

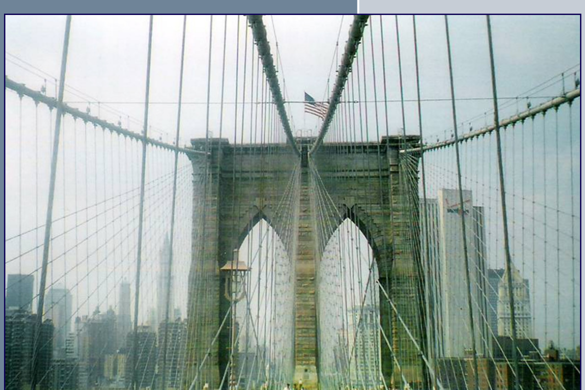


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Framework for Improving Resilience of Bridge Design

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16. Abstract

Bridges are an integral and important part of the highway infrastructure system and need to be designed to provide the necessary safety for the traveling public. Bridge failures can result in the disruption of commerce and services, significant repair costs, and most importantly the loss of human life. Performing a failure analysis during design, coupled with the review of past bridge failures, can help to avoid the need to initiate investigations and perform forensic engineering after a failure. This is the motivation for the development of this *Framework*.

The development of this *Framework* considers bridge failures that resulted in collapse, service closures, major repairs, or other significant issues that occurred while the bridge was in service or during construction. A fault tree methodology is adopted in the *Framework*, where lessons from past bridge failures are used extensively to identify potential events that could lead to a bridge failure. A bridge designer, conscientiously or unconscientiously, goes through a fault-tree analysis mentally while ensuring that the design is devoid of weaknesses that could lead to bridge malfunction or failure.

This *Framework* can provide a starting point for the less than senior engineer to jump start a conscience evaluation process, and is expected to be of interest to students and instructors of bridge engineering, bridge owners, bridge designers, inspectors, fabricators, contractors, and maintenance personnel.

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FOREWORD

The motivation for developing the "Framework for Improving Resilience of Bridge Design" is to perform failure analysis during design to avoid the need to initiate failure investigation and perform forensic engineering after an in-service failure. Lessons from past bridge failures are used extensively in the fault-tree analysis to identify potential events that could lead to a bridge failure.   A bridge designer goes through a fault-tree analysis mentally in making sure that the design is devoid of weaknesses or hot-spots that could lead to bridge malfunction or failure. This is generally adequate for the smaller, simpler and more common types of bridges. For the more complex bridges, it is desirable to perform a fault-tree analysis to systematically determine all contributing factors or events that could lead to a bridge failure. The contributing factors or events can then be considered and carefully addressed in the design by the bridge designer. The fault-tree methodology is also conducive to using the collective knowledge, experience, and skills of engineering professionals in a group environment to develop or review a fault-tree analysis of a more complex design and take steps to enhance the resilience of bridge design for safety, quality, and economy.

Illustrative examples of fault-tree analysis are given in the "Framework for Improving Resilience of Bridge Design" for several types of bridges, including superstructures and substructures. The illustrative examples provide the tools for identifying potential failure modes and give suggestions for giving due considerations in the design to prevent such potential failures and to improve the resilience of bridge design.

The bridge designer has the most influence in the quality and performance of his/her design. However, the bridge designer needs the cooperative and collaborative efforts of the fabricators, contractors, inspectors, and maintenance personnel to fully meet the intent of quality and resilience of the design.

The Framework is expected to be of interest and use to students and instructors of bridge engineering, bridge owners, bridge designers, inspectors, fabricators, contractors, and maintenance personnel.

The constructive review comments on the final draft from many engineering professionals are very much appreciated. The readers are encouraged to submit comments for enhancements of future edition of the Framework to Myint Lwin at the following address: Federal Highway Administration, 1200 New Jersey Avenue, S.E., Washington, DC 20590.

Myint Lwin

M. Myint Lwin, Director
Office of Bridge Technology

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The authors would like to acknowledge the encouragement and guidance provided by Myint Lwin, the Director of the Office of Bridge Technology, and Raj Ailaney, the Contract Manager of the Office of Bridge Technology during the development of the *Framework*. The authors also would like to thank the engineering professionals that provided very useful review comments on the final draft of the *Framework*.

1.0 INTRODUCTION

Bridge failures can result in the disruption of commerce and services, significant repair costs, and most importantly the loss of human life. Bridges rarely experience complete failure during non-extreme events, however when such failures do occur, the results can be catastrophic:

- The collapse of Schoharie Creek Bridge in 1987 resulted in the loss of 10 lives.
- The collapse of a curved I-girder ramp under construction in 2005 resulted in the death of a construction worker.
- The August 2007 collapse of the I-35W Bridge in Minneapolis, Minnesota killed 13 people and injured 145 others.

Bridges are an integral and important part of the highway infrastructure of the Nation and need to be designed to provide the necessary safety for the traveling public. The review of past bridge failures allows bridge designers to apply lessons learned to new design projects and to the preservation of existing structures which will help prevent future failures.

The intent of this document is to provide a framework that can be employed by bridge designers during the design process that can help to minimize bridge failures while in service and/or during construction. Use of this methodology is not a panacea for the prevention of bridge failures, but is intended to provide bridge designers with a tool for identifying potential failure mechanisms and design accordingly to prevent failures. Addressing potential failures during design is much less costly and painful than failure investigation and forensic engineering carried out after an actual failure event. This is accomplished by highlighting design considerations that could reduce failures that might not be readily apparent in current design specifications. The framework developed within this document will incorporate a review of past bridge failures either during construction or in service to identify and assess the causes and determine what bridge designers can do to reduce the likelihood of future failures. The main purpose of this framework is to advise bridge designers to "think outside the box" and potentially go beyond governing design specifications during the design process. Doing this addresses potential failures that a particular bridge design may be susceptible to, and make provisions during design to prevent such failures. It should be noted that not all of the potential failure scenarios described in this document can be designed for/investigated during the design phase for every bridge, but designers should be aware of these failure scenarios for general knowledge purposes. These failure scenarios deserve further study to make sure they are adequately addressed in design.

The development of this framework considers bridge failures that resulted in collapse, service closures, major repairs, or other significant issues that occurred while the bridge was in service or during construction, which could have been mitigated in the design process; similar to a lessons-learned approach. For each bridge type considered in this document, a fault tree is developed to describe the design issue/characteristic that may lead to a failure. Although fault trees can be used to numerically investigate the probability of failure, the use of fault trees in this framework is intended solely as a visual representation of potential bridge failure scenarios. Through these fault trees, the framework addresses design provisions and/or methods that can be employed to mitigate specific issues/characteristics that can lead to failures. Suggested guidelines and methods are developed from the current AASHTO *LRFD Bridge Design Specifications*, as well as from past specifications, the archival literature, and "rules of thumb."

The primary intended audience for this document is relatively less experienced bridge designers faced with the design of any number of bridge types, and are not familiar with past bridge failures that could have been prevented. However, the broader audience is truly all bridge professionals; the industry as a whole can benefit from the sharing of past bridge failure causes, which can aid in the prevention of future bridge failures. Furthermore, this document does not fully cover each type of failure with an in-depth discussion. The intent of this document is to provide the designer with a "first line of defense," and if the reader needs further information, references are cited. Potential failure scenarios are highlighted throughout this document, however it is up to the designer to determine whether or not the failure scenario needs to be investigated further as it relates to the particular bridge being designed.

1.1 Definition of Failure

Failure is defined herein as the inability of a bridge or one of its primary load-carrying components to no longer perform its intended function. For bridges under construction or in service, this framework considers the term failure in two different contexts: 1) collapse and 2) critical defect. Herein, a bridge *collapse* is the failure of all or a substantial part of the bridge where full or partial replacement may be required. The term *critical defect* refers to the condition in which the structure has undergone some deformation, section loss, or similar undesirable condition, but has not collapsed and can be repaired or retrofitted. Additionally, delays during construction and/or fabrication can be considered as a *critical defect* in the overall bridge construction process.

Failures can be caused by one, or a combination of the following (not inclusive): errors in design, detailing, or construction; unanticipated affects of stress concentrations; lack of proper maintenance; the use of improper materials or foundation type; or the insufficient consideration of an extreme event. Design and detailing errors, omissions, or flaws could lead to failure of a structure. Also, there could be a loss of the original design intent during the detailing process. A construction failure can occur due to the incorrect installation of structural members or temporary supports. A deviation from the approved construction procedures may also result in a failure. The lack of proper maintenance can result in corrosion leading to deficient members. The use of incorrect materials, concrete deck admixtures for example can also result in a type of failure. However, it has been shown through various studies [1][1] that a bridge failure is most likely to be caused by an extreme event, with the most prevalent type being flooding and scour.

1.2 Redundancy

Redundancy is typically defined as the ability of the bridge system to sustain damage without collapse. In a non-redundant bridge system, the failure of any one critical member may result in the collapse of all or a portion of the bridge system. In a redundant system, two or more components must fail before the bridge system collapses. There are three types of redundancy that can exist for bridges:

- *Internal Redundancy* - Internal redundancy relates to the fact that the failure of one element of a member will not result in the failure of other elements of the members. For

[1] Numbers in brackets refer to References at the end of this document.

example, riveted plate girders and multiple eyebar truss members have internal redundancy. In a riveted plate girder, if a crack begins in one of the elements, it will not propagate directly into adjacent elements. Welded plate girders and rolled sections do not have internal redundancy.

- *Structural Redundancy* - Structural redundancy refers to the redundancy that exists as a result of the continuity within the framing element. A statically indeterminate structure, such as a continuous beam, could be classified as being structurally redundant.
- *Load Path Redundancy* - Load path redundancy is related to the ability of the structure carrying load following the loss of a single member. A bridge such as a two-girder superstructure is classified as non-redundant because it does not have any alternative load paths. The failure of a single girder in a two-girder bridge could result in failure of the entire bridge system. Another example is a single column pier.

Redundancy has a significant role in the prevention of bridge failures. A non-redundant bridge is more susceptible to a failure since it is more likely to have a reduced number of members or no alternate load path. At the present time, the AASHTO *LRFD Bridge Design Specifications* [2] only recognize load path redundancy as described above. A redundant structural system is defined in the AASHTO *LRFD Bridge Design Specifications* as a system of element and components whose failure is not expected to cause collapse of the bridge. A redundant system can receive some benefit in the AASHTO *LRFD Bridge Design Specifications* [2] with the use of a load modification factor (η) to account for redundancy, if the designer and/or owner feel this is warranted. Good and cost-effective design practice incorporates as much redundancy as can be justified economically, sometimes going beyond the minimum design requirements.

Additional information regarding redundancy in highway bridge superstructures can be found in NCHRP Report 406 [3]; and likewise for highway bridge substructures in NCHRP Report 458 [4].

1.3 Bridge Inspection and Maintenance

In accordance with the National Bridge Inspection Standards (NBIS), every bridge, in general, is to be inspected at regular intervals not to exceed twenty-four months [5]. Regularly scheduled inspections enable bridge owners to routinely recognize the general conditions of the bridge and help to detect any potential problems that could lead to a failure. Bridge inspectors must meet defined qualifications and recognize the importance of observations made during an inspection. These qualifications are specified through the NBIS program, but may vary if more rigorous requirements have been implemented at the State level. Additionally, the bridge and the details themselves must be readily inspectable. The bridge designer must consider whether or not the details that have been developed can be inspected with relative ease. Obviously, some design details can not be inspected by typical means and methods. For example, prestressing tendons in a prestressed concrete girder, or internal pistons of a pot bearing, are virtually impossible to fully inspect. Post-earthquake inspection of some foundations would require earth moving equipment for inspection.

Bridges must also be maintained. A good maintenance program will help to reduce the potential for deterioration that leads to a bridge failure. Steel bridges often require cleaning and repairs associated with corrosion. Likewise, concrete bridges often require concrete repairs related to concrete degradation and reinforcement corrosion. If bridge maintenance is not routinely performed, deteriorated areas in need of repair will increase, resulting in the increased potential for a bridge failure.

1.4 Design Errors and Omissions

Errors and omissions during the design of a bridge can have serious consequences. The designer must provide calculations that are performed in accordance with the bridge owner's governing design specifications and standards and meet the appropriate standard of care. The structural details, such as shear connectors, reinforcing bars, or gusset plates, must be suitable and meet all design requirements. The bridge design plans must be developed in accordance with the design calculations performed by the bridge designer. In addition, the correct materials must be specified in the contract plans, and in accordance with the bridge owner's specifications.

Failure in any of these aforementioned tasks will result in, at the very least, an unsuccessful bridge process, or at worst, a failure of the bridge itself. The framework developed herein does not necessarily consider the effects of an incorrect design for each specific bridge type, since design errors and omissions can occur in many places during the design process. A well established QC/QA program can help to reduce errors and omissions.

1.5 Quality Control and Quality Assurance Programs

The Bridge Owner plays the most important role in the quality and success of a project. The Bridge Owner must clearly establish the requirements and expectations of a project. These requirements and expectations must be communicated and understood by the bridge designer and the contractor. The owner, the designer and contractor are then expected to work together to meet the requirements and expectations.

Quality Control/Quality Assurance (QC/QA) programs are formal office or organizational procedures or practices for ensuring that the owner's requirements and expectations are fully met. A QC/QA program provides checks and balances within an organization to assure quality in the final products. QC/QA programs are implemented at different levels or phases of work activities. For example, in the design phase, the bridge designer is responsible for making sure his/her calculations and drawings are accurate and meeting the requirements of the design. The bridge designer is performing QC of his/her own work by establishing a procedure for self-checking the work for accuracy and correctness. On the other hand, the reviewer, practicing QA, is responsible for independently checking the work of the bridge designer to assure accuracy and correctness in meeting the design requirements and expectations of the bridge designer. In construction, QC is the responsibility of contractor to ensure that the quality of the work is carried out in compliance with the contract provisions. On the other hand, the owner is responsible for practicing QA to assure that the contractor is carrying out the work in accordance with the contract.

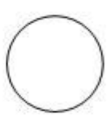
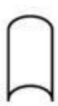
A good QC/QA program is a deliberate and systematic approach to reduce the risk of introducing errors and omission into a design. The likelihood of a failure in any design process is increased if standardized procedures are not established and followed. In some cases, the root cause of a bridge failure can be traced back to a failure to create or follow a good QC/QA program. The implementation and adherence to a good QC/QA program will likely reduce the possibility of failure of the overall bridge design process.

For major bridge projects involving unusual, complex, and/or innovative features, a peer review may be desirable to raise the level of confidence in the quality of design and construction. A peer review is generally a high-level review by a special panel of professionals specifically appointed by the Bridge Owner to meet the needs of the project. Peer review is an effective way to improve quality and to reduce the risk of errors and omissions.

1.6 Fault Tree Diagram

A fault tree diagram is a graphic model that shows parallel and sequential failure paths that can lead to an undesirable outcome: in this case a bridge failure. The fault tree diagram is helpful in determining potential failure modes and their interactions in a complex system such as a bridge. A fault tree diagram is developed in a top-down direction. In this application the top event is the failure of the bridge. The events immediately beneath the top event lead to the execution of the top event. Successor events and conditions that most directly lead to the predecessor events are then determined. This process is repeated at each successive level of the fault tree until the diagram is complete. Table 1-1 provides an explanation of the symbols used in the fault tree diagrams contained in this document. Specific information regarding the use of fault trees in engineering type applications can be found in Haasl et al. [6].

Table 1-1 *Symbols used in typical fault tree diagrams, describing Events, Basic Events, Or Gates and And Gates.*

Symbol	Usage	Usage
	Events	Represents the Top Event and the Intermediate Events in the fault tree
	Basic Event	Represents the Basic Event in the fault tree. Will be the lowest level of resolution in the tree.
	Or Gate	The output event associated with this Or Gate exists if at least one of the input events (preceding event) exists.
	And Gate	The output event associated with this And Gate exists if only all of the connected input events (preceding events) exist simultaneously.

Utilizing a fault tree diagram, a bridge is modeled to demonstrate/determine the critical failure paths. The fault tree diagram illustrates the structural component interactions, redundancy, actions/causes such as corrosion or fatigue, and environmental impacts such as flooding or scour.

The failure paths depicted in the fault tree diagrams are intended to provide bridge designers with a means to improve bridge designs and prevent future failures. If bridge designers understand critical failure modes related to the particular bridge being designed, they may be able to employ additional analyses or design calculations to investigate potential failures and determine strategies to assess and rectify issues not typically addressed during the design phase.

The general fault trees presented in this document are qualitative; however these fault trees can be used in a quantitative sense as well. The vulnerability of a bridge can be determined by a numerical evaluation that employs the probability values of each basic event. Once the probabilities of the basic events are determined, Boolean algebra can be performed. This will result in the probability of failure of the top event (in this case, a bridge failure) and the relative importance ranking of each path of the fault tree. LeBeau and Wadia-Fascetti [7] discuss the use of Boolean algebra to determine the probability of failure of different failure path mechanisms for the collapse of the Schoharie Creek Bridge. Similarly, Daniels et al. [8] perform a quantitative vulnerability assessment of several steel bridges, employing a method analogous to a fault tree analysis. Both of these studies show that fault trees have both qualitative and quantitative advantages that could be employed by a bridge designer during the design process. The reader can refer to these studies for information regarding a numerical evaluation associated with fault trees in bridge design.

2.0 STEEL GIRDER BRIDGE FAILURE FRAMEWORK

A general fault tree for the case of a steel girder bridge failure is developed and presented in this section. As discussed earlier, failure in this framework refers to a total collapse of the bridge system or an event that renders the bridge unfit for service. In general, a fault tree tends to be project specific, such as the case of the failure tree developed by LeBeau and Wadia-Fascetti [7] for the collapse of the Schoharie Creek Bridge. However, for this framework, in an attempt to include all steel girder bridges, both I- and box-girders, the fault tree presented in Figures 2-1, 2-2, 2-7, 2-8, and 2-12 are developed for a general case. The general fault tree developed below points out events that could cause a failure in the bridge, which could be addressed during the design of the structure.

