end{figure}
the collapse of the Haeng-Ju Grand Bridge as described previously. It was attempted to design a structural fuse within the cast-in-place joint between cantilever and drop-in girder. No safe way of automatically cutting the continuity tendons after collapse onset was found, however, and the idea was abandoned.

The preliminary design was therefore changed by inserting one additional hinge in every second span. Instead of using monolithic cast-in-place joints, the drop-in girder in those spans was connected to both cantilevers by hinges (Fig. 3). The additional hinge becomes the new segment border (instead of pier D); it isolates an incipient collapse acting like the expansion joints in the Haeng-Ju Grand Bridge. If hinge H2 fails, the drop-in girder extending between the inserted hinge and the hinge H2 will fall and disengage from the remaining structure in a predictable way: the inserted hinge will break away; the disengagement of the drop-in girder is forced and defined by the geometry of the hinge corbel (Fig. 4). To assure final separation before the free end of the drop-in girder hits the water, its length was reduced, a change that also proved beneficial regarding the loading on the remaining structure.

Verification of the remaining structure was performed for the load impulse acting on the cantilever tip during the fall and disengagement of the drop-in girder. For the sake of simplicity and because of modelling uncertainties, single-degree-of-freedom response spectra for the acting load impulse were developed. Based thereupon, an overall dynamic amplification factor was derived and a quasi-static analysis was performed. The formation of plastic hinges was deemed acceptable for this load case, and the plastic reserves of the structure were utilized. A detailed account of the progressive collapse study undertaken during the design of the Confederation Bridge is given in Ref. [18]; an abridged version can be found in Ref. [11].

This case study will be concluded by some more general comments. Limiting the consequences of a local failure without consideration of the possible cause or the probability of its occurrence corresponds to the design strategy "assume local failure" that is reflected in some standards and guidelines already.39 Accomplishing this goal by structural segmentation is a design option that has occasionally been used but not yet codified. Adopting the segmentation method for structural design can lead to the insertion of joints. Such a measure reduces the degree of static indeterminacy and thus the level of continuity. In the Confederation Bridge, the ability to activate alternative load paths, that is, its structural redundancy, was not lessened, however, by this measure because it did not remove any feasible alternative path—having in mind that designing a 500 m span, even though a theoretical option offered by the original structural system, was impracticable. Furthermore, the robustness of the structure, that is, its insensitivity to local failure, was increased. This shows that associating continuity with redundancy and equating redundancy with robustness, even if valid for particular types of structures, is not generally justified. These terms should be carefully distinguished.

**Fig. 3: Insertion of a hinge in every second span**

**Fig. 4: Forced disengagement of drop-in girder at hinge**

**Segmentation versus Alternative Paths**

It is noted that the continuity required for the formation of alternative paths may, in certain circumstances, not prevent but promote collapse progression. This can be the case when the alternative paths are not provided with the strength required to withstand the forces transmitted by continuity. This remark also applies to prescriptive design rules that are based on the idea to provide alternative paths by increasing continuity. If it is impracticable to provide alternative paths with sufficient strength, the segmentation method (implemented, if necessary, by selectively eliminating continuity) has the advantage. This is also the case if alternative paths (or segment borders) are strong enough but the corresponding verification proves difficult or unconvincing.

The alternative-path method, however, has the advantage if the fall of structural components or debris must be prevented. This applies to cases in which falling parts could strike key elements of the remaining structure because the impact loading produced by such an event is difficult to design for. Such conditions are found in structures with primarily vertical alignment such as high-rise buildings; they are less typical for horizontally aligned structures. The efficiency of the alternative-path method decreases with an increase in initial failure size. The efficiency of the segmentation method tends to be insensitive to this parameter, although, in this case, a comparatively large extent of collapse must be accepted. Both methods can be combined.

**Prevent Local Failure**

The methods discussed in the previous two sections aim at achieving collapse resistance through structural robustness. Another way of achieving collapse resistance is to provide maximum safety against failure of key elements. This high level of local safety is preferably assured by providing specific local resistance in the key elements. If specific local resistance cannot be achieved or would require disproportionate effort, an enhanced safety of key elements can also be provided by nonstructural protective measures. Such measures include protection against vehicle impact or bomb blast, limitation or control of public access, and other protective measures like aerial surveillance or anti-aircraft defence. In either case, the measures to be taken are guided by the assumable accidental circumstances established for a given project.

