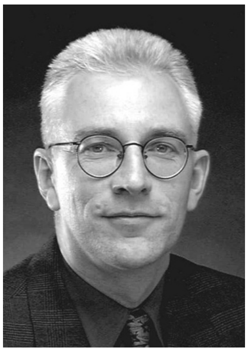
Progressive Collapse of Bridges—Aspects of Analysis and Design

Uwe STAROSSEK
Professor, Dr.-Ing., P.E.
Hamburg University of Technology
Hamburg, Germany



Uwe Starossek, born 1956, received his civil engineering degree from RWTH Aachen, Germany in 1982, and his doctoral degree from the Univ. of Stuttgart, Germany in 1991. Since 1999, he is a Professor of Structural Engineering at Hamburg University of Technology.

Summary

It is shown that current design methods are inadequate to prevent progressive collapse. Definitions for the terms *collapse resistance* and *robustness* are proposed. An approach for designing against progressive collapse is suggested and a set of corresponding design criteria is presented. These include requirements, design objectives, design strategies, and verification procedures. In addition to the better-known design methods providing *specific local resistance* or *alternate load paths*, an approach based on *isolation by compartmentalization* is presented and discussed. It is found that the terms continuity, redundancy, and robustness should be carefully distinguished. The general concepts and findings presented here are applied to bridges.

1. Introduction

Progressive collapse is characterized by a distinct disproportion between the triggering event and the resulting widespread collapse. If we take this disproportion as the defining feature of progressive collapse, then the cause of initial failure, be it a local action or a local lack of resistance, is irrelevant to this definition. Progressive collapse has played a role in such catastrophic events as the collapse of the Alfred P. Murrah Federal Building (Oklahoma City, 1995) and the World Trade Center towers (New York, 2001), but in a large number of less dramatic failures as well which also include some bridge collapses. Following the Ronan Point failure in 1968, progressive collapse has received more widespread attention. Likewise, since the events of September 11th, 2001, research on progressive collapse has intensified.

The term *robustness* is defined here as insensitivity to local failure. Different structural systems exhibit different degrees of robustness. Such differences are neglected even in modern design procedures using partial safety factors. Other problems of current design approaches are that low probability events are neglected and that admissible failure probabilities cannot be specified for risks of the type “low probability / high consequence.” Additional considerations are therefore necessary to ensure structural safety after the occurrence of accidental circumstances. Such additional considerations have in the past been made only in particular cases, e.g., for embassy buildings or very long bridges, i.e., for obviously exposed or vulnerable structures, and mostly at the engineer’s discretion.

The following begins with a presentation of failure events and an overview of previous research in progressive collapse and of the current state of codification to deal with this problem. To prepare the further discussion, the two terms *robustness* and *collapse resistance* are defined. Based on a detailed analysis of the deficiencies in current design methods, a pragmatic design approach is suggested and a set of corresponding design criteria is presented. These include possible design methods such as the alternate-load-paths approach and the hitherto little-noticed compartmentalization approach. It is referred to various examples which include a large multi-span girder bridge. The various design methods are compared in regard to their adequacy for different types of structures. Finally, the application of these general concepts to various types of bridges is discussed in more detail.

2. Failure Events

Naturally, structural failures provide the strongest reason to investigate the presented problem. Descriptions and investigations of structural failures can be an important basis for research. Wearne (2000) describes a number of failures out of recent years, ranging from Ronan Point (a multi-story building in London, 1968) to the Sampoong Superstore (a department store in Seoul, 1995). Progressive collapse as partial cause is evident throughout the described failure events.

Wittfoht (1983) analyzes the collapse of a bridge under construction. The description is convincing in its accuracy and conclusiveness and the description of the sequence of failure appears to be appropriate. It was not sufficiently emphasized, however, that progressive collapse was part of the problem. The structure's collapse was not solely caused by the failure of the formwork but was inherent to the structural system due to its lack of robustness. Another progressive collapse of a bridge occurred during the construction of Haeng-Ju Grand Bridge in Seoul in 1992. After the failure of a temporary pier, an 800 m section of the bridge collapsed. 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 collapse of structural elements (Lee 1998). The high degree of robustness of the *material* coupled with the continuity of the tendons over the length of the bridge worked against the robustness of the *structure*. A chain reaction ensued where the forces transmitted by the tendons led to the collapse of all eleven continuous spans. The collapse did not stop until it reached the transition joints on both ends of the bridge.

Prendergast (1995) describes the collapse of the Alfred P. Murrah Federal Building in Oklahoma City in April 1995. It was initiated by the detonation of a truck bomb outside the building. The enormous degree of destruction (and the large number of casualties) seems related, however, to insufficient structural robustness. Every second exterior column was indirectly supported by a continuous transfer girder. These and other weaknesses in design were presented in the official report (Corley et al. 1996). Recommendations for structural design were derived including, among others, the provision of continuity in the concrete reinforcement. In view of other findings discussed in this paper, however, the value of continuity appears ambiguous. Under certain circumstances, continuity can even be harmful.

In terms of tragedy and losses the above mentioned cases of damage were far exceeded by the collapse on September 11th, 2001 of the twin towers of the World Trade Center. The impact of the airplane and the subsequent fire initiated local failures in the area of impact. The ensuing loss in vertical bearing capacity was limited to a few stories but extended over the complete cross section of the tower. The upper part of the structure started to move downwards and accumulated kinetic energy. The subsequent collision with the lower part of the structure, which was still intact, caused enormous impact forces which were far beyond the reserve capacities of the structure. This, in turn, led to the complete loss of vertical bearing capacity in the area of the impact. Failure progressed in this manner and led to a total collapse (Bǎzant and Zhou 2001, Starossek 2001, FEMA 2002).

3. Previous Research

Time and again progressive collapse has been a topic of investigations as well as a theme at conferences (Breen 1975), (Breen and Siess 1979). The publications address specific aspects of the phenomenon or its occurrence in specific structures (Hawkins and Mitchell 1979), (Mitchell and Cook

1984), (Pekau et al. 1984). Further papers focus on particular actions, such as fire (Andrews and England 1995) or impact and blast loads (Gilmour and Virdi 1998), all of which can trigger a progressive collapse.

Yokel et al. (1989) examined the U.S. embassy in Moscow in terms of its sensitivity to progressive collapse. They compared analysis methods, considered alternate load paths, and made recommendations of measures to increase the collapse resistance. Other project-based publications focus on the design of the Confederation Bridge, Canada (Ghali and Tadros 1997, Starossek 1997, 1999, Starossek and Sauvageot 1998). The possible progressive collapse of a bridge structure running over many spans and the corresponding design improvements are discussed in these papers.

The publications mentioned so far comprise investigations of certain types of structures or particular projects submitted to specific triggering actions. They are independent of each other and do not lay the foundation for a comprehensive theory of progressive collapse. The approaches, results and recommendations vary from case to case.

A comprehensive account of the phenomenon and the deduction of general rules for design and verification have rarely been attempted. To prevent progressive collapse, Leyendecker and Ellingwood (1977) suggest providing either specific local resistance or alternate load paths. Gross and McGuire (1983) compare design methods to avoid progressive collapse. They recommend checking the stability of the structure after selected elements have been removed and thus also embrace a deterministic alternate-load-paths approach. Proposals for a probabilistic assessment of structures have been made by Ellingwood et al. (1983) and Bennett (1988). Progressive collapse is characterized by dynamic effects induced by local failure and failure progression. Such effects were not yet considered in these papers.

Since the events of September 11th, 2001, research into progressive collapse has been intensified and more strongly coordinated (MMC 2003), (PCI 2004). The effort to address all remaining questions, to define objectives for research and codification, and to develop a collective research program is clearly expressed in MMC (2003). In contrast to earlier publications a number of points are emphasized: the necessity to establish performance expectations as design criteria; the differentiation of requirements with regard to progressive collapse resistance and investigative accuracy depending on the structure's importance and exposure (tiered approach); the distinction between prescriptive design rules (indirect design; e.g. the requirement of continuity in concrete reinforcement) and more elaborate design strategies based on design criteria and analysis (direct design, performance-based design). The necessity to improve and verify the calculation tools is also pointed out. As for verification, comparison with full model tests, with structures which despite serious damage have not sustained collapse, and with controlled demolitions by detonation is suggested.

