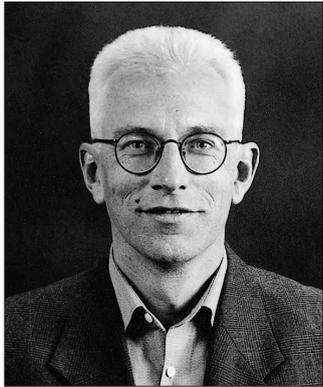


Progressive Collapse Study of a Multi-Span Bridge



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Summary

The overall structural response to local failure has become an issue of attention for certain types of large-scale structures. The significance of such considerations is discussed here. A progressive collapse study of a multi-span prestressed concrete bridge is presented. The analysis strategy, the results obtained, and the ensuing impact on the design of this bridge are discussed.

Introduction

Local failure of a structure, i.e. the failure of one of its constituent elements, may result in the failure of another structural element. In such a way, failure might progress throughout a major part or all of the structure.

Different structural systems exhibit different degrees of sensitivity toward progressive failure. These different degrees of sensitivity are neglected when conventional design approaches are used, which typically investigate sectional forces and focus on the safety of the individual structural elements rather than on the safety of the entire structure.

Progressive collapse may have catastrophic consequences. For example, the Alfred P. Murrah Federal Building in Oklahoma City collapsed on 19 April 1995, causing the deaths of 168 people. The collapse was triggered by a car bomb outside the building, but subsequent investigations showed that the extent of the tragedy was largely due to a property of the structural system. Only one in three of the building's outer columns was supported on its own foundation. The other columns rested on a transfer girder that ran across the face of the structure on the second floor [1]. Such a structural system has little redundancy, and the loads carried by a failing column cannot be redistributed to neighbouring columns. Thus, failure will not remain locally limited but will spread further throughout the structure.

We can assume that the designers of this building had followed applicable

codes and current rules of practice. Nevertheless, had they based their design on a more redundant structural system with all columns being extended to foundation level, the resulting overall safety level would have been higher and the consequences of the bomb attack would have been less disastrous. (Alternatively, accidental failure of individual main columns should have been considered as additional design load cases.)

Further events of structural collapse involving mechanisms of failure progression intrinsic to the structural system have occurred recently. Present codes give no guidance on how to prevent progressive collapse or, more precisely, how to provide a homogeneous level of global safety to different kinds of structures. The problem has been identified though, and is addressed in some codes. The first paragraph of the CEB-FIP Model Code 1990 [2], for instance, states that structures should "withstand accidental circumstances without damage disproportionate to the original events". Following this general rule, however, requires that engineers use judgement and that owners are wise enough to accept or even encourage engineering advice beyond that required by the explicit code requirements.

In the following, a large multi-span bridge project is presented. A progressive collapse study of this project is outlined, and its impact on the final design is discussed. This study was carried out by the author while working for J. Muller International, San Diego.

Northumberland Strait Crossing Project

The Northumberland Strait Crossing Project, now called the Confederation Bridge, is a prestressed concrete bridge between Prince Edward Island and the mainland of New Brunswick, Canada. Construction was started in spring 1994, final closure was achieved in autumn 1996, and the bridge opened to traffic in summer 1997.

The bridge is 12.9 km long, and consists of a main bridge of 43 continuous 250-m spans and approach viaducts on either side of the main bridge (Fig. 1). The cross section of the superstructure is a mono-cellular box with deck-slab cantilevers (Fig. 2). The girder depth changes continuously from 14.0 m at the piers to 4.5 m at midspan. The deck rises up to +59 m above mean sea level. The piers are supported on ring foundations down to -38 m [3].

The entire main bridge, including its substructure, is made of large-scale

components pre-fabricated on shore. The principal components are the pier bases, which rise to +4.0 m above sea level, the pier shafts, which include massive conical ice shields extending down to -4.0 m, the 192.5-m-long cantilever main girders, and drop-in girders 52 m or 60 m in length (Fig. 2).

Moment-resistant connections are provided between the cantilever main girders and the pier shafts by means of post-tensioning. Every second main span is closed with a drop-in girder made continuous with both cantilevers, thus creating a series of two-column portal frames. Continuity of these drop-in girders to the cantilever ends is accomplished through cast-in-place joints and external post-tensioning. The remaining spans are completed with drop-in girders that are simply supported on the cantilever ends.