Ensuring high safety against local failure requires more than the use of high design loads or the recourse to measures which protect against mechanical action. More thought is required concerning the structural resistance. Safety factors for material and soil resistance must be chosen higher than usual; the soil exploration should be done with particular care; structural design and construction of all key elements should meet stringent quality requirements; the site engineer should be aware of all key elements identified in the design process. However, local failure can also be caused by corrosion or fire—occurrences that are more effectively countered by corrosion protection, regular inspection, fire protection, and fire fighting systems than by structural measures.

The development and codification of equivalent loads (accidental design actions) has started but is incomplete. Eurocode 1, Part 1–7 specifies accidental design actions for some specific accidental circumstances, that is, internal explosions and impact of road vehicles, rail traffic, ships or helicopters. Comprehensive provisions on the impact of road vehicles on structures are given in a Swiss guideline. Further, the construction stages are of importance although they can hardly be covered in a standardized way. Temporary bracings and auxiliary piers can become key elements. Instead of specifying equivalent loads, it might be more expedient to generally increase the safety factors or to prevent the failure of key elements due to accidental actions, such as falling construction equipment or collapsing framework, by using loads specifically determined for such occurrences. If such an approach proves impractical, the onsite safety requirements for the construction stages identified as crucial should be raised.

It must be noted that although high safety against local failure can be produced, this safety cannot be absolute and in face of unknown future actions may not even be as high as hoped for. Moreover, an inherent lack of robustness of a structure is not eliminated. Nevertheless, application of the design strategy “prevent local failure” is justified when the significance and exposure of the structure are not extremely high or when other methods are inapplicable—provided the key elements are clearly and fully identifiable. This strategy will be cost-effective if the critical areas of the structure and the number of key elements are small. If it is to be applied to structures of extremely high significance and exposure, however, the decision to do so should not rest with the engineer alone. Instead, a public and political consensus on the acceptability of the residual risk must be reached.

**Design of Collapse-Resistant Bridges**

The few papers on collapse-resistant design of bridges published to date mostly focus on preventing local failure. The preceding discussion shows, however, that there is a set of design methods to choose from or to combine when a structure is to be designed against disproportionate collapse. It was seen that the suitability of the various design methods depends on the design objectives and on the type of structure and its alignment in space. The understanding of these dependencies is still rudimentary to date. A characteristic feature of bridges is that they are primarily horizontally aligned structures. Compared with buildings, there is less need to provide alternative paths to prevent the fall and impact of structural components. This is just as well as it is often difficult to provide alternative paths in structures that have one main axis of extension, such as bridges.

Other common properties shared by large bridges, but also by high-rise buildings, are large internal forces and a high degree of structural interaction. Furthermore, such structures are mostly unique and expensive. It is concluded that for such structures direct design methods are mostly preferable in terms of safety and economy over prescriptive design rules. Other common properties and conclusions with respect to avoiding disproportionate collapse are difficult to find. The subsequent discussion therefore focuses on particular bridge types. The level of detail varies among the bridge types according to the different levels of analysis performed so far.

**Continuous Girder Bridges**

Relevant examples considered above are the Viadotto Cannavino and the Haeng-Ju Grand Bridge, which collapsed in a progressive manner following an initial failure, the Tasman Bridge, which did not, and the Confederation Bridge, which was designed against disproportionate collapse. On the basis of this experience and the preceding discussion, it is concluded that there are two design approaches to convey collapse resistance to continuous girder bridges. On the one hand, the segmentation method can be used. For the bridge type considered here, the segmentation borders are the regions around selected piers and those piers themselves. For short spans up to, say, 40 m, it will be possible to provide these elements with high local resistance to enable them to act as segment borders. For longer spans, the segmentation approach requires selective elimination of continuity at the envisaged segment borders (Fig. 3). One way of doing so is the insertion of breakaway hinges (Fig. 4).

On the other hand, an attempt can be made to prevent local failure. Thought must then be given to possible accidental circumstances that endanger key elements. Such circumstances should be minimized or designed for. They can include a ship or an aircraft that crashes into the bridge, an unexpected thick ice formations that collide with a bridge pier, fire caused by a traffic accident that damages the cantilever tendons in the top slab, a bomb explosion at a vulnerable location, other accident-related or malicious actions, design and construction flaws, or simply corrosion. The probability of ship impact is regularly evaluated by specialist consultants when designing major bridges. The decision to protect the bridge piers through strengthening or impact-resistant barriers is determined from such an assessment. The normal design loads for ship collision and environmental loading should be increased when designing key elements. Construction stages must likewise be considered as noted before. For small to medium-sized bridges (up to, say, 1000 m long), this design strategy might be preferable to the segmentation approach.