4. Standards

The requirement for structural resistance to progressive collapse is not yet consistently embodied in the structural design standards. Dusenberry and Juneja (2003) subject the building codes of the U.S. and Canada to a critical assessment. Even when progressive collapse is mentioned in those documents (in particular, in NRC (1996), ASCE (2002), ACI (2002)), it is done so without giving much practical guidance and, if so, without general applicability. The ASCE guideline 7-02 (ASCE 2002) states: "… buildings and structural systems shall possess general structural integrity, which is the quality of being able to sustain local damage with the structure as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage …" More specific design rules, where given, are of prescriptive nature and limited to specifications for detailing the concrete reinforcement and connections. Design objectives and rules for analysis and checking are not further specified. The implementation of these codes in individual cases is left to the engineer's discretion and ability.

The arguably most detailed design rules, even if limited to buildings, can be found in the guidelines for U.S. federal office buildings (GSA 2003). Requirements are defined, design methods based on analysis but using simplified calculation methods are described, and prescriptive design rules are given.

The CEB-FIP Model Code 1990 (CEB 1991), requires that structures should withstand accidental circumstances without damage disproportionate to the original events—which is a more general requirement than that of ASCE (2002). Eurocode 0 (EC 0 2002), by stressing that the failure of one element shall not lead to the failure of the entire structure, is in line with ASCE (2002). A more sophisticated approach containing detailed design rules, even if limited to buildings, has been drafted for the future Eurocode 1, Part 1-7 “General Actions – Accidental Actions” (EC 1-1-7 2004).

At present, precise design and analysis rules only exist for special types of structures. For the design of cable-stayed bridges, for instance, the PTI Recommendations (PTI 2001) require that the sudden rupture of one cable shall not lead to structural instability and specify a corresponding load case “loss of cable.” Beside this, requirements concerning progressive collapse are mainly found in building standards. In this regard, the British standards have a long tradition starting with the Ronan Point collapse. Detailed prescriptive design rules for steel buildings can be found in BS5950-1 (2001). Moore (2003) gives a survey of the historical and current development of British building standards. The most recent British provisions are given in The Building Regulations 2000 (2004); they are similar in scope and content to the draft of Eurocode 1 (EC 1-1-7 2004).

5. Two Definitions

The term robustness is often used in papers and discussions on progressive collapse. Still, this term is used differently and there is no generally accepted agreement to date on its exact meaning (JCSS 2005). The following two definitions prove useful within the context of the discussion presented here.

5.1 Robustness

It is suggested to define the term *robustness* as insensitivity to local failure, where “insensitivity” and “local failure” are to be quantified by the design objectives which are part of the design criteria (see that section below). Defined in this way, robustness is a property of the structure alone and independent of the possible causes of initial local failure. This definition is in contrast to a broader definition of robustness—as it is given, for instance, in the draft of Eurocode 1 (EC 1-1-7 2004)—which does include possible causes of initial failure. Such a broader definition is close to the term *collapse resistance* as defined in the next section. It is believed that clarity is served by distinguishing these two properties (which could be named differently if no consensus on a re-definition of the term robustness can be reached).

5.2 Collapse Resistance

It is suggested to define the term *collapse resistance* as insensitivity to accidental circumstances, where again the “accidental circumstances” are to be quantified by the design objectives. Collapse resistance is a property that is influenced by numerous conditions including both structural features and possible causes of initial failure. The structural system is of particular importance. It would intolerably limit the range of design possibilities, however, if only those structural systems were permitted that are clearly robust. Nor is such a limitation necessary because a structure whose system tends to promote collapse progression can be made sufficiently collapse resistant by other measures such as an especially safe design of key elements. Furthermore, collapse resistance may not be required for every structure.

6. Design of Collapse-Resistant Structures

6.1 The Inadequacy of Current Design Methods

Modern design codes and procedures of design and verification are based on reliability theory. Actions and resistances are statistically determined on the basis of empirical data. After choosing an allowable probability of failure, the design values for actions and resistances can be computed using probabilistic methods. Such an approach is based on a mathematically sophisticated and, as it seems, sound foundation. It is reflected in the design codes by partial safety factors and a series of load combination schemes. If the application of the ensuing code rules is often cumbersome, the designer might take comfort in the idea that, by working on a rational mathematical basis, a consistent safety level is reached. Still, it turns out that such an approach fails with regard to the identification and proper treatment of a potential for progressive collapse. There are three reasons for this failure (Starossek 1997).

The first reason lies in the fact that design codes are based on the consideration of local, not global failure. Correspondingly, design equations are usually defined and applied on a local level only (check of cross-sectional forces or element stability). Structural safety, therefore, is likewise accounted for on the local level only. The global safety, i.e., the safety against the collapse of the entire system or a major part thereof, is a function of the safety of all the elements against local failure but also of the system response to local failure. The latter influence is neglected. Different systems will respond differently to local failure. The underlying assumption that a consistent safety level of a structure is reached by an adequate safety of its elements, therefore, is not generally valid. Such methods when applied to non-robust structures will produce unsafe designs.

The second shortcoming of current design methods is that low probability events and unforeseeable incidents (accidental circumstances) are not taken into account. Within the scope of a probabilistic design concept, such a simplification is necessary because the supporting statistical data, derived from experience and observation, is unavailable. In the case of a non-robust structure, however, this simplification becomes inadmissible. Let's consider, for the sake of argument, a structure with primarily serial load transfer (say, a high-rise building) where the local failure probabilities are statistically independent. In that case, the probability of collapse will be in the order of the *sum* of the failure probabilities of all the elements (say, the building's individual stories) of the structure. If the number of elements is sufficiently large (simply, if the area of attack is large enough), even very low probabilities of initial local failure can add up to a probability of global failure which is high enough to be taken seriously. (For structures with primarily parallel load transfer, the probability of collapse is in the order of the *product* of the failure probabilities of the elements and thus very low.)

The third problem with current design methods is that the underlying probabilistic concept requires specification of an admissible probability of failure. Considering the extreme losses that often result from progressive collapse, it seems difficult to reach a true societal consensus on the admissible probability of such an outcome—a problem which risks of the type “low probability / high consequence” are typically up against (Breugel 1997).

6.2 Possible Improvements of Current Design Methods

The first problem outlined in the preceding section results from practical limitations which appear when reliability theory is applied to actual structural systems. The determination of a system's global safety has to take into account the system response to local failure. Within the framework of current design methods, one could attempt to consider this influence by an additional partial safety factor on the resistance side of the design equations. This factor would take the value one for robust structures and a value smaller than one for non-robust structures. Provisions in some codes are indeed equivalent to such an approach. In that case, however, the reduction of the design value of resistance of non-robust structures is based rather on judgment than on thorough analysis. Such reduction factor would actually have to be stipulated based on parametric analytical studies for all the different structural systems covered by the respective design codes. Another possibility would be to pursue a fully probabilistic analysis in a given design situation.

In either case, though, the system response to local failure needs to be considered. That response involves large deformations and displacements, separation of structural elements, falling separated elements striking other elements below, and other kinds of interaction which all require a fully nonlinear dynamic analysis in the time domain. These difficulties are compounded by the need to consider many initial failure scenarios and by the fact that, due to the nonlinear dependencies appearing here, small errors in the modelling assumptions can produce large deviations in the computational outcome. Even a deterministic analysis of the system response to local failure poses tremendous difficulties. A stochastic analysis of that response and the analysis of global safety would add further dimensions of difficulty, and, therefore, seems out of reach of today's analysis resources—at least when an exact computation of general reduction factors or an exact analysis of a specific structural system in a design situation is expected. On the other hand, an analysis of the system response to local failure, be it deterministic or stochastic, could give some qualitative indication on the degree of robustness of a certain type of structure or of a specific structural system.

The second and third problems outlined in the preceding section are fundamental challenges to a purely probabilistic design approach. If a low probability of local failure can add up to a large probability of global failure, we need to know that quantity. Also, if societal consensus on the admissible probability of a catastrophic event cannot be reached, another basic ingredient to a numerical stochastic computation is missing.

