Multi-Load-Path Structures for Highway Bridges

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The failure of a component or connection that had been considered vital to the structural integrity of a bridge does not always result in the collapse of the bridge as a whole. When a bridge survives such a failure, it has been possible to reconstruct the bridge safely and to reopen it to use after a short time. This survival of the structure when total collapse would be expected is believed to be due to the ability of most structures to redistribute loads after the failure of a component or connection. Multi-load-path structures have this ability, but single-load-path structures do not. Although this ability to redistribute loads is the result of an unintentional backup system, no present bridge design criteria require consideration of such an ability. The objectives of this paper are to establish, by providing proper definitions, a framework of reference for further discussion, to explore the merits of excluding single-load-path structures from future designs, and to describe the extra design work required if such considerations become criteria for design.

It has frequently been observed that the failure of a component or connection considered vital to the structural integrity of a bridge did not result in the collapse of the whole bridge In such cases it has been possible to safely reconstruct the bridge and to soon reopen it. Although temporary closure is a definite inconvenience and emergency reconstruction may be costly, these disadvantages are insignificant in comparison with the consequences of a total collapse. For example, should the bridge span a waterway, the wreckage could conceivably block navigation for a long time. Similarly, the collapse of an overpass onto an important highway artery could result in dire consequences regarding life, property, and traffic.

The reason for this phenomenon of a structure's surviving when total collapse was expected is believed to be the ability of most structures to redistribute loads after a component or connection fails. Multi-load-path structures have this ability, while single-load-path structures do not. Most structures seem to possess this ability, as the result of an unintentional backup system, but no present bridge design criteria require it.

The objective of this paper is to establish, by providing proper definitions, a framework of reference for further discussion, to explore the merits of excluding single-load-path structures from future designs, and to describe the extra design work required if such considerations become criteria for bridge design.

HISTORICAL BACKGROUND

The total number of bridge collapses in the United States and Canada is estimated to have been about 250/year in the last few years. The number of structures that have had component or connection failure is undoubtedly much higher. The collapses may be attributed to one or a combination of the following causes:

1. Overload due to live load and impact,
2. Collision of heavy vehicles with the structure,
3. Fatigue, with or without brittle fracture,
4. Failure or excessive movement of substructures or both, or
5. Deterioration of structural components or connections or both.

It is part of human nature to hide one's faults, and this is just as true of those who deal with bridges as of anyone else. Consequently, most failures and collapses are not reported at all, and it is conceivable that a researcher's attempt to unearth the details of such events would not be welcomed. In the following, only a few cases will be given, enough to identify the problem. These cases are either in the public domain or are known to us at first hand.

Silver Bridge

The most notorious collapse, and one that made a profound impact on the consciousness of a complacent American public, was that of the Silver Bridge. From the structural point of view, the bridge is a combination of a three-span chain-suspension system and stiffening trusses. The redundancies and, for that matter, the stability of the bridge are conditional on the presence of tension in the chains.

The story of the collapse is well documented (1). It began with the fatigue failure of one of the two eyebars in a section (Figure 1). The failure of the one resulted in the connecting pin's being pulled out of the other. With the eyebar chains gone, the trusses proper were simply supported and, being unable to resist the combined dead and live loads present, they collapsed.

Some of the chain bridges in Europe and South America are rather old but in no appreciable danger of collapse. Figure 2 illustrates one of these bridges, which is located on the Danube in Budapest and was completed in 1849. The chains consist of alternating 12 and 13 eyebars to a section, so that the failure of one bar would be of no significance as far as structural safety is concerned.

The collapse of the Silver Bridge resulted in 46 deaths, a long and costly investigation, a monumentally inconvenient detour, and a multimillion-dollar reconstruction. Authorities immediately closed another bridge on the Ohio River that was similar in construction, for fear of collapse. This bridge also was later replaced at a high cost.