**Cable-Stayed Bridges**

This bridge type is a good illustration of the statement made above that the terms continuity, redundancy, and robustness should be carefully distinguished. Although such structures possess a high degree of static indeterminacy and internal continuity, this does not mean that they are redundant or robust. The sudden loss
of one or more cables and a possibly ensuing zipper-like collapse is of particular concern. The problem is aggravated by a number of factors: the cables are easily accessible and exposed to accident-related or malicious action (Fig. 5); their cross section is relatively small which makes it difficult to provide them with specific local resistance; a cable loss could happen nearly instantaneously producing an impulsive dynamic loading; the unzipping of cables is likely to interact with and be reinforced by a stability failure of the bridge girder that is in compression; the system might respond in a nonductile manner.

To meet such concerns, the PTI recommends that a cable-stayed bridge shall be capable of withstanding the loss of any one cable. A loss-of-cable load case, an associated load combination, and applicable partial safety factors are specified. The impulsive dynamic loading resulting from the sudden loss of a cable is recommended to be determined in a quasi-static analysis using a dynamic amplification factor (DAF) of 2.0. Application of such a recommendation in the design of recently erected cable-stayed bridges in the United States showed that the loss-of-cable load case can become a controlling requirement for the design of the stiffening girder increasing construction costs. For this reason, the recommendation that a DAF determined by nonlinear dynamic analysis, but not smaller than 1.5, can be used as an alternative was added to the fifth edition of the PTI Recommendations.

Numerical Study of Loss-of-Cable Load Cases

Numerical studies were performed for a cable-stayed bridge with a main span of 600 m, side spans of 210 m, and end spans of 60 m (Fig. 6). The two cable planes are slightly inclined in the transverse direction. They consist of 80 cables each in semi-fan configuration. The cable anchors in the stiffening girder are spaced at 15 m. The latter consists of an orthotropic deck supported on two steel edge beams with a height of 2,60 m. The effect of loss-of-cable load cases is investigated in nonlinear dynamic analyses of a spatial model of the structure. The loss of any one cable is modelled as a sudden failure and the ensuing extreme responses of state variables (displacements and sectional forces) are determined. By relating these extreme dynamic responses to the corresponding static responses to cable loss, DAFs are derived that can be used in quasi-static analyses. Because the comparison of dynamic and static responses is done case by case for individual cable losses and state variables, the resulting DAF is valid for the considered case only and will generally be different for other cable losses and state variables.

The numerical results confirm that it is impossible to specify a global DAF. The DAF strongly depends on both the location of the failing cable and the location and nature of the considered state variable. For the vertical displacement of the stiffening girder in the vicinity of the failing cable, the DAF is between 1.5 and 1.8. The DAF is between 1.3 and 1.6 for the positive bending moments—and between 1.4 and 2.7 for the negative bending moments—in the stiffening girder in the vicinity of the failing cable (Fig. 7).
(More precisely, these values refer to the bending moments in the stiffening girder edge beam in the plane of cable failure.) For the forces in the cables adjacent to the failing cable, the DAF is between 1,35 and 2,0. The DAFs for state variables at locations distant from the failing cable are often much larger than 2,0. This, however, results from the generally small static response at distant locations—and not from a particular large or controlling dynamic response. The numerical values of DAFs just specified were determined for an undamped system. The attenuating effect of damping was found to be small.

Remarkably, the DAFs for the bending moments in the pylons are found to be much larger than 2,0 (i.e., by one order of magnitude). Again, the corresponding static responses are small. Contrary to the findings for the stiffening girder, however, the dynamic responses are also large and can become controlling (Fig. 8). These results were obtained for an undamped system. The effect of damping was found to be more pronounced for the pylon moments than for the other state variables. When assuming a damping ratio-to-critical of 1%, the design bending moments are reduced by approximately 15%. These findings indicate that, for the bending moments in the pylons due to cable failure, a quasi-static analysis with a DAF of 2,0 can be unsafe and should be replaced by proper dynamic analysis.

Although the sudden loss of one cable can result in controlling bending moments in the stiffening girder, it is not a controlling load case for the design of the cables. In other words, if the loss-of-cable load case were neglected in the design, the failure of one cable would not lead to a zipper-like failure progression but potentially to a more complicated collapse mode that is initiated by bending and stability failure (i.e., buckling of the stiffening girder). For the case of a simultaneous failure of two adjacent cables, the numerical study indicates that the ultimate load is reached in the neighbouring cables. Nevertheless, an immediate collapse requires a still larger number of initially failing cables when ductile material behaviour is taken into account. In that case, the failure would progress as an instability-type collapse or a mixed-type collapse in which the features of zipper-type and instability-type collapses interact.