6.3 Suggested Design Approach

It follows from the discussion above that the shortcomings of current design methods can at best only partly be overcome within the framework of reliability theory. The possibilities of improvement which do exist are not yet explored today and might prove insufficient in the future. Still, guidance is needed on how to design a collapse-resistant structure that is insensitive to accidental circumstances. It is therefore suggested to use, for the time being, the following pragmatic approach.

On the one hand, the design methods as described in the current codes are applied. They are based on reliability theory and reflected in the codes by partial safety factors and load combination schemes. In view of the inconsistencies outlined above one could argue that the number of load combinations prescribed by some codes can be reduced because it is exaggerated when compared to the actually achieved accuracy.

On the other hand, an additional assessment with particular regard to collapse resistance is made. This procedure is further described in the subsequent sections in which thoughts on an appropriate additional design process are put forward along with possible avenues of investigation and enhancement of collapse resistance. This additional investigation is not necessarily based on reliability theory but rather on judgment and a decision-making process. Analyses are carried out deterministically.

7. Design Criteria

In the assessment and the design of a structure with regard to its collapse resistance, the following additional design criteria are of importance:

I. Requirements
II. Design objectives
III. Design strategies
IV. Verification procedures

First, the *requirements,* particularly the question if collapse resistance is necessary, should be clarified. The necessity depends on the structure's *significance* with respect to the consequences of a collapse, including the immediate material and immaterial losses but also indirect effects, e.g., the possible impairment of the infrastructure and of civil and national defense. Another criterion for the determination of requirements is the structure's degree of *exposure* to hazards of war, malicious action, and natural disasters. The exposure can be considered particularly high for public buildings, major bridges, and other lifeline structures. If collapse resistance is deemed necessary, the following *design objectives* must be specified:

1. Assumable extent of accidental circumstances
2. Assumable extent of initial local failure
3. Acceptable extent of collapse progression
4. Acceptable extent of damage to the remaining structure
5. Applicable load combinations and safety factors

Design objectives 2, 3, and 4 can be used when testing for *robustness*, design objectives 1, 3, and 4 can be used when testing for *collapse resistance* according to the respective definitions given above. The following *design strategies* to prevent progressive collapse are mentioned in the literature and have at least partially made their way into the design codes:

1. High safety against local failure
   - 1.1 Specific local resistance of key elements (direct design)
   - 1.2 Non-structural protective measures (event control)
2. Design for load case "local failure" (direct design)
   - 2.1 Alternate load paths
   - 2.2 Isolation by compartmentalization
3. Prescriptive design rules (indirect design)

These methods are further discussed in Section 9 below. The prediction of the structural behavior following a local failure requires suitable *verification procedures*. Accurate analysis will require a high degree of expertise and modeling effort. Thus, development and validation of simplified but admissible verification methods would be a worthwhile undertaking.

The design criteria I to IV listed above are to date only partially addressed in codes and guidelines. As far as applicable design criteria are not available in codified form, they should be agreed upon by the contracting and other affected parties or established by the building authorities. It is anticipated that the design criteria can only partly, at best, be developed from first principles and reliability theory. There will remain necessity for engineering judgment and a decision-making process, most notably when stipulating the acceptable extent of collapse progression. On the other hand, the choices to be made here are relatively transparent, at least when compared to the choice of a safety index β, so that an informed societal consensus is in principle possible—even when that consensus leads to the conclusion that certain kinds of structures should better not be build.

8. Investigation of the Structure

If collapse resistance must be ensured, the structure is to be investigated with respect to its robustness. The procedure of verification follows from the definition and from the design objectives given above. If an initial local failure leads to a collapse progression or to damage larger than acceptable, the finding would be "non robust." Consequently, the structure's response to local failure is to be analyzed. Local failure can be produced by, e.g., the removal of a structural element. The cause and probability of the local failure is irrelevant to the assessment of robustness which is a characteristic feature of a structure; hence a high safety against local failure is also irrelevant.

Even though the cause of local failure does not need to be specified, it is still necessary to carefully select the local weakening and to model it realistically. It can be necessary to investigate various cases of initial failure. If the local weakening is caused by the removal of a structural element, the removal should be modeled as a dynamic action. The system's response to all dynamic and static loading is to be examined as realistically as possible, including effects such as inertia and impact forces, large deformations, inelastic material behavior, and damages. The more realistic the modeling and analysis, the more confidence can be had in the finding "robust." In reverse: if simplified methods of modeling and analysis are used which have not yet been validated, confidence can only be had in the finding "non robust."

9. Design Strategies

The finding "non robust" can be dealt with in several ways which will be exemplified in the following. The Alfred P. Murrah Federal Building, Oklahoma City collapsed following the detonation of an ammonium nitrate fuel oil mixture equivalent to 1,800 kg of TNT inside a truck in front of the building. According to the building's structural design concept, every second exterior column rested on a transfer girder that ran across the face of the structure on the second floor (Fig. 1). For the sake of argument, it is assumed that the detonation led to the immediate destruction of only one main column. (In reality, further main columns might have been directly affected (Corley et al. 1996); in that case, the collapse would have been rather due to the power of the bomb blast than to a lack of structural robustness.) The structure did not provide enough redundancy to redistribute the loads to the neighboring main columns. The failure of the transfer girder and the failure of the adjacent secondary columns resting on the transfer girder led to a collapse progression which, possibly promoted by impact loads and horizontal forces, could spread through the neighboring spans finally affecting a major part of the structure.

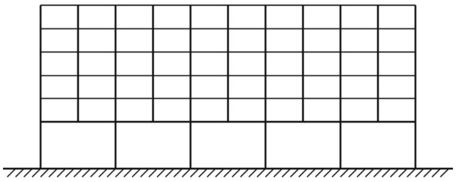


Fig. 1 Structural design concept

The starting point of a performance-based design (direct design), again, is the definition of progressive collapse. The structure is weakened by removing selected structural elements; the structure's response to this modification is analyzed. Concerning type and quality of this investigation, the preceding section is referred to.

9.1 High Safety against Local Failure

If the acceptable extent of collapse progression is exceeded, the removed structural element is identified as a *key element*. One possibility for achieving collapse resistance is to provide maximum safety against failure for all key elements. This high level of safety is preferably assured through providing specific local resistance in the key elements. In the case of the Murrah Federal Building, this would require strengthening the main columns (Fig. 2), e.g., by designing for blast or for equivalent loads (accidental design actions) as envisaged in (EC 1-1-7). If specific local resistance cannot be achieved or would require disproportionate effort, high safety against failure of key elements can also be provided by non-structural protective measures (event control). Such measures

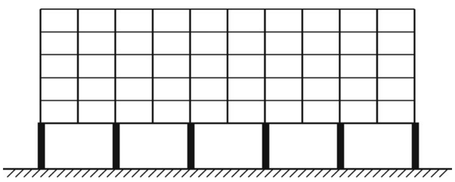


Fig. 2 Specific local resistance

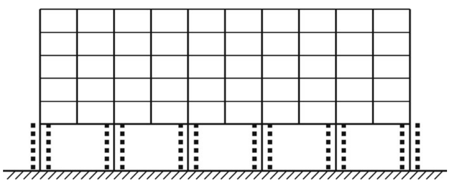


Fig. 3 Protective barriers

include barriers to protect against vehicle impact or bomb blast (Fig. 3), limitation or control of public access, and other protective measures like aerial surveillance or antiaircraft systems. In either case, the measures to be taken are guided by the assumable extent of accidental circumstances to be defined as a design objective.

It should be kept in mind, however, that ensuring high safety against local failure requires more than the use of high design loads or, alternatively, recourse to measures which protect against mechanical action. On the one hand, local failure can also be caused by occurrences such as corrosion or fire which are more effectively countered by corrosion protection, regular inspection, fire protection, and fire fighting systems (which in a wider sense are also non-structural protective measures) than by increasing design loads. On the other hand, more thought and measures are required concerning the structural resistance. Safety factors for material and soil resistance must be chosen higher than usually; 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 during the design process.