LaFayette Street and I-79 Bridges

The main girders of two bridges, the LaFayette Street Bridge over the Mississippi River in Minneapolis-St. Paul (1974) and the I-79 bridge over the Ohio River (1977), failed due to a combination of brittle fracture and fatigue that originated from incomplete fusion of welds of a wind-bracing gusset plate and an electroslag flange joint, respectively. Both superstructures consist of two welded-plate girders 3.5 m (11 ft) high and continuous over three spans. The plate girders are interconnected by cross frames, wind bracings, and a composite concrete deck. Because of the similarity between the two, only the I-79 bridge is shown in Figure 3. Both failures occurred in one girder in the central span, close to midspan. Although in both cases the tension flange and approximately 90 percent of the web fractured, neither bridge collapsed.

The failure of each bridge was discovered in time by the respective authorities. They were closed and quickly reconstructed. In preparation for the reconstruction of the I-79 bridge, an extensive study (2) was carried out regarding the distribution of loads following the failure. After several attempts that used traditional methods of investigation, the structure was idealized as a space frame for a STRUDL-type analysis. This analysis clearly indicated that, because of torsional stiffness and longitudinal continuity of the superstructure, a significant redistribution took place that permitted the bridge to carry all dead loads with some margin to spare after the failure of one main load-carrying component.

Figure 1. Connection of eyebars of the chain.

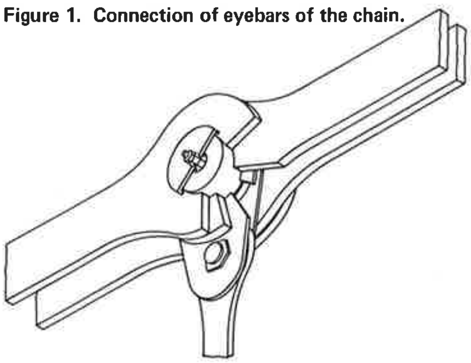
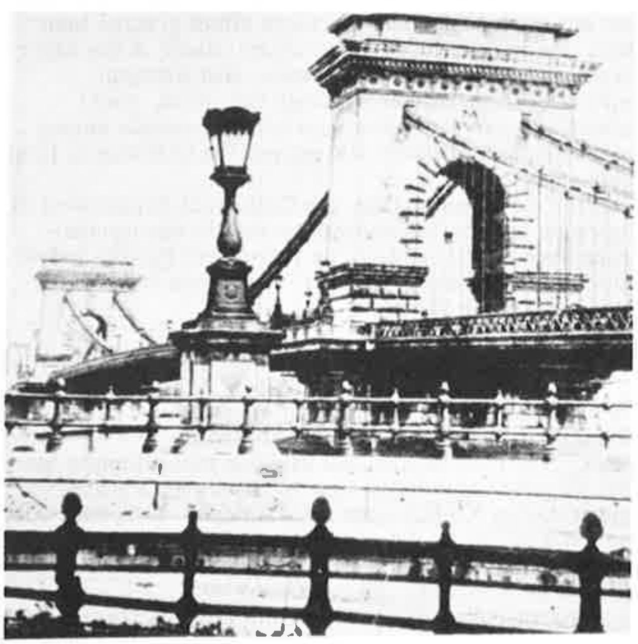


Figure 2. Lanchid over the Danube, Budapest, Hungary.



Ontario-35 Bridge

During the summer of 1976, a bridge on Ontario-35 at Minden, which consisted of four steel girders continuous over three spans, was discovered to have one girder completely fractured in the northern span. A photograph of the bridge, after discovery of the failure, is shown in Figure 4. The fracture, located about 2 m (6 ft) from the pier, was caused by two major inclusions in the welds of the girder web. As usual, the failure was identified as a combination of brittle fracture and fatigue.

After discovery of the failure by a boatman, the fractured girder was supported by an emergency falsework and later reconstructed by installing a fully bolted connection. The bridge was closed to traffic for only one day. It is obvious that the three intact girders, helped by their continuity over the internal piers, had been able to redistribute the loads safely after the failure.