These results were obtained for a long-span multicable system with steel girder. It needs to be investigated to which degree they are representative for cable-stayed bridges in general. In future, it might become possible and more convenient for the engineer to directly determine the design forces in a nonlinear dynamic analysis instead of performing a quasi-static analysis with predetermined DAFs.

**Initial failure of multiple cables**

Another concern is that the assumption of just one cable failing at a time might be insufficient. Having in mind traffic accidents as possible triggers of initial failure, it has been suggested that the sudden loss of all cables within a 10 m range measured along the cable anchors should be assumed. On the basis of this suggestion, the sudden and simultaneous loss of any two adjacent cables was assumed in the design of the Taney Bridge—a recently erected cable-stayed bridge in Ireland carrying two tracks of the Dublin Light Rail Transit system.25 These dynamic load cases were accounted for in quasi-static analyses using a DAF of 2,0. They were combined with a specified set of other loadings including live load on the track adjacent to the failed cables. A specified set of load and resistance partial safety factors was used. Additionally, the loss of two adjacent cables was considered statically in combination with a larger live load to account for the possibility that a second train crosses the bridge soon after the failure of the cables while the first train is still standing on the bridge. These design criteria were developed jointly by the designer and the owner.

**Other Design Methods**

Verifying loss-of-cable load cases is an application of the alternative-path approach. When examining the other design methods discussed above, it turns out that the segmentation approach does not generally make sense for cable-stayed bridges. Except for multispan systems, the minimum segment size, and thus the minimum extent of collapse, corresponds to the size of the entire bridge (or at least half the bridge when a centre hinge is introduced in a three-span system). The specific-local-resistance approach, too, seems inappropriate, at least for the currently preferred multicable systems with narrow cable spacing. This is due to the small cross-sectional area of the load-bearing elements of the cables, their presumably small resistance to lateral action, and their significant impact on construction costs.

When finally considering the possible provision of non-structural protective measures, it is found that this approach should be employed more, not as a substitute for but rather as a complement to the verification of loss-of-cable load cases. Protective measures could include barriers to fend off vehicles, fencing to deter trespassers from approaching the cables, and protection against explosions. The latter could include a thick composite-material sheath over the first 4 to 5 m of cable from deck level to mitigate the effect of a shaped charge mounted directly on a cable. To prevent cable loss due to corrosion, efficient corrosion protection systems of cables and anchors, and regular inspection, ideally complemented by constant monitoring, are needed.

**Suspension Bridges**

The hangers of a suspension bridge are secondary load-bearing elements contrary to the cables of a cable-stayed bridge, which are constitutive elements of the primary load transfer system. Nevertheless, the sudden loss of one or a few hangers would likewise lead to an impulsive dynamic loading on the remaining system, and a zipper-like failure of adjacent hangers is possible.26 Such a collapse progression can be seen in the
footage of the Tacoma Narrows Bridge failure in 1940 where the first hangers snapped due to excessive wind-induced distortions of the stiffening girder and then the entire girder peeled off from the hangers and suspension cables. In the case of a self-anchored suspension bridge, any unzipping tendency is likely to be reinforced by the susceptibility to buckling of the stiffening girder.

Progressive collapse initiated by the sudden loss of one or a few hangers has not been a major concern in the past. With the emergence of new types of threats, it is sensible to take a different stance. The challenge can be met by making hanger loss less probable and, at the same time, by designing for such a scenario. The corresponding measures for cables of cable-stayed bridges outlined in the previous section are applicable. In an earth-anchored suspension bridge, there is a further possible design method unfeasible for cable-stayed and self-anchored suspension bridges: the stiffening girder could be provided with a number of breakaway hinges that give way at the beginning of a collapse. In other words, the segmentation approach could be used. A collapse initiated by hanger loss would be isolated by the hinges and limited to one segment.

The primary load transfer system of a suspension bridge is formed by the suspension cables. Using the segmentation approach to confront the possibility of a suspension cable failure could at best be an option in multi-span systems. The alternative-path approach, entailing the force transfer to the remaining cables, only appears feasible in bridges with more than two suspension cables. It is a design option in large suspension bridges where more than two cables are provided already or where the number of cables can more easily be raised. Increasing the specific local resistance of the suspension cables by increasing the cross-sectional area of their load-bearing elements could be appropriate for short spans. The suspension cables of long-span bridges have large cross sections and masses already, which provide them with high local resistance, including against lateral action, even without further strengthening. They are also less sensitive to a local lack of resistance caused by wire breaks because each suspension cable consists of many wires and even a broken wire will carry load again at a certain distance from the breakage point.