The fault tree developed for steel girder bridges assumes that the bridge is designed and constructed according to the governing specifications for normal design loads as well as required extreme events. It also assumes that regular inspections and maintenance are performed over the service life of the bridge.

The fault tree is established with the top event, the *Steel Girder Bridge Failure*, as shown in Figure 2-1. The failure can develop from four different categories; *Design/Operation, Inspection, Construction, or Fabrication.* These four categories are joined by an Or Gate, which means any one of the four conditions can result in a failure. A bridge specific fault tree will be more refined than the general case provided in this framework. Also, not all of the aforementioned conditions will necessarily apply to the specific bridge in question, but the designer should be aware of all of the possible events on the fault tree. For example, there will be some construction and steel erection aspects that do not necessarily fall under the bridge designer's control, but more so for an engineer working for a contractor, fabricator, or steel erector. These aspects are presented here so bridge designers understand and take into account the entire process of the design, fabrication, construction, and inspection of the bridge.

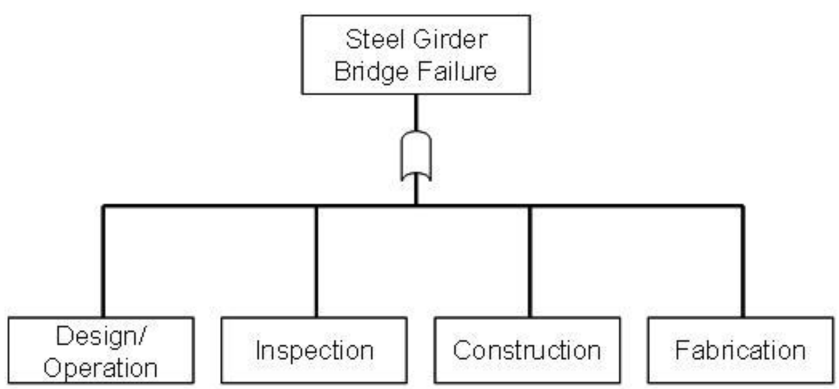


Figure 2-1 *Steel girder bridge fault tree, showing top categories only which include Design and/or Operation, Inspection, Construction, and Fabrication.*

The *Design/Operation* category alludes to the fact that a failure, either a collapse or a critical defect, can occur while the bridge is in service. *Inspection* refers to the fact there may be a problem with the regular inspection resulting from a design that does not facilitate inspection of the bridge components. The Inspection category does not intend to encompass problems with the actual bridge inspection, but highlight issues that can be addressed during design that will facilitate bridge inspection. A failure of the bridge process can occur during the *Construction* of the steel girder bridge, whether it is a collapse or a problem that results in delays. The steel *Fabrication* process is also subject to errors and problems, which could result in a failure of the bridge process. These are all conditions that the bridge designer must be aware of, and give due consideration to, when designing a steel girder structure. Each of the four categories is developed into a more detailed fault tree.

2.1 Design/Operation Category

The fault tree that follows the *Design/Operation* category is shown in Figure 2-2. While in service, a bridge failure can result from either a failure of the superstructure or substructure. A detailed representation of the superstructure fault tree is shown in Figure 2-2, while the substructure is shown later in Figure 4-1.

A failure of the steel girder superstructure can be caused by a failure in any one of the superstructure components; the most severe are those occurring in the girders, cross frames or diaphragms (in curved girder bridges), bearings, or concrete deck. Again, an Or Gate is used to join these fault scenarios, meaning that a failure of anyone of these components will cause a failure of the superstructure. A failure of the superstructure will then trigger a design/operational failure of the bridge.

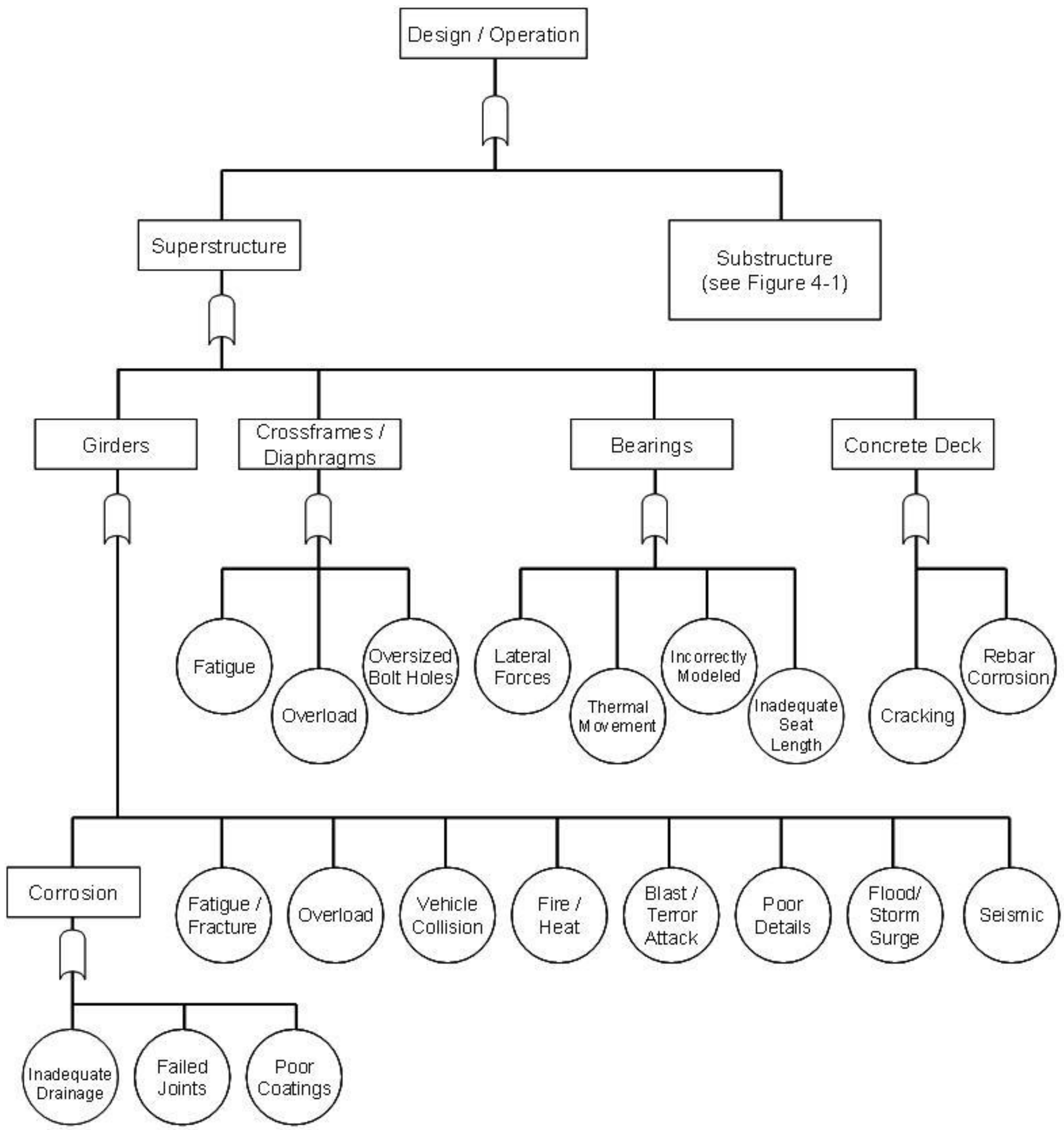


Figure 2-2 *Portion of the steel girder bridge fault tree showing the Design and/or Operation category for superstructures only, with several Events and Basic Events provided.*

2.1.1 Superstructure – Girders

2.1.1.1 Corrosion / Adequate Drainage Details

Girder failure can be caused by one or more events, as shown in the fault tree, Figure 2-2. For example, inappropriately detailed drainage components can often become clogged and not provide for the removal of water and deicing salts from the structure. Thus, deterioration (corrosion) caused by the combination of water and deicing salts can cause the failure of the main bridge girders. This is depicted in Figure 2-2 in the lower left corner of the figure.

The importance of adequate drainage was evident in the collapse of the I-95 Mianus River Bridge in Greenwich, Connecticut, in 1983. The bridge, completed in 1958, used floor beams that framed into two main girders, and employed pin-and-hanger assemblies in the two main girders. Maintenance personnel testified that drains in the bridge were difficult to keep open because the scuppers and downspouts were too small and downspout slopes were too shallow and changed direction too abruptly [9]. Additionally, much of the down-spouting was inaccessible or difficult to repair. In a 1973 resurfacing project, the curb drains were covered with steel plates and asphalt, forcing the water, deicing salts, and debris to drain through the expansion joints. The troughs beneath the expansion joints, and just above the pin-and-hanger connections and bearings systems, became clogged with debris.

The failure investigators determined that one of the pin-and-hanger assemblies was subjected to a significant amount of corrosion resulting from the clogged troughs and drainage modifications. It was concluded that corrosion packout of the pin-and-hanger assembly was primarily responsible for the lateral displacement of the hanger on the pin, and subsequent collapse of the particular suspended span of the bridge [9, 10].

It is important to locate drainage scuppers and connecting elements in regions of the superstructure where, if they become clogged with debris, they will cause the least damage. It is also important that the down-spouting be accessible for inspection, maintenance, and repair, and has the necessary slopes and connections to prevent clogging. In addition, it may be advantageous to make the steel superstructure continuous and eliminate expansion joints, thus eliminating drainage details that cause problems. An example of where simple spans are made continuous, to eliminate poor drainage details is shown in Figure 2-3. Adequate drainage and the layout of the drainage elements will help to prevent girder corrosion.

a) Before Rehabilitation

b) After Rehabilitation

Figure 2-3 *Photos showing the before and after of the elimination of trough-type drainage details by making simple steel spans continuous during a rehabilitation project.*

In addition, girders that are part of an overpass structure are prone to deicing salt-spray. Once deicing salts are applied on the roadway below, traffic on that roadway will cause the salt and water combination to become airborne, with some of the deicing salt/water solution splashing onto the steel bridge girders overhead. If the girders in these splash zones are not adequately protected against corrosion, such as protection through proper painting methods, the girders will deteriorate, leaving the bridge system vulnerable to critical defects.

2.1.1.2 Fatigue and Fracture

There have been several reported steel girder bridge failures caused by either fatigue or fracture of the girder steel. A significant amount of research has been performed regarding fatigue over the past 50 years. This research has led to the categorization of various bridge details in the current design specification that can be used by the bridge designer to investigate fatigue in the design process. The AASHTO *LRFD Bridge Design Specifications* [2] has separated fatigue into two categories: load induced fatigue and distortion induced fatigue. In general, when provisions concerning fatigue sensitive details are followed, the potential failure of a bridge due to fatigue is significantly reduced. However, it should be noted that fatigue cracking due to out-of-plane distortions of the girder web, at the diaphragm connection plates of multiple girder bridges, has resulted in the largest number of cases of fatigue cracking in steel bridges.

Distortion-induced fatigue is addressed in the current specifications, but not in as much detail as load-induced fatigue. Distortion-induced fatigue crack growth generally results from small deformations, usually out-of-plane, in localized areas, and may not be readily apparent during the design process. Fisher et al. [11] provide several examples of details that can be susceptible to distortion-induced fatigue:

- Transverse stiffeners cut short of the tension flange.
- Rotation of a floor beam attached to the main girder web.
- Web gaps in multi-girder bridges at the diaphragm connections.
- Web gaps in box girder bridges at the internal diaphragms.
- Lateral bracing connections.

Distortion-induced fatigue details must be given consideration by bridge designers. For additional information regarding fatigue, other than the current specification, the reader is referred to Demers and Fisher [10], Fisher et al. [11], Fisher [12], and Fisher et al. [13].

A brittle fracture can result in the failure of the steel girder, causing a collapse of the entire system or a critical defect that must be repaired. Steel girder fractures can occur with no warning. Sometimes the details that lead to a fracture are not easy to inspect. Compared to fatigue cracking, the number of brittle fractures in highway bridges has been relatively small over the past 40 years [14].

Several brittle fracture failures have been noted in the archival literature:

- In January of 1977, a fracture was discovered through the welded bottom flange and web splice of an I-girder on the I-79 Ramp Bridge crossing the back channel of the Ohio River near Pittsburgh, Pennsylvania. The fracture was brittle with little or no apparent plastic deformation of the steel [15]. The crack in the 30 in. wide by 3.5 in. thick bottom flange opened approximately 1.75 in., and extended upward through the full depth of the 132 in. by 0.5 in. web plate. The crack terminated at the underside of the top flange. The fractured girder was a main girder of a three-span, two-girder-floorbeam type bridge system. The crack initiated in a butt weld detail that was prepared using an early version of the electroslag welding process. At the time, the electroslag welding process had been used in building design but not in bridge design. The I-79 Bridge was one of the first bridges to employ the electroslag weld process. Soon after this failure, the FHWA prohibited the use of electroslag welding in bridge members subjected to tension. However, in March 2000, the FHWA issued a memorandum allowing the use of Narrow-Gap Improved Electroslag Welding (NGI-ESW) that can be used as an alternative welding procedure in bridge structures [16].

- In 1975 the Lafayette Street Bridge in St. Paul Minnesota had a fracture in one of the main girders near the center span inflection point. The Lafayette Street Bridge was also a three-span, two-girder-floorbeam type bridge system. This structure, similar to the I-79 structure, was capable of carrying traffic after the fractures and without collapse. This was attributed to the bridge system behaving like a torsionally stiff box [11]. Redundancy was provided by the continuity of the three span structure, as well as the lateral bracing, floorbeams/stringers, and concrete deck and shear connectors, even though the bridge was a two-girder system.

- In December 2000, the Hoan Bridge, in Milwaukee, Wisconsin, experienced a brittle fracture in all three girders in a cross section of the three-span continuous plate girder south approach unit. Two of the girders had full depth fractures. The lateral gusset plates were slotted to fit around the transverse connection plates. It was determined that all three girder web fractures initiated from the geometric restraint and resulting triaxial stress at the intersection of the lateral bracing gusset plate and transverse connection plate welded connection with both intersecting and overlapping welds [17].

- In 2003, a fracture was found during a routine inspection on one of the main girders of the US 422 Bridge over the Schuylkill River in Pottstown, Pennsylvania. The bridge is a six-span two-girder steel structure, and employed a lateral bracing system near the bottom flange. The fracture, shown in Figure 2-4, was located in a positive moment region, and initiated at the intersecting welds connecting the web, lateral bracing gusset plate, and transverse connection plate approximately 3 in. above the bottom flange [17]. The fracture propagated upward in the web, above the gusset plate, and also propagated downward and completely severed the bottom flange.

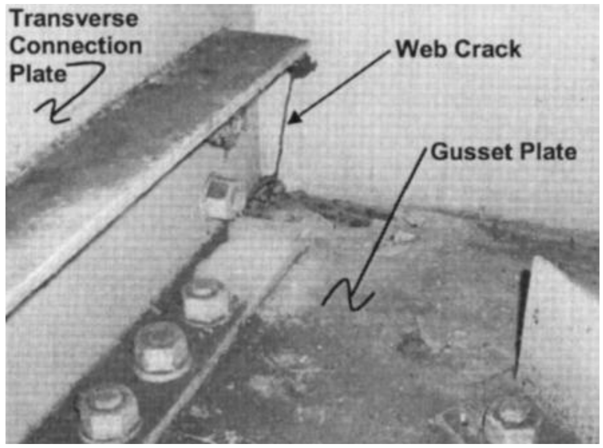


Figure 2-4 *Photo of fractured girder of the US 422 Bridge showing a crack in the web emanating for intersecting welds (taken from [17])*

The US 422 Bridge and Hoan Bridge fractures resulted from constraint-induced fracture, CIF [17]. Conner et al. [17] states that three conditions must be present for a detail to be vulnerable to CIF.

1. There must be an area of high stress concentration that locally magnifies the stress level occurring in the web plate.
2. The local stress concentration must occur in an area where the plate is under high constraint, preventing local yielding.
3. There must be sufficiently high tensile stresses present at the detail, including dead load, live load, and residual stresses.

Furthermore, Connor et al. [17] provide prevention strategies that can be employed during design to mitigate constraint-induced fracture.

During the design process, and more so for a non-redundant system, designers may need to investigate the potential failure of a steel girder due to a brittle fracture in a flange and/or web. Finite element programs and computer simulations can be used to investigate the loss of continuity in a bottom flange for example. Alternatively, when possible, connections should be detailed and material specified appropriately to avoid fracture critical details and/or members.

Fracture toughness requirements are specified in the AASHTO *LRFD Bridge Design Specifications*, Article 6.6.2 [2] for given temperature zones (specifically Table 6.6.2-2). All main components and connections subjected to tensile forces, including those subject to a reversal of stress, are subject to mandatory Charpy V-notch fracture toughness testing and should be specified as such on the design plans. Another primary reference for designers regarding fracture in steel structures is by Rolfe and Barsom [18].

Also, per the AASHTO *LRFD Bridge Design Specifications,* Article 6.6.2 [2], the bridge designer is responsible for determining if any steel bridge component is a fracture critical member. The designer is also responsible for clearly specifying those fracture critical members on the design plans. Fracture critical members are defined as a pure tension member or tension components of a bending member in which the failure of that member would result in a collapse of the bridge, i.e., no alternate load path.

2.1.1.3 Overload

An overload, such as an illegal non-permitted load or an overload during construction, could cause the steel girder to fail, leading to a failure of the bridge. All bridges are designed for a given design load and possibly permit load, in accordance with current AASHTO *LRFD Bridge Design Specifications* and as modified by State bridge design specifications. Pedestrian bridges need to be designed for pedestrian loads, and some vehicle loads, depending upon the owner's requirements. In most cases, there is an inability to monitor whether or not a vehicular bridge has been overloaded by a non-permitted load.