Ontario-33 Bridge

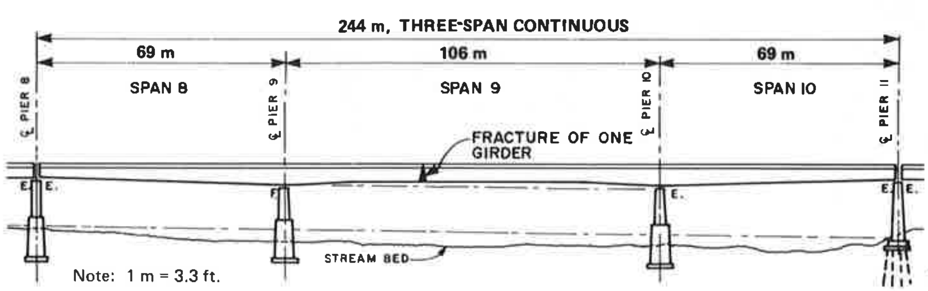
In 1969, during an extremely cold winter night, the lower chord of one steel truss of a bridge fractured completely at the end of a welded cover plate. This bridge is located on Ontario-33 at Frankford (Figure 5). Since this bridge is simply supported, complete separation in a chord member would be expected to precipitate its collapse. Such was not the case. The separation resulted in a one-sided deflection of about 13 cm (5 in), which helped to bring the problem to light. When the separation was found, the structure was jacked back to its original shape by an ice-supported temporary falsework and the member was rehabilitated by means of a bolted connection. The bridge was closed for only a week.

An analytical investigation carried out later revealed that the deck system, which consisted of cross beams, stringers, and an unintentionally composite concrete deck, had taken over the role of the tension chord of the main truss on a temporary basis after some considerable but harmless deformation of the superstructure.

Truss Bridges

As described by Sanders, Elleby, and Klaiber (3), a number of bridges made obsolete by the building of the Saylorville Dam on the Des Moines River were subjected to load tests in Iowa during 1974. One test on the single-lane truss bridge illustrated in Figure 6 included cutting its vertical members at one cross section. Theoretically, the discontinuity of one web member would result in the collapse of the truss, since it would have lost its entire load-carrying capacity in shear. The test indicated that the actual load-carrying capacity of the bridge had not been appreciably de-

Figure 3. I-79 bridge over the Ohio River.



creased by this induced discontinuity. Subsequent investigations revealed that the combined effect of the frame action inherent in trusses and the semicontinuous nature of the deck system could provide an adequate level of shear capacity.

In contrast to the encouraging outcome of the Iowa tests, a through-truss bridge in the eastern part of the United States was reported to have collapsed after a commercial vehicle collided with one end-diagonal member in 1975. The end diagonal plays three roles: It carries maximum shear in the truss, it is part of the compression chord, and it also is the leg of the end portal frame. Being at the entrance to the structure, it is the structural component most likely to be hit by an errant vehicle. When such a component fails, due to overload or collision, the system cannot offer alternative load paths and the bridge collapses.

Figure 4. Ontario-35 bridge at Minden.

Figure 5. Ontario-33 bridge at Frankford.

Figure 6. Hubby bridge over the Des Moines River.

Excessive Movement

The collapse of a major bridge that was continuous over several supports, in the western part of the United States, was reported in 1977. An investigation revealed that the chain reaction that destroyed at least three spans was precipitated by the excessive movement of one pier due to scour. It is not entirely clear whether design and construction errors or wrong assessments of the hydraulic conditions were responsible for the collapse.

Because of the excessive movement of the pier, the expansion bearings jumped their seats, so that the end span became a cantilever unable to carry its own weight. The failed span apparently created force effects, for which the structure was not designed, that resulted in the subsequent failure of two more spans and piers. In retrospect it is obvious that a stopper device applied to the expansion bearing could have prevented the collapse.

A similar phenomenon, although on a much larger scale, was observed in California after the 1971 earthquake, which caused the collapse of at least six bridges and damaged scores of others. One of these bridges is shown in Figure 7. In order to prevent a similar happening in the future, the California Department of Highways ordered a major investigation that included the construction of a number of silicon rubber models that were properly compensated for mass distribution. Excitation of the models was attained by using specially designed shaker tables that permitted any combination of frequency, amplitude, and direction of application.

The model tests revealed that it was rather unlikely that any earthquake-induced force effect greater than those due to dead loads could cause failure of the superstructure. It was found, however, that bridges, especially those that incorporate tall piers, could exhibit excessive relative dynamic movements among their components that would permit the bearings to jump their seats.

After the investigation, the California Department of Highways ordered all bridges located in earthquake-prone areas of the state to be retrofitted (4) with safety devices to limit these relative movements to predetermined maximum values.