The development and codification of equivalent loads (accidental design actions) for general structures has still to be made. For bridges, the design loads for ship collision, waves, current, and ice pressure should be increased. The construction stages are of particular 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 an occurrence. If such an approach proves impractical, the on-site safety requirements for the construction stages identified as crucial could 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. Also, the structure's inherent lack of robustness, in the sense defined in Section 5.1, is not eliminated. Nevertheless, application of the design strategy "high safety against local failure" is justified in certain cases, in particular, when the structure's significance and exposure 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 structure's critical areas and the number of key elements are small.

For instance, small to medium-sized buildings with transfer girders and indirect load transfer can be made sufficiently collapse resistant through local strengthening of the main columns if the alternatives "modification of structural system" or "strengthening of transfer girder" (see Section 9.2.1) are unsuitable for architectural or other reasons. Likewise, for small to medium-sized bridges (up to, say, 1000 m in length), the design strategy "high safety against local failure" might be preferable to alternatives like "insertion of hinges" (see Section 9.2.2). If this design strategy, however, is to be applied to structures of extremely high significance or exposure (e.g., the Freedom Tower or the Messina Bridge), the decision to do so must not rest with the engineer alone. Instead, public and political consensus on the acceptability of the residual risk should be reached

9.2 Design for Load Case "Local Failure"

Absolute safety against local failure cannot be achieved. Therefore, the design strategy discussed in the previous section should preferably not be applied to extremely significant or exposed structures. For other structures, a sufficiently high level of safety against local failure may be achievable but would result in disproportionately high costs. In such cases, it is better not to increase local safety but structural robustness. This requires postulating local failure, which may occur for any reason

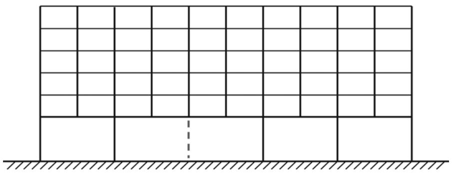


Fig. 4 Column failure introduced as load case "local failure"

and with any probability whatsoever, and limiting the collapse progression which follows this triggering event to an acceptable extent (design objective). The investigation should be based on the failure of the structural elements identified as key elements, in line with the assumable extent of initial local failure (design objective). In the case of the Murrah Federal Building, the lower part of a main column could be considered a key element (Fig. 4).

9.2.1 Alternate Load Paths

If the collapse shall be limited to the initially failing structural element, at least two design measures are possible for a structure like the Murrah Federal Building. On the one hand, the structural system could be modified. When extending all the exterior columns down to the foundation, the need for a transfer girder is eliminated (Fig. 5). In case of failure of the lower part of one column (dashed in Fig. 5), the upper part of that column remains supported by the transverse girder, which now becomes a transfer girder with an effective span as in the original structure. It should thus be possible to design for the additional loading. The girder becomes part of an alternate load path for the forces otherwise carried by the lower part of the column. The existence of alternate load paths shall be referred to as *redundancy*, which means redundancy of the structure with regard to its ability to carry loads. The redundancy is increased by the measure just described. This assessment is, at least in this case, intuitively corroborated by the increase of the system's statical indeterminacy.

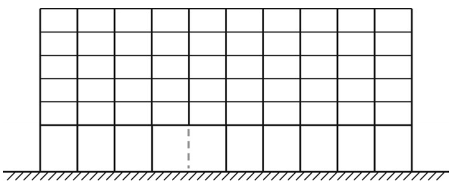


Fig. 5 Modification of structural system

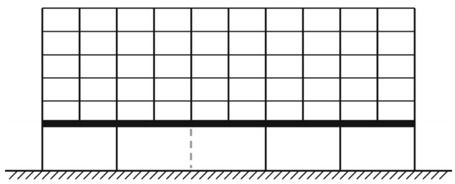


Fig. 6 Strengthening of transfer girder

An alternative measure would be to strengthen the transfer girder by designing it for the load case "column failure" (Fig. 6). Since an alternate load path is provided, the structure's redundancy would again be increased, although, in this case, the increase is not accompanied by an increase in statical indeterminacy. In view of the relatively large span, it may prove difficult to strengthen the transfer girder sufficiently. This problem could be alleviated by using relaxed design objectives when checking for this special load case, e.g., by permitting a certain amount of damage in the remaining non-collapsed structure.

The elements and modes of load transfer of alternate load paths can vary widely. In the aforementioned case, the flexural capacity of a main load-bearing element was mobilized. In other cases, alternate load paths can form through the mobilization of axial or torsional resistance. The utilization of plastic reserves in the structure, the transition from flexural to tensile resistance (catenary action), or from plane to spatial structural behavior, e.g., in one-way slabs turning into two-way slabs, are all possible. If plastic reserves are utilized, a sufficient degree of ductility must be ensured.

9.2.2 Isolation by Compartmentalization

For certain structures, the alternate-load-paths approach will reach its limits. For instance, if the Confederation Bridge, Canada (Fig. 7) were to be made robust and collapse resistant in this way, the initial local failure to be considered would be the failure of a bridge pier; this in turn would require designing a prestressed concrete frame with a span of 500 m—arguably a vain endeavor. The design method chosen was to spatially limit local failure by isolating the collapsing section (Fig. 8). In consultation with the supervising authority, Public Works Canada, the acceptable extent of collapse progression was determined and, based thereupon, the location of collapse boundaries was derived (Fig. 8, pier D and hinge H1). The collapse must not transgress these boundaries; the collapsing section is thus isolated from the remaining structure. The structure is compartmentalized by the collapse boundaries which are stipulated based on the design objectives and become compartment borders (Starossek 1997, 1999).

Fig. 7 Confederation Bridge, Canada

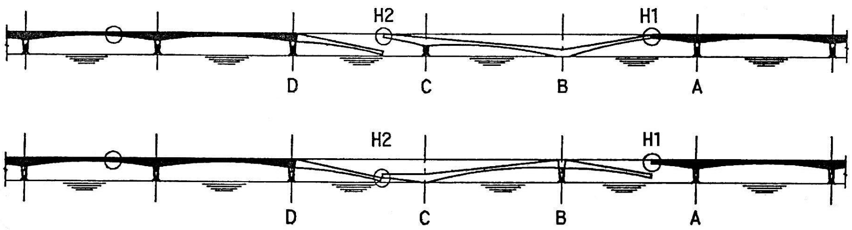


Fig. 8 Limitation of local failure by isolating collapsing sections

Such an approach requires 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 compartment borders (in the considered case pier D and the region between and including hinge H1 and pier A); they isolate the collapse and become the key elements in this approach. One design possibility would be to provide these elements with specific local resistance. Verifying that their resistance is adequate, however, may be difficult because of the high loading and because of analytical uncertainties (Starossek 1997).

In the case of the Confederation Bridge, the response of the structure to the left of hinge H2 (Fig. 8) after a sudden loss of that hinge was investigated. The firstly affected region between H2 and pier D consists of a cantilever beam and a precast drop-in girder connected to it by a cast-in-place joint and continuous prestressing tendons. The sequence of collapse, according to static and dynamic analysis, is marked by several distinct events:

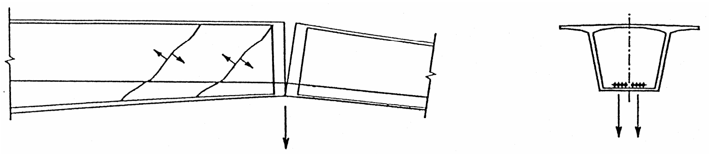


Fig. 9 Bending rupture at cast-in-place joint

– The girder fails in bending under its own weight at the cast-in-place joint between the cantilever and the drop-in girder (Fig. 9).

– The drop-in girder rotates around this point, remaining connected to the cantilever through the continuity tendons.

– The free end of the drop-in girder hits the water; the drop-in girder ruptures due to bending under the inertia forces induced by its own mass.

