SOME CONSIDERATIONS IN THE DESIGN OF LONG SPAN BRIDGES AGAINST PROGRESSIVE COLLAPSE

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Abstract

Long span bridges have not been designed to resist progressive collapse explicitly; many long span bridge forms, due to reasons of structural efficiency, are intrinsically non-redundant, i.e. they incorporate elements whose localized failure would precipitate collapse. There are also long span bridge forms that are susceptible to progressive collapse due to the loss of a series of adjacent members as a result of a single loading event. In either case, this class of structures may be termed to have single point vulnerability. Herein, aspects of long span bridge design as they relate to single point vulnerability and progressive collapse are discussed together with some suggestions for potential improvements in design strategies.

Overview

Given structural efficiencies intrinsic to long span bridges, designing against progressive collapse has not been a major consideration in the development of bridge form. Many long span bridges, if not most, are designed as non-redundant structural systems with single point vulnerabilities. This inherent lack of redundancy covers the full range of long span bridge forms including suspension, stay cable arch and truss bridges. Recent events, such as the collapse of the I35W deck truss bridge in Minneapolis, Minnesota, bring into sharp focus the need to incorporate progressive collapse into the design of major bridges, regardless of structure type.

In US practice, cable stayed bridges are the only long span bridge form routinely designed for member (cable) loss. The Post-Tensioning Institute's *PTI Recommendations for Stay Cable Design, Testing and Installation* provide explicit design guidance for abrupt cable loss as well as cable replacement [1]. However these design provisions are not federalized standards - they apply only to cable stayed bridges; similar provisions for member loss (even cable loss) are not included in the American Association of State Highway and Transportation Officials (AASHTO) provisions. Even the *PTI Recommendations* provide little guidance in the design of the global structural system, though analysis techniques (both quasi-static and dynamic) are discussed in the commentary. General design requirements for cable-supported bridges to resist such an extreme event do not exist, nor is the subject of progressive collapse broached in any substantive way in the *PTI Recommendations*.

There appears to be some implicit awareness of the potential vulnerability of suspenders / hangers to precipitate progressive collapse, given the use of high safety factors (3 or 4) for the design of suspenders in suspension bridges (and in some cases

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the hangers for arch bridges). The use of high factors of safety could be argued as an effective design strategy against progressive collapse. However to design a specific element such as a suspender/hanger to a higher factor of safety without regard to the remaining structural system (stiffening girder or tie girder) is inappropriate if not illogical, particularly when we consider a structural system where the hangers are an integral component to system stability as in the case of tied arches. In any case, the design of suspenders / hangers to high margins of safety is an implicit strategy, these factors of safety are chosen without explicit regard to progressive collapse.

Our experience with the application of the PTI recommendations for cable stayed bridges indicates that cable loss governs design, particularly for shallow depth superstructures with relatively large cable spacing. This is worrisome, given that cable stayed bridges are intrinsically more redundant (hyperstatic) than other bridge forms, and thereby have enhanced resistance to abrupt member loss as a class of structures. Arches with vertical hangers (particularly tied arches) and trusses, by comparison, have much less internal redundancy and are, by definition, more sensitive to member loss. It is not common practice to design these bridge types for abrupt member loss or more generally, to have adequate resistance against progressive collapse.

Finally, design provisions for progressive collapse are often applied narrowly, with the assumption of single abrupt member loss. These assumptions are often inappropriate when applied to long span bridges; overly conservative in some cases, unsafe in others. As will be discussed below, the duration of the member loss event, the number of members directly engaged in the event, the damage to the member(s) sustained prior to the point of loss and finally the response of the structural system are key aspects of progressive collapse as they apply to the design of long span bridges.

Redundancy & Progressive Collapse

Progressive or disproportionate collapse has been an integral part of many building codes since the early 1970's. Much of the code development work on disproportionate collapse stemmed from the aftermath of the Ronan Point collapse in England, in 1968, and subsequent design provisions against progressive collapse found their way into building regulations for the United Kingdom, Sweden, Denmark, West Germany, Netherlands, France, Eastern Europe and Canada. However, no standardized approach for design to prevent progressive collapse was adopted in the United States [2]. Following the terrorist attacks of the Alfred R. Murrah Building in 1995, and the World Trade Center on September 11th, 2001, there has been renewed interest in the design of buildings against progressive collapse, and more prescriptive design provisions for buildings are in various stages of implementation, with the General Services Administration's Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects the most comprehensive to date.