Bridge designers need to be aware that overloads do indeed occur on most bridges. During design it may be necessary to investigate owner specified loads, as well as a potential overload condition for vehicular routes that are more likely to experience non-permitted vehicles, based on an owner's recommendation.

2.1.1.4 Vehicle and Vessel Collisions

A vehicle or vessel collision with a steel girder superstructure could cause a failure of the bridge. Overpass bridges, which cross over an interstate for example, can be subjected to an over-height vehicle collision. Instances of these do occur, such as special loads, or oversized loads, which may exceed the clearance provided. Similarly, a bridge over a navigable waterway could be hit.

During the bridge design process, the bridge designer must verify that the design team has adequately addressed the clearances required by the owner. If a risk of impact from over height vehicles or vessels reasonably exists, steps could be taken to either mitigate the potential collision (use of a sacrificial beam to protect load-carrying girders) or evaluate the impact on the structural system.

2.1.1.5 Fire / Extreme Heat

During an extreme heat event there can be a significant loss of stiffness in the girder and its connections. A vehicle or vessel fire below a steel girder bridge could cause the girders, as well as other bridge components, to fail resulting in a failure of the bridge.

In April of 2007, an extreme fire event did occur on a highway in Oakland, California which caused the collapse of the over passing multi-I-girder steel structure. A gasoline tanker truck caught on fire, and came to rest under the overhead structure, which funnels traffic onto the San Francisco – Oakland Bay Bridge. A thermal threshold was exceeded, causing the steel girders to buckle, leading to the collapse of the span.

Bridge designers need to be aware of the fact that a fire can occur below a steel girder superstructure. Depending on the importance of the structure, it may be necessary to investigate the bridge behavior due to an extreme heat event to ensure that a collapse does not happen. Alternatively, a design could be developed that would allow some delay before a collapse would occur, which would minimize the potential loss of life.

2.1.1.6 Blast / Terrorist Attack

Blast loads resulting from explosions, triggered during a terrorist attack for instance, that occur below the bridge deck will impart large uplift forces on the superstructure. The forces may cause the girders to slip off of the bearings, deformations of the steel girders, or reduce the composite behavior of the deck and steel girders. Similarly, an explosion on the bridge deck may cause large areas of the deck to fail, the girders to deform, or reduce the composite behavior of the deck and steel girders. In both cases, the explosion may result in a sustained fire, which could lead to failure of the superstructure.

Depending on the importance and vulnerability of the structure being designed, the bridge designer may need to consider the effects of blast loads on the superstructure during the design process. The designer, owner, and/or security personnel should perform a risk assessment to determine the threats that a particular bridge may be subjected to. If the risk is low, a simple overpass for example, then the consideration of blast loads may not be warranted. However, if the bridge is an important structure, and/or the risk of a terrorist attack is high, the designer may need to consider the effects of blast loads on the superstructure. The effects of blast loading can typically be investigated through computer simulations and the use of finite element analysis procedures. It may be difficult to develop a design that would resist all types of terror attacks, however a design could be developed that would delay collapse and allow the preservation of human life.

For additional information regarding the consideration of blast loads and terrorist attacks in the design process, the reader is referred to the following references:

- The FHWA assembled a Blue Ribbon Panel on Bridge and Tunnel Security to address issues related to the vulnerability of bridges and tunnels to terrorist attacks [19]. The panel investigated funding requirements associated with future research and implementing countermeasures against terrorist attacks.
- Through risk management techniques, Williamson and Winget [20] outline methods to mitigate the risk of terrorist attacks against critical bridges providing cost effective security measures, discussion of blast effects on bridges, and structural design guidelines.
- Ray [21] describes a risk based methodology that can prioritize the needs for terrorist attack mitigation for specific bridge components.
- Winget et al. [22] discuss how incorporating physical security and site layouts in the design process can help to mitigate terrorist attacks. The authors also discuss the potential effects of blast loads on bridges, and provide structural design solutions that may be employed by the bridge designer to reduce these effects.

2.1.1.7 Poor Details

For some steel girder bridges, certain details may prove unusually costly, unusually time consuming, or difficult to fabricate and/or erect. If the detail is too difficult to fabricate or erect, then delays may result in the fabrication process or construction. For more complicated details, for example, cross frames at a severely skewed pier, it is worthwhile for the bridge designer to contact fabricators and obtain their opinion on the ability to fabricate the detail in question. Similarly, a designer should contact a steel erector concerning a complicated erection scheme or connection assembly. It will help to prevent delays, extra costs, and potential failures during the bridge project if the designer investigates complicated details during the design process. Additional information regarding general steel girder bridge details can be found in the AASHTO/NSBA Steel Bridge Collaboration document *Guidelines for Design Details* [23], and in *Practical Steel Tub Girder Design* [24] specifically for steel box girders.

2.1.1.8 Flood / Storm Surge

A bridge with a small vertical clearance over a waterway could be vulnerable to damage resulting from a debris flow in a flood situation. If the vertical clearance is small, it is possible that the girders of the structure will cause flood debris to be stopped at the bridge. This debris stoppage and water flow could lead to additional lateral loads on the steel girders that were unanticipated during design. Wardhana and Hadipriono [1] found that 16 cases of bridge failure (any type of bridge) derived from debris flows in the same year, 1995, resulting from flash flooding in Madison County, Virginia. For additional discussion on storm surge, see section 3.1.1.10.

2.1.1.9 Seismic

Seismic loads can be one of the most critical extreme events that must be considered by bridge designers. Depending on the geographic location of the structure, seismic loads may govern the design. Several bridge failures, both steel and concrete, have been reported on in textbooks and the archival literature, most notably are bridge failures associated with the 1989 Loma Prieta earthquake in California. It is not the intent of this document to list all of the noted bridge failures due to seismic activity, potential design procedures, and load mitigation techniques, but to highlight the need to consider earthquake loads when necessary. However, it should be noted by bridge designers that in many cases bridge failures due to seismic loads are caused by inadequate bridge seat lengths and/or the use of non-ductile details in both the super and substructure elements.

The reader can refer section 3.10 of the current AASHTO *LRFD Bridge Design Specifications* [2] for earthquake design loading. However, the bridge designer should be aware that local and/or individual State specification may take precedence, or be more restrictive than the AASHTO *LRFD Bridge Design Specifications*. It is the responsibility of the bridge designer to use the correct seismic design specifications.

Additional information regarding seismic loading and design in bridges can be found in the following references (not an all-inclusive list):

- "NCHRP Report 472: Comprehensive Specification for the Seismic Design of Bridges," [25]
- "Recommended LRFD Guidelines for the Seismic Design of Highway Bridges, Part I: Specifications and Part II: Commentary and Appendices," [26]. These guidelines are based on NCHRP Report 472.
- *Seismic Design and Retrofit of Bridges* [27].

2.1.2 Superstructure – Cross Frames / Diaphragm

Cross frames and diaphragms are generally considered secondary members in straight girder bridges. However, in horizontally curved steel girder bridges cross frames and diaphragms must be considered primary members, and designed as such. The cross frames in a horizontally curved girder bridge transmit forces necessary to provide equilibrium, therefore these forces must be considered in design.

A failure of a cross frame in a steel girder superstructure does not necessarily mean that the structure will collapse, but it does result in a critical defect that must be assessed. Similar to the girders in a steel girder superstructure, the cross frames can fail due to fatigue and overload considerations.

2.1.2.1 Use of Oversized Bolt Holes

Oversized bolt holes are sometimes used in the connections between the cross frames and the girders in steel girder bridges. It is believed by some in the bridge industry that oversized bolt holes facilitate the steel erection, providing additional flexibility to make the connections. While this practice may be suitable for straight steel girder bridges, it is, in most cases, not suitable for horizontally curved and/or skewed steel girder bridges. The control of geometry is extremely important during the steel erection of a horizontally curved and/or skewed steel girder bridge. The use of oversized holes will likely compromise the geometric control necessary to successfully assemble the bridge components. As the girder lines are erected across a curved I-girder bridge, the girders will displace and rotate more due to the extra space of the oversized holes. This additional displacement and rotation will often cause the girders to be at a location that is not in accordance with the final top of steel elevations typically shown in design plans. The bridge designer must understand the potential pitfalls associated with the specification of oversized bolt holes.

2.1.3 Superstructure – Bearings

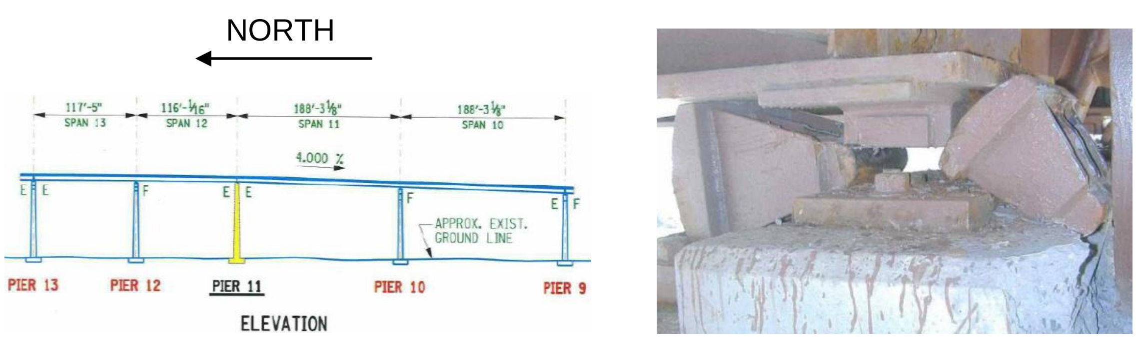
Bearing failure is a critical defect that could lead to collapse or partial collapse of a bridge. One primary cause of a bearing failure is a longitudinal and/or lateral force that exceeds the capacity of the bearing to resist that force and subsequently causing the girders to slip off the substructure seat. Such a force could result from an extreme wind event, water (associated with flooding), an earthquake loading, or vessel/vehicle collision. Rocker bearings are particularly susceptible to longitudinal forces, and are therefore not typically employed in new bridge designs and are often replaced as part of bridge preservation programs.

Elastomeric bearing pads can exhibit failure modes such as crushing, delamination, and slippage. Slippage tends to be a significant serviceability issue for neoprene bearing pads, as reported by McDonald et al. [28]. Refer to the paper by McDonald et al. [28] for recommendations on preventing slippage of elastomeric bearing pads.

Unless significantly exceeding allowable displacements, thermal movements alone don’t typically cause bearing failure, but can be a major contributor. Longitudinal and lateral restraint or provision for displacement must match the assumptions used in the analysis. This is especially true when modeling curved and/or skewed girder bridges, since an inconsistency in the analysis may exceed the bearing's load capacity or permitted motion, or cause distress in other structural elements.

Rocker bearings can be susceptible to failure resulting from what is known as "ratcheting." Debris and/or corrosion material can build-up on the rocker seat area and can prevent the bearing from moving freely as intended by design, thus imparting longitudinal forces into the support structure. The build-up of material, coupled with the movements caused by thermal cycles can result in the "ratcheting" effect of a rocker bearing. Two recent rocker bearing failures highlight this issue:

- On July 27, 2005, in Albany, New York, two steel I-girder spans of the I-787 Ramp AC structure (Dunn Memorial Bridge Interchange) fell of their support bearings at Pier 11 supporting the expansion ends of two different two-span units (see Figure 2-5). The authors of the forensic investigation indicated that the failure can be most likely attributed to the fact that the Span 12 bearings became overextended, due to their inability to return toward vertical during time of expansion of span 12, thus pushing Pier 11 southward to accommodate expansion [29]. During periods of Span 12 contraction, the bearings could tip further north becoming more overextended, and repeated thermal cycles would cause the bearings to "ratchet" further northward, until they became unstable [29].
- A similar rocker bearing failure occurred in a steel plate girder approach span of the Birmingham Bridge in Pittsburgh, Pennsylvania, in February 2008. The rocker bearings at Pier 10S tipped over, dropping the steel girders and roadway approximately 8 inches. Based on the forensic investigation, the rocker bearings most likely tipped over at the pier due to a combination of events, including improper installation and a leaking expansion joint above the bearings that contributed to and accelerated the corrosion of the bearings. Collecting debris also allowed the ratcheting effect [30]. A photo of the tipped bearing at Pier 10S is shown in Figure 2-6.



(a) (b)

Figure 2-5 *Dunn Memorial Bridge Interchange Rocker Bearing Failure: (a) Sketch of the bridge elevation from Pier 9 to Pier 13; (b) Photo of the failed rocker bearings at Pier 11 showing them tipped over (taken from [29]).*

Figure 2-6 *Photo of failed rocker bearing at Pier 10S of Birmingham Bridge Approach Span showing the rocker tipped over with girder sitting on top of failed bearing (taken from [30]).*

2.1.4 Superstructure – Concrete Deck/Railings

The failure of a concrete deck in a steel girder superstructure is not likely to cause a collapse of the bridge, but could result in a critical defect on the bridge. Cracking of the concrete, combined with corrosion of the steel reinforcement could lead to a serviceability issue and poor ride quality for the users of the bridge. To prevent a deck failure, the designer must take steps to reduce cracking in the deck and corrosion of the reinforcement. For example, concrete admixtures to reduce shrinkage cracking, corrosion inhibiting devices, use of epoxy-coated reinforcement, and/or providing additional concrete cover to the steel reinforcement are methods that can be employed by the designer, in consultation with the owner's specifications, to reduce the likelihood of a concrete deck failure. The reader can refer to NCHRP Synthesis 333: *Concrete Bridge Deck Performance* for additional information regarding design and construction practices intended to improve the performance of bridge decks [31]. Topics in NCHRP Synthesis 333 include factors that contribute to the durability of concrete, performance of various deck protection systems, and lessons learned in design, construction, and maintenance of concrete bridge decks.

Additionally, bridge traffic barrier systems, otherwise known as railings, must meet a certain crash test level requirement in accordance with the AASHTO *LRFD Bridge Design Specifications* [2]. There are six bridge railing test levels specified, and named TL-1 though Tl-6. The various test levels are intended to evaluate performance factors of the railing, including structural adequacy, occupant risk, and post-impact behavior of the test vehicle. TL-4 bridge railings are typically satisfactory for interstate bridge designs. In several States, bridge owners have standard bridge railings that are to be used in by the bridge designer. However, if there are no specific State standards, the owner must specify which of the test levels is most appropriate for the bridge site [2].

The reader is also referred to section 3.1.1.2 of this document for discussion of creep and shrinkage in concrete, and section 3.1.1.3 regarding concrete properties.

2.2 Inspection Category

The fault tree that follows the *Inspection* category is shown in Figure 2-7. Inadequate inspection can contribute to a failure of the bridge by not finding, assessing, reporting, and initiating repair actions for a problem. Therefore details should be incorporated into the design by the designer to facilitate inspection of the bridge.

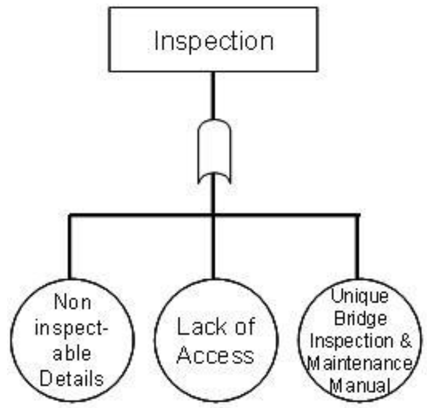


Figure 2-7 *Portion of the steel girder bridge fault tree showing the Inspection Category only, with three Basic Events provided.*

In addition to providing access, the details themselves must be inspectable. Elements that are difficult to inspect are typically problematic to maintain, which could lead to a condition in which fatigue cracks or other flaws may go undetected. During the design process, the designer should verify that all areas of the structure can be reasonably accessed, inspected, and maintained.

Adequately sized manholes must be provided in steel box girder bridges, which allow bridge inspectors access into the box itself. In a continuous box girder bridge, manholes must also be provided in the internal full depth diaphragms at the supports, which allow bridge inspectors to pass through from span to span. Information regarding manhole details in steel box/tub-girder bridges can be found in *Practical Steel Tub Girder Design* [24].

Furthermore, for bridges where access to the girders and bridge components may be difficult from underneath or by a snooper truck, it may be necessary to have walkways installed, or at least hand rails installed on the girders. Forethought by the designer during the design process, concerning the inspection of the structure, can help to prevent a potential failure of the bridge system.

In some cases, for unique steel girder bridges, it may be advantageous to have a bridge maintenance manual developed specifically for the bridge in question. This document would include items that the designer knows may be potential causes for structural problems in the future. Additionally, the manual should show what structural elements are fatigue sensitive and/or fracture critical so that the bridge inspectors know what elements should be given additional attention during the bridge inspection.

2.3 Construction Category

The fault tree that follows the Construction category is shown in Figure 2-8. This part of the overall fault tree may not necessarily apply to the bridge designer, but may be more applicable to a specialty engineer working for a contractor or steel erector. However, it is necessary that the bridge designer is aware of the potential construction issues when developing a new bridge design in order to facilitate (and/or simplify) bridge construction.

