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Gusset Plates in Steel Bridges-
Design and Evaluation

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Gusset Plates in Steel Bridges— Design and Evaluation

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Abstract—This Steel Technical Information and Product Services (Steel TIPS) report provides information and technologies that can be used to design as well as to evaluate gusset plates in steel truss bridges. The first chapter of this report is devoted to an introduction to the steel truss bridges and gusset plates and their types as well as to the objectives and scope of this document. The second chapter provides a historical background on gusset plates used in steel truss bridges. Chapter 3 presents a background on development of equations and concepts used in design of gusset plates and provides procedures and specifications for design of gusset plates used in steel trusses of bridges. The last chapter, Chapter 4, provides notes on evaluation of gusset plates in existing bridges. The report has an appendix on J. A. L Waddell's First Principles of Designing.

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In several places, it was necessary to quote a certain provisions of the AASHTO Specifications (AASHTO, 2007/08 and earlier editions) related to the gusset plates in order to discuss them. These verbatim quotes are used with a written permission of the AASHTO (American Association of State Highway and Transportation Officials) which holds copyright to those provisions.

ACKNOWLEDGMENTS

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Many practical and technical aspects of this report are credited to the reviewers and if there are shortcomings, errors or omissions, they are mine. I hope that the readers will forgive those shortcomings and will find the time to notify me at astaneh@ce.berkeley.edu of the errors and omissions so that I can post corrections.

I am indebted to the developers of the web sites: www.historicalbridges.org , www.bridgehunter.com and www.bridgemeister.com for their dedicated work and time spent in collecting and posting extensive information and thousands of photographs on bridges, especially truss bridges and their gusset plates. The sites provided me with many valuable photographs to use and visually demonstrate many technical items discussed in the report. The technical information on the web sites was also very valuable in understanding the historical development of steel bridges, in particular the truss bridges during the 18th, 19th and 20th centuries. Similarly, I would like to acknowledge my enormous benefit from Google's scanned old books with expired copyrights posted on its books site (http://books.google.com/).

The opinions expressed in this document are solely those of the author and do not necessarily reflect the views of the University of California, Berkeley, where he is a professor of structural engineering, the Structural Steel Educational Council of which he is a Participating Member, or other agencies and individuals whose names appear in this report.

GUSSET PLATES IN STEEL BRIDGES — DESIGN AND EVALUATION

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Notations

A_g = gross area
A_{gw} = gross area of gusset plate at Whitmore's section = $(W)(t_g)$
A_n = net area
A_{nt} = net area in tension (in block shear failure equations)
A_{nW} = net section of the gusset plate at Whitmore's section
A_{tg} = gross area along the plane resisting tension stress
A_{tn} = net area along the plane resisting tension stress
A_{vg} = gross area along the plane resisting shear stress
A_{vn} = net area along the plane resisting shear stress
c = distance from extreme fiber of gusset plate to its neutral axis = $t/2$
d_h = diameter of bolt hole
E = modulus of elasticity of steel
F_t = allowable tensile stress on the critical section
F_{cr} = critical stress
F_{max} = maximum stress
F_u = specified ultimate strength of steel
f_t = applied tensile stress on the critical section (un-factored)
f_v = applied shear stress on the critical section (un-factored)
F_v = allowable shear stress on the critical section
F_y = specified minimum yield stress of steel
F_{yg} = specified minimum yield strength of gusset plate
I = moment of inertia of critical section = $tL^3/12$
L = length, also length of critical section of gusset plate
L_{fg} = free length of gusset plate
l_i = length of i^{th} failure section
M = bending moment acting on the critical section of gusset plate under consideration
M_p = plastic moment capacity of critical section of gusset plate under consideration = $Z_x F_y$
M_u = ultimate moment capacity of critical section of gusset plate under consideration = $Z_{x\text{-}net} F_u$
N = axial force acting on the critical section of gusset plate and normal force (axial force) acting on the critical section of gusset plate under consideration or acting on an element of gusset plate
N_i = force parallel to axis of tension member acting on i^{th} failure section
N_u = tensile fracture strength of net area of critical section of gusset plate under

consideration = $A_n F_u$

N_y = yield capacity of critical section of gusset plate under consideration = $A_g F_y$

Q_i = force effect

Q&T = Quenched and Tempered steel

R_n = nominal resistance

R_{nbr} = nominal bearing strength of bolt group

R_{nbs} = nominal block shear resistance

R_{nbs} = nominal shear strength of the bolt group connecting the member to the gusset plate

R_{nc} = nominal capacity of gusset considering buckling failure mode

$(R_n)_i$ = tension capacity of i^{th} failure section in direction parallel to axis of tension member

R_{nW} = nominal yield strength of a Whitmore section of the gusset under direct axial tension or compression

R_{nWu} = nominal ultimate strength of a Whitmore section of the gusset plate under direct axial tension

R_r = factored resistance: ϕR_n

S_x =elastic section modulus

s = length

t = thickness of a plate

t_g = thickness of a gusset plate

V = shear force acting on the element or on the critical section of gusset plate under consideration

V_{nu} = nominal shear resistance of the net area of the critical section of gusset plate under consideration = $(A_n)(0.58F_u)$

V_{ny} = nominal shear yield capacity of the gross area of the critical section of gusset plate under consideration = $A_g(0.58F_y)$

V_y = shear yield capacity of the critical section of gusset plate under consideration = $A_g(0.58F_y)$.

V_u = ultimate shear capacity of the net section of the critical section of the gusset plate under consideration = $(A_n)(0.58F_u)$.

W = width of the Whitmore's section

Z_x = plastic section modulus of the critical section of gusset plate under consideration

$Z_{x\text{-}net}$ = section modulus of the net area of the critical section of gusset plate under consideration

α_i = angle between axis of the tension member and i^{th} failure section

γ = load factor

γ_i = load factor: a statistically based multiplier applied to force effects

η = a factor relating to ductility, operational importance, or redundancy as specified in

AASHTO LRFD Specifications (AASHTO, 2007)

η_D = a factor relating to ductility , as specified in Article 1.3.3 of the AASHTO LRFD Specifications (AASHTO, 2007)

η_I = a factor relating to operational importance as specified in 1.3.5 of the AASHTO LRFD Specifications (AASHTO, 2007)

η_i = load modifier: a factor relating to the ductility, redundancy, and operational importance

η_R = a factor relating to redundancy, as specified 1.3.4 of the AASHTO LRFD Specifications (AASHTO, 2007)

θ = angle of member with horizontal line

ρ_i = a parameter equal to $1/\sqrt{1+2\cos^2\alpha_i}$

$\sum \eta_i \gamma_i Q_i$ = applied load effect, the demand side of equation of design

σ = normal stress acting on an element

σ_1 = principal stress acting on an element

σ_2 = principal stress acting on an element

σ_{allow} = allowable normal stress for the material of gusset plate

σ_d = normal stress created by force in the diagonal member

σ_b = bending stress

σ_i = normal stress acting on all i^{th} failure section, see Figure 3.19

σ_i' = normal stress parallel to axis of the tension member acting on i^{th} failure section

σ_{max} = maximum normal stress acting on the critical section of gusset plate

σ_v =normal stress created by the force in vertical member

σ_{red} = equivalent (Von Mises) stress acting on i^{th} failure section

τ = shear stress acting on the element

τ_{allow} = allowable shear stress for the material of gusset plate

τ_i = shear stress perpendicular to axis of the tension member acting on i^{th} failure section

ϕ = resistance factor: a statistically based multiplier applied to nominal resistance

ϕ_{br} = resistance factor for bearing failure of member due to bolt pressing against it = 0.80

ϕ_c = resistance factor for buckling of gusset plate equal to 0.80

ϕ_{bs} = resistance factor for bolt shear equal to 0.80

ϕ_i = resistance factor

ϕ_y = resistance factor for yielding of steel equal to 0.95

ϕ_u = resistance factor for tension fracture in net section equal to 0.80

ϕ_y = reduction factor for ductile yield failure mode = 0.90

$(\phi R_n)_{Conn.}$ = reduced resistance of connection

$(\phi R_n)_{Member}$ = reduced resistance of member

Preface

My main motivation to write this report was the tragic collapse of the I-35W bridge on August 1, 2007 which resulted in the loss of 13 precious lives and the injuries to more than 140 people. The first photos of the collapsed bridge showed pieces of gusset plates fractured through their net sections and still attached to the end of members extruding from the waters of the Mississippi River. For weeks after the collapse, as the information began to stream from the bridge engineering community and investigative agencies to the general public, the terms *deck trusses, non-redundant* members, *fracture critical* bridges, *loss of section due to corrosion, fatigue cracks* and especially *structurally deficient* bridges and *gusset plates* entered the press releases and news broadcasts on the collapse. Still, the failure of gusset plate remained the probable cause of the collapse. The term *gusset plates,* in the eyes of public became a symbol of the tragedy synonymous to the *O-rings* of the space shuttle Challenger disaster of 21 years earlier. Devastated surviving family members of the victims and the injured as well as a shocked population, newscasters, public officials and policy makers were demanding answers on why the tragedy occurred and how can we avoid similar ones in the future?

In the aftermath of the collapse the Federal Government charged the National Transportation Safety Board, a respected engineering investigative body, with investigating the collapse and its cause(s). The NTSB in its final report released in December of 2008 established the cause of the collapse the failure of gusset plates U10. Those gusset plates, which were originally under-designed, had reached their failure load on the day of the collapse due to 1970s addition of a 2-inch thick layer of concrete to the deck and the presence of more than 250 tons of construction load at the time of the collapse on the bridge deck in the areas above the gusset plates U10.

With this report on design and evaluation of gusset plates in bridges, I make an attempt to provide the readers, especially the bridge engineers, with a background on steel truss bridges and gusset plates as well as a summary of historical developments of how gusset plates were designed and evaluated. Furthermore, I provide procedures for design and evaluation of gusset plates in steel truss bridges. The report also honors the knowledge and wisdom of the great bridge engineers of the 19th and 20th centuries, such as Squire Whipple (1804-1888), George Shattuck Morison (1842-1903), and John Alexander Low Waddell (1854-1938), who developed much of the design procedures for gusset plates we use today.

To prepare the report I have used many publications published as far back as early 1800s when the first cast iron and wrought iron truss bridges were built in the United States. Throughout the report the experience, knowledge and wisdom of Dr. John Alexander Low Waddell, one of the founding fathers of modern bridge engineering, and in my opinion the father of steel truss bridge engineering, had the most influence not only on the items related to the subject of the report, but to general design concepts and issues related to the safety of bridge structures.