In U.S. bridge design practice, there has been no parallel development in design guidelines against progressive collapse, although a number of historical instances of progressive collapse events exist, as will be discussed presently. It is not clear why these bridge collapse events have not resulted in the development of comprehensive guidelines for bridges, in a manner parallel to building design.

On December 15th, 1967, the collapse of the eyebar suspension Silver Bridge over the Ohio River during rush-hour traffic killed 46 people. The collapse was due to a fracture of the eyebar suspension chains (two eyebars per chain) associated with stress corrosion cracking and corrosion fatigue, prompting national concern about bridge conditions and safety. Congressional hearings resulted in a mandate that required the U.S. Secretary of Transportation to develop and implement National Bridge Inspection Standards (NBIS). The Ronan Point Apartment Tower Collapse occurred just 6 months later, killing only 4.


Silver Bridge Collapse – December 15, 1967

A little over 15 years later, on June 28, 1983, a section of the Mianus River Bridge catastrophically failed due to abrupt fracture of a pin and hanger detail (a failure mode eerily similar to the Silver Bridge eyebar failure). This failure resulted in only three fatalities (fortuitously the collapse occurred at 1:30am) and disrupted commerce in the Northeastern U.S. for several months. Following this event, a significant body of research into fatigue of steel connections was undertaken and resulting in more rigorous inspection, design, and fabrication procedures for fracture-critical structures.[3] In addition, retrofits to enhance the redundancy of pin-hanger type connections were implemented on a broad scale throughout the United States.


Mianus River Bridge Collapse – June 28, 1983

The development and adoption of comprehensive AASHTO LRFD design provisions in 1996 represent the first explicit consideration of redundancy and ductility (and operational importance) in the form of load factor modifiers. The overall consideration of these three factors has a small effect on design, particularly for bridges with single point vulnerabilities and the potential for abrupt member loss. In the current code, non-redundant elements must be designed to resist an added 5% (1.05 times) the factored design loads to which all other members are subjected. If we consider a suspender as an example: assume a factor of safety of 2 results in a designation of the suspenders as non-redundant (i.e. the loss of a suspender would precipitate progressive collapse), we then would be required to provide a factor of safety of 2.1. This design approach will do little to enhance safety. Of additional concern is that no rigorous guidelines exist to assess whether the member is redundant (fracture critical) and warrants the increased load factor, nor any discussion of the dynamic aspects of member loss. Why progressive collapse (presumably considered in the *redundancy* load factor modifier) is not worthy of stand-alone design provisions is unclear. What is clear is that current AASHTO LRFD design provisions will not lead to enhanced safety, particularly for long span bridges with single point vulnerabilities.

To be fair to code development in the US, fundamental research into redundancy of superstructures and substructures has recently been completed [4], [5] and may yet result in revisions to AASHTO LRFD. However, this work does not draw significantly on past research on progressive collapse (particularly from the perspective of collapse dynamics) and is not generally applicable to long span bridge forms.

It may be that the failure to develop and implement progressive collapse design guidelines in bridge codes is somehow related to the entrenched idea that member failure is related to corrosion and fatigue, the clear contributors to the major collapses of the Mianus and Silver bridges discussed above. While fatigue and brittle fracture remain a significant concern particularly for the large inventory of steel bridges built in the US prior to the 1970's, it results in a focus on routine inspection of existing bridges and member design, detailing, and fabrication of new bridges instead of the global structural system design. This fatigue and fracture based framework for member loss results in code specified requirements for Charpy V-notch fracture toughness of steels used for members identified as FCM's (fracture critical members) instead of holistic requirements for system design to resist progressive collapse. The commentary in AASHTO LRFD in section 6.6.2 is particularly relevant:

> *"The criteria for a refined analysis used to demonstrate that part of the structure is non-fracture critical has not yet been codified. Therefore, the loading cases to be studied, the location of potential cracks, degree to which the dynamic effects associated with a fracture are included in the analysis, and the fineness of the models and choice of element type should be agreed upon by the Owner and the Engineer…"*

Substantial work remains to develop a comprehensive treatment of progressive collapse for bridges, and in particular, for long span bridges. At the very least, an adaptive use of the strategies used in progressive collapse design for buildings represents an appropriate starting point.