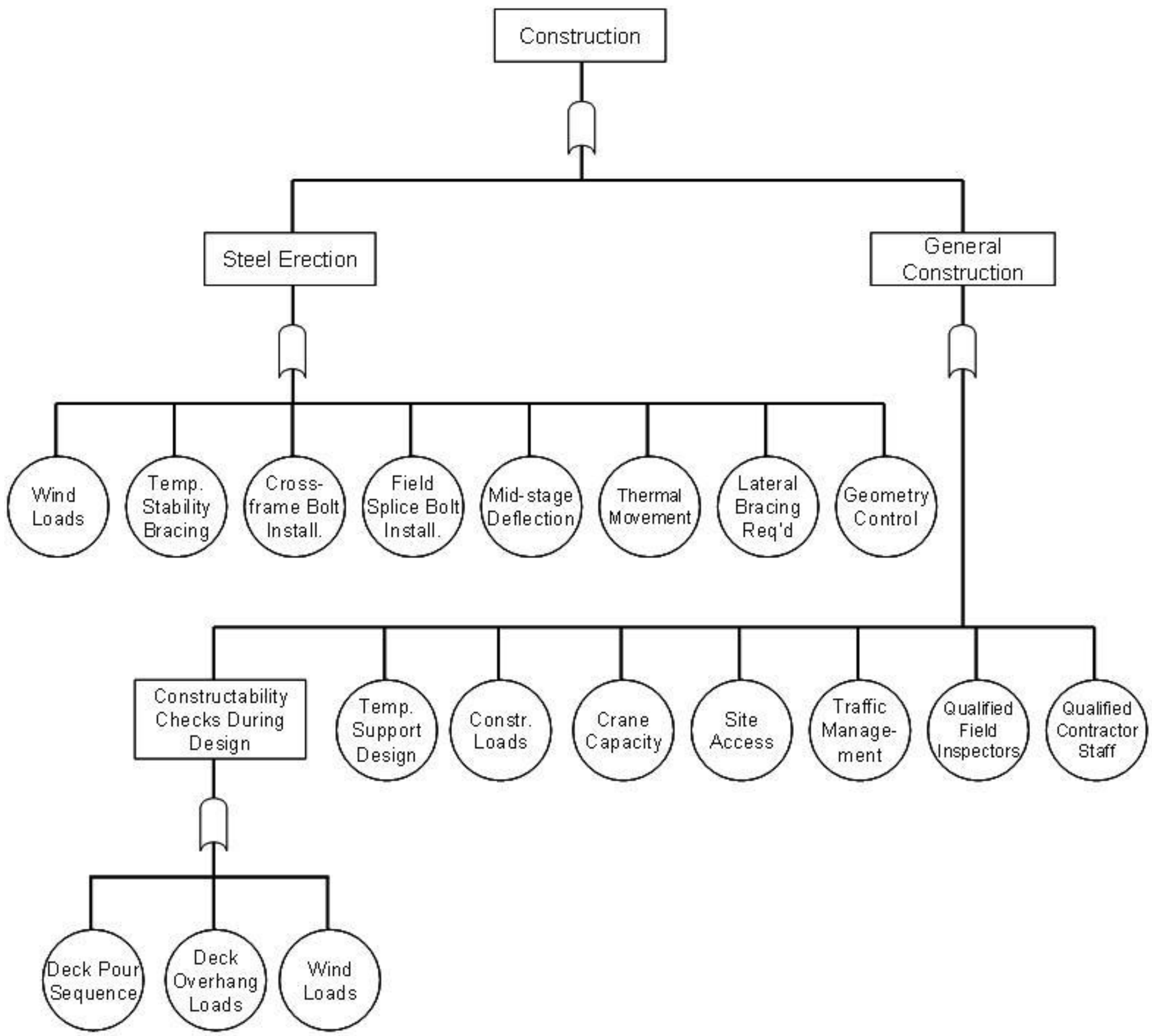


Figure 2-8 *Portion of the steel girder bridge fault tree showing the Construction Category only, with several Events and Basic Events provided.*

The Construction category is divided into two failure paths: *Steel Erection* and *General Construction*, as shown in Figure 2-8. This approach is used because the general construction path is typically applicable to all other bridge types, and will be referenced in latter portions of this framework. In the case of a steel girder bridge, a failure during the steel erection or during the general construction of the bridge can result in an overall failure of the bridge system. A failure may be a total or partial collapse during steel erection, or may be classified as a critical defect, such as a connection misalignment or limited crane access.

2.3.1 Steel Erection

A failure during the *Steel Erection* of a bridge will trigger a failure on the construction path of the framework, as depicted in Figure 2-8 and 2-1. As shown in Figure 2-8, there are several events that can cause the steel erection to be considered as a failure. These events acting alone may cause a failure, or the combination of any set of these events can cause a failure.

Wind loads, for example, can cause lateral forces that can overstress the girder flanges, or overload tie-down assemblies. If a girder is not properly braced during erection, wind loads may in fact cause the girder to roll over and possibly off of the supports. The girders need to be adequately braced for all lateral loads.

Bracing (temporary or permanent) is typically needed to resist wind loads and to provide stability for the girders. During steel erection, I-girders are often placed onto the supports with large unbraced lengths. These large unbraced lengths cause steel plate I-girders to be susceptible to lateral torsional buckling during steel erection. If the erected girder has not been designed for this temporary condition, has an inadequate number of brace points, or the bracing does not provide the necessary stiffness, the girder will buckle.

Several instances of inadequate bracing during steel erection have been reported. One such incident occurred on May 15, 2004, in Golden, Colorado, at the I-70/C-470 interchange, when a steel I-girder collapsed during a highway bridge-widening project. The girder, which was erected two nights prior, fell onto the roadway below, killing three occupants of a vehicle. The NTSB determined that the probable cause of the girder collapse was the failure of the girder's temporary bracing system and deficiencies in the installation of the girder and the bracing [32].

On May 16, 1995, a partially erected steel girder bridge on Tennessee Highway 69 collapsed. Wojnowski et al. studied the collapse, providing insight into the state of stress and buckling stability of bridge superstructures during steel erection [33]. The paper illustrates the effect that the lack of bracing has on the stability of a partially erected structure.

Bolted splices and connections require special attention during the steel erection. In accordance with AASHTO *LRFD Bridge Construction Specifications* [34], Article 11.6.5, splices and field connections shall have one-half of the holes filled with bolts and cylindrical erection pins (half bolts and half pins) before installing and tightening the balance of high-strength bolts [34]: this is a minimum condition. These bolting requirements, as well as the tightening requirements, must be specified by the bridge designer, in accordance with the owner's specifications, in the steel erection procedure. If these requirements are not stated in the steel erection procedure, field bolts may not be properly installed. The incorrect installation of field bolts could lead to unpredicted girder displacements or a girder collapse.

Displacements during the steel erection should be evaluated during development of the steel erection procedure by the specialty engineer working for a contractor. This is especially critical for horizontally curved and/or skewed girder bridges. Curved girders will rotate out-of-plane, as shown in Figure 2-9, causing the top and bottom flanges to displace out-of-plane, unless the rotation is prevented by supports or bracing. Large rotations and displacements may make connecting components difficult. Additionally, for straight, curved, and skewed bridges, deflections and rotations at field splice locations should be investigated to ensure that the connections can be made.

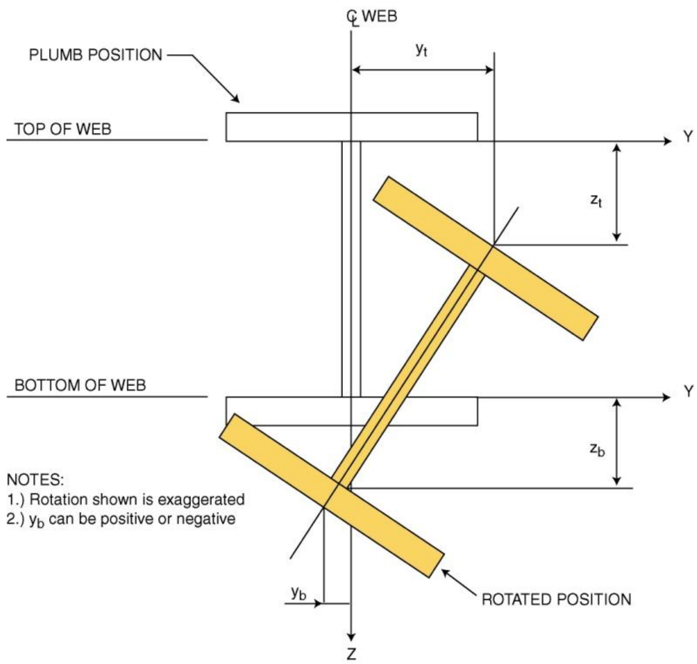


Figure 2-9 *Curved girders displace vertically and rotate out-of-plane causing the top and bottom flanges to displace out-of-plane unless the rotation is prevented by supports or bracing.*

Thermal movements during steel erection need to be considered by the specialty engineer engaged by the contractor. Bridges are typically designed for an ambient temperature of 68°F. If the bridge is erected when the temperature is higher or lower, there is potential for connection and component misalignments.

In steel I-girder bridges, lateral bracing (temporary or permanent) may be required to control lateral deflection due to wind loads. Longer spans (greater than 300 feet) generally have issues related to deflections due to wind. In their *Bridge Design Standards*, PennDOT [35] has requirements for lateral bracing applicable to erected steel superstructures, before the placement of the concrete deck. PennDOT Bridge Design Standard, BD-620M, states the permissible lateral deflection of a span due to wind loads is the span length divided by 150 (L/150). (Other States may have similar standards that may need to be followed; the PennDOT example is provided for reference only.) The bridge designer should investigate the need for lateral bracing during the design process to verify that the bridge, as shown on the design plans, is constructable. The engineer that is developing the steel erection sequence must consider wind loads, and their effect on the erected steel superstructure as well as during stages of the erection sequence.

During steel erection, especially for horizontally curved and/or skewed girder bridges, it is important that the erector control the geometry of the structure throughout the erection sequence. The use of temporary support towers (as shown in Figure 2-10), bracing, guy wires, or holding cranes are methods that can be used to control geometry. If the erected geometry of the superstructure deviates from the anticipated geometry, misalignments of the bearings, cross frames, or expansion joints can result. In more complex cases, the specialty engineer should provide the steel erector the anticipated superstructure geometry at each erection stage.

Figure 2-10 *Photo showing the use of temporary supports during erection of a curved steel I-girder bridge. The use of temporary supports help to control the geometry of the structure during steel erection.*

2.3.2 General Construction

As shown in Figure 2-8, there are several events related to the *General Construction* of a steel bridge that may cause a failure of the bridge system. Some of these events are required to be investigated during the design process by the bridge designer, such as the girder constructability checks.

In accordance with the AASHTO *LRFD Bridge Design Specifications* [2], Article 6.10.3, constructability checks of the steel girders are required during the design process. These checks indicate whether the girder will have adequate strength during critical stages of construction. Insufficient consideration of constructability during design could result in a failure during construction. Some items that need to be considered as part of the constructability checks include:

- *Deck Placement Sequence* - The concrete deck is often placed in multiple pours, resulting in girders being composite in some locations, and noncomposite in others at various times. A particular deck placement sequence can create temporary moments in the girders that are larger than the final noncomposite moments after the entire placement is complete.
- *Deck Overhang Loads* - Cantilever forming brackets (shown in Figure 2-11) are typically used along the exterior girders to construct the concrete deck overhangs. The eccentricity of the deck weight and other construction loads (screed, formwork, construction live load) acting on the bracket applies a torsional moment on the exterior girder. The overhang bracket will transmit loads to the top flange and the girder web, depending on the bracket and web height. The overhang bracket should terminate at the web-bottom

flange junction. However, if it does not (a deep girder for example) then the potential for web buckling needs to be investigated.

- *Wind loads* - Wind loads also need to be considered during the deck placement. The combination of loads associated with the deck placement and wind can result in an overstress condition. Typically, the specialty engineer may need to specify a certain wind speed at which the deck can be placed, in order to reduce the lateral loads acting on the girder.

Figure 2-11 *Photo showing the cantilever forming brackets which will support formwork that will be used to construct the concrete deck overhang on a simple span steel girder superstructure.*

A study related to a failure of a steel plate I-girder bridge during deck placement has been reported on by Waddle and Wang [36]. A three-span continuous composite plate girder bridge having end spans of 215 ft and a center span of 290 ft was studied. It was determined that local buckling in the web and that lateral torsional buckling of the plate girder occurred during or immediately following a particular stage of deck placement, before sufficient composite action was reached. The authors noted that a critical process to be accounted for in design is during the construction when all or parts of the structures act in a noncomposite fashion.

In 2002, workers were placing a concrete deck for a new pedestrian bridge in Marcy, New York, when the bridge suddenly collapsed, killing one construction worker and injuring nine others. The steel superstructure was a single trapezoidal box girder which spanned 170 ft, with no top flange lateral bracing. The girder buckled due to global elastic-torsional buckling when approximately 60 percent of the wet concrete had been placed. Recommendations resulting from a forensic investigation of the collapse [37] are: 1) require top flange lateral bracing in trapezoidal box girders; 2) require stability checks because long span girders can have stability limitations, with or without top flange lateral bracing.

Yura and Widianto [38] performed additional studies regarding the Marcy Pedestrian Bridge collapse. The authors further investigated the causes related to the lateral torsional buckling of the cross section, the need for lateral bracing in trapezoidal box girders, and the similar buckling case of a twin I-girder system. A twin I-girder system could be susceptible to global lateral torsional buckling at various stages of a steel erection sequence. A relatively simple equation was developed that could be used to approximate the buckling strength of a twin I-girder system.

During steel construction, temporary support towers, also known as falsework, may be used to reduce loads in the girders and help control geometry. Often, a temporary support may be selected from a manufacturer's catalogue or stock available at local contractor equipment rentals, based on the vertical load the supports must resist. The specialty engineer must check that the assumptions made in the capacity of the temporary support tower by the manufacturer are consistent with the conditions in the field. Temporary support towers from a catalogue often do not provide any lateral support so additional provisions are often needed if lateral restraint is required. Furthermore, the specialty engineer may need to specify how the ground is to be prepared for the foundations of temporary support towers.

During the design process, the bridge designer should consider limitations on the size and weight of field pieces. They should not be too heavy to be lifted by typical erection equipment which can access the site or too large (height, length) to transport and deliver by normal means. If the bridge can not be constructed by typical methods, this should be noted in the design plans.

Traffic management during the construction must also be considered. If road or lane closures are required to erect the bridge components, the bridge designer, in coordination with traffic engineers, should verify if closures will be allowed by the governing agency. Coordination early in the design process will help to eliminate potential conflicts when the structure is being erected.

Qualified field inspectors are required during erection to verify that components are placed within the specified tolerances at each stage of erection. If the components are not placed in the correct locations at the beginning of erection, it can compound problems as additional field pieces and/or components are erected. The inspectors need to be familiar with the given type of construction, and the known pitfalls that can occur.

2.4 Fabrication Process Category

The fault tree that follows the Fabrication category is shown in Figure 2-12. This part of the overall fault tree will apply to the bridge designer as well as the specialty engineer working for a contractor or steel erector. As shown in Figure 2-12, there are four basic events shown that can trigger a failure in a bridge project related to the fabrication of the steel superstructure.

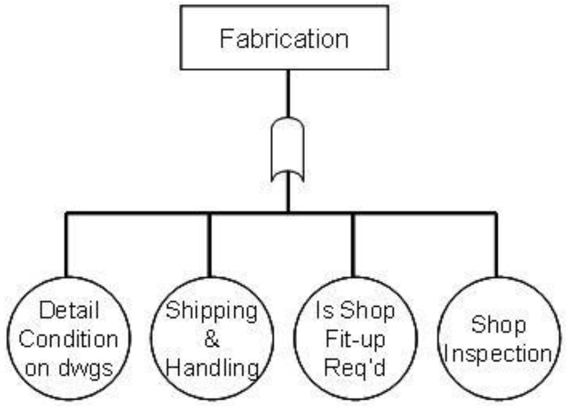


Figure 2-12 *Portion of the steel girder bridge fault tree showing the Fabrication Category only, with four Basic Events provided.*

In accordance with the AASHTO *LRFD Bridge Design Specifications* (2008), Article 6.7.2, for horizontally curved and/or skewed I-girder bridges, the contract documents are to clearly state the intended erection position of the girders and under which load condition that position is to be achieved. The intended erected positions are: girder webs theoretically plumb, or girder webs out-of-plumb. Common conditions at which the erected positions can be theoretically obtained are: no-load condition, steel dead load condition, or full dead load condition. When designing a curved and/or skewed I-girder bridge, the bridge designer needs to consider the detail condition that will be specified and how the fabricator can achieve that detail condition. If the intended erected position and load condition are not clearly identified in the contract plans by the designer, a failure of the erection process or long term problems may result. Article 6.7.2 and the commentary of the AASHTO *LRFD Bridge Design Specifications* [2], discusses this topic in great detail, and it should be reviewed by the designer for additional information. Further discussion on this topic is available in the AASHTO/NSBA Steel Bridge Collaboration document *Guidelines for Design for Constructibility* [39].

Additionally, if the bridge designer does not fully consider the consequences of detail conditions, a failure or critical defect may occur. This has been shown in studies conducted by Chavel and Earls [40, 41] regarding the detailing and erection of the horizontally curved span of the steel I-girder Ford City Veterans Bridge. The intended detail condition for the subject structure was for the girder webs to be plumb under steel dead load. This was to be achieved by detailing the cross frames for a web plumb position at the steel dead load condition, while the girders were detailed for the web-plumb position at the no-load condition, creating a detailing inconsistency, resulting in formidable fit-up problems and an increase in erection costs [41]. The choice of detail condition and methods of achieving the detail condition (for example, webs plumb at steel dead load) could have a significant effect on the success or failure of a curved and/or skewed I-girder bridge project.

Shipping and handling of girder field pieces must be considered by the bridge designer during the design process. Permits may be required and could put constraints on the design. If the fabrication is to be done in one State, and delivered to another, the fabricator must verify permit requirements for each State. The designer will need to assess length, weight, curved girder sweep, and allowable times of transit when necessary. The steel I-girder Lake Creek Bridges in northern Idaho required that the girders be shipped between different States. The permits in one State allowed shipment during the day, while the adjacent state only allowed shipment of the same piece at night [42]. This transporting issue resulted in construction delays. Longer girder field pieces may be too heavy to lift with typical erection cranes and transported by typical methods. If longer girder field pieces are used in the design, the bridge designer should contact a fabricator and/or transporter to verify that the necessary equipment and permit capabilities are available.