Preliminary Design

The final design described above deviates from the preliminary design (Fig.

3) in several respects. In the preliminary design, the lengths of the cantilever main girders and the drop-in girders were 150 m and 100 m, respectively. The cantilever depth changed linearly, and the depth of the drop-in girders was constant. Instead of simply supported drop-in girders in every other span, the girders were designed with continuity at one end and a hinge at the other.

This concept seemed to be advantageous in terms of ease of construction, construction costs and maintenance. However, it was inadequate in that the structure would not have been robust enough to prevent progressive collapse following accidental loss of one span.

Progressive Collapse Study

The need to perform a progressive collapse study for the final bridge design was formally established in cooperation with Public Works Canada, the supervising authority. The engineers responsible for the final design (J. Muller International-Stanley Joint Venture, San Diego) studied possible mechanisms of progressive collapse and means of protecting the structure against such an outcome.

Philosophy of Investigation and Design

Progressive collapse could be triggered by a number of stimuli, for example:

- a ship or an aeroplane might crash into the bridge

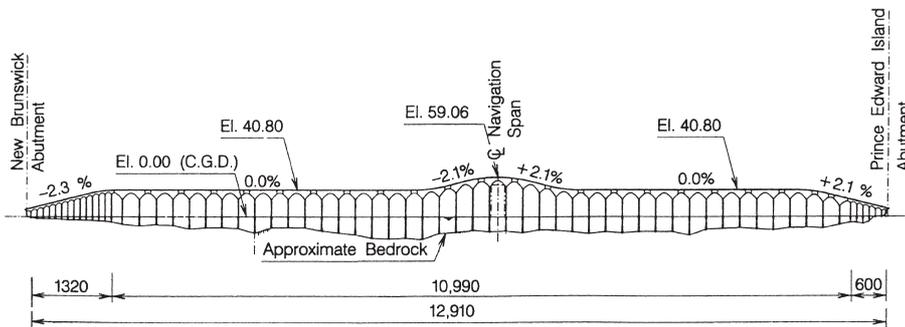


Fig. 1: The Northumberland Strait Crossing Project

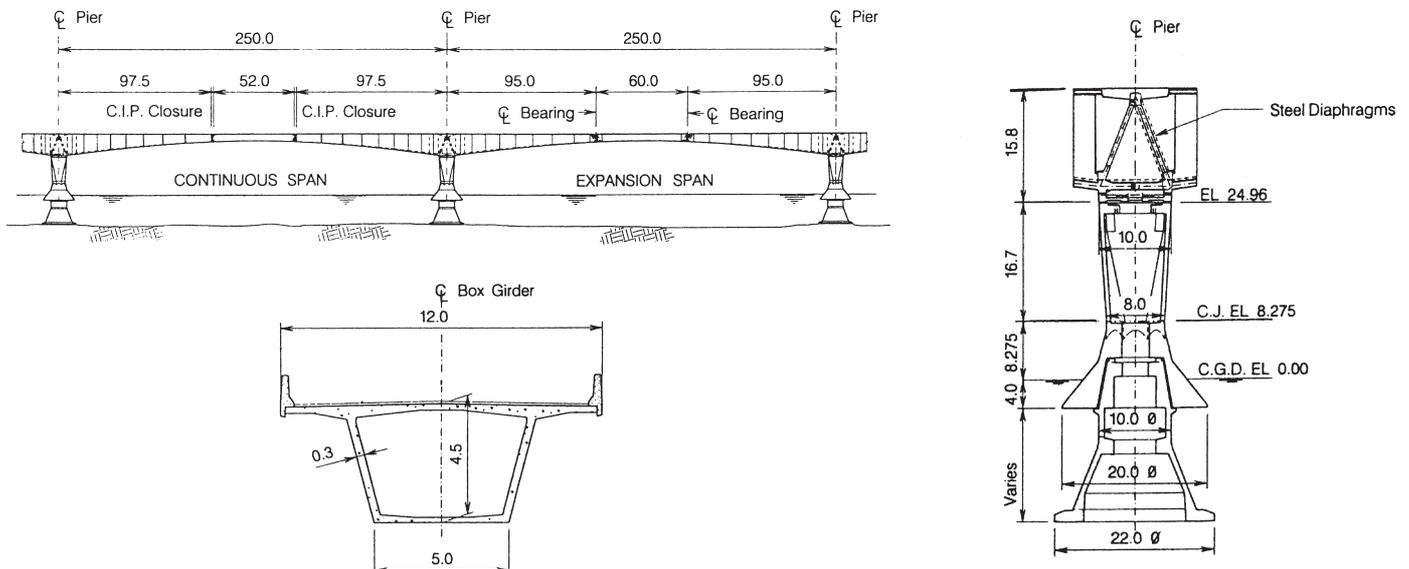


Fig. 2: Elevation, cross section of superstructure, and longitudinal section at the piers of the main spans

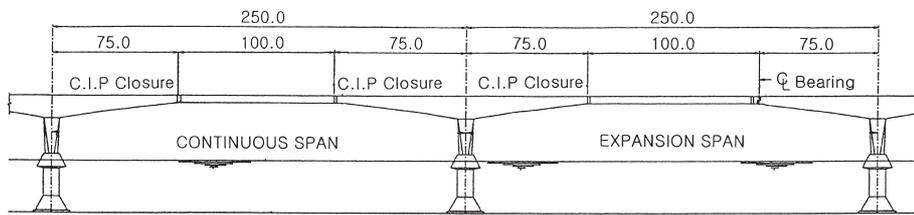


Fig. 3: Preliminary design elevation of the main spans

- unexpectedly strong ice formations might collide with a pier
- fire caused by a traffic accident might damage the cantilever tendons in the top slab
- a bomb placed at a vulnerable location might explode.

In view of the accidental nature of the potential triggers and the large dimensions of this structure, it would be unrealistic to design against progressive collapse by preventing local failure at any expense. Instead, the possibility of a local failure must be accepted, and this should then become the starting point for further investigation.