Element Vulnerability

Given that corrosion and fatigue are well understood as potential causes of element failure and progressive collapse, they will not be discussed in detail here. It should be mentioned, however, that there are many cases where fatigue and corrosion have not resulted in progressive collapse, particularly in structures where the failed elements would be classified as fracture critical, highlighting our inability to predict progressive collapse. A fairly comprehensive discussion of failures of members identified as FCM's that did not lead to progressive collapse is presented in *NCHRP 354* [6] Notable examples include the Lafayette Street Bridge in Minneapolis MI (1976), the Neville Island Bridge in Pittsburgh, PA (1977), the Hoan Bridge, Milwaukee, WI (2000) and the US 422 Bridge in Pottstown, PA (2003).

Following the events of September 11th, 2001, there has been considerable concern that major bridges may be the target of a terrorist attack. Over the past three years, we have conducted a reasonably comprehensive series of studies to assess the vulnerability of bridge elements, particularly cable elements to terrorist attack. These studies are ongoing and include tensioned and un-tensioned tests of seven wire strand stay cables, wire rope and structural strand suspenders, even tests of a large diameter parallel wire main cable (untensioned). A wide array of threats have been considered in these evaluations including explosive threats (contact explosives, shape charges, explosively formed penetrators) fragmentation (primary and secondary fragmentation associated with large improvised explosive devices) as well as thermic and mechanical cutting tools widely used in the construction industry. While a detailed discussion of this research cannot be presented for reasons of security, it can generally be concluded from these tests that there are a number of potential terrorist attack methods that are capable of inducing abrupt member loss, particularly for small structural elements such as stay cables, suspenders, and truss members.

Barge Impact to Stay Cables, Fuzhou, China

In addition, we have conducted research on the behavior of bridge members under accidental (or intentional) extreme events such as impact (vehicular, vessel, aircraft) and fire (vehicle or vessel delivered). Unfortunately, this research has also demonstrated the vulnerability of cable and truss elements to these types of threats. Highlights of this research are presented below.

The first example is vessel impact with a stay cable. During a hurricane in June of 2001, a runaway 1000 ton crane barge struck 3 or more stay cables of the Ching Chau Ming Jang Cable Stayed Bridge in Fuzhou, China. Only 3 strands were found to have ruptured in a single 73 strand stay [7]. The entire cable was replaced, given that the remaining strands had varying forces and the potential that these strands had been damaged could not be ruled out. It is noted that the bridge was nearing construction completion at the time of the impact; the cable was replaced with no difficulty and the bridge was opened to traffic soon after.

Tethered barrage balloons were used extensively in World War II, particularly by the British as a deterrent against attacks by German aircraft. In many circumstances, aircraft that collided with cables were damaged significantly; over hundreds of kills including 231 V-1 Bombs [1][8] have been attributed to barrage balloons. The British Royal Air Force maintained approximately 2000 operational balloons throughout the war. The physics of aircraft impact to a tethered cable is attributed to Housner, as described in [9] where the tethered cable's impact resistance (in terms of aircraft impact speed) is related to ultimate strain capacity and stress wave velocity of the cable. World War II aircraft did not have adequate speed at impact to severe the cables; instead the tethered cables were an effective means of damaging aircraft. A more recent example is the Cavalese Cable Car disaster in Italy on February 3rd 1998, where a US Military aircraft (EA 6B Prowler) struck and severed a cable resulting in the death of 20 people. It appears likely that the velocity of the aircraft was a major contributor to the cable loss, especially given the extensive damage to the wing of the aircraft. A similar incident occurred in France in 1961 also involving a military aircraft.

There are also two significant recent (last 10 years) examples of cable damage/loss associated with fire involving two multispan cable stayed bridges, the Rion Antirion Bridge in Greece and the Mezcala Bridge in Mexico. Less than six months after the Rion Antirion Bridge was opened to traffic, a reported lightning strike caused a fire resulting in cable failure, the cable below the failed cable also was caught fire locally but self arrested before significant damage. The bridge was closed and subsequently reopened to limited traffic prior to cable replacement. It was reported that cable clamps that were provided for later use of cross-ties (if necessary) contributed to the risk associated with a lightning strike and that a more comprehensive lightning suppression system was added (together with a damping system to address cable vibrations). It could be argued that the hydrocarbon based HDPE sheathing, with its high flammability and heat release characteristics, was more fundamentally to blame.