John Alexander Low Waddell, while continuing his successful practice of bridge design, especially designing and patenting movable truss bridges, wrote a monumental 2-volume book: *Bridge Engineering* (Waddell, 1916) with 80 chapters (2115 pages) on almost every conceivable aspect of bridge engineering including planning, design, cost estimation and construction. Chapter 78 is on the “Specifications Governing the Designing of the Superstructures of Steel Bridges, Trestles, Viaducts, and Elevated Railroads. “ The chapter has very specific provisions on design of the members and connections of steel bridges, which in those days were primarily riveted steel truss bridges mostly supported on piers and some suspended from the eye-bar chains and later from the cables. After reviewing Waddell’s 1916 Bridge Engineering book and comparing those specifications to the latest AASHTO Standard Specifications one realizes that the current AASHTO provisions (AASHTO, 2007/08) regarding the design of trusses and gusset plates are almost the same as those in Waddell‘s Bridge Engineering book, Chapter 78. As a tribute to J.A.L. Waddell, I have added an appendix to this report: *J.A.L. Waddell’s First Principles of Designing* which includes excerpts from his valuable 1898 bridge engineering book *De Pontibus* (Waddell, 1898).

I hope this publication will be of some use to bridge engineers and inspectors who are asked to review the gusset plates in steel truss bridges and repair and retrofit any deficiency they find to ensure that a tragic collapse similar to the I-35W Bridge does not happen. The current Steel TIPS report focuses on design and evaluation. A following report will focus on the repair and retrofit of the damaged or deficient gusset plates in steel truss bridges.

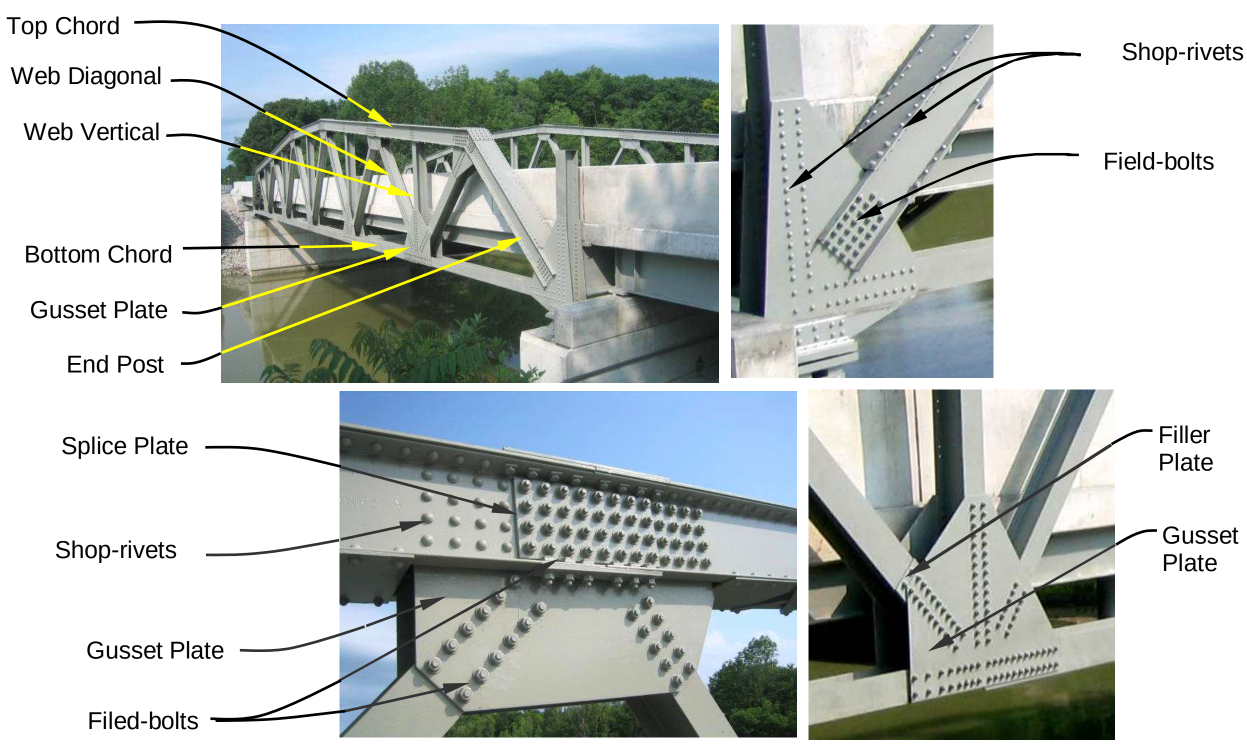
Abolhassan Astaneh-Asl,
Berkeley, April 18, 2010

1. Gusset Plates in Steel Bridges

Photo: www.historicalbridges.org (with permission)

1.1 INTRODUCTION

Gusset plates are used in steel bridge trusses and braced steel towers to connect the steel members to each other. They are also used in connections of the lateral wind bracings of the steel bridges. Figures 1.1, 1.2 and 1.3 show typical examples of short, medium and long span steel truss bridges and their gusset plates. Later this report will discuss various types of gusset plates and how they are designed and evaluated. An upcoming second part to the report (Astaneh-Asl and Tadros, 2010) will discuss how truss bridges and especially their gusset plates may be repaired or retrofitted if they are damaged or found to be deficient.



Photos courtesy of www.historicbridges.org/ (with permission)

Figure 1.1 The 1963 King's Highway 21 Bridge, Lambton County, Ontario is an example of typical short span riveted/bolted truss bridges

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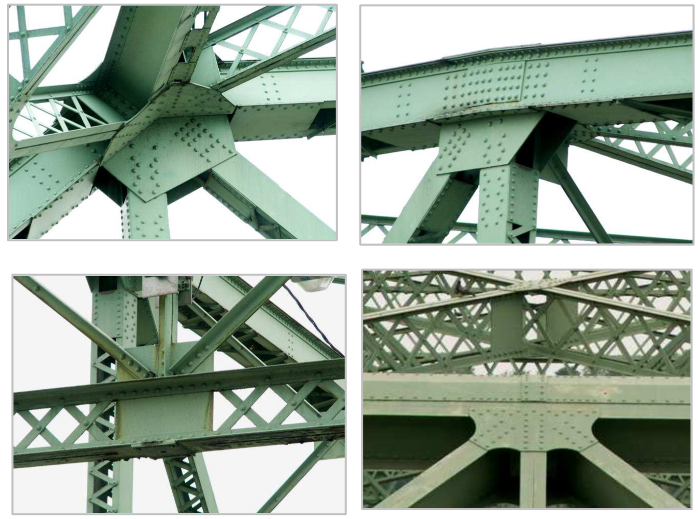

Figure 1.2 The 1932 Kittanning Bridge (Citizens Bridge) carrying US-422 Business over the Allegheny River in Pennsylvania, a typical and classic example of medium span riveted Parker through truss bridges

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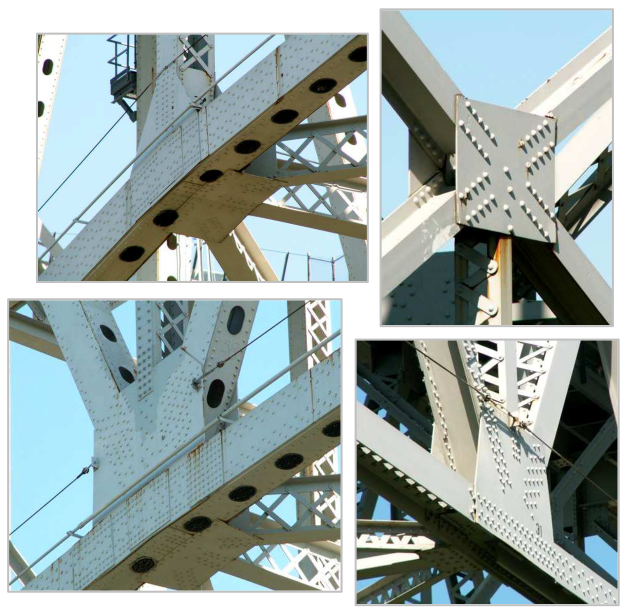


Figure 1.3. The 1938 Blue Water Bridge carrying the I-69 and I-94 / KH-402 over St. Clair River in Michigan (U.S./Canadian Border), an example of major truss span

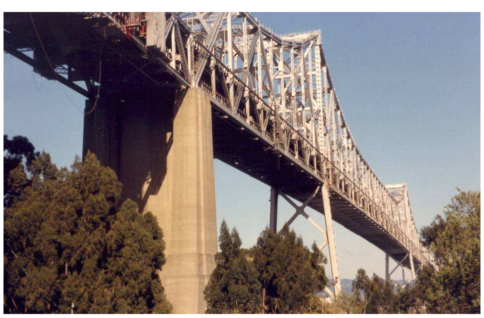
Like any component of a bridge, gusset plates in new truss bridges should be designed in accordance to the applicable provisions of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) which is currently the governing specifications. In this report, I will refer to this document as “AASHTO Specifications (AASHTO, 2007)”.

Article 2.5 of the AASHTO Specifications (AASHTO, 2007) is on *Design Objectives* and provides specific information and guidelines with regard to the safety, serviceability, constructability, economy, and aesthetics of bridges and their components. It is important to note that under *Safety Objectives of Design* AASHTO Specifications (AASHTO, 2007) have only a sentence that, in my opinion, is the most important and the strongest provision of the entire specifications. The statement is: "The primary responsibility of the Engineer shall be providing for the safety of the public." Throughout my career in bridge and structural design, research and teaching, this statement has been my guide. In writing this report as well, the above AASHTO statement was my guide in providing information that can be used by engineers in fulfilling their primary responsibility to provide for the safety of the public.

Riveted, bolted and to lesser degree welded gusset plates have been used in steel truss bridges. In the United States, welded gusset plates have been used occasionally in connections of the lateral bracing systems. These lateral bracing gusset plates are either double gusset or in most cases a single gusset plate welded to all members to be connected to each other. Bridges built prior to the 1940s, which the author refers to in this report as the *first generation* of steel bridges, generally have rivets as their connectors. Two important examples of these bridges are the Golden Gate Bridge and the San Francisco Oakland Bay Bridge, shown in Figure 1.4, which were completed in the late 1930s.

The Golden Gate Bridge
(Photo by and Copyright Abolhassan Astaneh-Asl

The West Crossing of the San Francisco Oakland Bay Bridge under Construction
Photo by California Public Works Department



The East Crossing of the San Francisco Oakland Bay Bridge
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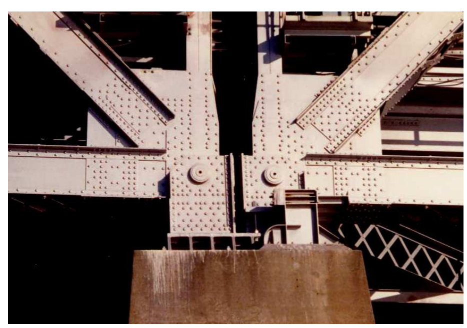

The East Golden Gate Bridge
(Photo by and Copyright Abolhassan Astaneh-Asl

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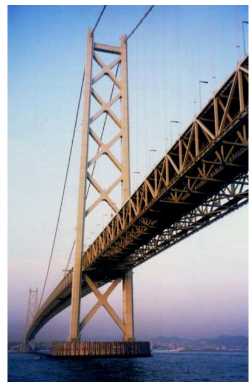
Figure 1.4. The Golden Gate Suspension Bridge and the San Francisco Oakland Bay Bridge are examples of magnificent bridges using steel trusses with gusset plates throughout their structure.