The need for a progressive collapse analysis can be demonstrated by using the stochastic concepts of risk and reliability theory [4]. However, application of such concepts to project-related design work is not only difficult, but in the context of accidental loading and progressive collapse it is controversial. The statistical data are not sufficient to reliably establish the probability of triggering events, and the magnitude of potential losses is not acceptable to society [5]. Thus, instead of a stochastic risk analysis, a deterministic analysis has been performed. It is based on certain assumptions and premises regarding failure mechanisms and maximum admissible failure progression.

Depending on the accidental triggering event, initial failure might occur in the transverse direction or in the vertical plane through the bridge axis. Because of the joints in the bridge deck, a transverse failure would not give rise to substantial horizontal forces in the adjacent bridge sections. However, it would produce large vertical forces and could continue as a failure in the vertical plane. Therefore, only the latter case has been investigated further.

The following approach has been developed for the preliminary structural system (Fig. 4). A collapse triggered by the failure of pier B or C should stop,

at the latest, at hinge H1 and at pier D. It is assumed that the drop-in girders slide off their respective bearings at hinges H1 and H2 so that the vertical supports at these locations are suddenly lost. The responses of the remaining structures (to the right of H1 and to the left of H2) to these dynamic loads are investigated.

Loss of Hinge H2

The response of the structure to the left of hinge H2 (Fig. 4) after a sudden loss of this hinge has been investigated. The sequence of collapse, according to static and dynamic analysis, is marked by several distinct events (Fig. 5):