Mezcala Bridge – Fire and Cable Failure

On the Mezcala bridge in Mexico, a traffic incident on March 17th, 2007 involving two school busses and a truck transporting coconuts produced a fire at deck level resulting in the failure of one stay cable and limited damage to an adjacent cable. Similarly, the bridge was immediately closed to traffic and reopened to limited traffic prior to cable replacement. Here again, it seems clear that the HDPE as a corrosion protection element is poorly suited to resist fire and becomes part of the fire load, if ignited, enhancing the potential for damage. In both these instances fire produced a load that resulted in cable loss. In addition, it appears likely that multiple cables could participate in a deck level fire or a fire associated with a lightning strike. Would these bridges have survived the loss of two or three adjacent cables? A fire event that caused the failure of three adjacent stays resulting in loss of the span (or spans) is a progressive collapse event that involves multiple elements. This problem of single point vulnerability, where an extreme event results in multiple element loss is discussed in more detail below.

One might conclude that suspenders for suspension bridges and hangers for arches are much less susceptible to fire damage, given that HDPE corrosion protection coating is not used. However, there is enhanced susceptibility of these cable types to fire, not associated with the corrosion protection system, but with the zinc speltering used in the anchorages of these cable types.

We have also conducted research into the behavior of cables under rapid loading near the limit state. This is of particular concern in progressive collapse design where it is vital to have an accurate assessment of the capacity of adjacent members to resist forces associated with member loss.

Wire Rope Socket Premature Failure

A series of tests were conducted by the British Health and Safety Executive (HSE)[10] to assess the behavior of wire rope assemblies to static and dynamic impact loads. Both 25mm and 32mm diameter cables were tested, and premature socket failures were noted in many cases, particularly for larger diameter cables. This is clearly an area of concern and suggests that further research be conducted. Another important consideration is the condition of the cable at the time it is subjected to an extreme event associated with member loss. The strength reduction factors that are typically applied to account for material property variations and environmental degradation may not be sufficient to capture behavior under dynamic loading. Also, when suspenders are removed and tested to estimate remaining life, static tests often indicate a significant loss in element ductility (if not premature fracture) suggesting enhanced sensitivity of in-place cables to a member loss extreme event.

Wedge Unseating – Explosive Tests

As part of our explosive tests, we have also witnessed wedge unseating with 7 wire prestressing strand used in many (if not most) stay cable systems. While these tests were intended to evaluate cable behavior under explosive threats, there were clear indications that wedge unseating occurred. However, it was difficult to assess whether wedge unseating was limited to the strands that were severely damaged or not and subsequent tests are planned to evaluate the potential for wedge unseating without significant damage to the individual strands. It may be both necessary and prudent to incorporate wedge retainer plates in all stay cable anchorages to avoid this mode of failure (particularly if it results in premature cable loss).

It would appear prudent that similar dynamic tests be conducted with bolted connections typical of trusses, to ensure similar behavior is not a significant concern, particularly given the general practice that truss connections are not designed for the ultimate strength of the connected elements. The basis for vulnerability assessment is often times based upon limit state behavior, and any premature failure, particularly at the connections can substantially enhance the overall vulnerability of the structure to progressive collapse.

The Design of Long Span Bridges for Progressive Collapse

Insights gained from work that we have done on the progressive collapse of long span bridges types will be presented below. Each bridge type will be discussed separately with a focus on network tied arches and cable stayed bridges given their intrinsic redundancy and therefore enhanced resistance to progressive collapse associated with member loss.

Truss Bridges

We have conducted a detailed progressive collapse study for a major truss in the northeast United States to assess its vulnerability to terrorist attack. For security reasons, detailed information on these analyses cannot be presented. While we did identify a few fracture critical truss members, the majority could be classified as redundant given that the lateral and sway bracing provided an effective alternative load path in many circumstances. In contrast, for the I35W bridge in Minneapolis, MI a detailed study undertaken by others to assess the vulnerability of the truss determined that 25% (52) of the bridge's truss members were identified as fracture critical [11]. It is noted that compression only members were excluded from consideration given the focus on fatigue as the principal risk to the structure (this is of course not the case in a terrorist evaluation where a compression member is just as likely a target). This represents a structure that does not have adequate safety as originally designed with such a disproportionately large number of elements that are fracture critical.