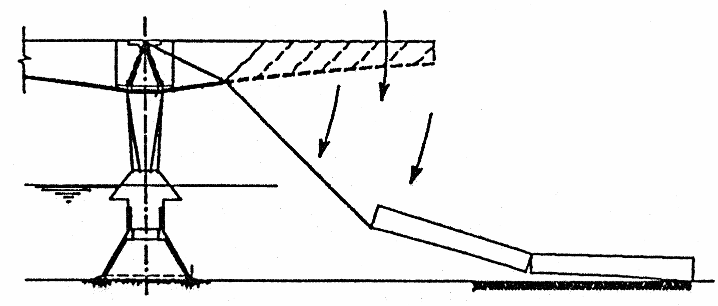


Fig. 10 Failure progression toward pier D

- Large forces are transmitted to the cantilever during this violent event. Shear failure occurs at the cantilever end.
- The tendons cut through the bottom slab, thus crippling the cantilever's bending resistance.
- Rupture progresses throughout the cantilever towards the pier (Fig. 10).

The result of further analysis seemed to depend more and more on modeling assumptions. Further analytical prediction was therefore deemed inaccurate and unreliable. Verification of adequate resistance of the compartment borders, and thus verification of collapse resistance, was impossible because of both the high loading and the analytical uncertainties.

Both problems can be solved or mitigated by selectively eliminating continuity at or close to the compartment borders. By inserting joints, break-away hinges or structural fuses, or by providing plastic hinges, the loading of the compartment borders will be reduced and analysis will be simplified.

In the case of the Confederation Bridge, it seemed 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 system. Otherwise, the collapse could progress into the adjacent span (left of pier D), and an outcome similar to the collapse of Haeng-Ju Grand Bridge could ensue (see Section 2). It was attempted to design a structural fuse within the cast-in-place joint between cantilever and drop-in girder. However, no secure way of automatically cutting the continuity tendons (after collapse onset) was found, and the idea was abandoned.

The preliminary design was therefore changed by inserting additional hinges in every second span. Instead of using monolithic cast-in-place joints, the drop-in girders in those spans were connected to both cantilevers by hinges (Fig. 11). If support H2 fails, the drop-in girder, extending between the inserted hinge and hinge H2, will fall and separate from the remaining structure in a predictable way. The separation would be forced and defined by the geometry of the hinge corbel (Fig. 12).

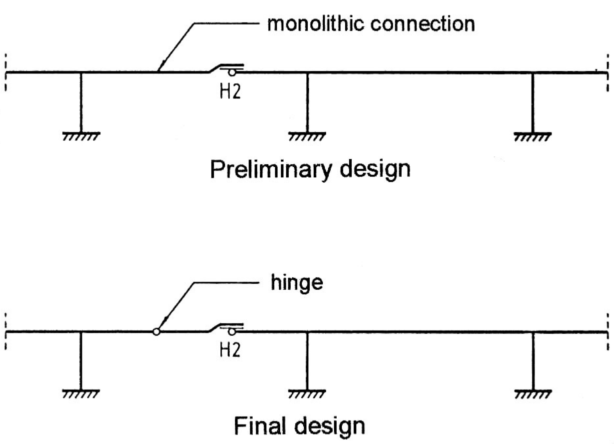


Fig. 11 Insertion of hinges

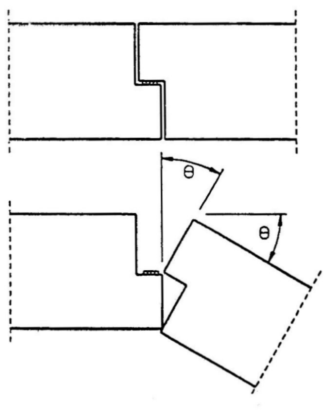


Fig. 12 Disengagement of drop-in girder

Verification of the remaining structure was performed for the load impulse acting on the cantilever tip during the fall of the drop-in girder. For sake of simplicity and because of modeling 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, and the plastic reserves of the structural system were utilized in the detailed design against progressive collapse. A detailed account of the progressive-collapse investigation of the Confederation Bridge is given in Starossek (1997); an abbreviated version of that account can be found in Starossek (1999).

This section on isolation by compartmentalization shall be concluded by the following more general remarks. Limiting a local failure without consideration of the possible cause or the probability of its occurrence is in line with standards such as GSA (2003) or EC 0 (2002). Accomplishing this goal by compartmentalization and, in particular, through the insertion of hinges, seems to be less conventional. Inserting hinges reduces the degree of static indeterminacy, and thus, the level of continuity. The structure's redundancy, however, is not lessened by this measure because it does not remove any feasible alternate load path (having in mind that designing a frame with a span of 500 m, even though a theoretical option offered by the original structural system, is impracticable). On the other hand, the robustness of the structure, i.e., its insensitivity to local failure, is increased. This shows that associating continuity with redundancy and equating redundancy with robustness, even if possible for particular types of structures, is not generally justified. These terms should be carefully distinguished.

9.2.3 Redundancy versus Compartmentalization

The design method “isolation by compartmentalization,” including the possible consequence of eliminating continuity, has so far been described only in publications related to the Confederation Bridge. Its application to that particular case was substantiated by the infeasibility of alternate load paths. This approach may still be preferable, though, even when alternate load paths could be provided. Furthermore, the continuity required for the formation of alternate load paths may, in certain circumstances, not prevent but rather promote collapse progression. This view is supported by eyewitness accounts of controlled demolition experts and fire fighters who have observed the collapse of buildings (MMC 2003, page 21). According to these accounts, local failure may actually pull down a greater portion of the building when structural components, like frames or diaphragms, are too well tied together. Such observations seem plausible when considering that collapse progression requires interaction, which in turn could mean a certain degree of connectivity, between structural elements.

In light of these considerations, the failure of Ronan Point, an often cited example of progressive collapse, can be interpreted differently. Triggered by an explosion in one of the upper stories, one building corner collapsed over nearly the entire height of the building. The larger part of the building, however, remained undamaged. This progressive collapse of floor slabs has been ascribed to a lack of continuity in slab reinforcement. On the tentative premise that an overall collapse of the building must be prevented, and contemplating the design method “isolation by compartmentalization,” such a lack of continuity does not seem so bad after all.

Stimulated by the collapse of Ronan Point, requirements for continuity have been included in building codes in the form of prescriptive design rules (see Section 9.4 below). These provisions were intended to increase the robustness of a structure. If, however, the resulting alternate load paths become overloaded, the design objective cannot be achieved. In this context, an observation made by Corley et al. (1996) concerning the collapse of the Murrah Federal Building is of interest. If only one main column was immediately destroyed by the bomb blast (one of the possibilities discussed in that report), it is argued that the two adjacent main columns could have been pulled down by the connections to the falling structural components in-between (in case the reinforcement were continuous). This assessment is supported by the fact that the collapse stopped at a main column shortly after a discontinuity in the transfer girder's top reinforcement (Fig. 13).

Fig. 13 Partial collapse of the Alfred P. Murrah Federal Building, Oklahoma City

Two more examples where compartmentalization accomplished by discontinuity has possibly prevented widespread collapse are the Pentagon Building and the Charles de Gaulle Airport Terminal in Paris. The Pentagon Building consists of three building rings each divided in five compartments separated by expansion joints. The airplane impact near an expansion joint caused several columns on both sides of the joint to fail. The more affected section—the outer ring on the right of the joint—collapsed while the less affected section—the outer ring on the left of the joint—did not (Fig. 14). A connection might have promoted a collapse progression, since the left section was heavily damaged as well and might not have been able to carry additional loads. The isolation of collapse on the other side of the collapsed section was achieved by strong structural elements which resisted the collapse loads and thus likewise formed collapse boundaries and compartment borders.

The partial collapse of the Charles de Gaulle Airport Terminal was initiated by the failure of a portion of the roof due to poor workmanship and deficiencies in design. The collapse came to a halt at the two joints which separated the collapsing section from the adjacent structures on both sides (Fig. 15). It seems unlikely that the forces which occurred during collapse could have been sustained, in case of continuity, by the adjacent sections since these sections suffered from construction deficiencies as well.

Fig. 14 Partial collapse of the Pentagon Building

Fig. 15 Partial collapse of the Charles de Gaulle Airport Terminal, Paris

The potential value of continuity shall not be called into question. It should be kept in mind, however, that continuity can be harmful when the resulting alternate load paths are not provided with the strength required to withstand the forces transmitted by continuity. If it is impossible, or overly expensive, to provide alternate load paths with sufficient strength, the design method "isolation by compartmentalization"—if necessary, by selectively eliminating continuity—has the advantage.