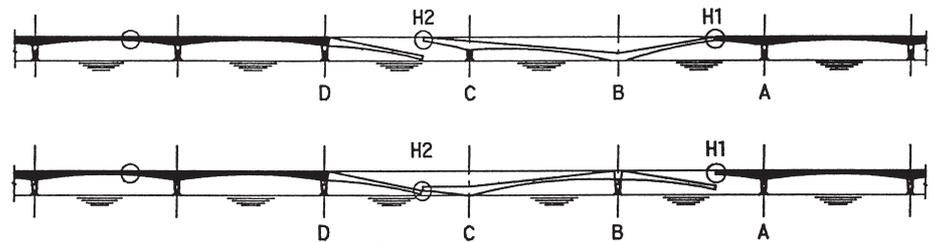


Fig. 4: Working hypotheses on the onset of progressive collapse

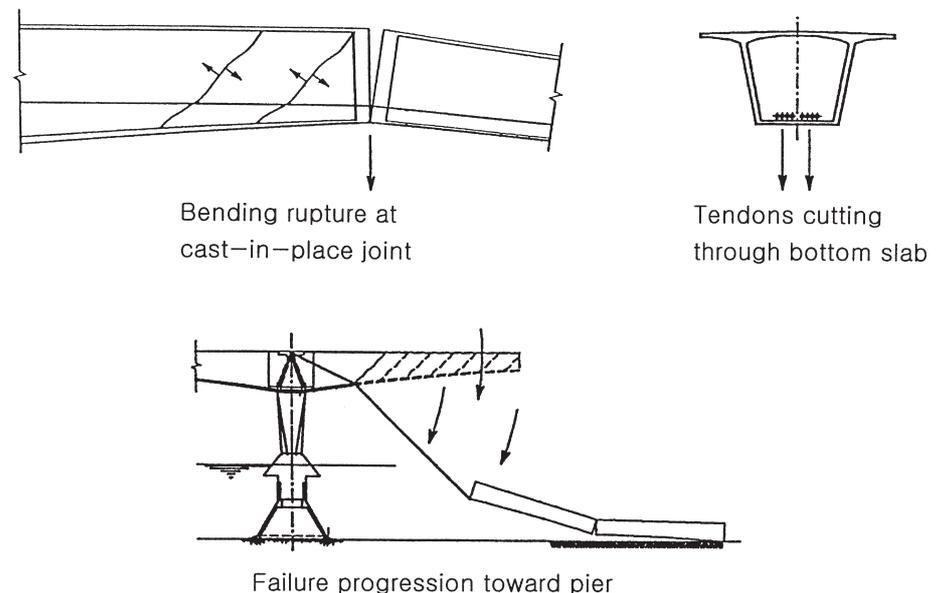


Fig. 5: Collapse after loss of hinge H2

- The girder fails in bending under its own weight at the cast-in-place joint between the cantilever and the drop-in girder.
- 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, and the drop-in girder ruptures due to bending under the inertia forces induced by its own mass [6].
- 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.

Further analytical prediction was deemed to be inaccurate and unreliable. Nevertheless, failure of the adjacent span (to the left of D; Fig. 4) and, thus, progressive collapse seemed possible.

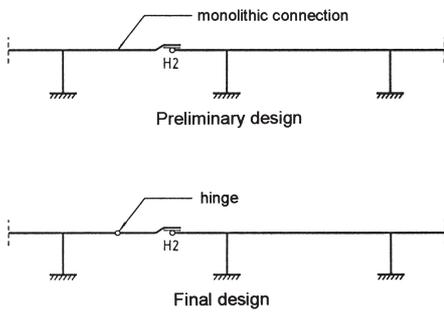


Fig. 6: Modification of structural system

The only way to achieve a predictable response was to allow for an early separation of the falling drop-in girder from the remaining system. It was attempted to design a structural fuse within the cast-in-place joint between the cantilever and the drop-in girder. However, a reliable method to automatically cut the continuity tendons (after collapse onset) was not found, and the idea was abandoned.

The toughness of prestressing steel tendons against lateral cutting action became apparent during the progressive collapse of the Haeng-Ju Grand Bridge, Seoul, in 1992. It has been reported that none of the deck tendons broke in that incident. Instead, the tendons cut through the reinforced concrete cover at the point of deck rupture [7]. The high integrity of the tendons resulted in the collapse of all 11 continuous spans between adjacent expansion joints.

An early separation of the drop-in girder from the remaining system appeared to be guaranteed only by insertion of additional hinges. Therefore, the structural system of the preliminary design was modified (Fig. 6).

In the modified system, the drop-in girders in every other span are simply supported on the cantilever ends. The length of these girders was reduced from 100 m to 60 m in order to ensure separation before the free end of the girder hits the water. As additional benefits from this length reduction, the forces generated by the rotating drop-in girder are lower, and the progressive failure resistance of the adjacent span is increased.