Cape Girardeau Truss Demolition

This significant difference in progressive collapse potential has very much to do with the effectiveness of alternative load paths, particularly the deck and the lateral system. The unusual width of the I35W bridge together with its configuration of floor trusses and upper lateral bracing made it particularly vulnerable to progressive collapse. The results of the forensic studies currently underway will undoubtedly yield insights into the collapse mechanism and may yet serve as a catalyst in the development of progressive collapse guidelines for bridges.

An important example of progressive collapse of a truss bridge during demolition appears to be a circumstance where the dynamics of member loss were improperly considered. In the early fall of 2004, the Cape Girardeau truss bridge river spans were to be demolished after completion of an adjacent cable stayed bridge.

Explosive demolition was planned to drop a span at a time to ensure that Mississippi River navigation would not be impacted. On September 9th, demolition of the side span resulted in near instantaneous failure and subsequent collapse of the main span, as well as the other side span and the adjacent approach span trusses. This accident resulted in interruption to navigation as well as substantially complicating the completion of truss demolition, which was not completed until over 1 month later.

The Gene Hartzell Memorial Bridge in Easton, PA, designed by URS and completed in 2002 represents an example in the design of a deck truss designed to avoid fracture critical members, using composite action between the deck and top chord and redundant tie plates for selected bottom chord tension members. This represents a new direction in the design of trusses that will hopefully lead to a class of safer structures of this type.

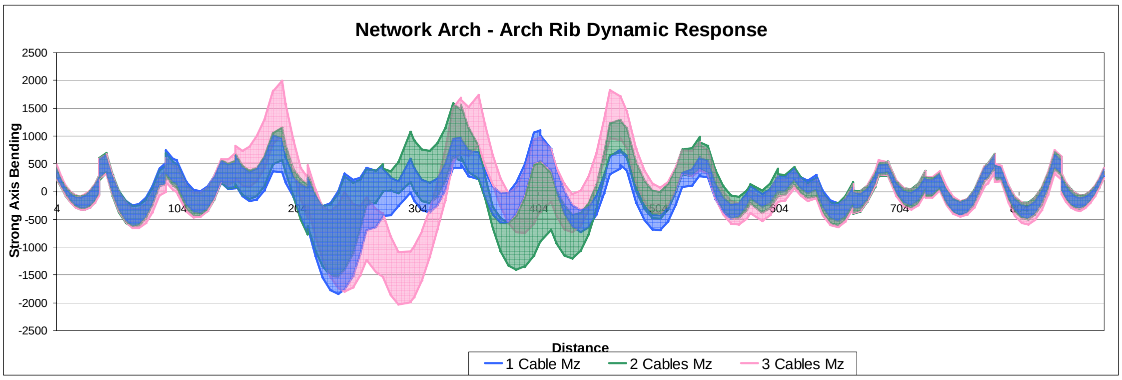
Arch Bridges

Blennerhassett Island Network Tied Arch

In a recent design for a 268m network tied arch in West Virginia, we together with Michael Baker, Jr. Inc conducted comprehensive analyses to evaluate cable loss and partial tie girder fracture. This project presented an ideal opportunity to assess aspects of progressive collapse on arch bridge design with a particular emphasis on cable loss dynamics. Early on in the design process, it became clear that cable loss governed the design of the arch rib and that the tie girder redundancy strategy (the use of a built up box section, such that a crack could not propagate from flange to web or vice-versa) was highly sensitive to live load moment demands. From these evaluations, it was clear that the only viable strategy was a network configuration of hangers (i.e. inclined hangers that cross at least once). This strategy significantly reduced moment demands in the tie girder under plate loss scenarios (whereby the three remaining plates are designed to have sufficient capacity) as well as reducing moment demands and enhancing the stability of the arch rib under cable loss scenarios.