DEFINITIONS AND INTERPRETATIONS

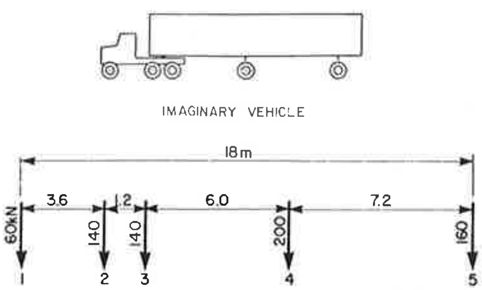
The case histories presented above are intended to provide the reader with a glimpse into the complexity of the issue. The first design specification that attempts to deal with the issue is the new 1979 Ontario highway bridge design (OHBD) code (5). Unless it is specifically approved by the minister of transportation and communications, the construction of single-load-path structures is prohibited by the code.

When approved, the permissible fatigue stresses in conjunction with type 1 serviceability limit states, which deal with a single design truck (Figure 8), are to be reduced by 25 percent for all welded-steel structures. This penalty is intended to further discourage the use of welded components, whose past performance has left much to be desired, in single-load-path structures. On the other hand, the code permits an increase of 20 percent in stresses at ultimate limit states for laminated timber decks, in which numerous planks share the concentrated wheel loads and the failure of a single plank would cause only an insignificant reduction in load-carrying capacity.

The Ontario code includes various provisions and definitions. To explain the fundamental principles

Figure 7. A bridge collapsed by the San Fernando earthquake.

Figure 8. The Ontario highway bridge design truck.



Note: 1 m = 3.3 ft; 1 kN = 224.8 lbf.

underlying the code, four key definitions are listed in alphabetical order as follows:

1. Collapse—A major change in the geometry of the structure that renders it unserviceable,
2. Component—A structural element or combination of elements that requires individual design consideration,
3. Failure—A state in which the load-carrying capacity of a component or connection has been exceeded, and
4. Multi-load-path structure—A structure in which the failure of a component or connection does not result in the collapse of the structure.

One important point here is the clear distinction between the collapse of the structure as a whole and the failure of a component or connection. To avoid confusion in nomenclature, this distinction has been meticulously followed throughout the code and in this paper. Other parts of the code reinforce the generally accepted practice of comparing load-carrying capacity with demand, expressed as a combination of load effects at the component and connection level.

The above definitions are reasonably applicable to traditional structural systems such as truss and girder bridges constructed of steel, concrete, or wood. Their components, including thin decks, stringers, cross beams, diaphragms, bracings, beams, girders, truss members, and bearings, are easily distinguishable; the effect of their failure and that of their connections on the safety of the structure as a whole can usually be assessed with certainty. This aspect will be discussed later with design considerations.

The current code definitions of collapse and failure tend to converge on one another in the case of monolithic superstructures such as solid and voided concrete slabs, single-cell and multicell steel and concrete boxes, and rigid frames. This is because the component becomes the structure as far as these integrated systems are concerned. An unintended restriction could arise in such cases, but a way of avoiding this difficulty is suggested below.

The Ontario code absolutely forbids the use of simply supported single-cell steel box bridges and virtually eliminates, by implication, the use of continuous ones. While this feature of the Ontario code (which on most matters is generally viewed as very liberal) may appear suprisingly conservative, one should recall the various disasters that have occurred during construction and the serious maintenance problems experienced after construction of some of these bridges in England, Austria, and Australia.

The provisions of the British code, for instance, are so complex and demanding of the designer that they in fact act as a deterrent. The Ontario code achieves the same goal in a simpler way.

The serviceability record of monolithic concrete superstructures (solid, voided, or cellular in cross section) is known to be outstanding, and it is obvious that their construction should not be discouraged by the placing of obstacles to them in writing the appropriate specifications. The problem, as mentioned above, lies in the fact that it is rather difficult to isolate a component in a monolithic structure. In order to resolve the problem, the following argument is suggested.