This is also the case if alternate load paths (or collapse-isolating elements) are strong enough, but the corresponding verification proves difficult or unconvincing due to the structure's high complexity, the effects of dynamic forces, or the utilization of plastic reserves (see the above discussion concerning the design of the Confederation Bridge).

The design method "alternate load paths," on the other hand, is indicated (provided that the alternate load paths can be shown to be sufficiently strong) if the fall of components or debris must be prevented by any means. This applies particularly to cases in which falling parts could strike key elements of the remaining structure because the impact loading produced by such an event are difficult to be designed for. Such conditions are found in structures with primarily vertical alignment, such as high-rise buildings; they are less typical for horizontally aligned structures, such as bridges. The suitability of the two design methods compared here will thus depend on the type of structure and its alignment in space.

The alternate-load-paths approach requires an increase of either or both continuity and strength. Compartmentalization, on the other hand, can be accomplished by less continuity or more strength. Other differences between these two methods concern the spatial distribution of design measures, the dependency of their efficiency on the size of initial failure, and the minimum extent of collapse. The alternate-load-paths approach leads to changes that are distributed throughout the structure; its efficiency decreases with in increase in initial failure size; it is therefore preferable for small initial failure size; the minimum extent of collapse decreases with initial failure size. The compartmentalization approach requires changes at discrete locations; its efficiency tends to be insensitive to initial failure size; it is preferable for large initial failure size; the minimum extent of collapse is fixed and comparatively large. Both methods can be combined. When the alternate-load-paths approach is used within individual compartments, structural robustness is increased for both small and large initial failure sizes.

9.3 Local Failure: Prevent or Presume?

If local failure is presumed, the design methods "alternate load paths" and "isolation by compartmentalization" can be pursued to make the structure robust and limit a beginning collapse to an acceptable extent. Again, the safe performance of certain key elements is crucial and must be verified. In contrast to the design strategy "high safety against local failure," these key elements are under the control of the engineer: they are selected by choosing the alternate load paths or the locations of compartment borders, a design freedom whose magnitude depends on the design objectives. Thus, the number of key elements can be comparatively small, particularly when the compartmentalization approach is utilized.

A further advantage of that approach is that the key elements do not need to be provided with maximum safety against failure (Starossek 2005). Even if one compartment border fails and collapse progresses into an adjacent compartment, the collapse will in all probability come to a halt there because that compartment is also isolated by borders selected and enforced by the engineer. The situation resembles a condition, discussed in Section 6.1, concerning the effects of different types of load transfer (serial vs. parallel load transfer). When using the design strategy "high safety against local failure," the probability of progressive collapse is in the order of the *sum* of the failure probabilities of all key elements, and thus increases with the number of key elements (which for that design strategy can be rather high). When using the design method "isolation by compartmentalization," on the other hand, the probability of progressive collapse is in the order of the *product* of the failure probabilities of the key elements, and therefore decreases with an increasing number of key elements and compartments. (Even if these compartments are all similar by design, the probability of progressive collapse will at least not exceed the failure probability of one key element.) The advantage stated above is shared, to a lesser extent, by the alternate-load-paths approach.

For these reasons, design methods based on the presumption of local failure seem preferable for structures of high significance or exposure. They allow high safety against progressive collapse at relatively low additional cost, as long as such a design is possible. Moreover, they are more satisfying from the standpoint of reliability theory because their efficiency is relatively independent of the failure probabilities of key elements (a statement based on the preceding paragraph); uncertainties related to triggering actions are altogether irrelevant, at least in the case of the compartmentalization approach.

9.4 Prescriptive Design Rules

The design methods studied in the preceding sections are based on analysis. The structure's response to the removal of selected structural elements is investigated; the results are compared to the design objectives. Such a performance-based procedure demands a great deal of time, skill, and computational resources on the part of the engineer. In the case of small to medium-sized structures, that kind of commitment may become disproportionate. If therefore a detailed investigation is dispensed with, the desire to achieve a certain level of collapse resistance, preferably by using prescriptive design rules which are codified and simple to use (indirect design), still remains.

Concerning the design of buildings, a couple of design rules have been developed. Some of them have made their way into the codes (GSA 2003, EC 1-1-7 2004 (draft), ACI 2002)]. A comprehensive review can be found in Dusenberry and Juneja (2003). The most common rules are as follows:

1. Tying together all main structural elements
2. Enabling catenary action
2. Providing ductility

According to ACI (2002), tying together the main structural elements can be achieved by making continuous a certain amount of reinforcement in the perimeter beams; connections of precast concrete members shall be designed for a given tensile force. Enabling catenary action is a requirement applying to floor slabs. In case of failure of intermediate columns resulting in the destruction of floor slabs, the debris is held in place by the tensile forces within the sagging remnants of the slab. A collapse progression due to the impact loading of falling debris striking other structural elements below is thus avoided. Catenary action is to be achieved by continuity in the slab's top and bottom reinforcement. It might be appropriate to require catenary action in beams as well. Ductility in structural elements and connections is achieved by proper detailing. In case of local failure, ductility allows for the utilization of plastic reserves and the dissipation of kinetic energy. The applicability of seismic design rules to collapse-resistant design is being discussed in Hayes et al. (2005).

All prescriptive design rules, as far as presented in codes, strive to ensure structural integrity through continuity and provision of alternate load paths. In light of the redundancy-versus-compartmentalization discussion above, these rules should be applied with discretion and only to structures within the scope of the respective code (i.e., to buildings, not to bridges). In any case, the force transfer, of forces assigned to alternate load paths, should be checked down to the foundation. In all measures arising from prescriptive design rules (tension ties, floor slabs stabilized by catenary action, plastic hinges, etc.), these forces should be determined based upon the overstrength of the respective tensile elements or the overstrength plastic moments of the plastic hinges.

The idea to isolate a beginning collapse by compartmentalization has not yet led to any prescriptive design rules. For developing such rules, the scope of application, requirements concerning the size of the compartments, and the detailing of the compartment borders must be investigated and specified.

10. Design of Collapse-Resistant Bridges

It follows from the preceding discussion that there is a variety of design methods to choose from or to combine when a structure shall be designed against progressive collapse. It was also 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 characterizing feature of bridges apparently is that they are primarily horizontally aligned structures. Impact loading produced by falling structural components or debris should therefore be less of a concern for bridges (when compared to buildings). It follows that there is less need to provide alternate load paths in bridges, and to tie together structural elements, in order to prevent the fall and impact of components.

Other common properties shared by large bridges, but also by high-rise buildings, are that they exhibit large internal forces and a high degree of structural interaction, and, therefore, are demanding in regard to structural analysis. Furthermore, such structures are mostly unique and expensive. It is concluded that for such structures direct design methods (performance-based design) are mostly preferable, in terms of safety and economy, over indirect design methods (prescriptive design). Further common properties and conclusions with respect to the question at hand are difficult to find. The further discussion will therefore focus on particular bridge systems.

10.1 Continuous Girder Bridges

Typical examples are the Viadotto Cannavino and Haeng-Ju Grand Bridge which collapsed in a progressive manner (see Section 2), and the Confederation Bridge which was designed against progressive collapse (see Section 9.2.2). Based on this experience and the preceding discussion, it seems that there are two design methods to convey collapse resistance to continuous girder bridges. On the one hand, the compartmentalization approach can be used. For the bridge type considered here, the key elements and compartment borders are the regions around the piers and the piers themselves. For small spans up to, say, 40 m, it might be possible to provide these elements with specific local resistance to enable them to act as compartment borders. For larger spans, the compartmentalization approach requires selective elimination of continuity at or close to the envisaged compartment borders. A safe way of doing that is the insertion of unlockable hinges. Other possibilities for eliminating continuity are mentioned in Section 9.2.2.

On the other hand, it can be attempted to provide high safety against local failure. In that case, careful consideration must be given to possible accidental circumstances which should be minimized or designed for. These can include a ship or an airplane that crashes into the bridge, unexpectedly strong ice formations which 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 accidental or malicious actions, deficiencies in design or construction, or simply corrosion. The probability of ship impact is regularly studied 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. The reader is referred to Section 9.1 for further discussion of these and other considerations. As suggested there, the decision to pursue one or the other design method will depend on the bridge size.