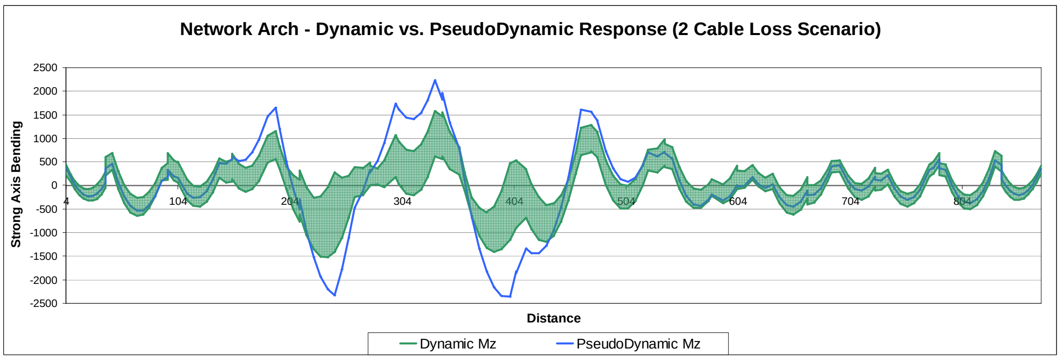
This work also resulted in the development of insights into the dynamics of cable loss, particularly in terms of analysis methods and the impact of loss duration (i.e. the abruptness of the loss of an element). This work is summarized elsewhere [12] and will not be discussed herein. However additional studies of network arches under single and multiple cable loss scenarios highlight the hyperstatic nature of these forms, highlighting the ability for load redistribution throughout the structural system in the event of even multiple localized failures. It is particularly interesting to compare peak structural response associated with the loss of 1,2, and 3 adjacent cables. Notice that the peak flexural demands for the arch rib (design critical element for cable loss) do not show marked increases as the number of cables that are lost increases. Also note the flexural response of nearly the entire arch. This clearly demonstrates the ability of the form to engage the resistance of the entire structural system to resist the effects of a localized cable loss event.

Also of interest is a comparison of the dynamic response of cable loss as compared to the pseudo-dynamic response with the technique described in the *PTI Recommendations*, whereby the lost element is applied with an impact factor of 2 with the force applied in the opposite direction of the force in the lost cable. It can be seen that the pseudo-dynamic response does capture behavior accurately, over-predicting response in some locations and under-predicting response in other locations. This is consistent with the findings of others on progressive collapse, that modeling the dynamics of loss is a necessity.



Envelope of Dynamic Response – Abrupt Cable Loss

We have also evaluated a major truss arch in terms of progressive collapse from terrorist attack and cannot discuss many details of the project for security reasons. The arch is unusual in that the deck system has very little strength or stiffness longitudinally, with all live load moments carried directly into the truss arch. Although this structure utilizes vertical hangers and is less intrinsically redundant, it is resistant to progressive collapse since cable loss precipitates local superstructure failure, such that it serves as a fuse to protect adjacent suspenders from unzipping and the arch from buckling.



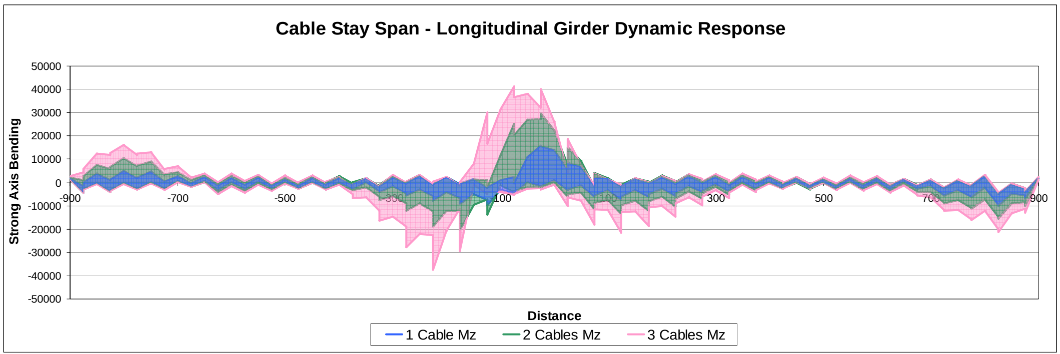
Dynamic Envelope versus Pseudo-Dynamic Response

Based upon the insights gained in our work in arch bridges, we would recommend that early in the design process a strategy be identified for progressive collapse resistance, in terms of arch buckling, tie girder failure and cable loss. Arches with vertical hangers are intrinsically vulnerable to progressive collapse and, in most cases, should not be used. This represents a significant change in current design practice where network tied arches are the exception, instead of the rule.

Network tied arches are ideal structural forms from the perspective of cable loss and have significant intrinsic benefits in terms of arch stability and tie girder localized failure. For engineers unfamiliar with the Network tied arch we recommend visiting http://pchome.grm.hia.no/~ptveit/, a site developed by the Norwegian engineer who first conceived of the network tied arch, Per Tveit. His conceptual design of network tied arches, where a large number of cables are used at close spacing, together with a prestressed concrete superstructure results in an unusually efficient long span bridge form with a high degree of resistance to progressive collapse, particularly from the perspective of cable loss.