For complex structures, such as horizontally curved and skewed I-girder bridges, the designer should determine if full shop or partial assembly is appropriate prior to shipment of the bridge components. Verifying bracing connections in a full shop assembly will add confidence that the same pieces will fit under field conditions. However, this added confidence will come with an additional cost. AASHTO *LRFD Bridge Construction Specifications* [34], Articles 11.5.3 and 11.8.3.7, provide the designer with requirements concerning shop assembly.

Proper inspection of the bridge components in the fabrication shop will help to prevent failures in the fabrication process and subsequent failures during steel erection. If an error is undetected during fabrication, additional failures may result throughout the bridge project. The inspectors need to be familiar with the given type of bridge being fabricated, and be aware of the difficulties that can occur in the fabrication process.

3.0 PRECAST CONCRETE GIRDER BRIDGE FAILURE FRAMEWORK

A general fault tree for the case of a precast concrete girder (prestressed and/or post-tensioned) bridge failure is developed and presented in this section. As discussed earlier, failure in this framework refers to a total collapse of the bridge system or an event that results in a critical defect. In their study of recent bridge failures, Wardhana and Hadipriono [1] noted that concrete beam/girder bridge failures have the third highest number of occurrences, following steel girder and steel truss bridges. Twenty-nine of the 503 failures recorded and studied were failures of concrete beam/girder bridges. Similar to the steel girder bridge framework, this concrete bridge framework is developed for a general case, as shown in Figures 3-1, 3-2, 3-7, 3-8, and 3-9.

The fault tree developed for precast concrete girder bridges assumes that the bridge is designed and constructed according to the governing specifications for normal design loads as well as required extreme events. It also assumes that regular inspections and maintenance are performed over the service life of the bridge.

The fault tree is established with the top event, the *Precast Concrete Girder Bridge Failure*, as shown in Figure 3-1. The failure can develop from four different categories; *Design/Operation, Inspection, Construction, or Fabrication.* These categories are joined by an Or Gate, which means any one of the four conditions can result in a failure. Similar to the Steel Girder Bridge Failure Framework, not all of the aforementioned conditions will necessarily apply to the bridge designer, but the designer should be aware of all of the events on the fault tree. These aspects are presented here so bridge designers understand and take into account the whole process of the design, fabrication, and construction of the bridge.

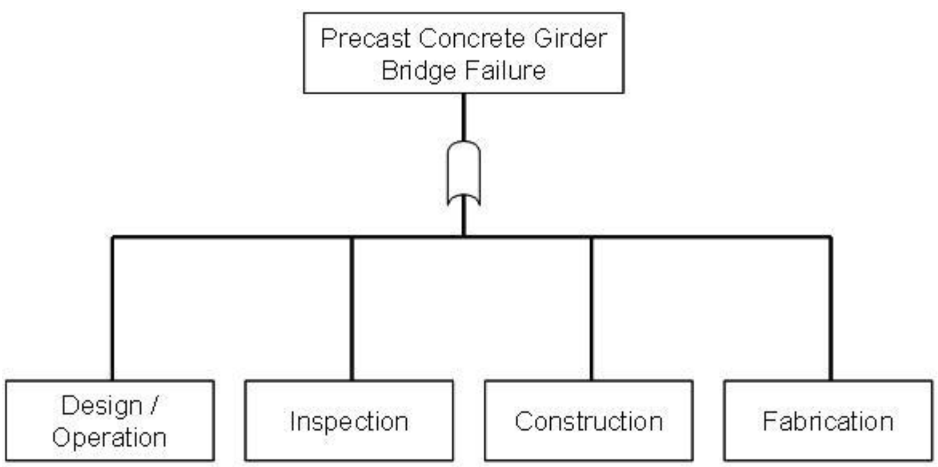


Figure 3-1 *Precast Concrete Girder Bridge fault tree showing the top categories only which include Design and/or Operation, Inspection, Construction, and Fabrication.*

The *Design/Operations* category alludes to that fact that a failure, either a collapse or critical defect, can occur while the bridge is in service. *Inspection* refers to the fact that there may be a problem with the routine inspection such that the design does not permit inspection of some of the bridge components. A failure can also occur during the *construction* of the concrete girder bridge, whether it is a collapse or a problem that results in delays (a failure of the overall process). The *fabrication* process is also subject to errors and problems, which could result in a failure of the overall bridge process. These are all categories that the bridge designer should be aware of, and give due consideration to, when designing a concrete girder superstructure. Each of the four categories is developed into a more detailed fault tree in this section.

Some of the events shown on the fault trees associated with a concrete girder bridge failure are the same as those shown previously for a steel girder bridge failure. The reader will be referred to previous discussions concerning failure events which are similar between the bridge types.

3.1 Design/Operation Category

The fault tree that follows the Design/Operation category is shown in Figure 3-2. While in service, a bridge failure can result from either a failure of the superstructure or substructure. A detailed representation of the superstructure fault tree is shown in Figure 3-2, while the substructure is shown in Figure 4-1.

A failure of the concrete girder superstructure can be caused by a failure in any one of the superstructure components, but mainly the girders, bearings, or concrete deck. Again, an Or Gate is used to join these fault scenarios, meaning that a failure of any one of these components may cause a failure of the superstructure. A failure of the superstructure will then trigger an operational failure of the bridge. For example, poor reinforcement and/or duct detailing, too much water in the concrete mixture, or corrosive constituents can cause an operational failure of a concrete girder bridge.

3.1.1 Superstructure – Girders

3.1.1.1 Corrosion / Inadequate Drainage Details

The failure of the girders can be caused by one or more events as shown in the fault tree, Figure 3-2. In addition to locating drainage scuppers and connecting elements based on design criteria, it is important to locate drainage elements in regions of the superstructure where they will cause the least damage if they become filled with debris. It is also important that the down-spouting be accessible for inspection and repair, and have the necessary slopes and connections to prevent clogging with debris. In addition, it may be advantageous to eliminate drainage details at support and/or expansion locations by making the concrete superstructure continuous and eliminating trough type drainage details when possible. Adequate drainage and the layout of the drainage elements will help to prevent corrosion of the reinforcing elements. Deterioration (corrosion) of the mild reinforcement or tensioned strands caused by the combination of water and deicing salts, can cause the failure of the main bridge girders. The corrosion of the mild reinforcement or tensioned strands will cause the girder concrete to crack and spall, furthering the possibility of a main girder failure.   This is depicted in Figure 3-2 in the lower left corner of the Figure.

Additionally, girders that are part of an overpass structure are prone to deicing salt-spray. Once deicing salts are applied on the roadway below, traffic passing through will cause the salt and water combination to become airborne, with some of the deicing salt/water solution splashing onto the bridge girders overhead. Salt-spray, and its effect on rate of corrosion, was one of the potential causes of the collapse of the exterior girder of the Lake View Bridge over I-70 in 2005 in Washington, Pennsylvania. Based on the pre-test inspections conducted, it appeared that the most exterior prestressing strand damage resulted from vehicle impacts damaging the concrete cover, followed by corrosion of the exposed strands [43]. A photo of the another beam in the structure, that did not collapse, showing the damaged girder and corroded strands is provided in Figure 3-3.

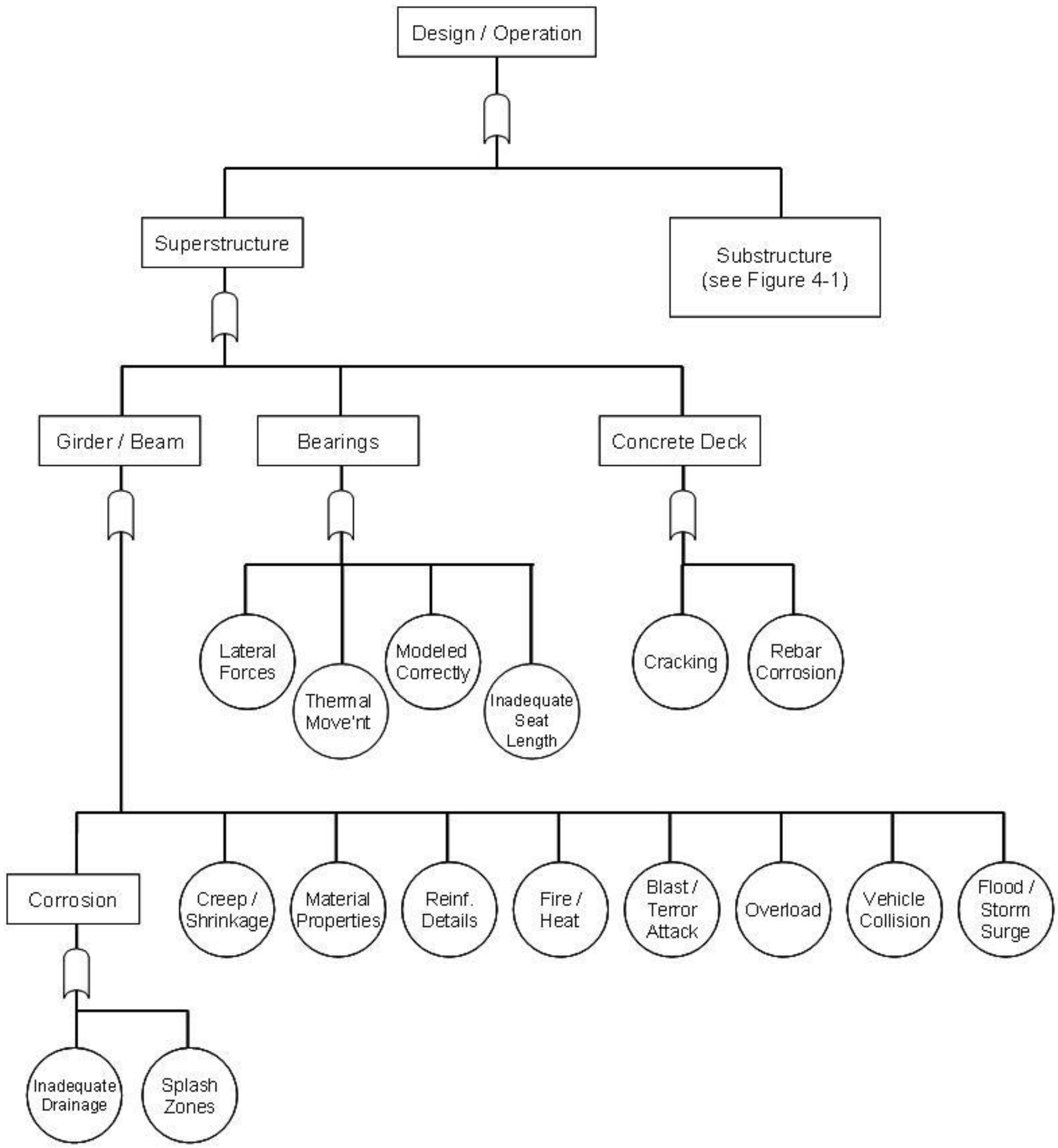


Figure 3-2 *Portion of the precast concrete girder bridge fault tree showing the Design and/or Operation category for superstructures only, with several Events and Basic Events provided.*

Woodward [44] reports on a segmental post-tensioned concrete bridge that collapsed in 1985 due to corrosion of the post-tensioning tendons. The investigation showed that corrosion of the tendons occurred where they passed through the segmental joints. Corrosion more than likely occurred because the tendons were inadequately protected at the concrete segment joints, allowing chlorides from deicing salts to penetrate the ducts.

Additional information regarding concrete bridges and corrosion can be found in *Concrete Bridges in Aggressive Environments* [45], and similar documents available through American Segmental Bridge Institute (ASBI) and the Portland Cement Association (PCA).

Figure 3-3 *Photo showing results of vehicle impact and subsequent corrosion damage to prestressing strands from a prestressed concrete box beam in Lake View Bridge over I-70 (taken from [43]).*

3.1.1.2 Creep and Shrinkage

Bridge designers must take into account the effects of creep and shrinkage during the design of concrete girder superstructures (and decks of steel girder superstructures). Creep and shrinkage can lead to cracks in the concrete, allowing water, salt, and other materials to penetrate the concrete. Once these materials penetrate the concrete, a corrosive environment can develop, and result in a failure of the concrete girder. Also, the effects of creep and shrinkage can increase deflections in beams and/or cause loss of prestressing forces in prestressed girders.

Creep is the increase in strain with time due to a sustained load. When concrete is initially loaded an instantaneous elastic strain develops. If this load remains on the member, creep strains will develop over time. Creep is affected by the ratio of the sustained stress to the strength of the concrete, the humidity of the environment, the dimensions of the concrete element, the composition of the concrete, and temperature. At high temperatures, such as in a fire, large creep strains can occur. Additionally, creep is most significant when the concrete has a high cement paste content. Concrete containing large aggregate fractions creep less because only the cement paste creeps and is restrained by the aggregate.

Shrinkage is the shortening of concrete during curing. The primary type of shrinkage, called drying shrinkage is the decrease in volume of concrete when it loses moisture, occurring after the concrete has attained its final set, and most of the chemical hydration process in the cement paste has occurred.

There are several factors that affect drying shrinkage:

- Aggregate content - the aggregates act to restrain the shrinkage of the cement paste, therefore concrete with high aggregate content tend to be less vulnerable to shrinkage.
- Water to cement ratio - the higher the water to cement ratio the more vulnerable the concrete element is to shrinkage.
- Humidity - shrinkage strains are largest for relative humidity of 40 percent or less.
- Temperature - shrinkage is less likely to occur at low temperatures.
- Admixtures – admixtures used to accelerate the hardening of the concrete tend to increase shrinkage effects. Air-entraining agents have little effect on shrinkage. The effect that an admixture has on shrinkage varies depending on the particular admixture.
- Surface Area / Volume – as the ratio of surface area to volume increases, the effects of shrinkage will increase.

More detailed discussions regarding creep and shrinkage and their effects can be found in concrete design textbooks, such as MacGregor [46] and Nawy [47].

3.1.1.3 Concrete Properties

The quality of the concrete, and its components, are particularly important to the behavior and endurance of a concrete superstructure. The bridge designer and/or owner must ensure that appropriate materials are specified and the contractor must provide components, including aggregate, water, admixtures, or grout, which meet specifications and do not adversely affect the concrete quality. There are several factors that affect the strength and behavior of concrete that bridge designers should be cognizant of, with the more important ones being:

- Water to cement ratio – the strength of concrete is significantly dependent upon the water to cement ratio. Typically, a water to cement ratio of 0.45 corresponds to a compressive strength of 5000 psi, while a ratio of 0.65 corresponds to a compressive strength of 3500 psi.
- Type of cement – different cements have a different rate of at which they gain strength.
- Cementitious materials – a portion of the cement is sometimes replaced with a supplementary material, such as fly ash or silica fume. These materials, referred to as pozzolans, possess little or no cement-like properties, but will react with calcium hydroxide to form cement-like compounds.
- Aggregate – the concrete strength is affected by the strength, texture, grading, and size of the aggregate.
- Other factors include moisture and temperature conditions during curing, age of the concrete, and the rate of loading.

3.1.1.4 Grout Properties and Grouting

In post-tensioned superstructures, the grout used will have a significant effect on the behavior and endurance of the bridge. The post-tensioning ducts and anchorages must be completely filled to provide a permanent protection for the post-tensioning steel and to develop a bond between the steel and the surrounding concrete. If voids remain due to accumulated bleed water, improper venting, or carbonated grout, corrosion of the post-tensioning strands can result. The quality of the grouting and the use of appropriate details for ducts and anchorages are critical for the performance of the structure.

The ASBI Grouting Committee [48] has reported on several structures in Florida that have experienced corroded post-tensioning tendons directly resulting from problems related to grouting techniques. In 1999, two corroded strands were found in a tendon anchorage at an expansion joint in a span of Niles Channel segmental bridge. It was concluded that initial corrosion resulted from the absence of grout due to accumulated bleed water that separated from the grout. This resulted in voids in the grout at the tendon anchorage, which then filled with water leaking through the concrete cover at the anchorages. In 2000, a failed external tendon and strands in an additional tendon were found in the Mid-Bay Bridge. Additional inspection resulted in additional tendons being replaced, mainly near expansion joints. It is believed that the nature of the grout mix, coupled with bleed water accumulation, and the filling of voids with water during construction may have contributed to the observed tendon corrosion. Also in 2000, inspections of the I-75/I-595 Sawgrass Interchange had found efflorescence at some tendon anchorages. In fact, during repair processes, it was found that some tendon ducts did not contain any grout at all.

Based on these findings, the ASBI Grouting Committee [48], summarized that most segmental concrete bridge grouting problems were related to: voids caused by accumulation of bleed water at tendon anchorages; salt contaminated water filling ungrouted tendon anchorages due to improper sealing of the ducts during construction; and substandard quality of grout installation and material. The bridge designer must be aware of these issues and consider how to avoid their potential effects during the design of post-tensioned concrete structures.

Additional information related to grouting materials, post-tensioning systems, and construction practices can be found in the FHWA publication Post-Tensioning Tendon Installation and Grouting Manual [49] and the American Segmental Bridge Institute's (ASBI) Construction Practices Handbook for Concrete Segmental and Cable Supported Bridges [50].

Grout is also used to fill cavities created by the removal of mandrels used for the harping of prestressing tendons in prestressed concrete girders. The bridge designer must verify that the grout specified is: approved by the governing agency and does not have an adverse affect on the performance of the superstructure. On May 20, 2000, at the Charlotte Motor Speedway in Charlotte, North Carolina, one span of a four-span, simply supported precast, pretensioned concrete pedestrian bridge collapsed, injuring 107 people [51]. Each span consisted of two precast, pretensioned double-T beams located side-by-side and connected longitudinally by welded shear connectors along the flange. A harped prestressing strand profile was employed, using a single hold-down at the midspan. Once the concrete hardened, the mandrel used to create the harped pattern was removed and the resulting cavity filled with grout. The forensic investigation indicated that severe corrosion of the prestressing strands occurred at the midspan of the double-T beams at the location of the mandrel hold down, where the cavity was filled with grout. The resulting loss in steel cross-sectional area reduced the strength of the superstructure. The aggressive corrosion of the strands was apparently caused by excessive additives (15 times that recommended) of a grout admixture containing chloride [51]. It should be further noted the grout admixture utilized by the contractor was not approved for use in precast concrete construction by NCDOT [51].