A more modern bridge where gusset plates are used in connections of steel bridge trusses is the 1998 Akashi Kaikyo Bridge in Japan, shown in Figure 1.5. The bridge span at 1991 meters (6532 feet) is currently the longest in the world. The gusset plates are used not only in connections of the super-structure stiffening trusses, but in trusses used inside the anchor blocks as well as in trusses embedded in the caissons supporting the tower anchor bolts, Figure 1.5. Another modern and important bridge with gusset plates as its connections is the Tatara bridge in Japan which is currently the longest cable-stayed span in the world.

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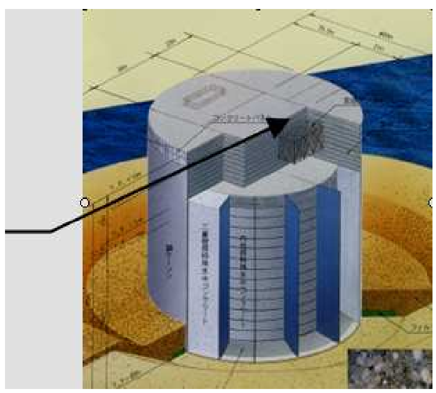
Tower anchor bolts are attached to these trusses embedded inside the caisson.



Trusses inside anchor blocks for cable anchorage

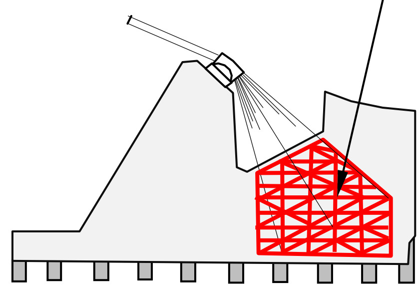


Figure 1.5. The 1998 Akashi Kaikyo Bridge, currently the longest span bridge in the world, is almost entirely a truss system. Even the main suspension cables are attached to trusses embedded inside the concrete anchor blocks as shown in the figures to the right. The anchor bolts of the main towers are attached to a truss embedded inside each caisson

Bridges built prior to the World War II were almost always riveted. In fact, the early editions of the bridge design specifications issued by the American Association of State Highway Officials (AASHO, 1927) prior to the 1st Edition issued in 1931 discouraged the use of bolts in the connections of steel bridges by stating that “Bolts- Unless specifically authorized, bolted connections will not be permitted". Of course this was prior to the development of high strength bolts and their standardization by the ASTM (American Society for Testing and Materials) into A325 and A409 bolts. After the standardization, the high strength bolts gradually replaced rivets by late 1960s, although some riveted bridges continued to be designed and built until early 1970s. In those later era riveted bridges, we find gusset plates and splice plates that are quite often shop-riveted and field bolted, see Figure 1.1. Field riveting, especially in relatively high bridges and low temperatures, was a difficult and time consuming process. Therefore, shop-riveting combined with some field bolting was done to make the erection of the superstructures easier, faster and more economical.

Welds were not commonly used in steel bridges prior to 1950s. At the end of the World War II, when thousands of welders left their jobs in manufacturing war machines and redirected their skills to building civilian structures, the steel bridges gradually went through a transition from all riveted to shop-welded and field-bolted construction. These steel bridges, many of them truss bridges, were mostly built on the Interstate highway system during 1950s and 1960s. Still, even during this period welds were not used as the connectors in the gusset plates of the main trusses but were used in fabrication of steel bridge built-up members and sometimes in the secondary gusset plates such as the gusset plates in lateral bracing systems, Figure 1.6. I have classified these bridges as the *second generation* of steel truss bridges. In these bridges, members were in general either riveted or welded built-up sections while the primary gusset plates in the main trusses were riveted or riveted/bolted and the secondary gusset plates of the lateral bracing system were either riveted/bolted or fully welded as shown in Figure 1.6.

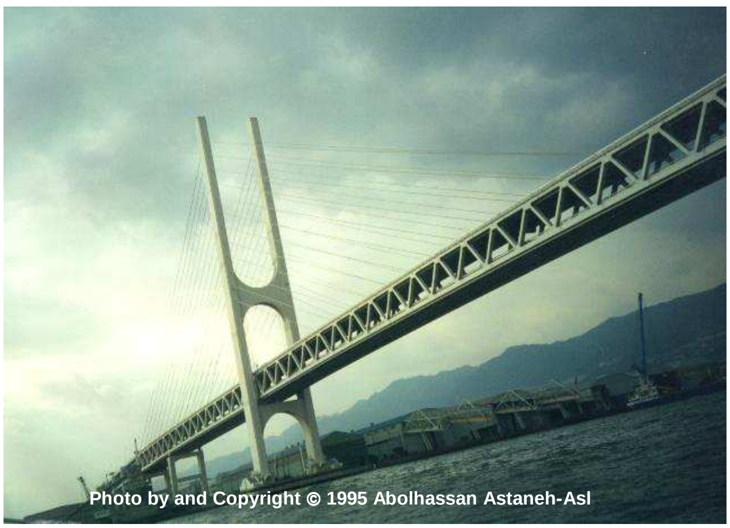
Above Photo by and Copyright John A. Weeks

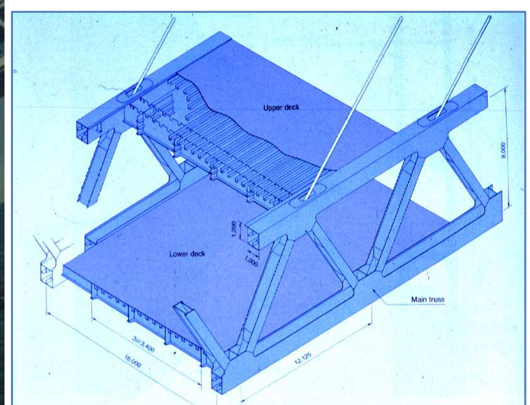
Above 2 Photos by and copyright 2008 Abolhassan Astaneh-Asl.

Figure 1.6. The I-35W bridge, had welded gusset plates in its transverse trusses. Note that the photos of gusset plates on the right were taken by the author after the collapse of the bridge.

At the time of design and construction of these *second generation* steel truss bridges our knowledge of fatigue behavior of welded steel bridges was somewhat limited. Although, the AASHO Specifications (the predecessor to the AASHTO Specifications) from the first edition in 1931 had a provision with regard to adverse effects of alternating live load due to passage of trucks, it was not until the 9th Edition of the AASHO Specifications (AASHO, 1965) that more detailed provisions regarding fatigue of welded steel bridges were included in the AASHTO Specifications. As a result, some weld details used in steel bridges built primarily during 1950's through early 1970s, which I have called *second generation* of steel truss bridges are quite vulnerable to developing fatigue cracks.

After the 1970s and the development of technologies and details that could prolong the fatigue life of steel bridges, shop-welded/field-bolted bridges, including truss bridges, became a standard practice and the use of rivets discontinued. I have called these modern bridges the *third generation* of steel bridges, where shop-welding/field-bolting was done and continues to be done to achieve the fatigue life expectancy of the design. Figure 1.7 shows typical examples of these modern bridges where trusses with shop-welded gusset plates are the main structural system.





Source: Akashi-Kaikyo Bridge Authority Publications

Figure 1.7. A view of the 1992 Higashi-Kobe Bridge in Japan, a modern elegant cable-stayed steel bridge with double-deck truss (top) and details of its truss deck system (bottom). The gusset plates are shop-welded.

Steel truss bridges were quite popular in the past due to their relatively high stiffness and strength per weight ratio. However, since the 1970s, steel trusses have not been used so frequently in the United States compared to other systems, such as the plate girders for short and medium spans and cable-stayed and suspension bridges. In modern times, steel trusses still prove to be quite efficient as the stiffening trusses of suspension or cable-stayed bridges, Figures 1.5 and 1.7. All of these bridge trusses, old and new, have gusset plates connecting their truss members to each other.

Gusset plates were always an important part of steel truss bridges. The evolution of gusset plates is discussed in Chapter 2. Since the early 20th century, gusset plates have been used not only in the lateral bracings but also as the connectors of the members of the main gravity load carrying steel trusses. However, it was the buckling failure of a few gusset plates in the I-90 deck truss bridge in Ohio in 1996 that brought the name *gusset plate* to the attention of the press and public.. In addition, although the buckling failure of the gusset plates in this bridge did not result in collapse of the span or any injury or death, the bridge sagged a few inches. As a result the repair and replacement of the damaged gusset plates became a relatively costly effort. More information on the collapse of the I-90 bridge is given in Chapter 3 of this report when I discuss the design of the gusset plates to prevent buckling failure mode.

In the aftermath of the 2007 tragic collapse of the I-35W bridge, another deck truss bridge, the National Transportation Safety Board became the agency in charge of investigating the collapse. On August 2, 2007, the day after the tragic event in Minneapolis, the Federal Highway Administration (FHWA 2007a) issued the first of its advisories regarding steel truss bridges and called for the state departments of transportation to *immediately inspect all steel deck truss bridges* (emphasis added). In its second technical advisory, titled *Construction Loads on Bridges*, the FHWA (2007b) stated that "we strongly advise the State Transportation Agencies and other bridge owners who are engaged in or contemplating any construction operation on their bridges to ensure that any construction loading and stockpiled raw materials placed on a structure do not overload its members." Although this technical advisory did not specifically mention gusset plates, the third technical advisory issued by the FHWA (2008) on January 15, 2008, also included gusset plates by stating that:

> "…the National Transportation Safety Board (NTSB) recommends that bridge owners conduct load capacity calculations for all non-load path-redundant steel truss bridges to verify that the stress levels in all structural elements, including gusset plates, remain within applicable requirements whenever planned modifications or operational changes may significantly increase stresses." (FHWA, 2008)

Issuing the above recommendation was a very positive step by the NTSB and the FHWA to bring to the attention of the transportation agencies and bridge owners the importance of the gusset plates, especially in fracture-critical steel bridge trusses, in stability of the bridge as well as in preventing a catastrophic collapse. The current AASHTO Specifications (AASHTO, 2007) has a section on gusset plates used in steel truss bridges as well as a section on connections and splices in steel bridges. In the bridge engineering textbooks and design manuals, some listed in the bibliography of this report, there are also discussions of the design of gusset plates in steel truss bridges. In Chapter 2 of this report I have summarized historical background and developments of gusset plates and their design. In Chapter 3, after reviewing the literature and available methods, I have attempted to develop and propose gusset plate design procedures that are consistent with the research results in the literature as well as with the experience with the field performance of steel truss bridges and their gusset plates. The proposed procedures in Chapter 3 for gusset plates are in the Load and Resistance Factor Design, LRFD format, which is currently used in the AASHTO Specifications (AASHTO, 2007). I hope that the proposed methods will assist bridge engineers in their efforts to evaluate gusset plates in existing steel truss bridges as well as in design of new bridges. The procedures given in Chapter 3 for the Load and Resistance Factor Design of the truss gusset plates need to be considered as a first step in developing rational guidelines for safe and economical design of gusset plates based on the information currently available in the literature on the behavior of gusset plates.