Experience with existing concrete bridges seems to indicate that failure never occurs as a result of overstressing the concrete proper. There are several reasons for this phenomenon:

1. Most design codes limit the permissible concrete compressive stress to 40 percent of the cylinder strength at 28 days.
2. Actual live-load stresses are usually less than 50 percent of those predicted by the design analysis.
3. Components in bridge construction are usually underreinforced.
4. The strength of concrete in a bridge is higher than that indicated by the cylinder tests because of partial confinement.
5. Concrete increases in compressive strength with age.

Accepting the argument that the concrete proper does not ordinarily govern design, the monolithic cross section can be broken down into components on the basis of individual reinforcing bars or prestressing cables. This approach would not only resolve the conceptual problem at hand, but would also lead to more economical structures. Since the cross section of such a bridge contains a large number of bars and cables, the increase in permissible stresses (of the order of 20 percent) allowed for laminated timber decks—due to load sharing—can also be applied. Such a provision would justifiably reflect the superior behavior of monolithic concrete structures in highway bridges.

This new concept could also be applied to tied arches, a structural system in which the failure of the tie would certainly cause collapse even if the deck system, if any, participated. The concept could, however, be interpreted to mean that a tie fabricated as a single member would no longer be permitted; it would have to be made from several elements or a single element substantially strengthened by prestressing cables or bars.

The second conceptual problem arises in conjunction with the definition of component failure. A number of cases were previously discussed in which the collapse of the structure was precipitated by the excessive relative movement of the substructure due to scour or earthquake. In the strict sense of the word, the substructures did not fail; therefore the failure definition above does not apply. Nevertheless, the bridges did collapse.

This particular problem could be conveniently resolved if the failure definition were extended to read: Failure is a state in which the load-carrying capacity of a component or connection has been exceeded or in which a component stops providing adequate support for other components due to excessive relative movements. This would permit the designer in the majority of cases to isolate components and to identify multi-load-path structures.

PROBABILITY OF COLLAPSE

In the preparation of the Ontario code, a variety of existing bridges were investigated for their safety index (β), which is defined as a numerical assessment of margin of safety, expressed as a number of standard deviations from the mean. It has been found that bridges that were free of any load-induced distress had a minimum value of $\beta = 3.5$. In the calibration process, i.e., in adjusting load, combination, and performance factors for design, this target value was used.

The probability of failure, expressed in the form $k \times 10^{-4}$, is directly related to β, and a value of $\beta = 3.5$ corresponds to a value of $k \cong 2.0$, as taken from standard tables. This value of probability, however, is meaningless in itself and will only be used in this paper to indicate the increase in the margin of safety obtained by permitting only multi-load-path structures.

If the number of critical components in an average bridge is N, the probability of collapse of a single-load-path structure is approximately $Nk \times 10^{-4}$. Considering the internal backup system as an independent variable, the joint probability of failure of both the primary and backup systems (the latter's presence makes the structure a multi-load-path structure) is $N^2k^2 \times 10^{-8}$. Assuming that $Nk = 10$, the probability of collapse of any multi-load-path structure is 10^{-6}, or approximately one bridge in the whole western world.

This simple calculation, although only approximate, nevertheless provides a feeling of magnitude regarding the general improvement in structural safety by introducing multi-load-path systems and indicates that no further backup systems are warranted.

DESIGN CONSIDERATIONS

Multi-load-path structures do not play a part in the design procedure at the present time. In preparation for their eventual incorporation, the following aspects of the issue should be discussed.

An investigation regarding multi-load-path capability of a structure presupposes that the designer is reasonably familiar with likely failure modes and subsequent failure mechanisms. In North America, unfortunately, this is not necessarily the case. The general tendency of university education is to produce generalists who know something about everything within the scope of a given engineering profession but who understand relatively little in depth.

Profound structural knowledge can only be attained at the postgraduate level or after many years of intense professional work. Furthermore, structural design codes tend to be set up as design aids rather than as performance criteria. The result is that structural engineers are not called on to achieve developmental and diagnostic capabilities. Accordingly, the introduction of such a process should preferably be preceded by some target-oriented educational drive.

Concrete structural components, especially those that are underreinforced, exhibit a remarkable margin of ductility and thus provide sufficient warning before they fail. This phenomenon is even recognized in the generally conservative AASHTO specifications (6), which do not require the posting of a bridge found substandard by analysis if the bridge does not show signs of distress. In any case, the yield-line theory, if properly applied, could reveal the failure mechanisms sought.