10.2 Cable-Stayed Bridges

This bridge type is a good illustration to the statement, made in Section 9.2.2, that the terms continuity, redundancy, and robustness should be carefully distinguished. Although such systems possess a high degree of static indeterminacy and internal continuity, their redundancy and robustness deserve careful consideration. The sudden loss of one or more cables and a possibly ensuing zipper-like collapse progression is of particular concern. The problem is aggravated by a number of factors: The cables are easily accessible and exposed to accidental or malicious action; 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 system might respond in a non-ductile manner.

To meet such concerns, the PTI Recommendations (PTI 2001) require that a cable-stayed bridge shall be capable of withstanding the loss of any one cable without the occurrence of structural instability. A loss-of-cable load case and an associated load combination and applicable partial safety factors are specified. The impulsive dynamic loading resulting from the sudden rupture of a cable is recommended to be determined in a quasi-static analysis using a dynamic amplification factor of two. Application of the PTI Recommendations in the design of recently erected cable-stayed bridges in the U.S. showed that the loss-of-cable load case, as specified, can become a controlling requirement increasing construction costs. It is currently being discussed to modify this specification in the upcoming 5th edition of the PTI Recommendations so that a dynamic amplification factor determined by non-linear dynamic analysis, but not smaller than a certain minimum value, can alternatively be used. Corresponding parametric studies are presently being performed by the author.

Another concern is that the assumption of just one cable rupturing at a time might be insufficient. Having in mind traffic accidents as possible triggers of initial failure, it has been suggested to assume the sudden loss of all cables within a 10-m range measured along the cable anchors. Based on this suggestion, the sudden and simultaneous rupture of any *two* adjacent cables was assumed in the design of Taney Bridge—a recently erected cable-stayed bridge in Ireland carrying two tracks of the Dublin Light Rail Transit system (O'Donovan et al. 2003). These dynamic load cases were accounted for in quasi-static analyses using a dynamic amplification factor of two. They were combined with a specified set of other loadings including live load on the track adjacent to the ruptured 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 traffic loading in order to account for the possibility that a second train crosses the bridge soon after the rupture of the cables while a first train is standing on the bridge. These design criteria had been developed jointly by the designer and the client.

Verifying loss-of-cable load cases is an application of the alternate-load-paths approach. When inspecting the other design methods listed in Section 7, it turns out that the compartmentalization approach does not generally make sense for cable-stayed bridges. Except for multi-span systems, the minimum compartment size, and thus the minimum extent of collapse progression, corresponds to the size of the entire bridge (or at least half the bridge when a center hinge is introduced in a three-span system). The specific-local-resistance approach seems inappropriate due to the small cross-sectional area of the cable's load-bearing element, its presumably small resistance to lateral action, and its major contribution to construction costs. This might become an option at best for small-span bridges. When finally the possible provision of non-structural protective measures is considered, it seems that this approach could be explored more in the future—not so much as a substitute but rather as a complement to the verification of loss-of-cable scenarios. Protective measures should include barriers to fend off vehicles, and fencing to deter trespassers from approaching the cables. To prevent sudden cable rupture due to corrosion, efficient corrosion protection systems of cables and anchors, and regular inspection are needed. As such, these provisions are also non-structural protective measures. Again, it is referred to Section 9.1 for further considerations.

10.3 Suspension Bridges

The hangers of a suspension bridge are secondary load-bearing elements—contrary to the cables of a cable-stayed bridge which are part of the primary load-bearing system. Nevertheless, the sudden rupture of a hanger would likewise lead to an impulsive dynamic loading on the remaining system and a zipper-like failure of adjacent hangers and a collapse progression can be envisaged. Such a collapse progression can be seen in the movie of the Tacoma Narrows Bridge failure in 1940 where the first hangers ruptured due to excessive wind-induced distortions of the stiffening girder and then the entire girder peeled off. Progressive collapse initiated by sudden rupture of a hanger has not been a major concern in the past. With the recent advent of new kinds of threats, it seems sensible, however, to take a different stance. The challenge can be met by making hanger rupture 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 preceding section are applicable.

In the case of a suspension bridge, there is a further possible design method unfeasible for cable-stayed bridges: The stiffening girder could be provided with a number of unlockable hinges which give way in the case of a commencing collapse. In other words, the compartmentalization approach could be used with the hinges forming compartment borders. A collapse initiated by hanger rupture would be isolated by the hinges and limited to one compartment.

The main load-bearing elements of a suspension bridge are the suspension cables. The compartmentalization approach to confront the possibility of a breaking suspension cable could at best be an option in multi-span systems. The alternate-load-paths approach might be feasible in bridges with more than two suspension cables. Increasing the specific local resistance of the suspension cables by increasing the area of its load-bearing element could be an appropriate measure for small spans. For large-span bridges, it is impracticable because of the suspension cables' major contribution to quantities and costs. On the other hand, the suspension cables of large-span bridges exhibit a huge cross-sectional area and mass anyway, and they seem quite resistant to local action even without further strengthening. They also are less sensitive to local lack of resistance caused by wire breakages because there are many wires, and even a broken wire will carry load again at a certain distance from the breakage point.

Nevertheless, and again invoking the advent of new kinds of threats, it seems advisable to take non-structural protective measures to protect the suspension cables—independently of the cables' size. For bridges of high significance or exposure, these measures might have to include appropriate shielding and security systems to safely deter invaders from approaching the suspension cables or their load-bearing components.

10.4 Arch Bridges

Arch bridges bear similarities with suspension bridges in terms of topology and flow of forces. Much of the statements presented in the preceding 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. The same solutions are possible with the exception that the compartmentalization approach is inapplicable to a tied arch. In a true arch, the bridge deck is supported by columns instead of suspended from hangers. A column failure due to accidental circumstances appears less probable. The columns are more resistant to lateral action and not exposed to traffic. On the other hand, they are also more hidden from public view which could favor malicious action. Concerning the arch of an arch bridge, some of the previous statements on suspension cables apply. There are two important differences, though. First, an arch can exhibit stability failure within the arch plane or in lateral direction. This opens up further possibilities of failure initiation and collapse progression. Second, all kinds of cross sections and different types of material are possible and in use for arches, contrary to suspension cables. Concerning local action, which might induce local stability failure and collapse, solid cross sections are more resistant than hollow ones, and concrete seems preferable over steel.

11. Conclusions

It was found that clearer and more practical definitions are arrived at when the term robustness is distinguished from the term collapse resistance, the former being a property of the structure alone, the latter including possible causes of initial failure. In regard to progressive collapse, non-robust structures are of particular concern and require specific consideration. The necessity of such consideration follows from an inspection of current design methods which are based on reliability theory. Because of fundamental difficulties and due to the number and complexity of influencing factors which appear after failure initiation, a purely probability-based design of real structures seems impracticable. A pragmatic design approach was therefore proposed in which the usual probability-based design procedures, as described in the codes, are complemented by an additional assessment and particular design measures with regard to progressive collapse in which analyses are carried out deterministically.

This approach is contrary to the recommendation made in EC 1-1-7 (2004) to perform nothing but a risk assessment for structures of the highest consequence class. It follows from the preceding discussion, however, that such a study can at best supplement, not replace, a design approach as it is presented in this paper. The structural designer, not the reliability theorist, should be in charge of the collapse-resistant design of a real structure. On the other hand, reliability theory can play a role in a more detailed comparison of the design methods discussed here and in determining some design criteria like exposure, assumable extent of accidental circumstances, or applicable safety factors. Nevertheless, these and other design criteria might in the end be left to engineering judgment. The dependency of the accuracy of such judgment, and the importance of reliability theory, is relatively high when using the design strategy “high safety against local failure,” less so for the design method “alternate load paths,” and minimum for the design method “isolation by compartmentalization.” Still other design criteria need to be stipulated in a decision-making process. The choices to be made in that process, e.g., concerning the acceptable extent of collapse progression, are relatively transparent so that an informed societal consensus should be possible.