3.1.1.5 Reinforcement Details

The detailing and subsequent installation of reinforcement will have a significant effect on the behavior and long-term performance of a concrete superstructure. The bridge designer must develop reinforcement details that can be constructed as the design intends. If details are too cumbersome and complicated, it is possible that the detail will not be constructed as intended. Improper detailing may cause a weakness in the superstructure, provide a location for crack initiation, or create inadequate concrete cover, leaving the reinforcement susceptible to corrosion. Furthermore, details with significant reinforcement congestion can lead to poor flow and distribution of concrete when it is placed.

The collapse of the de la Concorde overpass shows that the bridge designer must fully consider the effects of reinforcement details in a concrete superstructure. An inquiry into the collapse of a portion of the overpass in Laval, Quebec, Canada, on September 30, 2006, found that improper detailing and installation of the steel reinforcement was a contributing factor to the bridge failure [52]. The simple single span of the bridge consisted of adjacent box girders, which rested on a beam seat. The beam seat was built into the thickened slab that cantilevered from the abutments at each end, as shown in Figure 3-4. The commission determined that in the as designed structure, the concentration of numerous reinforcement bars on the same plane in the upper part of the thickened cantilever slab created a plane of weakness where horizontal cracking could occur [52]. Top bars in the thickened slab were not sufficiently anchored. Additionally, the improper placement of several reinforcement bars during construction created a much larger zone of weakness extending into the thickened slab [52].

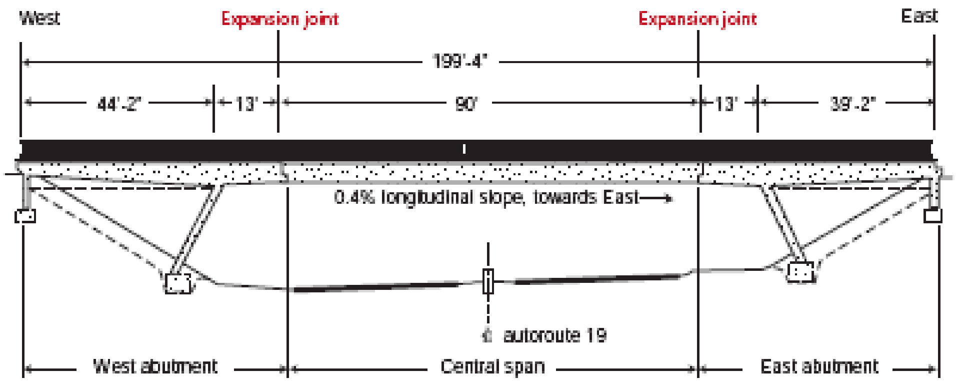


Figure 3-4 *Sketch showing the elevation of de la Concorde overpass; the simple central span of the bridge consisted of adjacent box girders, which rested on a beam seat that was built into the thickened slab that cantilevered from the abutments at each end (taken from [52]).*

Similar to reinforcement details, the details associated with prestressing and post-tensioning tendons will have a significant effect on the durability of a concrete superstructure. As discussed previously, the ASBI Grouting Committee (2000) reported on the corroded post-tensioning tendons in Florida. Some of the failures reported were due to grouting techniques as well as inadequate detailing of the ducts, especially at the anchorages. In the case of the Niles Channel Bridge, it was noted, at some locations, that water leaked through the concrete cover at the anchorages, resulting in corrosion of the post-tensioned tendons at this location. The ASBI Grouting Committee (2000) highlighted duct detailing issues such as:

- Providing effective sealing of tendons from water ingress in the interval between stressing and grouting.
- Providing anchorage details that are protected from water ingress, and showing such details on the design plans.
- Providing adequate ducts for internal tendons of concrete structures located in salt water environments or exposed to de-icing solutions.

Adequate details for the post-tensioning ducts and anchorages must be provided by the bridge designer. Particular attention must be given to anchorages located at expansion joints, which are susceptible to water ingress. Consideration of these issues at the design stage will potentially eliminate failures associated with the failure of post-tensioning tendons. Additional information regarding reinforcement details, including detailing of post-tensioning tendons and anchorages can be found in the Florida Department of Transportation's *Structure Manual,* Article 4.5 [53].

3.1.1.6 Fire / Extreme Heat

A fire or extreme heat event will cause high thermal gradients in a concrete girder, and as a result the surface layers of the concrete girder could expand and eventually spall off the cooler, interior portion of the girder. During an extreme heat event, when temperatures approach 800°F to 1200°F, there can be a significant loss of strength in a concrete girder. The temperature, at which a concrete beam will fail, is mainly dependent upon the type of aggregate used. Concretes made with carbonate aggregates, such as limestone of dolomite, are relatively unaffected by temperature until they reach 1200°F to 1300°F, at which time they rapidly lose strength. Aggregates such as quartzite, granite, and sandstones undergo plastic change at about 800°F to 1000°F, which causes a sudden change in volume and spalling of the concrete. Lightweight aggregates gradually lose their strength at temperatures above 1200°F [46].

Several fires on or below precast concrete girder bridges have occurred in recent years, in which the fire did not cause a collapse of the structure, but did necessitate some repairs and/or replacement. Several of these events are reported on in articles by Shutt [54] and Stoddard [55]:

- On June 20, 2007 near Nashville, Tennessee, a fuel tanker rear-ended a loaded dump truck resulting in a fire below a two-span, two-celled, hollow box-beam bridge. Analysis showed the bridge endured much heat but sustained little damage [54].
- On July 28, 2006 in Parker, Arizona, a fuel tanker crashed on a multi-span bridge that consisted of AASHTO Type III precast, prestressed girders. Fuel from the tanker spilled onto the bridge, beneath the structure, and through deck drains and expansion joints. The fire-damaged girders did not show visible signs of loss of prestress, but experienced various degrees of spalling. The repair plan included restoring girders where the reinforcement was exposed, and adding protective coating to deck to mitigate corrosion [54].
- In December 2002, a railroad tanker collision caused a fire under a prestressed concrete girder bridge in Tacoma, Washington. Stoddard [55] describes the inspection, testing, and analysis that occurred after the fire. All of the girders in the affected span had damage to the corners of the bottom flanges that could easily be removed to expose the outermost strands. However there was no noticeable loss of prestress.

On July 12, 2005, a significant fire incident occurred near Ridgefield, Connecticut which caused significant damage to an adjacent box-beam bridge. The Connecticut DOT coordinated with the FHWA’s Turner Fairbank Research Center to investigate the flexural capacity of the beams removed from the bridge after the fire. The results of the investigation showed that the beams still had sufficient flexural capacity but the long-term viability of the beams was questionable. The visual and petrographic examinations showed that the damage to the bottom flange concrete was sufficient to allow pathways through the concrete to the depth of the bottom prestressing stands [56]. This could potentially lead to the accelerated deterioration of the bottom row of strands due to water ingress and subsequent corrosion. A photo of the bottom face of one of the tested beams is provided in Figure 3-5.

Figure 3-5 *Photo of the bottom face of a tested beam subjected to fire while in service showing that the concrete has deteriorated such that sufficient pathways through the concrete to the bottom layer of prestressing strands have developed (taken from [56]).*

Bridge designers need to be aware of the fact that a fire can occur below a concrete girder superstructure. Depending on the importance of the structure, it may be necessary to investigate the bridge behavior due to an extreme heat event to ensure that a collapse does not happen. A design could be developed that would allow some delay before a collapse would occur, which would allow human lives to be saved.

3.1.1.7 Blast / Terrorist Attack

See section 2.1.1.6 of the Steel Girder Failure Framework.

3.1.1.8 Overload

See section 2.1.1.7 of the Steel Girder Failure Framework.

3.1.1.9 Vehicle and Vessel Collisions

See section 2.1.1.8 of the Steel Girder Failure Framework.

3.1.1.10 Flood/Storm Surge

A bridge with a small vertical clearance over a waterway could be vulnerable to damage from a debris flow and storm surge in a flood. If the vertical clearance is small, it is possible that the girders of the structure will cause flood debris to be stopped at the bridge. This debris stoppage and water flow could lead to additional lateral loads on the girders that were unanticipated during design. Wardhana and Hadipriono [1] found that 16 cases of bridge failure (any type of bridge) derived from debris flows in the same year, 1995, resulting from flash flooding in Madison County, Virginia.

In August 2005, several concrete girder bridges were damaged or completely destroyed in Alabama, Louisiana, and Mississippi during Hurricane Katrina. Padget et al. [57], used data from 44 damaged bridges to develop relationships between storm surge elevation, damage level, and repair costs. The authors point out that several traditional fixed spans displaced due to a combination of buoyant forces and pounding by waves. The US-90 Biloxi-Ocean Springs Bridge suffered severe damage due to a combination of storm surge and wind/wave induced loading [57]. The four-lane, 1.9 mile, multi-span concrete girder bridge had low-lying spans. The storm surge caused severe damage to the bearings, and most connections between the deck and the pier caps were destroyed, allowing free movement of the spans. In fact, several spans on the western half of the bridge became completely unseated and were submerged in the bay, as shown in Figure 3-6. In traditional hurricane prone areas, the designer should take measures to reduce the likelihood of failure caused by storm surge: the bridge could be designed to a higher elevation, provide details such as transverse shear keys to prevent lateral movement, or tie downs to prevent upward movement.

Figure 3-6 *Photo showing the spans of the US-90 bridge completely unseated and submerged in the water due to storm surge induced loading (taken from [57]).*

3.1.1.11 Seismic

See section 2.1.1.9 of the Steel Girder Failure Framework.

3.1.2 Superstructure - Bearings

See section 2.1.2 of the Steel Girder Failure Framework.

3.1.3 Superstructure - Concrete Deck/Railings

See section 2.1.3 of the Steel Girder Failure Framework.

3.2 Inspection Category

The fault tree that follows the Inspection category is shown in Figure 3-7. Similar to a steel girder bridge, the lack of inspection or inadequate inspection can lead to a failure of a concrete girder bridge. Obviously, for a concrete bridge, it is difficult to inspect the steel reinforcement, prestressing and/or post-tensioning steel. However, when possible, details should be incorporated into the design by the bridge designer to ensure that the bridge can be inspected by normal methods. Details that are difficult to inspect may lead to a condition in which reinforcement corrosion or concrete cracks go undetected. During the design the bridge designer must consider whether certain details need to be inspected. Some details, such as prestressing strands and internal post-tensioning tendons, can not be inspected by typical means and methods. If future evaluation is needed, provisions like externally accessible strain gauges, or corrosion potentiometers, must be built in. External post-tension ducts are more inspectable than internal ducts.

Some concrete girder bridges are much more difficult to inspect than others. For example, adjacent prestressed concrete box girder bridges tend to be difficult to inspect. There is not a practical manner in which to asses the condition of shear keys and grouting between the adjacent box girders. The shear keys and grouting can deteriorate due to the use of deicing salt solutions. If the bridge is being designed for an environment in which deicing salts are used, it may be necessary to consider the loss of the shear keys for load rating purposes.

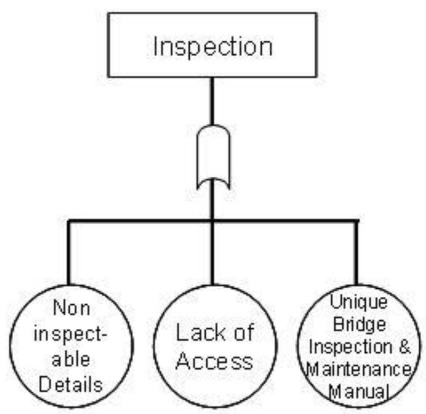


Figure 3-7 *Portion of the precast concrete girder bridge fault tree showing the Inspection Category only, with three Basic Events provided.*

The application of architectural treatments to a concrete girder may make it difficult to inspect a suspect area. The presence of architectural treatments will impair sounding techniques (hammer tapping, for example) often used to determine if the concrete, mild reinforcement, and/or prestressing strands are deteriorated.

Access holes must be provided in larger concrete box girder bridges, which allow bridge inspectors access into the box itself. In a continuous box girder bridge (segmental), access holes must also be provided in the internal full depth diaphragms at the supports, which allow bridge inspectors to pass through from span to span. Also, access is needed to all cells in multi-cell structures. As an example, the Florida Department of Transportation's Structures Manual, Article 4.6.1 [53] provides details regarding access in concrete box girders. FDOT recommends providing access doors at intervals of no more than 300 feet; access openings should be no less than 32 inches wide by 42 inches tall; and access entrances should not be placed over traffic lanes or locations that could endanger bridge maintenance personnel or the traveling public.

3.3 Construction Category

The fault tree that follows the Construction category is shown in Figure 3-8. Similar to the steel girder bridge framework, this part of the overall fault tree may not necessarily apply to the bridge designer, but may be more applicable to a specialty engineer working for a contractor. However, it is necessary that the bridge designer is aware of the potential construction issues when developing a new bridge design in order to ensure that the bridge is constructible.

Many of the basic events shown in the Construction category of the fault tree in Figure 3-8 are similar to the basic events for steel girder bridges. A few items will be highlighted in this portion of the framework for concrete girder bridges, however the reader is directed to section 2.3 for basic events not discussed.

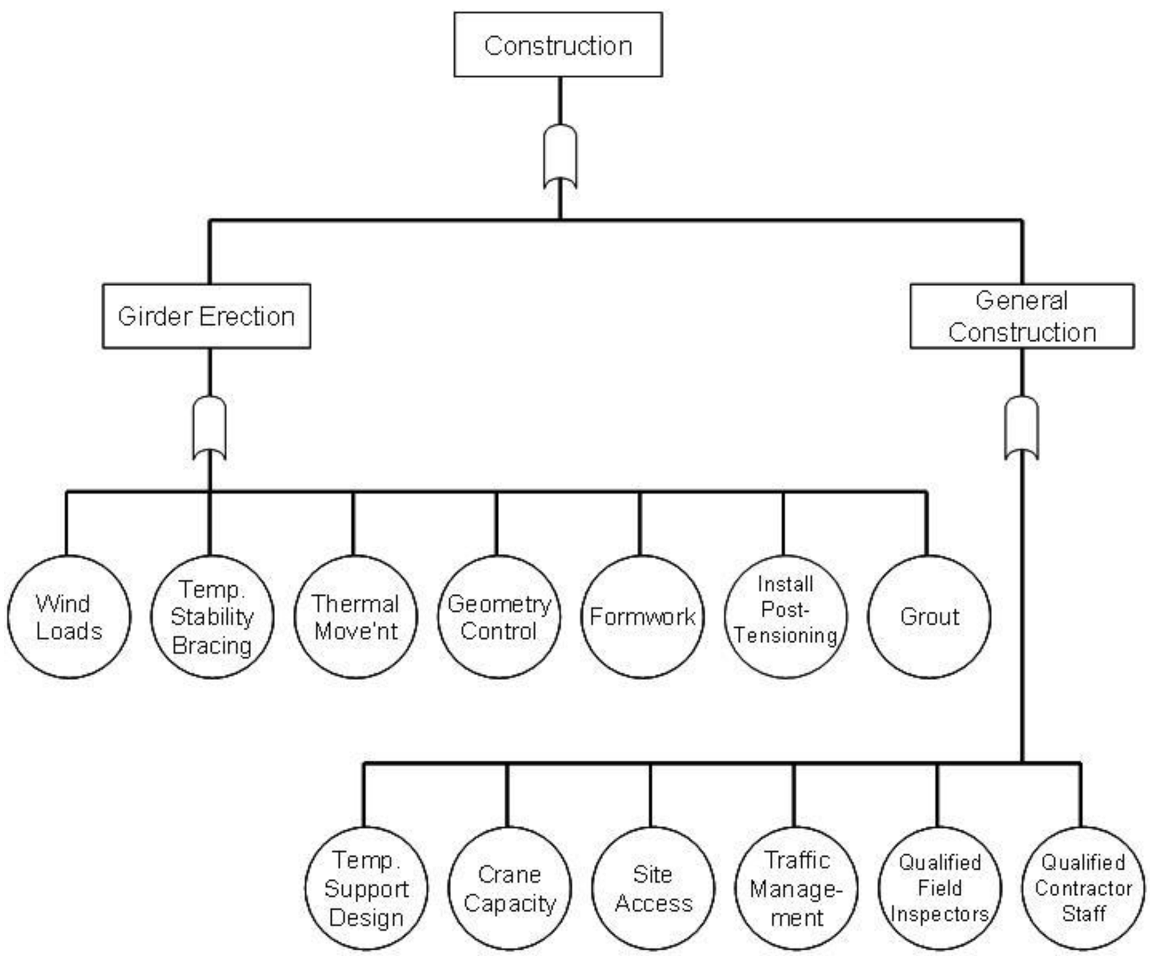


Figure 3-8 *Portion of the precast concrete girder bridge fault tree showing the Construction Category only, with several Events and Basic Events provided.*

3.3.1 Girder Erection

The lack of proper temporary bracing during girder erection can lead to a failure of a concrete bridge. Tremblay and Mitchell [58] report on a concrete bridge which utilized precast pretensioned AASHTO Type V girders that had two spans collapse during construction. On June 18, 2000, the four-span Souvenir Boulevard Bridge in Laval, Quebec, had four girders slide off their pot bearings in the two interior spans. The girders had been erected and formwork placement nearly completed for diaphragms and the deck slab, but the deck was not to be placed for a few more days. Guided and non-guided pot bearings were used at several locations throughout the structure. The authors concluded that the girders were not adequately braced during construction for the pot bearings used [58]. The use of sliding pot bearings coupled with insufficient temporary bracing resulted in the girders being in an unstable state of equilibrium, where collapse could be triggered by a very small load or disturbance [58].