In view of the emergence of new types of threats, in particular aggressive and well-resourced malicious action, it is nevertheless advisable to take non-structural measures to protect the suspension cables regardless of cable size. For bridges of high significance and exposure, these measures should include appropriate fire protection and electronic security systems to reliably deter trespassers from approaching the suspension cables. This applies especially to locations where the cables are easily accessible (mid span) or where their load-bearing elements are exposed (splay chamber).

**Arch Bridges**

Arch bridges have similarities with suspension bridges in terms of topology and flow of forces. Much of the statements presented in the previous section apply or can be adapted to arch bridges. A through arch or a tied arch has hangers like a suspension bridge and the same potential problem of a zipper-like failure. Similar design approaches can thus be used. As for a self-anchored suspension bridge, segmentation of the bridge girder is inapplicable to a tied arch. In a true arch, the bridge girder is supported above the arch by columns and not suspended under the arch by hangers. A column failure due to accidents appears less probable; the columns are more resistant to lateral action and not exposed to traffic. However, they are also more hidden from public view, which facilitates malicious action.

Concerning the arch, that is, the primary load transfer system, there are two important differences when compared with a suspension cable. First, an arch can exhibit global stability failure, which can occur within the arch plane or in lateral direction. This opens up further possibilities of failure initiation and collapse progression. Second, various kinds of material and cross sections are possible and in use for arches. These design options should be evaluated with respect to actions that induce local failure (possibly facilitated by local instability, i.e., plate buckling) leading to collapse. In this regard, solid cross sections are more resistant than thin-walled or hollow ones and concrete is preferable over steel. On the other hand, the susceptibility to local action of a hollow steel tube arch can be reduced by filling the regions accessible to traffic or trespassers with concrete.

**Conclusion**

As evidenced by actual failures, disproportionate collapse triggered by accidental circumstances and initial local failure is an important aspect in bridge design. This question is first studied in terms of structures in general. Definitions for the terms robustness and collapse resistance are presented, and the shortcomings of current design methods are addressed. A pragmatic approach for designing structures against disproportionate collapse and a framework of design criteria are outlined. This includes a set of design methods that are described and compared.

If collapse resistance is to be based on the assumption of an initial local failure, the alternative-path method or the segmentation method can be pursued to make the structure robust and to limit an incipient collapse to an acceptable extent. As an illustration of the segmentation method, the collapse-resistant design of the Confederation Bridge is presented. If the design is based on reducing the probability of local failure, on the other hand, the specific-local-resistance method and non-structural protective measures can be considered. These methods do not aim at enhancing robustness. Nonetheless, they should be considered in cases where other methods are impracticable or overly expensive. In addition to these direct design methods, prescriptive design rules can possibly be developed but are of limited applicability to large bridges and other major structures.

Bridges are primarily horizontally aligned. Compared with buildings, there is less need to provide alternative paths to prevent the fall and impact of structural components. Various types of major bridges have been considered in more detail. It is found that continuous girder bridges are preferably made collapse resistant by the segmentation method—which can require a selective elimination of continuity at the segment borders—or by reducing the probability of local failure of key elements. The criteria for selecting one or the other method are discussed.

In the case of cable-stayed bridges, the stay cables are key elements that are particularly vulnerable. The scenario of a collapse triggered by the loss of one or a few cables is countered by providing alternative paths, that is, by designing the bridge for the corresponding

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loss-of-cable load cases. Results of corresponding nonlinear dynamic analyses indicate that such a design can partly be based on a quasi-static analysis using a predetermined dynamic amplification factor (DAF) of 2.0 or smaller. However, at some locations in the stiffening girder and even more so in the pylons, stresses much larger than those determined with a DAF of 2.0 can occur. The alternative-path approach should be complemented by nonstructural measures that protect the cables against vehicle impact, malicious action, and corrosion.

Similar conclusions are drawn for suspension bridges and their hangers. Hanger loss could be made less probable but, at the same time, it should be designed for such a scenario. A further option, in the case of an earth-anchored suspension bridge, is to pursue the segmentation method and to isolate a collapse initiated by hanger loss by breakaway hinges inserted into the stiffening girder. The primary load transfer system of large suspension bridges could be made more robust by raising the number of suspension cables, which facilitates the formation of alternative paths. A particular concern is the possible exposure of the unprotected suspension cable strands to malicious action. For bridges of high significance and exposure, appropriate shielding and security systems should be provided. Arch bridges have similarities with suspension bridges and much of the respective statements apply. Arches are prone to global stability failure, however, and they are often made of thin-walled cross sections—two particularities that warrant further consideration.

References