The proposed method for design and evaluation of gusset plates considers all known failure modes or limit states and, for each limit state of failure, provides the *demand* as well as the *capacity* side of the equation of design:

$$\text{Demand} \leq \text{Capacity} \tag{1.1}$$

Chapter 4 of the report includes notes on evaluation of the existing condition of gusset plates from the point of view of corrosion and strength.

1.2 OBJECTIVES OF THE REPORT

The main objectives of this report are:

- To introduce the reader, especially the current generation of bridge engineers and structural engineering students to the background, historical development and types of steel gusset plates in steel bridge trusses.
- To present design procedures for design of new gusset plates and for evaluation of existing ones.
- To provide notes on evaluation of existing condition of gusset plates.

1.3 SCOPE

Gusset plates are used in steel trusses for buildings and bridges as well as in the connections of the braced frames for buildings. This document focuses on the gusset plates for steel truss bridges. Information on the behavior and design of gusset plates for braced frames in buildings are found in the literature, including Chambers and Ernst (2005). Information on seismic behavior and design of gusset plates can be found in the Steel TIPS report by Astaneh-Asl (1998).

A typical bridge gusset plate, the focus of this report, is shown in Figure 1.8. The report provides information and technologies that can be used to design gusset plates in new steel truss bridges as well as to evaluate the gusset plates in existing steel truss bridges. Figures 1.9 through 1.16 show examples of steel truss bridges and their gusset plates which are the focus of this report.

Chapter 1, provides a brief introduction to the steel truss bridges, their gusset plates , as well as objectives and scope of this report.

Chapter 2 provides a historical background on steel truss bridges and their gusset plates as well as on their performance in the past. An attempt is made to make Chapters 1 and 2 not only useful to the engineers but also useful to non-engineers interested in steel truss bridges and gusset plates.

Chapter 3 presents information on design of gusset plates, their failure modes and a background on development of equations and concepts used in design of gusset plates in steel truss bridges. Chapter 3 is prepared for bridge engineers who are familiar with the bridge design concepts and specifications such as the AASHTO Specifications issued by the American Association of Highway and Transportation Engineers. Chapter 3 also provides design procedures and recommendations for design of gusset plates in steel trusses. The procedures are in the Load and Resistance Factor Design (LRFD) format consistent with the current AASHTO LRFD Bridge Design Specifications (AASHTO, 2007). It is hoped that the proposed procedures for design of gusset plates will receive the critical review of the profession and after necessary corrections and modifications , which are no doubt needed, will be incorporated into the future AASHTO Specifications.

Chapter 4 provides notes on evaluation of existing gusset plates. This chapter is also prepared with the hope that it will be somewhat useful to bridge engineers and inspectors in their evaluation of existing steel truss bridges and their gusset plates.

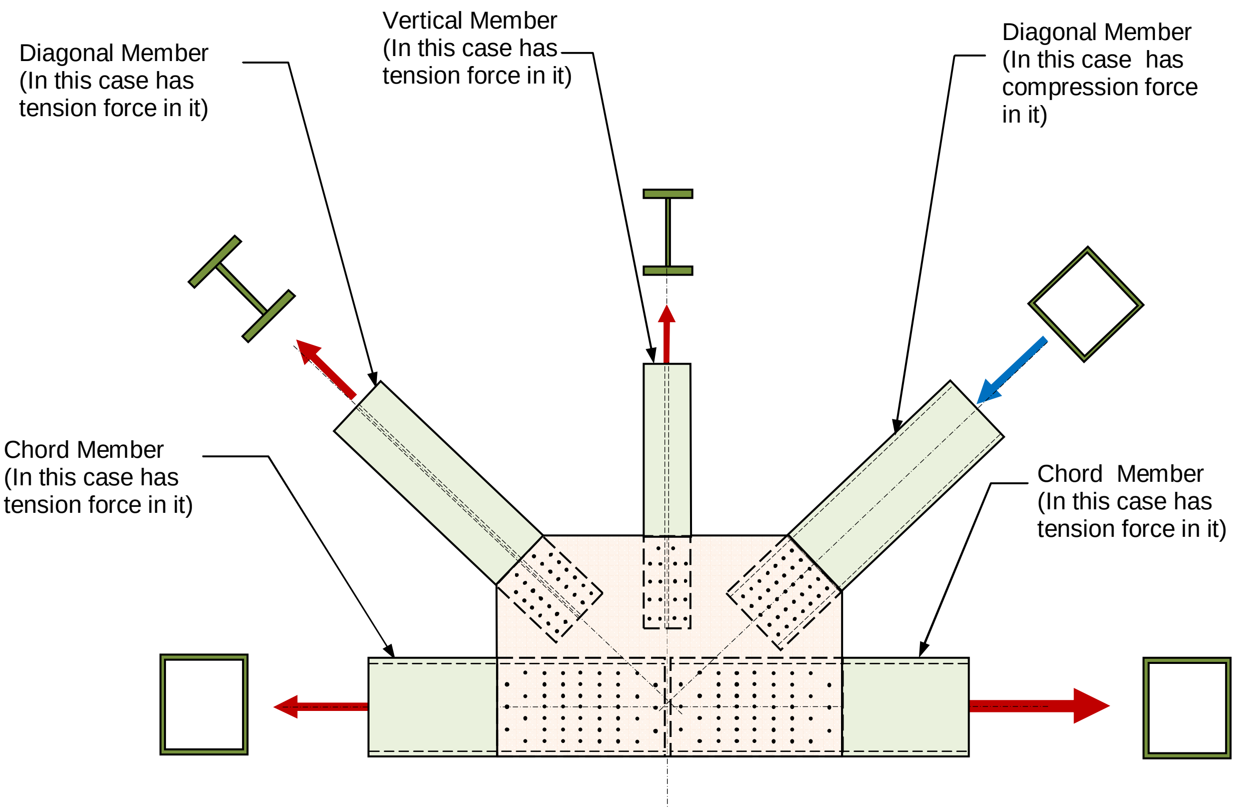
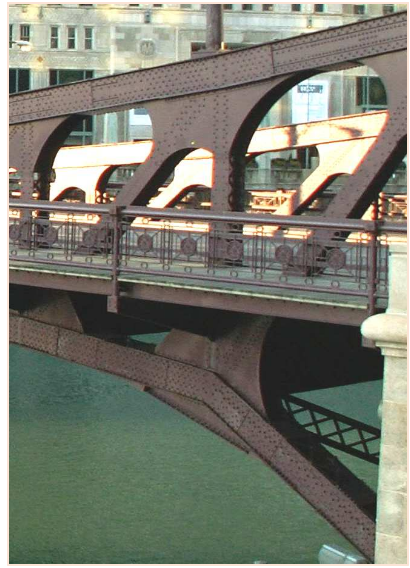


Figure 1.8. A typical riveted/bolted steel bridge truss gusset plate, the main focus of this report

Figure 1.9. The 1924 riveted King's Highway 3 Bridge in Ontario, Canada, and its riveted/bolted gusset plate connections. The top right photo shows a gusset plate which is also a splice plate for the bottom chord member

Figure 1.10. The 1947 Cut River Bridge on US 2 in Michigan and its riveted curved-edge gusset plate connections. The curved-edge gusset plates are not as common in the U.S. bridges as the straight-edge gusset plates.



Photos courtesy: www.historicalbridge.org (with permission)

Gusset Plates with curved edges

Figure 1.11. The 1920 Franklin Street Bridge in Chicago, IL. This riveted bascule truss bridge is a typical representative of a number of similarly beautiful bascule bridges built in Chicago over the Chicago River. Note the curved edges of the gusset plates.

Photos: www.historicalbridge.org (with permission)

All-bolted gusset plates

Gusset Plates in Main Trusses

Gusset plates in lower lateral bracing system

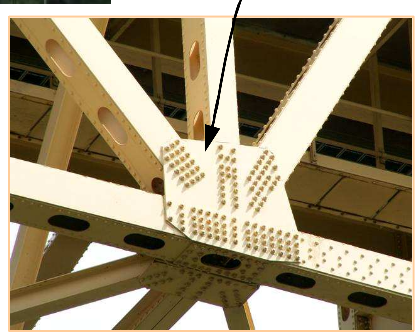


Figure 1.12. The 1962 International Bridge, Michigan, USA, and Ontario, Canada, and its bolted gusset plate connections. Members are riveted perforated box sections or rolled wide flange shapes

Figure 1.13. The 1951 riveted Pennsylvania Turnpike Allegheny River Bridge and its gusset plate connections