On the August 9, 2007, nine of the eleven precast, prestressed concrete girders in a span fell from the piers. The investigation established that the incident could have been prevented by installing adequate temporary lateral bracing to stabilize the concrete girders, individually and collectively, until permanent reinforced-concrete diaphragms and the deck were completed [59]. Furthermore, the investigation pointed out that inadequately braced concrete girders set on elastomeric bearing pads can become unstable over time [59].

In concrete girder bridges, especially those that are cast-in-place, the formwork design must be considered by the specialty engineer. Improper use of the formwork, or poor design, could lead to a failure during bridge construction. ACI 347-04: *Guide to Formwork for Concrete* [60] provides the engineer with guidance for the design and specification of formwork for concrete structures. Additional information regarding concrete formwork design can be found in Hurd [61]. The reference provides the engineer with formwork material properties, design data, construction suggestions, and has guidance relating to structural design details for formwork.

3.3.2 General Construction

The design of temporary support structures must be fully considered by the bridge designer and/or specialty engineer for a concrete girder project, as appropriate fir a given project. The use of improperly placed and/or designed temporary supports will lead to a failure during construction, as shown in Figure 3-4. Hart et al. [62] presented the results of an investigation of the collapse of a temporary support for a cast-in-place, post-tensioned concrete box girder bridge in California. The authors concluded that the temporary support collapsed because it was constructed with an excessive initial out-of-plumbness, resulting in an inability of resisting the gravity loads above.

3.4 Fabrication Process Category

The fault tree that follows the Fabrication category is shown in Figure 3-9. This part of the overall fault tree will apply to the bridge designer, construction inspector, as well as the specialty engineer working for a contractor or erector. As shown in Figure 3-9, there are several basic events shown that can trigger a failure in a bridge project related to the fabrication of the concrete superstructure.

Some of the basic events shown in the Fabrication portion of the fault tree in Figure 3-5 are similar to the basic events for steel girder bridges. A few items will be highlighted in this portion of the framework for concrete girder bridges, however the reader is directed to section 2.4 for basic events not discussed.

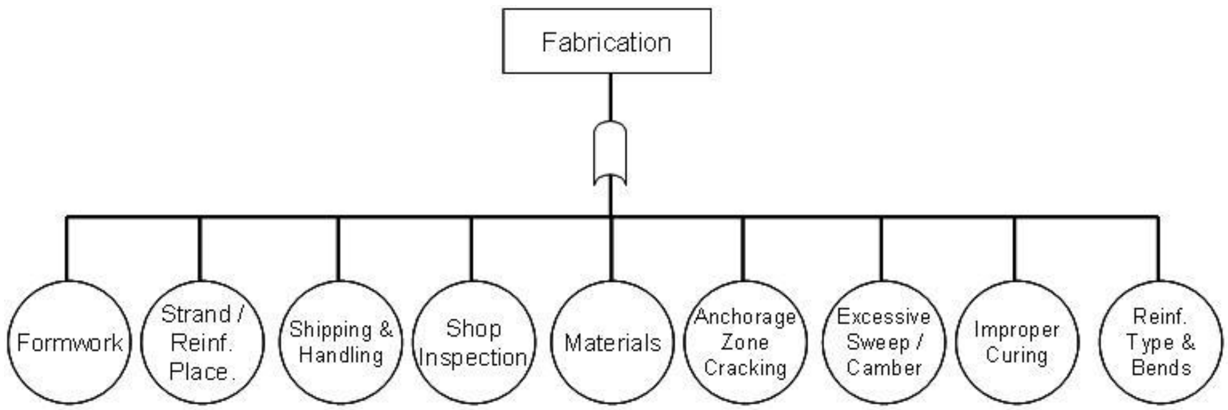


Figure 3-9 *Portion of the precast concrete girder bridge fault tree showing the Fabrication Category only, with several Basic Events provided.*

In some cases, it has been noted that formwork will tend to float or bow during precast concrete girder fabrication, especially for box girders. This float or bow will tend to cause the webs on one side of the box to be slightly wider than the other side. Although this float or bowing will not affect the flexural behavior of the girder itself, it could lead to inadequate concrete cover. It was noted in the report regarding the Lakeview Bridge collapse that in some locations, the webs of the concrete box differed in thickness by one inch [43].

The correct placement of prestressing/post-tensioning strands and/or mild reinforcement must be ensured during girder fabrication. Although a difference in the exact location of the reinforcement may not affect the girder's capacity, it could results in a smaller concrete cover than given by the design plans. For the precast prestressed concrete box-girder tested for the investigation of the Lakeview Bridge collapse, it was noted that the bottom strand was only ¾" from the edge of the beam, where the prescribed value per design was more than 1 ½" [43]. A condition such as this could lead to more significant spalling due to the reduced concrete cover.

Similar to steel girder bridges, the shipping and handling of concrete girder field pieces must be considered by the bridge designer during the design process. Permits may be required. Longer girder field pieces may be too heavy to lift with typical erection cranes and transported by typical methods. If longer girder field pieces are used in the design, the bridge designer should contact a transporter to verify that the necessary equipment and permit capabilities are available.

4.0 SUBSTRUCTURE FAILURE FRAMEWORK

A general fault tree for the case of a bridge substructure failure is developed and presented in this section. As discussed earlier, failure in this framework refers to a total collapse of the bridge system or an event that renders the substructure with a critical defect. Similar to previous frameworks, this substructure framework is developed for a general case, as shown in Figure 4-1.

As shown in Figure 4-1, several causes of substructure failure are related to extreme events, such as scour due to flooding, vessel collisions, and earthquake loading. These topics will be discussed in this section. However, the reader is referred to *NCHRP Report 489: Design of Highway Bridges for Extreme Events* [63] additional information regarding extreme event loading combinations.

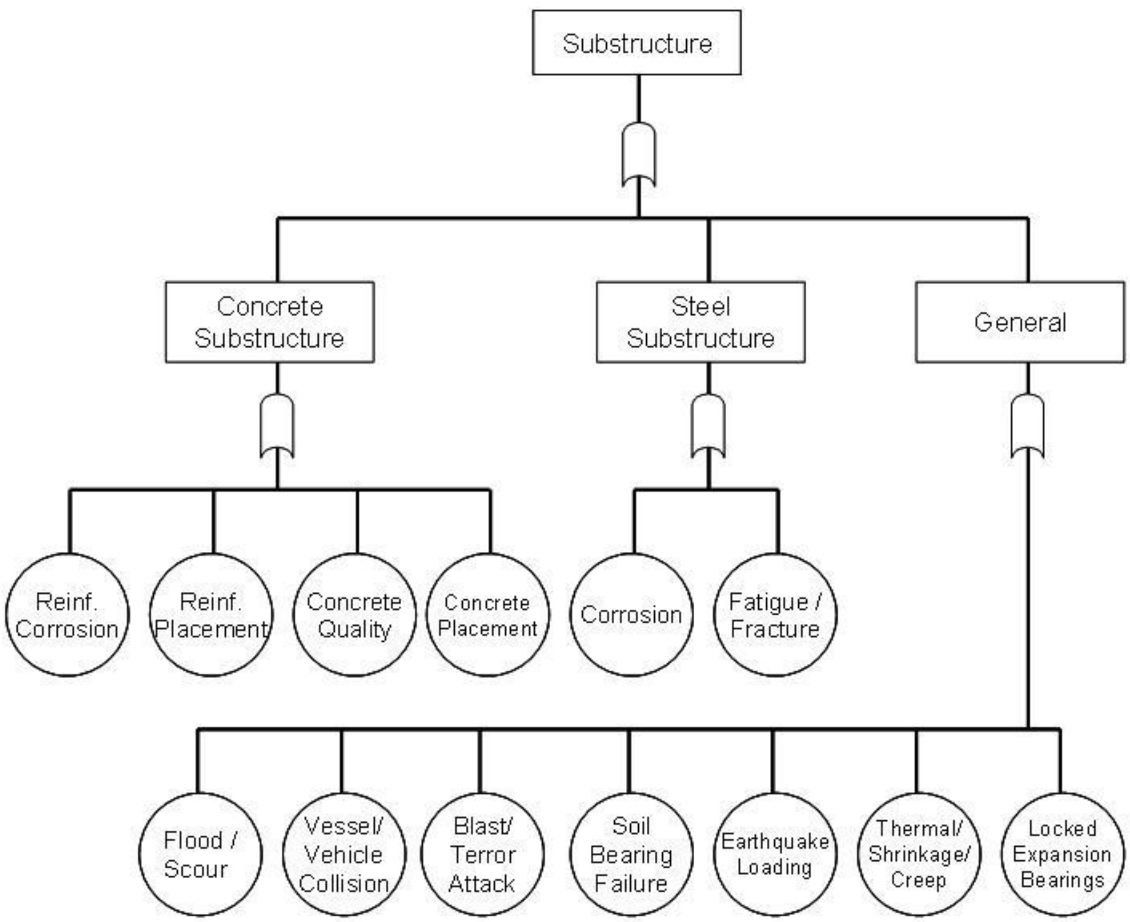


Figure 4-1 *General substructure fault tree with main categories of Concrete Substructures, Steel Substructures and General Substructures, and several basic events shown for each.*

4.1 General

4.1.1.1 Scour and Flood

Scour is defined as the erosion of streambed or bank material from bridge foundations due to flowing water, as illustrated in Figure 4-2. A majority of bridge failures in the United States and elsewhere are the result of scour [2]. Wardhana and Hadipriono [1] found that 243 bridge failures, out of a total of 503 recorded failures, were related to flooding and scour. The AASHTO *LRFD Bridge Design Specifications* [2] require that scour at bridge foundations be designed for 100-year flood event or from an overtopping flood of lesser recurrence interval. Additionally, the bridge foundations are to be checked for stability for the 500-year flood event or from an overtopping flood of lesser recurrence interval.

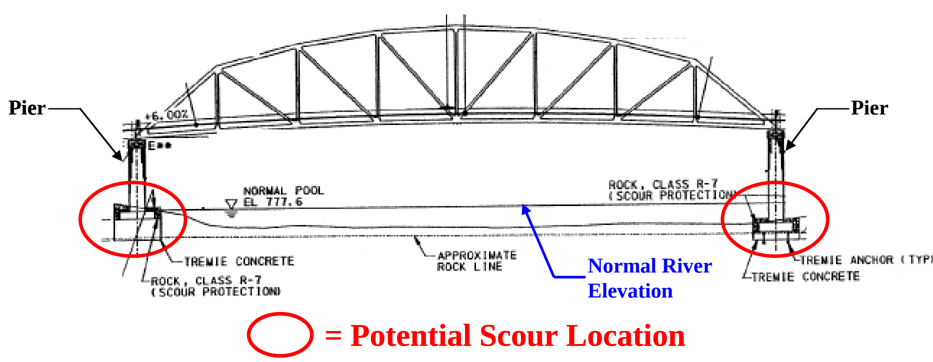


Figure 4-2 *Sketch showing potential scour locations for a general bridge crossing over a river in which the concrete supports are located within the limits of the waterway.*

FHWA *Hydraulic Engineering Circular No. 18: Evaluating Scour at Bridges (HEC-18)* [64] provides guidelines for designing bridges to resist scour, evaluating existing bridges for scour vulnerability, inspecting bridges for scour, and improving the estimation scour at bridges. Additionally, the HEC-18 document divides the total scour at a bridge foundation into three components:

- *Long-term aggradation and degradation:* elevation changes in the streambed of the waterway caused by erosion and deposition of materials.
- *Contraction scour:* resulting from the removal of material from the channel bed and/or banks caused by encroachment of the embankments into the waterway.
- *Local scour:* removal of material from around the area of the bridge foundation caused by acceleration of water flow associated with a flood or high-water event.

An example of a foundation scour failure related to severe flooding is the collapse of the Schoharie Creek Bridge in 1987. The Schoharie Creek Bridge carried the New York State Thruway over Schoharie Creek, near Amsterdam, New York. The bridge consisted of five simply supported steel spans, supported on two abutments and four concrete piers. Each concrete pier consisted of two columns connected at the top with a pier cap beam, and at the bottom the columns framed into a concrete plinth on top of a shallow spread footing. Pier 3 had failed due to scour deterioration mechanism: the riprap protection around the footing was inadequate against the stream bed erosion, and the plinth and footing fractured in tension causing the pier to collapse [7]. Additionally, the shallow foundations utilized in the pier design more than likely contributed to the scour condition.

The Schoharie Creek Bridge collapse illustrates the importance designing bridges to resist scour as well as the need to properly inspect and maintain bridges vulnerable to scour. Pier and abutment foundations must be designed taking into account the design scour depth. Scour design depth can be determined in accordance with *HEC-18* [64] and the AASHTO *LRFD Bridge Design Specifications* [2]. Placing spread footings below the scour design depth or extending piles or drilled shafts below the scour design depth can help to reduce the potential for scour. For additional information regarding the prevention of scour, the reader is referred to *HEC-18* [64], *NCHRP Report 587: Countermeasures to Protect Bridge Abutments from Scour* [65], and *NCHRP Report 593: Countermeasures to Protect Bridge Piers from Scour* [66].

In addition to scour, bridge designers must consider lateral loads caused by significant flooding events, as required by the AASHTO *LRFD Bridge Design Specifications* [2]. Weber et al. [67] report on a collapse of a temporary bridge during a flooding event. On May 26, 1989, a steel pile bent supporting two spans of a one-lane temporary ridge over the Great Miami River in Hamilton County, Ohio collapsed into the river, killing two people. Floating debris in the flooded river had struck the pile bent, leading to the collapse. The authors concluded that the primary cause of failure of the pile bent was the lack of structural capacity for lateral loadings [67]. The steel pile bent had been proportioned for vertical loads, but not necessarily for lateral loads, since some of the lateral load cases investigated resulted in a factor of safety less than 1.0. The bracing between the H-piles were mainly positioned to resist in-plane buckling of the H-piles and not necessarily to resist lateral loads resulting from flooding. The collapse of this bridge shows that bridge designers must give careful consideration to extreme lateral loads that can be placed on substructure units, for permanent and temporary bridges.

4.1.1.2 Vessel and Vehicle Collision

Bridge designers must consider the lateral loads imparted to bridge piers by ships and/or barges when the structure being designed crosses a navigable waterway. Similarly, for an overpass structure, lateral loads resulting from vehicle or train collisions must be considered when bridge piers are located near traffic lanes or a railroad below. Collisions with bridge piers by barges/ships, trains, or vehicles that cause a bridge to fail can not only result in the loss of human life, but will damage the transportation system and economy, especially for failures occurring on major thoroughfares. Wardhana and Hadipriono [1] note that 59 bridge failures, or 12 percent of the total number of bridge failures studied, resulted from land and marine vehicle collisions.

There have been several reported incidents of piers weakened or destroyed by vessel or vehicle collisions, subsequently causing a bridge failure. On May 9, 1980, during a significant storm, an oceanic freighter collided with a pier, causing more than 1200 feet of the southbound steel truss structure of the Sunshine Skyway Bridge to collapse into the Tampa Bay. The vessel collision resulted in the death of 35 people [68]. The new cable-stayed bridge utilized a main span that was 50 percent wider than the original main span that served as the shipping lane. The main span piers and some of the approach piers are surrounded by a dolphin protection system, which are circular cells of sheet piling filled with rock and/or concrete.

On May 28, 1993, a towboat pushing an empty barge collided with a support pier of the eastern span of the Highway Route 39 Judge William Seeber Bridge in New Orleans. The impact severed a river pier, causing two approach spans and a two-column bent to collapse onto the barge and waterway [69]. Two automobiles fell with the bridge, resulting in one fatality and two serious injuries. The canal was closed to navigational traffic for 2 days, and the bridge was closed to vehicular traffic for 2 months.

On May 26, 2002, a towboat with two barges collided with a pier of the I-40 Bridge in Webbers Falls, Oklahoma, killing 14 people and injuring several others [70]. The traffic in both directions of the major east-west national corridor was abruptly stopped. Reinforced concrete piers utilized spread footings and pier protection was constructed only on the upstream side of the main span, and one at each of the main span piers only. The pier that was struck by the barges was an unprotected approach span pier.

El-Tawil et al. [71] report on three events that led to the loss of human life and failure of an overpass bridge due to a vehicle collision.

- In 1993 a tractor with a bulk-cement-tank semi-trailer traveling on I-65 collided with a supporting column of County Road 22 in Evergreen, Alabama. The pier collision resulted in two spans of the overpass collapsing onto I-65, killing two drivers who collided with the collapsed bridge.
- In 2002, a tractor trailer traveling on I-45 in Dallas, Texas hit a concrete pier column for the Highway 14 overpass. The overpass collapsed, killing one individual.
- In 2003, a semi-trailer collided into a median support on a bridge crossing over I-80 near Big Springs, Nebraska. The collision caused the overpass to collapse, killing one person and severely disrupting traffic on I-80.

El-Tawil et al. [71] address the issue of vehicular collisions with bridge piers, and present results of detailed finite element analyses of various collision scenarios.

Computer analyses may be warranted for complex and/or significant structures that could be subject to collision loads. The bridge designer and/or owner must also consider the use of protective devices. Fender type systems consisting of rubber, steel, or concrete, or dolphin type protection systems can reduce collision force effects in the pier by absorbing the impact energy. Protective islands can be effective in vessel collision protection. Crash walls can be designed to resist vehicle and train loads for overpass structures. Furthermore, a framed structure such as a concrete box girder bridge with an integral pier cap is more likely to distribute a collision load throughout the structure than a girder-slab structure.

Article 3.14 of AASHTO *LRFD Bridge Design Specification* [2] addresses design guidelines for vessel collisions. The requirements in the AASHTO *LRFD Bridge Design Specifications* are adapted from the AASHTO *Guide Specification and Commentary for Vessel Collision Design of Highway Bridges* [72], using the Method II risk acceptance alternative. Both of these specifications provide designers with guidelines related to the risk of ship and barge collisions and the design of pier protection systems. Additionally, the AASHTO [2] and AREMA [73] Specifications provide guidance to bridge designers regarding required clearance envelopes.