The insertion of additional hinges makes the load case “loss of hinge H2” equivalent to load case “loss of hinge H1”, which is discussed below. Credit should be given here to the authors of an earlier conceptual design, which was based on the same structural system as that finally chosen [6].

Static Indeterminacy and Robustness

As stated above, insertion of additional hinges was the only way to achieve a predictable response. By assuming that not only the predictability but also the robustness is expected to be improved by this measure, the following question can be raised: how can a reduction of the system’s degree of static indeterminacy (which is considered a measure of redundancy) increase robustness?

A partial explanation might be that a progressive failure requires a certain degree of connectivity and interaction between neighbouring structural elements, properties usually associated with the system’s degree of static indeterminacy. The fact that progressive failure involves violent dynamic effects might also contribute to this counter-intuitive effect.

Loss of Hinge H1

The response of the structure to the right of H1 (Fig. 4) after a loss of this hinge has been investigated. Because of the modification of the structural system, this loss does not need to be a sudden event associated with a step-impulse type of loading. Instead, sudden loss of the hinge at the opposite end of the drop-in girder might occur, leaving that girder to rotate around hinge H1. Final separation will take place at an angle of rotation defined by the geometry of the hinge corbel (Fig. 7).

The vertical hinge force at H1 during this more gradual event is indicated in Fig. 8. The force was calculated by establishing and solving the non-linear equation of motion that describes the

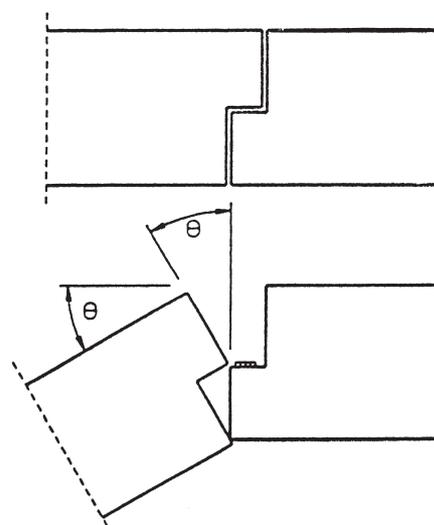


Fig. 7: Disengagement of drop-in span forced by corbel geometry

rotation of the drop-in girder [4]. At the start of rotation, the vertical force at the cantilever tip drops to 50% of its static value. During rotation, it increases gradually and eventually exceeds the static value. When the angle of disengagement is reached, the force suddenly disappears.

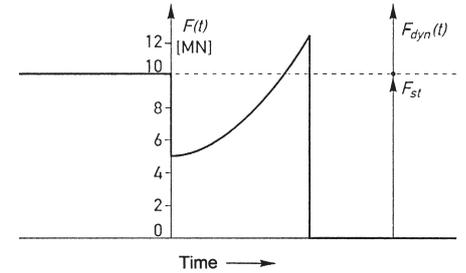


Fig. 8: Vertical force at cantilever tip during fall of drop-in girder

Fig. 9 shows the dynamic part of this load function (defined in Fig. 8) and the response of a single-degree-of-freedom system, calculated by means of a Duhamel integral. All quantities are made dimensionless by relating them to the respective static values. The load function’s initial step impulse of 0.5 excites the system to a maximum response of almost 1. The second loading step, to a final value of 1, causes the system response to exceed 2, which would be the maximum response value for a simple unity-step impulse loading [8]. Hence, a gradual separation of the drop-in girder can produce higher forces in the remaining structure than a sudden loss. The reason for this is the second step impulse, which results from final disengagement and might produce a resonance-like dynamic amplification.

The maximum response depends on the ratio of the time of final disengagement to the system’s period of vibration. The analysis was therefore repeated for different periods of vibration. Fig. 10 shows the resulting spectra of extreme responses, to the same dynamic loading, of a single-degree-of-freedom system as functions of its period of vibration. Based on this investigation, it was concluded that the dynamic response could be up to 2.6-fold higher than the static response (i.e. the response when the same load is applied very slowly).