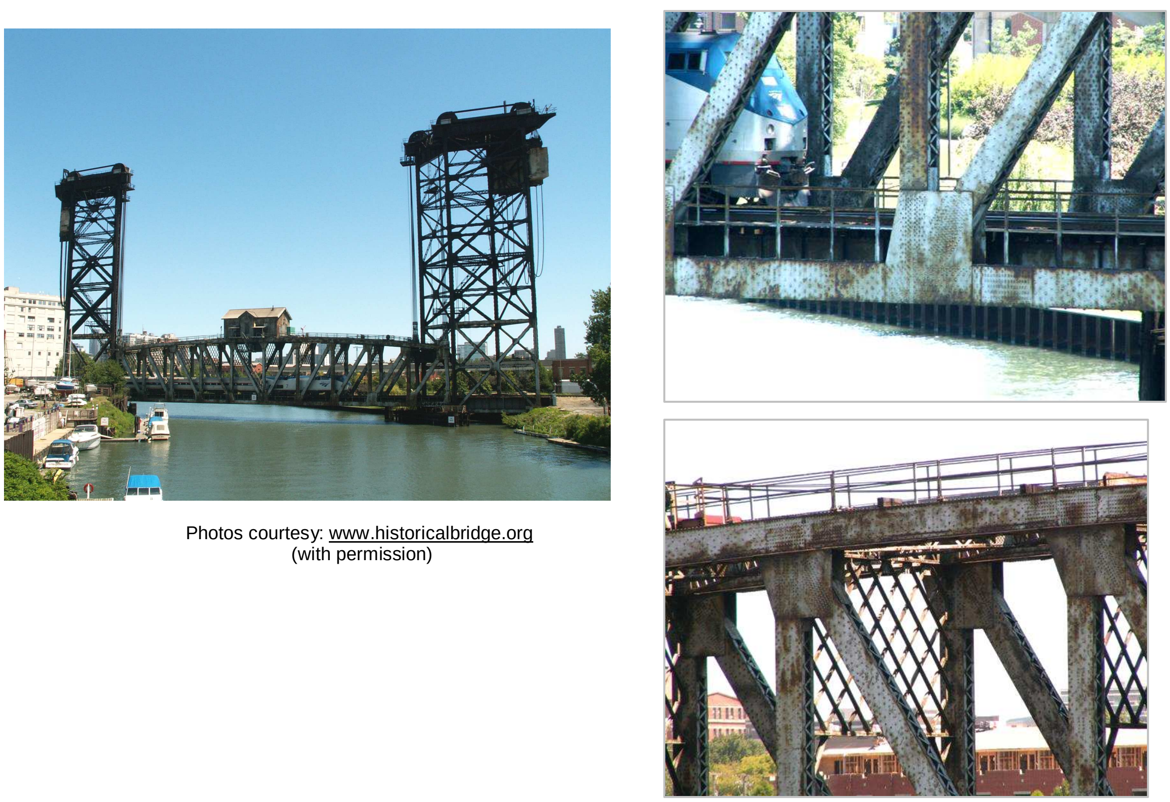
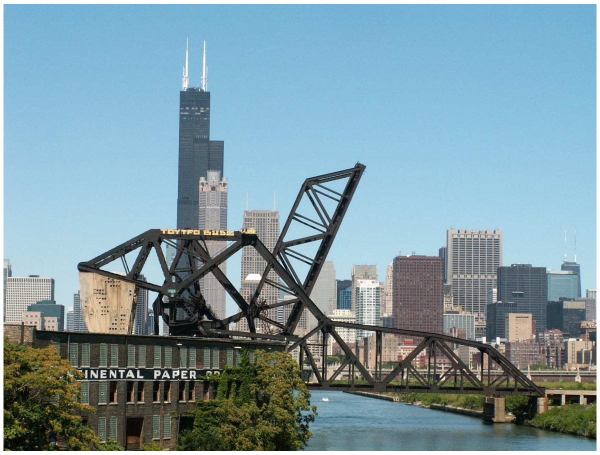


Figure 1.14 The 1915 Canal Street Railroad Bridge in Chicago and its gusset plates



Photos courtesy: www.historicalbridge.org
(with permission)

Figure 1.15 The St. Charles Air Line Lift Bridge and B&O RR / Chicago Terminal Lift Bridge

Photos and information courtesy: www.historicalbridge.org
(with permission)

According to www.historic bridges.org and the photo to the right by Greg Kish at the site the swing span collapsed in 1992 after the freighter H. Lee White carrying 67,000 kips of iron ore hit it.

Figure 1.16. The 1913 Grosse Ile Toll Bridge, a swing bridge parts of which was replaced after the swing span collapsed in 1992

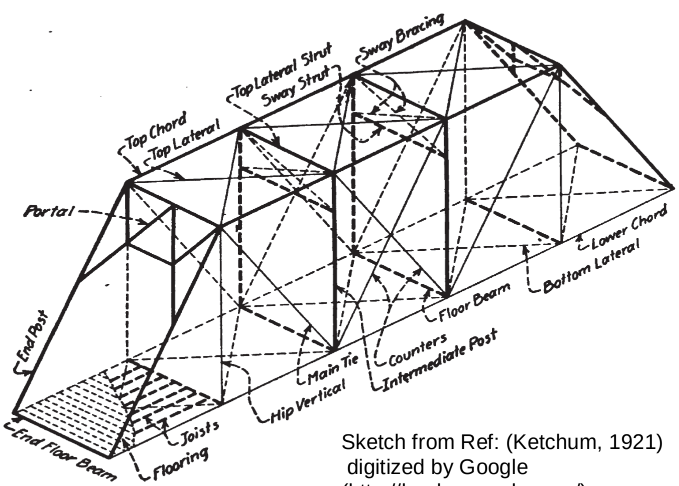
2. Historical Background on Gusset Plates in Steel Truss Bridges

Photo: Mark P. Thomas

2.1 A BRIEF INTRODUCTION TO BRIDGE TRUSSES

Since this report focuses on gusset plates, I do not intend to discuss extensively the issues related to the behavior and design of steel trusses but rather to provide a brief background introduction to the terminology of trusses and where and what type of gusset plates are used in these trusses. For a discussion of trusses, the reader is referred to the literature and bridge engineering publications, such as Waddell (1898 and 1916) and Ketchum (1920) for historical riveted bridge trusses, and to Bresler, Lin, and Scalzi (1968) and Brockenbrough and Merritt (1999) for more modern bridge trusses. Figure 2.1 shows a typical Pratt truss along with definition of components of typical trusses.

Photo of CR-43, Alleghany County, NY
from www.historicalbridges.org
(with permission)



Sketch from Ref: (Ketchum, 1921)
digitized by Google
(http://books.google.com/)

Figure 2.1 A typical Pratt truss and its members

Figures 2.2 and 2.3 show examples of the most common types of steel bridge trusses. The trusses are *through-deck* trusses, where the deck of the bridge is supported at the level of the bottom chord, such as those shown in Figure 2.2, or they are *deck trusses*, where the deck is supported on the upper chord level. For short span bridges, especially for railroads, through-deck trusses were used quite frequently in the past. For long span bridges crossing a valley or wide body of water, both deck trusses and through trusses were used. Gusset plates are used at the panel points (that is, the joints) of both through and deck trusses. In addition, gusset plates are also used in other truss geometries such as arch trusses and stiffening trusses of suspension and cable-stayed bridges. Trusses with gusset plate connections are also used in movable bridges such as lift and bascule bridges, Figure 2.4.

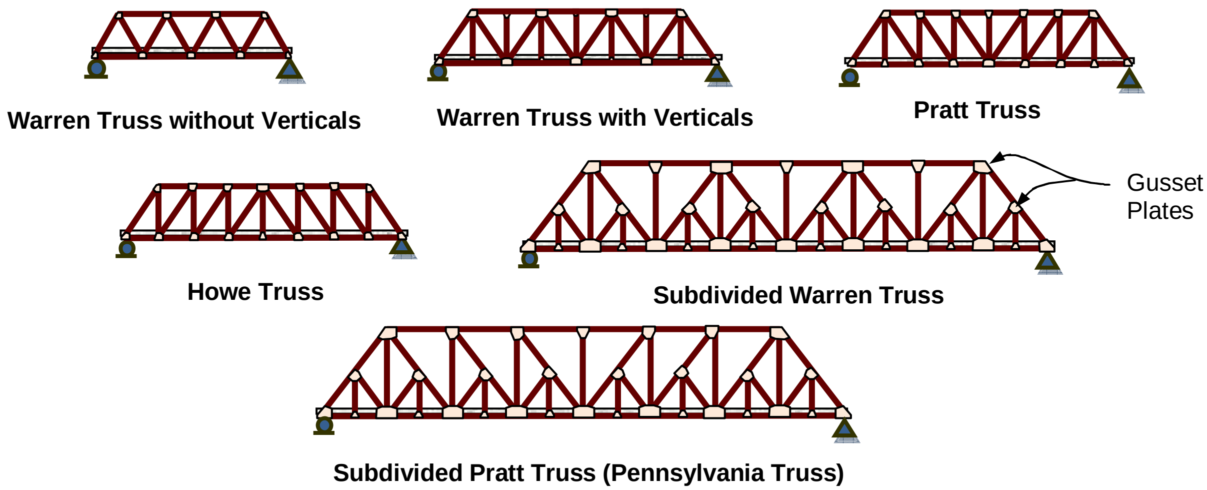


Figure 2.2 Common types of steel trusses used in bridges

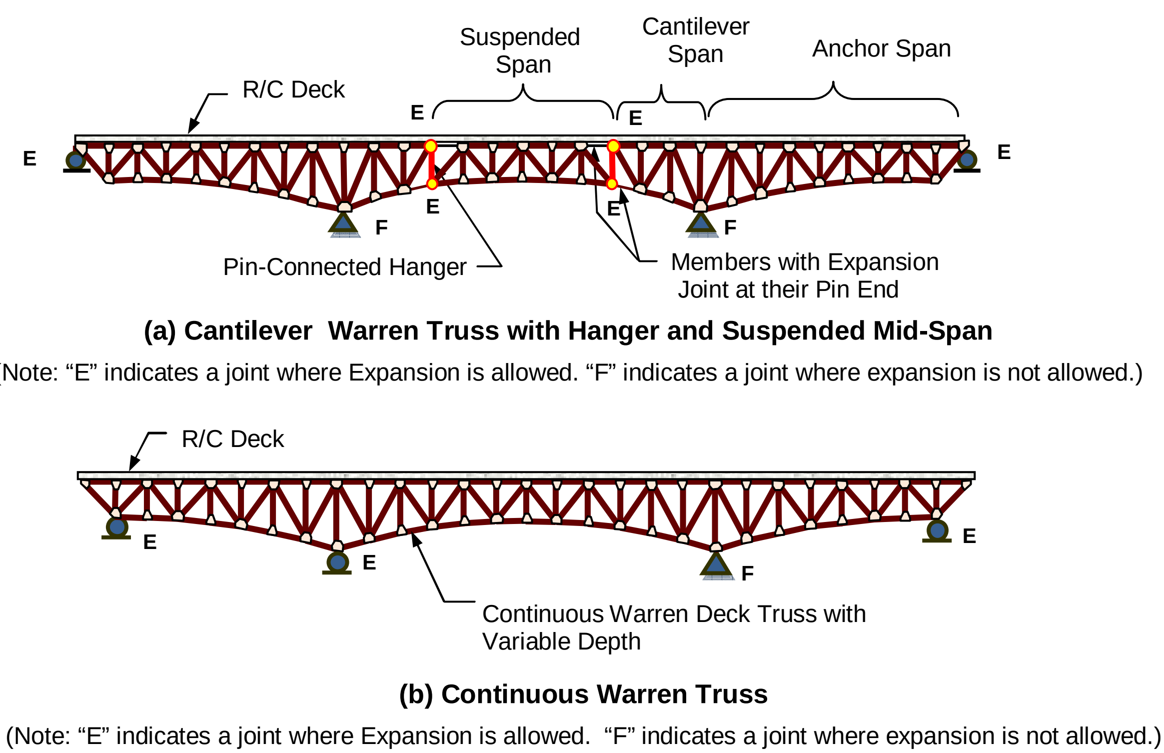
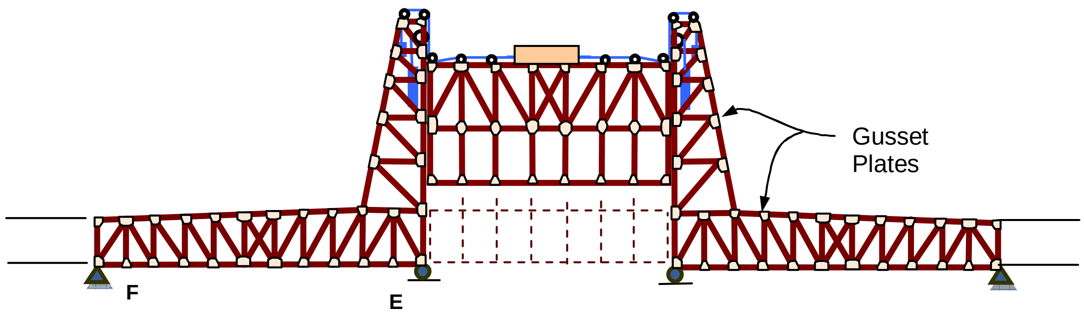
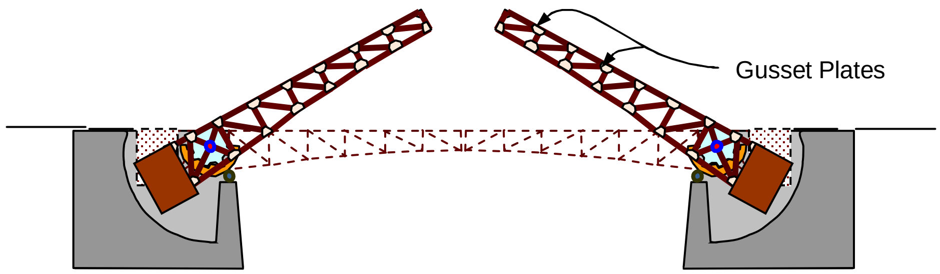