In the section on historical background, the cases of the Mississippi and Ohio bridges were cited. In both of these the failure, i.e., the total separation of the bottom flange and the web plate, occurred in the internal span. Continuity caused the broken girder to become two cantilevers and, because of the substantial load transfer through secondary members to the intact girder, the structure was capable of supporting all dead loads. It is natural to question whether collapse would also have been avoided had the failure occurred in the sidespan, but this question is difficult to answer.

The intent of this paper has been to deal with the issue in a philosophical manner; discussion of all structural systems is outside its scope. It is conceivable, however, that a number of them may require research work in this particular regard.

Another question relates to the criteria for which the internal backup system should be designed. Based on past experience, one may assume that the state of failure is a temporary condition that will soon be discovered. Because the probability of having two or more heavily loaded vehicles simultaneously present on a bridge is rather remote, it is suggested that, in terms of the Ontario code, type 1 serviceability limit states should be used. Type 1 live load is 80 percent of the OHBD truck shown in Figure 8.

For the evaluation of existing bridges, the Ontario code permits a reduction in the live-load factor from 1.40 to 1.25. It would seem appropriate to accept this reduction here. But since $0.80 \times 1.25 = 1.00$, the single OHBD truck could be used without any modification. Dead-load factors could be taken directly from the Ontario code as follows: (a) $D_1 = 1.10$ for steel, wood, and precast concrete; (b) $D_2 = 1.20$ for cast-in-place concrete; and (c) D_3 - 1.50 for asphalt concrete wearing surfaces.

Performance factors applied to nominal strength of a component or connection would be identical to those applied normally at ultimate limit-state considerations.

Finally, the question of economy of this proposition should be touched on. There are approximately 600 000 bridges in the United States and Canada at the present time. An estimated 250 of them collapse every year. If the average bridge replacement cost is $400 000, the total financial loss would amount to $100 million yearly. This, even at today's inflated prices, is a considerable sum.

Bakht and others (7) discussed the cost of designing bridges in Ontario over the past decade. The figures indicate that, on the average, the actual computational part of the design process does not exceed 1 percent of the construction cost of a bridge. It is difficult to predict, but one cannot imagine an increase due to multi-load-path considerations of more than 0.4 percent. If the total annual expenditure on new construction in the United States and Canada is estimated at $2.5 billion, the additional design cost would amount to $10 million.

It is relevant to speculate on why in the past so many bridges were designed as single-load-path rather than multi-load-path structures. We believe that there are two main reasons for this: ease of analysis and design and ease of construction. As far as the former is concerned, the introduction of the electronic computer and the consequent development of powerful analytical techniques such as the finite-element method have, in recent years, made it possible to analyze multi-load-path structures with comparative ease. As far as the second is concerned, the lack-of-fit difficulties of assembly of a redundant structure are freely admitted. It is our view that, because it is these same redundancies that prevent catastrophic collapse or failure of a structural member, the increased difficulty of assembly should be gladly tolerated. Thus, the balance of preference should swing decisively to the multi-load-path side.

Finally, the chain of argument can return to the Silver Bridge. The collapse and the attendant tragedy brought home the point that bridges do not last forever without proper inspection and maintenance. It is unfortunate that it took 46 lives to prove the obvious. Nevertheless, the lesson has been learned. Without trying to belittle the significance of inspection, the nature of the crack that caused the collapse was such that it was unlikely to be discovered. Had the bridge been a multi-load-path structure, however, 46 human lives might not have been lost.

CONCLUSIONS AND RECOMMENDATIONS

1. Many existing bridges are unintentionally of the multi-load-path type; that is, they have an internal backup system that prevents the collapse of the structure as a whole on failure of a critical component.

2. The definitions of the Ontario highway bridge design code can assist in isolating components and in identifying the presence of such an internal backup system.

Among our recommendations are the following: (a) single-cell steel box girder bridges should no longer be used; (b) the requirement for a compulsory backup system would extend the design process, although the cost of extra design work is only a fraction of the potential saving; (c) because the nature and reliability of certain failure mechanisms are not clear at present, further research work is warranted; and (d) the introduction of compulsory backup systems is expected to reduce the probability of bridge collapses to near zero.

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