4.1.1.3 Blast / Terrorist Attack

Blast loads resulting from explosions and/or terrorist attacks that occur below the bridge deck may impart large lateral forces on the substructure units depending on the proximity to piers or columns. The forces may cause large displacements, and shear or flexural failures, resulting in a failure of the bridge system. The designer, owner, and/or security personnel should perform a risk assessment to determine the threats that a particular bridge could be vulnerable to. Depending on the vulnerability, importance, and the risk of terrorist activity associated with the structure being designed, the designer may need to consider the effects of blast loads on the superstructure. The effects of blast loading can typically be investigated through computer simulations and the use of finite element analysis procedures. Additional information regarding blast loads and terrorist attacks on bridges can be found in Section 2.1 and 3.1 of this document.

4.1.1.4 Seismic

An earthquake will cause vertical, lateral, and/or rotational movements of the substructure that if excessive or unaccounted for, can result in collapse or partial collapse. As appropriate for the seismic zone, the bridge designer must consider these loadings and effects in the design of the bridge system. In addition, the designer must also provide proper for: steel connections, concrete column confinement, bearing restraints, shear keys, etc.

Some typical items, not an all-inclusive list, which bridge designers should consider with regard to substructure and seismic design are as follows:

- Provide adequate concrete column confinement
- Avoid diaphragm abutments that force failure underground, in which earth moving equipment may be required for a post earthquake inspection
- Provide plastic hinges in areas that may not directly lead to a collapse of the structure
- The use of approach slabs in case of settling of the substructure.

Additional information and references regarding seismic design in highway bridges is provided in section 2.1.1.9 of this document.

4.1.1.5 Other Substructure Failure Events

In addition to the above events, there are other failure events a bridge designer must consider in the design of substructure units, as shown in Figure 4-1. A soil-bearing failure that results in vertical and/or rotational movement of the substructure unit can cause a bridge to fail or develop a critical defect. If settlement or movement is a known possibility, the bridge designer should consider including the force effects this movement will have on the bridge system with all service and strength limit states.

4.2 Concrete Substructure

4.2.1.1 Reinforcement Corrosion

Bridge substructure typically consists of reinforced concrete columns and pier caps, concrete footings, steel piling, concrete shafts, or post-tensioned elements. Deterioration (corrosion) of the reinforcement caused by the combination of water and deicing salts can cause a failure of the bridge substructure. The corrosion of the reinforcement will cause the concrete to crack and spall, furthering the possibility of a failure.

Corrosion of the reinforcement can result from inadequate drainage of the superstructure. For example, inappropriately detailed drainage components can often become clogged and not allow the removal of water and deicing salts from the superstructure. The water and deicing salts can then escape the drainage piping, and flow along a portion of the substructure, creating a corrosive environment.

Substructure units that are part of an overpass structure are prone to deicing salt-spray because the substructure units are located in what is commonly referred to as a splash-zone. Once deicing salts are applied on the roadway being crossed by the superstructure, traffic passing through will cause the salt and water combination to become airborne, with some of the deicing salt/water solution splashing onto the nearby substructure. This solution will help to create an environment suitable for reinforcement corrosion.

Additionally, substructure units located in a coastal environment can be subject to ocean salt spray. Depending on the aggressiveness of the oceanic environment, the bridge designer may need to specify a type of corrosion inhibitor to be used in the substructure units. Adequate drainage elements and the consideration of deicing salt spray, or oceanic salt spray, will help to prevent corrosion of the concrete reinforcement, and subsequent deterioration of the concrete.

4.2.1.2 Thermal, Creep, Shrinkage

In concrete substructure units, forces related to temperature changes, creep, and shrinkage could have a significant effect on the behavior of substructure and the entire bridge system. For example, short multi-column bents where the columns are tied together at the top with a cap beam can have significant component force effects resulting from thermal, creep, and shrinkage loadings. If these force effects are not considered during design, the substructure can collapse or develop a critical defect. The bridge designer must consider the effects of temperature change, creep, and shrinkage in the design of reinforcement concrete substructure units, especially when the units become large.

4.3 Steel Substructure

Similar to concrete substructure units, steel substructure units are susceptible to corrosion caused by salt spray from deicing salts or a coastal environment. The bridge designer must be aware of this possibility given the location of the structure, as specific the proper corrosion protection.

Furthermore, steel substructure units may need to be investigated for fatigue and fracture. Given that steel substructure units are often non-redundant, a box-girder cross beam for example, members are typically designated on the design plans as fracture critical elements, and must be tested as such. Further information regarding fatigue and fracture in steel elements can be found in section 2.1.1.2 of this document.

5.0 COMPLEX BRIDGE FAILURES – LESSONS LEARNED

For complex bridges, such as trusses, arches, suspension, and cable-stayed, a fault tree will not be presented. Many of the potential failure mechanisms previously discussed in sections 2, 3, and 4 apply to these bridge types. For example, failure events such as scour; corrosion; fatigue and fracture; earthquake, blast and terrorist activity; detailing and construction issues; and vessel collisions should be investigated for complex bridges when applicable. Instead of repeating these events and others, a "lessons learned" approach is employed for these complex bridges. Specific failures and the lessons learned from those failures will be presented and discussed. It is anticipated that the discussion of these failures, and the lessons learned, will aid in the prevention of similar failures in the future.

5.1 Steel Truss Bridges

5.1.1 Quebec Bridge, 1907

One of the more well known failures of a steel truss bridge is the collapse of the Quebec Bridge on August 29, 1907, during construction, killing 75 workers. The Quebec Bridge was a three-span cantilever truss, with an 1,800 ft center span. Pearson and Delatte [74] provide a detailed account of the Quebec Bridge failure, including events that led up to the failure, highlights of the commission's report, causes of the failure, and ethical aspects. A distinguished panel was assembled to investigate the collapse of the Quebec Bridge. They found that the main cause of the bridge's failure was the improper design of the latticing of the compression chords. The allowable stresses used to design the members were considered unusually high for the time period. It was determined that the compression members did not have adequate stiffness to resist buckling. Additionally, member stresses were not recalculated and checked once the center span length was increased by 200 ft during the design phase, resulting in several overstressed members.

Pearson and Delatte [74] also highlight the failure of the engineer to recognize that a serious problem was developing as the structure was being erected. As the bridge was erected, ironworkers noticed significant midpoint displacements in some of the truss compression members. The lead engineers believed these deflections were small and not problematic, and were the result of some unknown preexisting condition, or hit by another component during erection. However, these noticeable displacements were the warning signs that something was wrong, and in this case, the start of compression member buckling.

As a result of the Quebec Bridge collapse, research regarding column buckling was initiated. The collapse of the bridge demonstrates the need to perform thorough calculations, especially when there is a change in bridge geometry and/or size. Furthermore, the collapse demonstrates the importance of communication during erection. If problems are noticed in the field by workers, supervisors, or inspectors, the problems need to be investigated immediately by the engineers. A successful bridge project, from design to final construction, requires the entire team to be working together and communicating.

5.1.2 Yadkin River Bridge, 1975

On February 23, 1975, the steel through-truss span of the Yadkin River Bridge collapsed in Siloam, North Carolina. One vehicle was on the bridge when it collapsed, and six additional vehicles drove off the approach spans after the main span truss had collapsed. The collapsed resulted in four deaths, and 16 injuries [75]. The Yadkin River Bridge was a 225 ft simple span steel truss, with the two truss planes 13 ft apart. The truss had a depth of 36.5 ft at midspan. The bottom chord consisted of eyebars; the top chord and end posts consisted of two channel sections with cover plates and lacing; vertical members were typically channels with lacing; diagonals were typically square bars used singly or in pairs; top and bottom laterals and sway frame bracing were round bars; and the lateral struts and portal members were single or double angle struts.

An investigation completed after the collapse concluded that the failure was due to a vehicular collision with one of the truss's end posts [76]. The resulting redistribution of dead load caused the failure of additional members, resulting in a torsional rotation of the structure. This rotation combined with the loss of upper laterals, allowed the top chord to buckle, resulting in the collapse of the bridge. The investigation also found that the truss members were adequately designed for their time period, and the corrosion that did exist on the members had no practical effect on the load-carrying capacity, and the physical and chemical properties of the steel were typical for the time period.

The collapse of the Yadkin River Bridge demonstrates the need for bridge designers to consider the loss of a single member during the design of a steel truss. The bridge designer must verify that there is enough redundancy built into the structure. Computer simulations, via finite element analyses, can be performed by the designer to investigate the behavior of a steel truss bridge after the loss of a single member. Several member loss scenarios should be investigated to verify that the structure is adequately redundant.

5.1.3 Minneapolis I-35W Bridge, 2007

On August 1, 2007, the eight-lane, three-span, steel deck truss I-35W Bridge in Minneapolis, Minnesota, collapsed into the Mississippi River, killing 13 people. The bridge was constructed in 1967, the main span of the bridge was 456 feet, and it carried approximately 140,000 vehicles daily through downtown Minneapolis. The collapse of the structure not only resulted in the loss of life, but created a major disruption of commerce. The bridge had been rated as structurally deficient since 1990, and had undergone annual inspections by the Minnesota Department of Transportation since 1993. The most recent inspection, complete in June 2006, indicated that there was cracking and fatigue problems, causing the bridge to receive a sufficiency rating of 50 percent on a scale of 0 to 100 percent [77].

At the time of the collapse, the I-35W bridge surface was being repaired requiring the closure of some traffic lanes. In addition, there was construction equipment and materials on the bridge when it collapsed. This led to the FHWA issuing a technical advisory regarding construction loads on bridges, Technical Advisory 5140.28 [78], since the NTSB had identified the construction loading on the bridge as a possible contributor to the collapse. The issuance of the technical advisory highlights the importance that the consideration of construction loads can have on a bridge project. The specialty engineer, working for a contractor on a resurfacing or complete rehabilitation bridge project, must consider the effects that construction loading will have on the structure. The specialty engineer must check that members are not overstressed or displacements are too large when construction loads are applied. Additionally, it must be verified that the structure and its components do not experience any instability during repair and/or rehabilitation operations.

At the time of this writing (December 2008), the National Transportation Safety Board (NSTSB) was conducting an investigation into the collapse and issued an interim report regarding findings of the investigation. The investigation discovered that the original design process led to an error in the sizing of some of the gusset plates in the main span trusses. The results indicate that eight of the 112 gusset plates were undersized and did not provide the factor of safety to be expected in a properly designed bridge. At these locations the gusset plates were approximately half of the thickness required. The results and calculations associated with this investigation can be found in the FHWA's interim report authored by Holt and Hartmann [79].

The findings of the interim investigation led to the NTSB issuing a safety recommendation to the FHWA, stating that "for all non-load-path-redundant steel truss bridges within the National Bridge Inventory, required that bridge owners conduct load capacity calculations to verify that the stress levels in all structural elements, including gusset plates, remain within applicable requirements whenever planned modifications or operation changes may significantly increase," [80].

Furthermore, the NTSB held a public meeting in November of 2008, and issued a synopsis of the Safety Board's final report, which was still under review at the time of this writing (December 2008). In the synopsis, the NTSB determined that the probable cause of the collapse was due to the inadequate load capacity of the gusset plates at node U10, due to a design error, and which failed under a combination of (1) substantial increases in the weight of the bridge resulting from previous modifications, and (2) the traffic and concentrated construction loads on the bridge on the day of the collapse [81]. The NTSB further stated that: (1) the failure of the original design firm's quality control procedures to ensure appropriate main truss gusset plate calculations were performed and inadequate review by the State and Federal transportation officials contributed to the design error, and (2) the generally accepted practice among transportation officials of giving inadequate attention to gusset plates during inspections and excluding gusset plats in load rating analyses contributed to the bridge collapse.

It is imperative in rehabilitation projects that the bridge designer investigates the capacity/demand of all members, including gusset plates, whenever there is a significant change in loading, an increase in deck thickness for example, or changes in operational use such as additional traffic lanes or vehicle types. Guidance for the load rating evaluation of gusset plates in truss bridges has been provided by the FHWA [82] for rating in accordance with the Load and Resistance Factor Rating Method (LRFR).

5.2 Suspension Bridges

5.2.1 Tacoma Narrows Bridge Collapse, 1940

Probably the most famous bridge failure is the collapse of the Tacoma Narrows Suspension Bridge on November 7, 1940, over the Puget Sound in Washington State. The bridge opened on July 1, 1940, had a total length of 5,000 ft, and a center span length of 2,800 ft. The Tacoma Narrows Bridge was the first suspension bridge to use steel plate girders to support the roadway while previous bridges typically used steel trusses to support the roadway. The plate girders used in the bridge were 8 ft deep. The dead load and stiffness of the bridge were much less than other suspension bridges built previously.

Shortly after the bridge opened it was discovered that under relatively mild wind conditions, the bridge deck oscillated severely. These vertical oscillations were a direct result of the slenderness, low self-weight, and low dampening ability of the bridge. On the day of the collapse, the bridge was closed after measuring a constant wind speed of 42 mph. The bridge deck was experiencing 38 oscillations per minute with a vertical amplitude of 3 ft [83]. The collapse was initiated by a failure in the center north suspender, followed by more severe vertical oscillations, then one of the girders buckled in the middle of the bridge, suspender cables failed, and sections of the main span fell progressively from the center to the towers. The Tacoma Narrows Bridge collapse is well documented and additional information can be found at a website sponsored by the University of Washington Libraries, www.lib.washington.edu/specialcoll/exhibits/tnb/ [84].

The collapse of the Tacoma Narrows Bridge initiated the consideration of aerodynamics in long-span bridge design. Wind not only causes static loads on a bridge, but results in a special dynamic behavior as well. The collapse shows bridge designers the importance of stiffness, rigidity, torsional resistance, and dampening in suspension bridges as they relate to wind. Addressing these issues can be accomplished through wind tunnel testing, and computer modeling that integrates the wind tunnel test data.

5.2.2 Silver Bridge, 1967

The Silver Bridge, which spanned the Ohio River between Point Pleasant, West Virginia, and Kanauga, Ohio, was the first eyebar suspension bridge built in the United States. The bridge, completed in 1928, had a center span length of 700 ft and end spans of 380 ft each. The suspension chain on each side of the bridge consisted of two eyebars having a thickness of 2 in. and width of 12 in. [85].

On December 15, 1967, the bridge collapsed during the evening rush hour, resulting in 46 fatalities, and nine injuries [86]. The NTSB concluded that the collapsed was initiated by a cleavage fracture in the lower limb of an eye bar on the north chain, on the Ohio side span, at the first chain joint away from the tower. The fracture was then followed by a ductile failure in the upper limb of the same eyebar, separating the eyebar from the chain. Immediately following this loss of this eyebar, the "sister" eyebar failed, resulting in a complete separation in the north chain and the onset of the collapse. The NTSB believed that the initial cleavage fracture was caused by the development of a critical size flaw over the 40-year life of the structure as a result of the combined action of stress corrosion and corrosion fatigue. It was noted by Lichtenstein that the eye of the eyebars, where the pin fits in, was elongated by one-eighth of an inch for ease of erection, and thus allowing a space for corrosion to develop [85]. The detail did not allow for visual inspection, without the dismantling of the joint.

The collapse of the Silver Bridge led to the initiation of routine inspections for all bridges, through the approval of the National Bridge Inspection Standards (NBIS) in 1968 by the U.S. Congress. The collapse also prompted research into metal fatigue, bridge management, and nondestructive inspection methods [87].

The failure of the Silver Bridge also shows the need to consider redundancy in design. With only two eyebars being used along the suspension chain, the failure of one eyebar resulted in a significant increase in loading on the adjacent eyebar and subsequent failure of the chain. The use of additional bars may have provided an increased redundancy since the load would have been transferred to more than just one adjacent eyebar. Furthermore, the collapse also shows the importance of details and connections, and the required ability to inspect them. Bridge designers have learned many lessons from the Silver Bridge collapse and these lessons must be continuously applied in future bridge designs.

5.3 Cable-Stayed Bridges

Although there have been no significant failures of cable-stayed bridges reported in the United States, there are failure events that the bridge designer must take into account during design. Similar to suspension bridges, cable-stayed bridges must be checked for aerodynamic effects to ensure that the bridge decks do not experience significant oscillations. Computer modeling and associated wind tunnel testing are typical requirements for cable-stayed bridges. Additionally, as reported on by Kumarasena et al. [88], serviceability problems related to large amplitude vibrations of stay cables under certain wind and rain conditions have been observed. In their report, Kumarasena et al. (2005) provide design guidelines for the mitigation of wind induced vibrations of stay cables that can be used by bridge designers experienced with cable-stayed bridge design and wind engineering.

Another potential failure mechanism that should be recognized by the bridge designer for cable-stayed bridges is the potential for corrosion of the stay cables. Hamilton et al. [89] provide an extensive survey of the stay-cable protection methods and experimental findings regarding durability testing of large-scale grouted stay specimens subjected to alternate wet and dry cycles with salt solution. Variables in the experimental study included temporary corrosion protection, galvanized strands, epoxy coated strands, and greased and sheathed strands. Additionally, Hamilton et al. [90] also report on corrosion testing of stay cables encased in grout, in which grout mixes were tested with various commonly used admixtures to optimize antibleed, viscosity, and corrosion protection.

6.0 CONCLUSIONS

This framework has been developed to advise bridge designers to consider potential failure scenarios during the design process. General fault trees describe potential contributory elements that designers can use to address potential sources of failure during the design process have been provided for several common bridge types.

This framework serves as a general checklist of issues that should be given attention by a bridge designer during the design process in order to minimize potential failures during the service life and/or construction of the specific bridge being designed. Fault trees allow the designer to graphically see various failure combinations and failure paths. The framework can help a bridge designer and/or owner to determine whether additional analyses investigating potential failures are warranted.

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