Figure 2.3 Examples of cantilever (top) and continuous (bottom) Warren bridge deck trusses



Waddell Lift Bridge

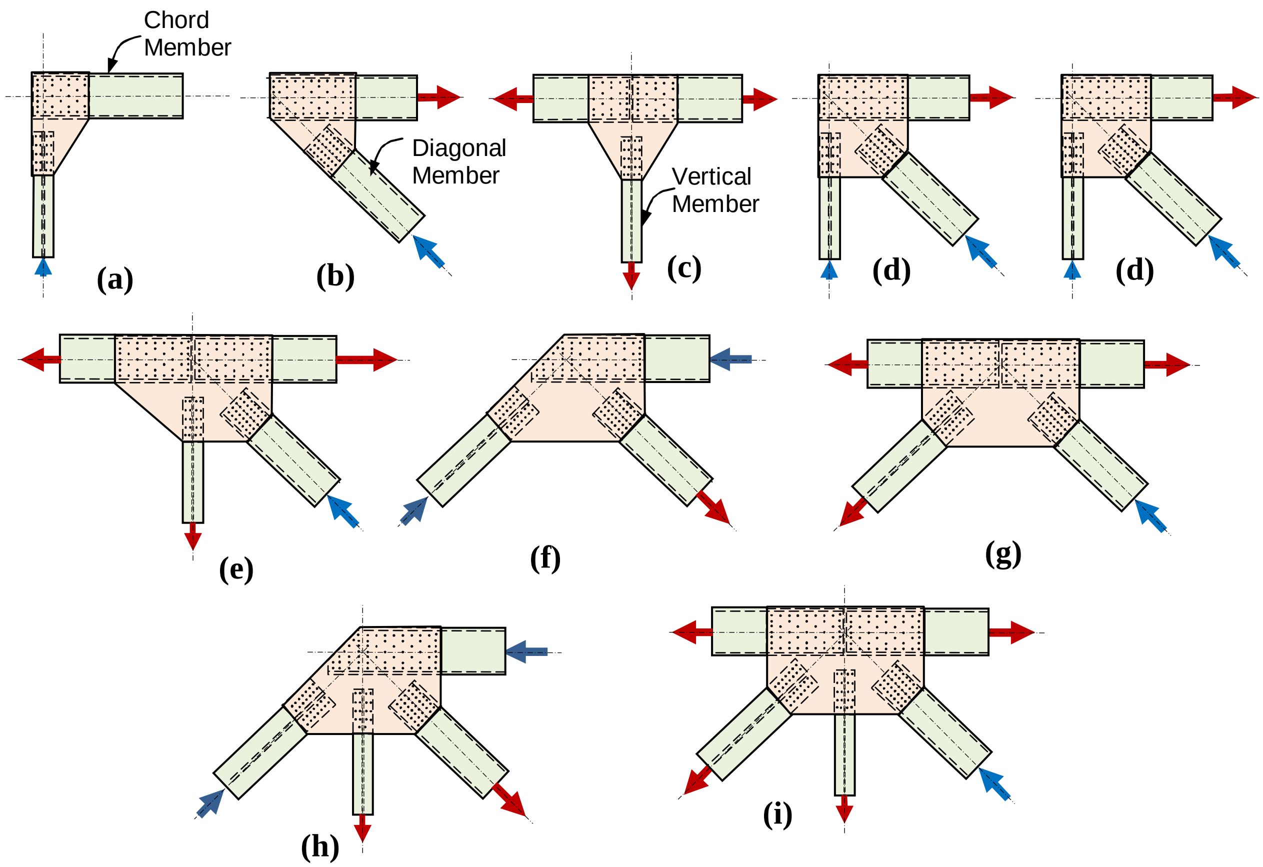


Chicago Type Bascule Bridge

Figure 2.4. Lift and *Bascule* truss bridges

Figure 2.5 shows typical gusset plate connections used in main trusses of steel truss bridges. All these details have double gusset plates. Figures 2.5(a), 2.5(b), and 2.5(c) show gusset plates with one vertical or diagonal web member attached to the chord. Figures 2.5(d) through 2.5(g) show two web members connected to the chord, and Figures 2.5(h) and 2.5(i) show gusset plate connections where three web members are connected to the chord. In some long span bridges such as the cantilever truss span of the San Francisco–Oakland Bay Bridge, four or more members are attached to one gusset plate, but these cases are relatively rare. The sketches of gusset plates in Figure 2.5 are shown for the upper chord nodes. For the bottom chord, the gusset plate details will be similar but vertically rotated. Similar details are found in old truss bridges but with the actual pins present in the connection as discussed in the following sections.

In most cases, gusset plates for the lateral bracing system are double gussets. In relatively rare cases of short span bridges, or when welds were used as connectors, we find single gusset plates. Unlike bridges, most gusset plates used in the lateral bracing systems in buildings are single gusset plates. A report on the gusset plates in buildings by the author (Astaneh-Asl, 1998) provides information on the behavior and design of gusset plates in braced frames in buildings. A report by Chambers and Ernst (2005) provides summaries of research results on gusset plates. The current report primarily focuses on riveted and bolted double gusset plates used in main trusses, steel braced towers as well as lateral bracing systems of steel truss bridges.



Note: The direction of forces shown in above details are for emphasis of the axial loads being the major force and do not necessarily represent general direction of forces in any detail.

Figure 2.5 Common types of gusset plates in steel bridge trusses

2.2 A LOOK AT THE HISTORY OF STEEL TRUSS BRIDGES AND THE EVOLUTION OF GUSSET PLATES

A review of the literature and information on the existing steel truss bridges in the United States, Canada, and elsewhere indicates that gusset plates have been used in riveted, bolted, and welded steel trusses as well as steel towers of bridges since the early 1800's. In early applications, gusset plates were used primarily in connecting the lateral bracing members to each other and to the main gravity load carrying trusses. The gravity load carrying main trusses of these early wrought iron or steel bridges, which mostly were for railroads, were pin-connected with no gusset plates in main trusses. Figure 2.6 shows Newport Southbank Bridge (also known as Purple People Bridge) on the Ohio River built in 1872. This bridge originally was a railroad and auto bridge and then in 2003 became a pedestrian only bridge(www.historicalbridge.org). The bridge is a good example of a major and multi-span riveted steel truss bridge of the late 19th century with actual pins in its truss connections and no major gusset plate used in its main gravity load carrying trusses.

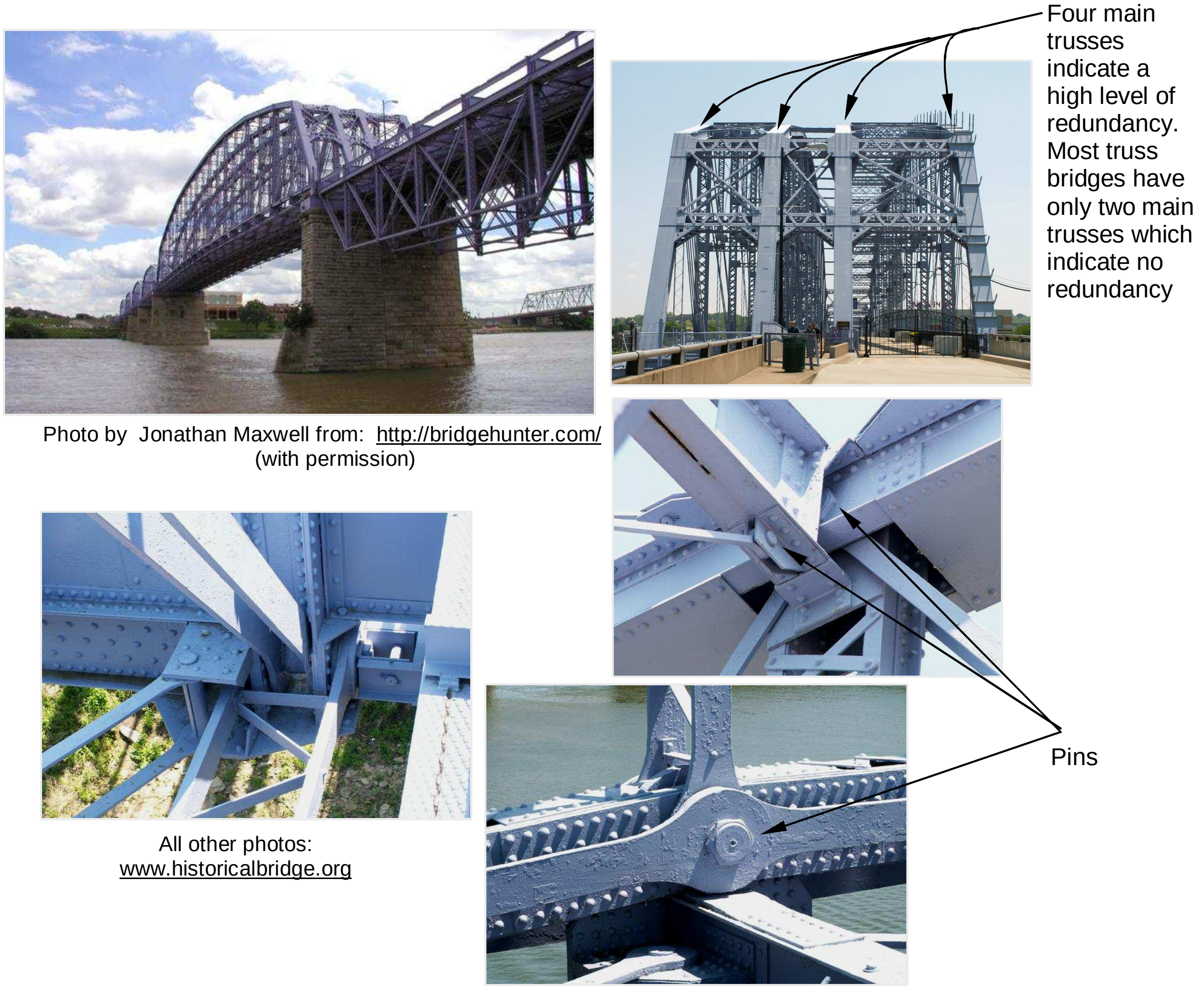


Figure 2.6 The 1872 riveted Newport Bridge in Ohio which has pin-connected truss joints