Cable Stayed Bridges

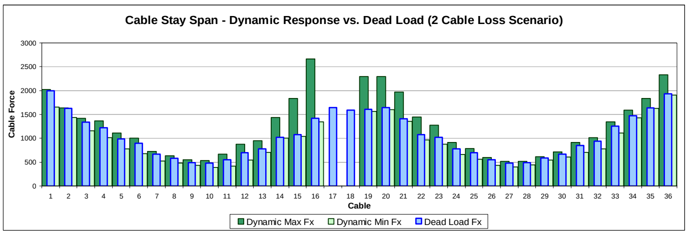
Cable stayed bridges generally have a high degree of internal redundancy and are designed for abrupt loss of a single cable which often governs the design. The two potential progressive collapse scenarios are i) flexural / buckling failure of the superstructure and ii) overloading of the adjacent stay cables sufficiently to result in unzipping (progressive failure of the adjacent stay cables. While these scenarios are evaluated for a single cable loss, the potential for multi adjacent cable loss cannot be ruled out either from an intentional attack (terrorism) or an accident (impact / fire). It is noted that fire related cable loss represents less risk since the dynamic effects of loss are reduced. However, if sufficient adjacent cables are lost in a fire event, progressive collapse may readily occur.



Girder Dynamic Envelope – Multiple Cable Loss

As part of the conceptual design of a new bridge in the northeast United States, a comprehensive evaluation of multi-cable loss was undertaken to assess the structure's resistance to progressive collapse. It is noted that this conceptual design is of an unusual configuration with parallel roadways supported by 4 planes of cables supported by a tower that has a high degree of lateral and torsional stiffness. These features were implemented to enhance overall resistance to progressive collapse.

It is clear from the longitudinal girder dynamic envelopes shown in the above figure that a significant increase in flexural demand (and similarly the likelihood of buckling) is associated with each additional cable lost. It is clear from these studies that multiple cable loss, whether a dynamic event (impact, terrorist attack) or a quasi-static event (fire) the potential for progressive collapse exists, and that superstructure depth (as a function of cable spacing) is the primary means of enhancing resistance to progressive collapse.



Dynamic Cable Loss – Cable Force Envelopes

Similar insights can be derived from evaluating the dynamic force envelopes associated with multiple cable loss scenarios. Depending upon the stiffness of the superstructure, cable force increases are localized adjacent to the lost cables resulting in the potential for unzipping. As will be discussed below in the response of suspension bridges, stiff superstructures tend to maximize the force demands at the adjacent cable. The lower factor of safety used in the design of stay cables as compared to suspenders results in enhanced vulnerability of cable stayed bridges to unzipping associated with multiple adjacent cable loss scenarios. Note that the cable stayed bridge form is intrinsically more resistant to progressive collapse, however the factors of safety typical of hangers / suspenders is nearly double that of stay cables (2.2 versus 4.0) and serves as the primary protection against unzipping.

Tacoma Narrows – Suspender Unzipping

As a final note, it has been suggested by some engineers that abrupt cable loss is overly severe given that a modern stay cable is comprised of multiple strands and is therefore internally redundant [13]. From our work on the behavior of large cables under terrorist attack and high velocity impact, it is absolutely clear that abrupt cable loss is readily achievable, particularly given intentional attack scenarios. Abrupt cable loss, while governing the design in many instances, is a valid consideration for cable stay bridges.

Suspension Bridges

The majority of our work on the progressive collapse of suspension bridges is related to protecting major bridges from terrorist attack, so for security reasons, details of this work cannot be presented. There are a few key observations that are worthy of discussion. It is envisioned that suspender loss, together with main cable and tower damage are the primary concerns associated with progressive collapse of suspension bridges.

We have had some key involvement in research to assess the performance of cellular steel structures to close-in explosive detonations and the potential for progressive collapse of damaged towers. This research includes both explosive tests and analytical predictions of tower cross sections typical of steel suspension bridge towers). We have also developed effective retrofit strategies to enhance the resistant of towers to progressive collapse for 6 major suspension bridges in the Northeast United States (3 are completed construction, 3 are at various stages of final design).