Time-history space-frame analyses of the remaining structure subjected to the same dynamic loading were performed. It has been corroborated that, for the sake of simplicity, a quasi-static

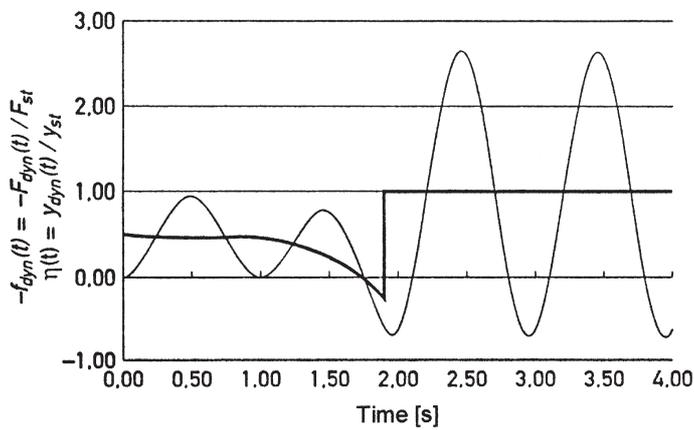


Fig. 9: Impulsive load (–) and response (–) of a single-degree-of-freedom system

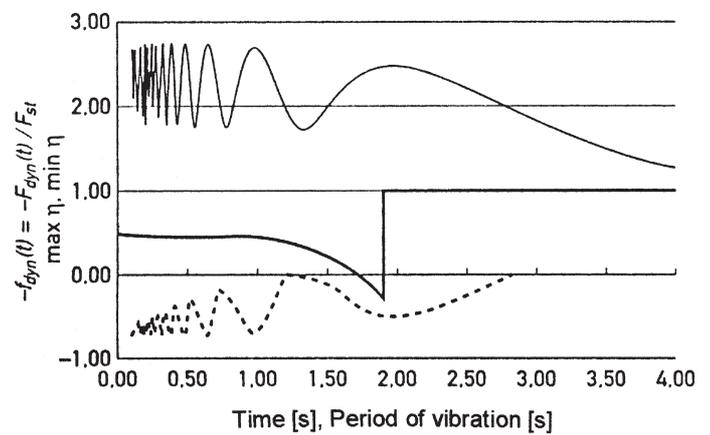


Fig. 10: Impulsive load and response spectra of a single-degree-of-freedom system (– dynamic load function; – extreme positive response; ... extreme negative response)

approach can be used in the analysis of the remaining structure and thus in the detailed design of the bridge. The structure is loaded with the static hinge force at H1, applied in the opposite direction and multiplied by a dynamic amplification factor of 2.6.

In view of the accidental nature of this loading, the formation of plastic hinges was deemed acceptable, and the plastic reserves of the structural system have been utilised in the detailed design against progressive collapse.

Additional Design Modifications

The final design of the Northumberland Strait Crossing Project was influenced strongly by the investigation on progressive collapse. Further to the design changes mentioned above, the following modifications were necessary in order to avoid progression of a local failure into the adjacent spans:

- post-tensioning between the superstructure and the piers was increased to the maximum level possible, considering the given pier geometry, in order to limit the moments that have to be redistributed into the superstructure after formation of a plastic hinge at the pier top
- the shape of the superstructure's soffit was changed from haunched to curved in order to increase the section depth and moment capacity where the drop-in girders are con-

nected monolithically to the cantilevers

- top and bottom reinforcement was added around the quarter points of the continuous spans to limit the number of plastic hinges in the superstructure to one
- transverse reinforcement was added in the regions of expected plastic hinges in order to provide sufficient rotational capacity.

A detailed account of the investigations outlined here and their impact on the final design of the Northumberland Strait Crossing Project can be found in [4].

Conclusions

The requirement to avoid progressive collapse in the event of local failure is an important design criterion for multi-span bridges and other complex structures. It can have a strong impact on both the conceptual design, including the choice of structural system, and the detailed design.

Current design codes do not strictly require the prevention of progressive collapse. Recent disasters and theoretical considerations on the basis of risk theory indicate that design codes should be improved to address this problem more clearly. In the meantime, owners and engineers should be

encouraged to use judgement and discretion to implement the necessary measures.

This article is dedicated to Prof. Jörg Schlaich on the occasion of his 65th birthday.

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