Figure 2.7 shows the Chagrin River Iron Bridge in Ohio built in 1881, in which the bottom chord and the web vertical and diagonal members are eye-bars with no gusset plate connections. Figure 2.8 shows the Twin Creek Bridge at Sonora Road (TR-455) in Ohio, built in 1902 with compression verticals and compression chords that are built-up members and tension diagonals and a bottom tension chord that are eye-bar. Still no significant gusset plate appears in this bridge. The Rommels Mill Bridge over the Clear Fork Mohican River at Benedict Road in Ohio, built in 1913, Figure 2.9, is a Warren type riveted, through- truss bridge. The bottom chords of the trusses of this bridge are built-up members instead of the eye-bars used in earlier steel truss bridges. Notice the small gusset plates connecting the web members of the truss to the top and bottom chords. In late 1800s and early 1900s gusset plates started to be used more often. Prior to this, when the tension members of steel trusses were generally made of eye-bar members, the connections were what we call today a *true pin* where an actual pin connects the web members to the truss chords. An example of this type of connection is shown in Figure 2.10.

Photo by Rick McOmber from http://oldohiobridges.com

Figure 2.7 The 1881Chagrin River Bridge

Photo by Nathan Holth from http://oldohiobridges.com

Figure 2.8 The 1902 Twin Creek Bridge

Photo by Ronald Jones at http://oldbridges.blogspot.com

Figure 2.9 The 1913 Rommels Mill Bridge

Photo from www.Fleckr.com

Figure 2.10 A "pin" connection on the compression chord of an old truss

During the late 1800s and early 1900s, gradually the actual pin connections in steel bridge trusses were replaced with similar-looking connections with members connected to gusset plates, but no longer were gusset plates connected to a pin. Instead, all members were connected to a pair of gusset plates in a way that still all centerlines of the members passed through a single work point. This was done to ensure that the members primarily had axial load in them. Of course, these gusset plates without the actual pin did not act as a perfect pin preventing development of bending moments in the members. As a result of rigidity introduced by the gusset plates in this new generation of *pin less* steel truss bridges, truss members were subjected not only to the axial forces but also to the bending moments as well. The moments created stresses not only in the members but in the gusset plates as well. More on secondary stresses in trusses, especially in gusset plates, is given later in this chapter. Today, with the availability of computer-based structural analysis software, one can easily model a truss with the ends of the members rigidly connected to each other and establish the axial load and bending moment in each member of a truss. But until the 1960s and the development of computer analysis software, truss analysis was still done assuming the truss gusset plate connections to act as pin connections, even though they were not. Then, by applying the method of joints or the method of sections to analyze the pin-connected truss the axial loads in the members were established.

During the 1920s and 1930s the use of eye-bars in steel truss bridges gradually decreased and the eye-bars were used only in tension diagonals and in some cases as the tension bottom chords of short and medium span trusses. Figure 2.11 shows views of the trusses of the San Francisco–Oakland Bay Bridge, completed in 1936, where gusset plates are extensively used in short (300-foot) simply supported, medium (500-foot) simply supported, and long (1400-foot main span) cantilever span riveted steel trusses. The bridge is one of the best examples of riveted steel truss bridges using a variety of trusses and steel braced piers with gusset plates. The bridge had many innovations implemented in it including development of a special type of mild carbon steel which later became the now well known ASTM A36 steel used in the majority of steel bridges, buildings and other structures since its development for the Bay Bridge. The concrete pump, which also is used in so many construction projects, was also invented for the construction of the Bay Bridge (CPWD, 1934-1939).

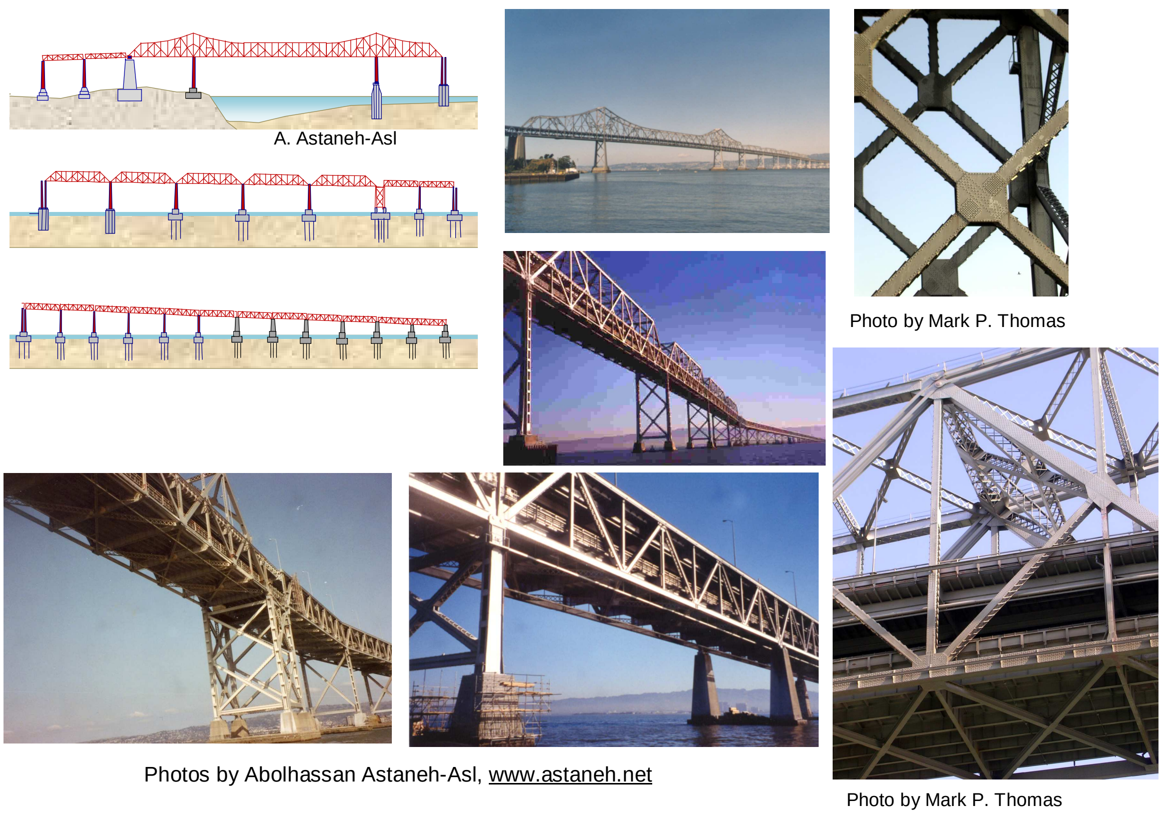


Figure 2.11. The San Francisco–Oakland Bay Bridge and its trusses and gusset plates

Figure 2.12 shows the 1968 Carquinez Bridge crossing the Carquinez Strait in the San Francisco Bay. The bridge is a very good representative of a steel through-truss bridge supported on the steel braced towers with almost all connections of the trusses and towers as well as the lateral bracing system being bolted gusset plates. The members in this bridge are welded I-shape or welded perforated box sections. The bridge was one of the first major bridges where the newly developed COR-TEN high strength quenched and tempered steel was used. More on the material used in steel truss bridges and their gusset plates is given later in Chapter 3. During the 1990s, extensive seismic retrofit was designed and implemented for this bridge including seismic retrofit of its truss members and gusset plate connections (Astaneh-Asl 1996) and (ICEC and Astaneh-Asl 1996).

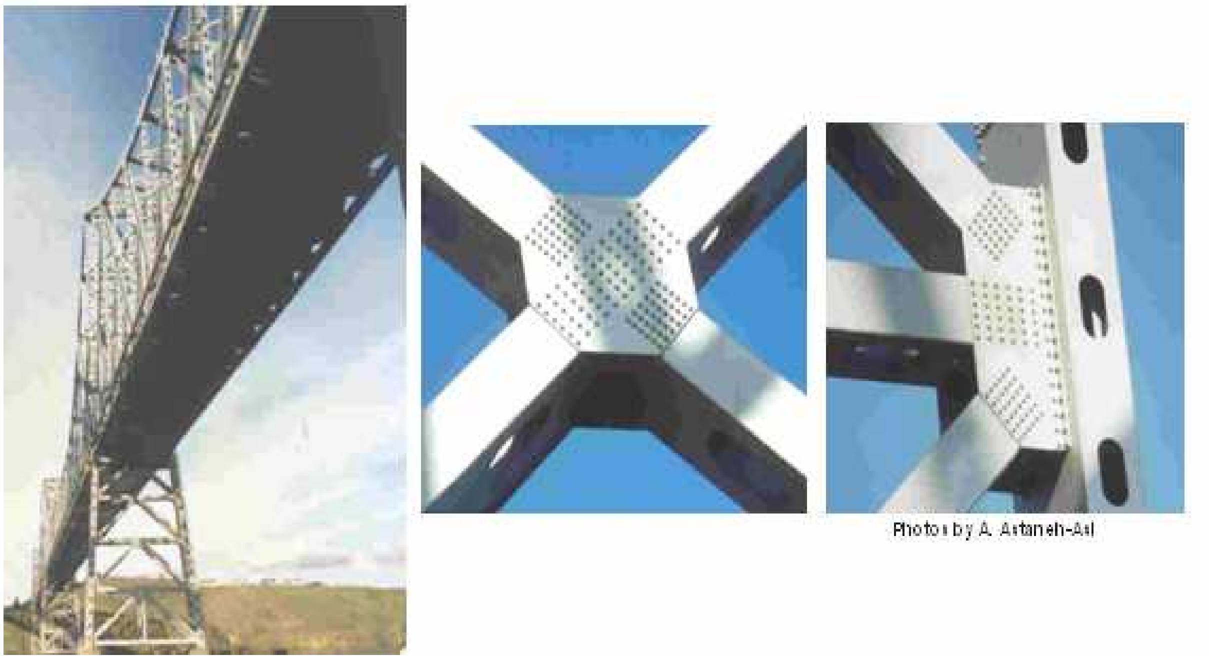


Figure 2.12. The 1968 Carquinez Bridge in California and its typical bolted gusset plates

2.3 REDUNDANCY IN TRUSSES AND *FRACTURE CRITICAL TRUSSES*

A historical item relevant to the discussion of steel trusses and their gusset plates is the issue of *redundancy* and *determinacy* of the steel trusses used in bridges. As mentioned above, the early steel bridge trusses had actual pin connections in their joints. In addition, to be able to analyze them using only a slide-rule, the trusses had to be determinate systems so that, using only the equations of equilibrium of forces at the joints, the axial forces in the members could easily and accurately be calculated. Being determinate systems, these steel trusses became what today we call *fracture critical* system. In a fracture critical system, fracture of a single member or connection results in progressive collapse of the entire span, as what tragically happened in the case of the I-35W bridge collapse in Minnesota in 2007 and in the collapse of the Point Pleasant/Silver Bridge 40 years earlier.