We have also completed progressive collapse evaluations associated with adjacent suspender loss with a focus on the potential for unzipping. Suspension bridges, particularly bridges with deep stiffening trusses, do not efficiently redistribute localized loading associated with member loss throughout the structure. The potential for unzipping becomes a concern when sufficient consecutive suspenders are lost and the redistributed force results in overload of the adjacent suspenders that remain. It is often the case that the superstructure has less strength than the ultimate strength of the suspender group, given that the factor of safety for the suspenders is much higher than for the design of the stiffening truss or girder. While damage to the superstructure may be anticipated with the loss of fewer adjacent suspenders, we would caution against the conclusion that the superstructure acts as a fuse to protect adjacent suspenders, rendering suspenders safe from the perspective of initiating progressive collapse, particularly in circumstances where abrupt cable loss is anticipated. The ability of the superstructure to transfer large loads prior to failure cannot be ruled out, particularly given the potential for membrane action at the flexural failure limit state. The video documentary of the failure of the Tacoma Narrows Bridge supports the potential for unzipping type failure, where wind induced damage to the superstructure resulted in the loss of nearly the entire main span.

It is interesting to note that the large factor of safety associated with the design of suspenders in suspension bridges is very desirable from the perspective of resistance to progressive collapse. While this bridge type is not hyperstatic and therefore inefficient at redistributing localized loads associated with member loss, this deficiency is readily overcome by increased safety margins to protect against an unzipping failure. It is therefore entirely appropriate that large margins of safety in the design of suspenders be maintained, with progressive collapse resistance as the primary basis for this recommendation.

Summary

It is high time that progressive collapse considerations be brought into the forefront of long span bridge design in order to enhance the reliability and safety of these major structures in the built environment. Progressive collapse resistance as a primary design consideration will lead to a more rational approach from assigning factors of safety to key elements such as suspenders and stay cables, to setting depth and stiffness requirements for cable stayed bridge and arch bridge superstructures, even to choosing appropriate design forms for major bridges (network tied arch versus conventional arches with vertical suspenders). A progressive collapse framework also provides a rational basis for assessing the safety and vulnerability of our existing bridge population, and will help us focus our resources in a manner consistent with reducing the risk catastrophic failure.

[1] *Recommendations for Stay Cable Design, Testing and Installation*, Post-Tensioning Institute, Phoenix, AZ, 2000, pp. 55-56.

[2] Dusenberry, D. *Review of Existing Guidelines and Provisions Related to Progressive Collapse*, Multihazard Mitigation Council of the National Institute of Building Standards, Washington, DC 2003, pp. 1-31

[3] *2002 Status of the Nation's Highways, Bridges, and Transit: Conditions & Performance*, Report to Congress, Federal Highway Administration, Chapter 11, Overview and Evolution of the Bridge Programs

[4] Liu, D et al, *NCHRP Report 458, Redundancy in Highway Bridge Substructures,* Transportation Research Board – National Research Council, National Academy Press, Washington. DC, 2001

[5] Ghosn, M. and Moses, F., *NCHRP Report 406, Redundancy in Highway Bridge Supertructures,* Transportation Research Board – National Research Council, National Academy Press, Washington. DC, 1998

[6] CONNOR. R.J. et al, NCHRP Synthesis 354 Inspection and Management of Bridges with Fracture Critical Details, A Synthesis of Highway Practices, TRB, Washington DC, 2005

[7] *Strand by Strand Cable Replacement,* in "News. The VSL-Intrafor Magazine", 2003, n. 1 v. 14 .

[8] Captain Norman MacMillan, The Royal Airforce in the World War, vol. 4, 1940-1945 (London: George G Harrap and Co, Ltd, 1950).

[9] Irvine, Max, *Cable Structures,* Dover Publications, New York 1981

[10] McCann, P. *Evaluation and Performance Deterioration of Compacted Strand Wire Ropes*, Health and Safety Executive, Research Report 487, 2006.

[11] URS Corporation, Fatigue Evaluation and Redundancy Analysis, I35W over the Mississippi River, Prepared for Minnesota DOT, July 2006.

[12] Zoli, T, and Woodward, R, *Design of Long Span Bridges for Cable Loss,* IABSE Symposium, Structures and Extreme Events, Lisbon, Portugal, September 14th -16th, 2005

[13] Abrams, et al, *Design for Cable Loss in Cable Stayed Bridges*, Bridge Engineering Association Seminar - Bridge Cables Assessment Design and Erection, February 2003