The alternate-load-paths approach and, in particular, the prescriptive design rules based on that idea should be applied with discretion. Forces should be determined based on the overstrength of elements introduced for continuity and the force transfer should be checked down to the foundation. Compartmentalization can be accomplished either by a strengthening or by a reduction of continuity at the compartment borders. In other words, the compartment borders must be able to sustain either large forces or large displacements. For certain structures, compartmentalization is the more suitable approach to prevent progressive collapse—a fact that has gone nearly unnoticed in the structural engineering community. If this option has been overlooked, one reason might be that the terms continuity, redundancy, and robustness are intuitively equated, a tacit assumption which is justified at best for particular types of structures.

The adequacy of a particular design method depends on the design objectives, and on the type of structure and its alignment in space. Bridge structures are primarily horizontally aligned whereas buildings can be horizontally or vertically aligned. Impact loading produced by falling structural components or debris is more of a concern for high-rise buildings, not so much for bridges. The need to provide continuity to prevent such impact loading is therefore different in these two kinds of structures. Various bridge systems were considered in more detail and some specific recommendations were given. This demonstrates the usefulness of the terms and general concepts developed in this paper.

References

ACI (2002). "Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02)." ACI Committee 318, American Concrete Institute, Farmington Hills, MI, January 2002.

Andrews, K. R. F., and England, G. L. (1995). "A computer program for the progressive collapse analysis of steel frames subject to fire." Proc., Int. Con. on Mechanics of Solids and Materials Engineering, Singapore, June 1995, 1010-1015.

ASCE (2002). "Minimum Design Loads for Buildings and Other Structures (ASCE 7-02)." American Society of Civil Engineers, Reston, VA, December 2002.

Bažant, Z. P., and Zhou, Y. (2001). "Why did the World Trade Center collapse?" *Society for Industrial and Applied Mathematics News*, 34(8).

Bennett, R. M. (1988). "Formulations for probability of progressive collapse." *Elsevier Science Publishers, Structural Safety*, 5(1).

Breen, J. E. (1975). Research workshop on progressive collapse of building structures. University of Texas at Austin, November 18-20, 1975, National Bureau of Standards, Washington, D.C.

Breen, J. E., and Siess, C. P. (1979). Progressive Collapse – Symposium Summary. *ACI Journal*, 76(9), 997-1004.

Breugel, K. van (1997). "Storage system criteria for hazardous products." *IABSE, Structural Engineering International*, 7(1), 53-55.

BS5950-1 (2001). BS5950-1:2000 "Structural use of steelwork in building – Part 1: Code of practice for design – Rolled and welded sections." August 2001.

CEB (1991). CEB-FIP Model Code 1990. Comite Euro-International du Beton, Bulletin d'Information No. 203, Lausanne.

Corley, W. G., Sozen, M. A., Thornton, C. H., and Mlakar, P. F. (1996). "The Oklahoma City Bombing: Improving Building Performance through Multi-Hazard Mitigation." Federal Emergency Management Agency Mitigation Directorate, FEMA Report 277.

Dusenberry, D. O., and Juneja, G. (2003). "Review of existing guidelines and provisions related to progressive collapse." Workshop paper, see MMC (2003).

EC 0 (2002). EN 1990:2002 – Eurocode 0. "Basis of structural design."

EC 1-1-7 (2004). Project Team Draft prEN 1991-1-7. "General Actions – Accidental Actions." July 2004.

Ellingwood, B., Leyendecker, E. V., and Yao, J. T. P. (1983). "Probability of failure from abnormal load." *Journal of Structural Engineering*, 109(4), 875-890.

FEMA (2002). *World Trade Center Building Performance Study*. FEMA Report 403, Federal Emergency Management Agency – Federal Insurance and Mitigation Administration, Washington, D.C. (Excerpt in *ASCE, Civil Engineering*, 72(5), May 2002.)

Ghali, A., and Tadros, G. (1997). "Bridge progressive collapse vulnerability." *Journal of Structural Engineering*, 123(2), 227-231.

Gilmour, J. R., and Virdi, K. S. (1998). "Numerical modelling of the progressive collapse of framed structures as a result of impact or explosion." 2nd Int. Ph.D. Symposium in Civil Engineering, Budapest, Hungary, August 26-28, 1998.

Gross, J. L., and McGuire, W. (1983). "Progressive collapse resistant design." *ASCE, Journal of Structural Engineering*, 109(1).

GSA (2003). "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects." U.S. General Services Administration and Applied Research Associates, June 2003.

Hawkins, N., and Mitchell, D. (1979). "Progressive collapse of flat plate structures." *ACI Journal, Proceedings*, 76(7), 775-808.

Hayes Jr., J. R., Woodson, S. C., Pekelnicky, R. G., Poland, C. D., Corley, W. G., and Sozen, M. (2005). "Can strengthening for earthquake improve blast and progressive collapse resistance?" *ASCE, Journal of Structural Engineering*, 131(8), 1157-1177.

JCSS (2005). Robustness of Structures. Workshop, JCSS & IABSE, Building Research Establishment, Watford, UK, November, 28-29, 2005.

Lee, M. S. (1998). Personal communication.

Leyendecker, E. V., and Ellingwood, B. R. (1977). "Design methods for reducing the risk of progressive collapse in buildings." Nat. Bureau of Standards, Building Science Ser., 98.

Mitchell, D., and Cook, W. D. (1984). "Preventing progressive collapse of slab structures." *Journal of Structural Engineering*, 110(7).

MMC (2003). Workshop on Prevention of Progressive Collapse. Chicago, July 10-12, 2002, Multihazard Mitigation Council of the National Institute of Building Sciences, MMC Report, Washington, D.C., http://www.nibs.org/MMC/mmcactiv9.html.

Moore, D. B. (2003). "The UK and European regulations for accidental actions." Workshop paper, see MMC (2003).

NRC (1996). "National Building Code of Canada 1995." National Research Council Canada, Ottawa, Canada.

O'Donovan, J., Wilson, K., and Dempsey, T. (2003). "The design and construction of Taney Bridge, Dundrum." Tom McCormack Memorial Lecture, January 21, 2003, The Institution of Structural Engineers, Republic of Ireland Branch.

PCI (2004). Blast and Progressive Collapse Resistance of Precast and Prestressed Concrete Structures. PCI Workshop, Chicago, April 21, 2004.

Pekau, O. A., Zielinski, Z. A., and Lee, A. W. K. (1984). "Progressive collapse in precast shear walls." Third International Conference on Tall Buildings, Hong Kong & Guangzhau.

Prendergast, J. (1995). "Oklahoma City aftermath." *ASCE, Civil Engineering*, 65(10).

PTI (2001). "Recommendations for Stay Cable Design, Testing and Installation." 4th Edition, Post-Tensioning Institute, Cable-Stayed Bridges Committee, Phoenix, Arizona.

Starossek, U. (1997). "Zum progressiven Kollaps mehrfeldriger Brückentragwerke." *Bautechnik*, 74(7), 443-453.

Starossek, U. (1999). "Progressive collapse study of a multi-span bridge." *IABSE, Structural Engineering International*, 9(2), 121-125, http://server.sh.tu-harburg.de/starossek/Index.htm.

Starossek, U. (2001). "Progressiver Kollaps von Tragwerken." *WKP Ingenieurforum, WKP König und Partner*, Hamburg, December 4, 2001.

Starossek, U. (2005). "Progressiver Kollaps von Bauwerken." *Beton- und Stahlbetonbau*, 100(4), 305-317.

Starossek, U., and Sauvageot, G. (1998). Discussion of "Ghali, A.; Tadros, G. (1997): Bridge progressive collapse vulnerability." *Journal of Structural Engineering*, 124(12), 1497-1498.

The Building Regulations 2000 (2004). "Structure: Approved Document A." The Stationery Office.

Wearne, P. (2000). "Collapse – When Buildings Fall Down." ISBN 1-57500-144-6, New York, TV Books, L.L.C.

Wittfoht, H. (1983). "Ursachen für den Teil-Einsturz des `Viadotto Cannavino´ bei Agro di Celico." *Beton- und Stahlbetonbau*, 78(2).

Yokel, F. Y., Wright, R. N., and Stone, W. C. (1989). "Progressive collapse: U.S. Office Building in Moscow." *Journal of Performance of Constructed Facilities*, 3(1), 57-75.