In fact, in fracture critical bridges, it is not necessary to have *fracture* of a tension member or a connection to initiate total collapse. Buckling failure of a single compression member or compression areas of a critical connection, such as a gusset plate of main trusses, can also cause total or partial collapse of a steel truss span. Therefore, instead of calling these non-redundant systems *fracture critical*, we might call them *failure critical* implying that the failure of one member or connection in these systems can result in progressive collapse of a large portion of the structure even the entire span.

There are three types of redundancy in bridge structures (NCHRP, 2005):

a. Internal Redundancy

b. Structural Redundancy

c. Load Path Redundancy

The following three sections discuss these redundancies in steel bridge trusses.

2.3.a Internal Redundancy of Bridge Trusses

According to the NCHRP (2005), "Internal redundancy, also called member redundancy, exists when a member comprised of multiple elements and a fracture that formed in one element cannot propagate directly into the adjacent elements". Again, as mentioned earlier, to have a more general definition, it might be necessary that in the above definition we replace the word *fracture*, which is associated with *tensile stresses* by the word *failure*, which is associated with both tensional stresses (fracture failure) and compressive stresses (buckling failure).

Steel bridges built prior to 1950's were riveted. The member sections in these bridges had several separate elements carrying the load in *parallel*. An example of a multiple load path cross section is a riveted built up truss members made of a number of angles and plates where if a single angle fractures due to fatigue or corrosion, the other elements of the cross section would carry the load of the lost element. Another example of members with multiple load path cross sections was multiple eye-bars used as single tension members. An example of non-redundant cross section is a single rolled wide flange shape where if a fatigue crack develops across the single section, it will propagate and result in total fracture of the cross section and the loss of member making these members *fracture critical* sections. Figure 2.13 shows examples of truss multi load path members made of a number of separate elements such as eye bars or angles and plates.

Figure 2.13 "Multi-element" riveted members used in riveted truss bridges in the past

The above discussion on member redundancy would also apply to some extent to the truss connections. The connections in most of the U.S. truss bridges are either pin connections or riveted or bolted double gusset plate connections. The pin-connected truss connections having a single pin do not possess internal redundancy since failure of the pin will result in failure of the connection which can lead to progressive collapse of the truss if the failed connection was a connection in the *primary* gravity load carrying system.

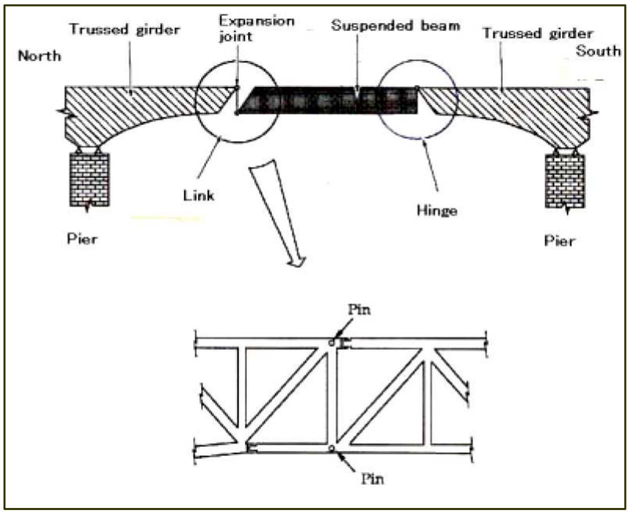
On the other hand, in the riveted or bolted gusset plate connections, the rivets and bolt groups that connect the members to the gusset plates, having multiple rivets or bolts, present multiple load paths and a high degree of internal redundancy where when a rivet or a bolt has failed, its load can be carried by others in the group.

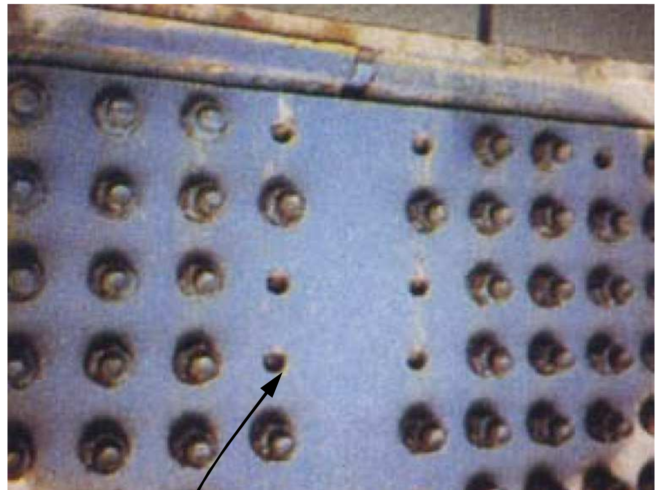
The tragic 1994 collapse of the suspended span of the Seougsu truss bridge in South Korea, Figure 2.14 which caused the death of 32 people, was blamed on initiation and propagation of fatigue cracks in the partially penetrated welds in the connection of suspended span to the end of the cantilever arm. The post failure investigation of this bridge indicated that the quality of construction was not as good as expected and as shown in Figure 2.15 some of the bolts in bolted splices were not installed in 1979 when the bridge was erected. Although this case of failure is related to poorly done and not properly inspected welds and does not represent the performance of properly done welded connections in trusses, nevertheless, as mentioned earlier, rivet and bolt groups because of their high degree of internal redundancy can be quite forgiving and would not fail if some rivets or bolts are missing or fatigue cracks develops in the elements of the connections. The loads carried by missing rivets or bolts , if they are only a few, can be carried by the remaining bolts in the group and the fatigue cracks can be arrested by the rivet or bolt holes.

In other words, riveted and bolted connections provide bridge owners with the opportunity to discover the missing, failed or corroded bolts during routine inspections and replace them. It is quite fortunate that most existing truss bridges in the U.S. are riveted or bolted.

Photo: Courtesy of Prof. Yozo Fujino

Figure 2.14 The collapse of the suspended span of the Seougsu Bridge in Seoul, S. Korea in 1994





Above sketch, photo at the right and information on collapse by Masahito Kunishama at: http://shippai.jst.go.jp/en/Detail?fn=2&id=C

Missing bolts in splices of the Seougsu bridge

Figure 2.15. The failure of Seougsu Bridge in Seoul was blamed on failure of welded connections of the cantilever arm to the suspended span and low quality construction and inspection.

Gusset plate connections in main trusses of bridges almost always have double gusset plates. As a result, these connections have two separate load path that in case of fracture or loss of thickness of one gusset, the other gusset, if intact can take the burden off the weakened gusset until the connection weakness is detected during inspection and remedied. In some of the old riveted bridges, built during the period of 1850-1930s, we find multiple plate gusset plates where each one of the two gusset plates is itself made of two or more plates as shown in Figure 2.16.

Of course having several thinner plates riveted to each other to have the required thickness of gusset instead of using a single plate with required thickness adds to the redundancy of the connection and creates additional and separate load paths similar to the case of built-up riveted members made of multiple elements as discussed in previous section. Since the 1960s and availability of thicker rolled plates as well as higher labor cost involved in drilling bolt holes in multiple plates instead of one plate, single plates are used for gusset plates in the steel trusses of the bridges, Figure 2.16.

2.3.b Structural Redundancy

According to the NCHRP (2005), “Structural redundancy is external static indeterminacy and can occur in a two or more span continuous girder or truss”. Establishing *Structural Redundancy* for girder bridges is relatively easy and straight forward. For example considering a three span continuous girder bridge, if a fracture occurs through a cross section in the middle span, the cantilevered segments of the fractured girder will be able to carry some of its load and shed the rest to the adjacent girder(s) through internal redundancy of the deck and presence of the cross bracings. Obviously, if a side span girder fractures, the end segment cannot carry any load and will collapse. So, depending on the location of the fractured section, the structural redundancy of a continuous girder can help to prevent the collapse of the span or a segment of it. If the girder instead of being continuous over the support consisted of three simply-supported spans, a fracture of any section in any of the three spans would result in total loss of the load

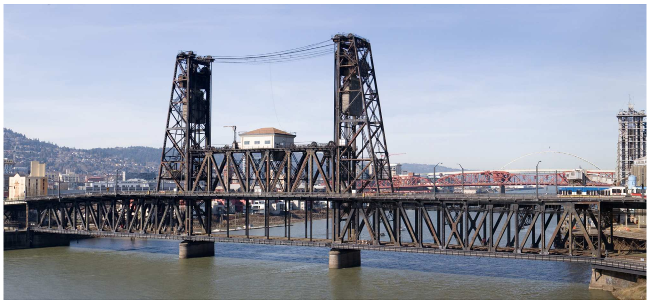
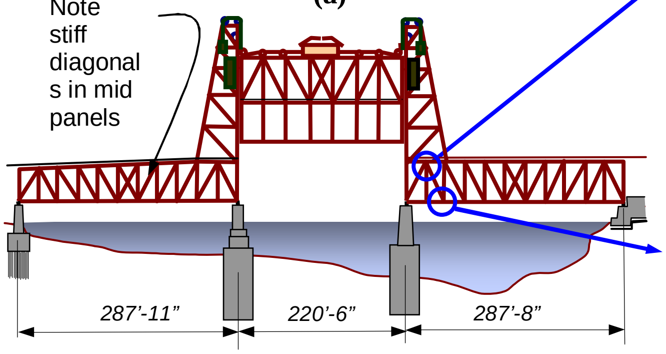


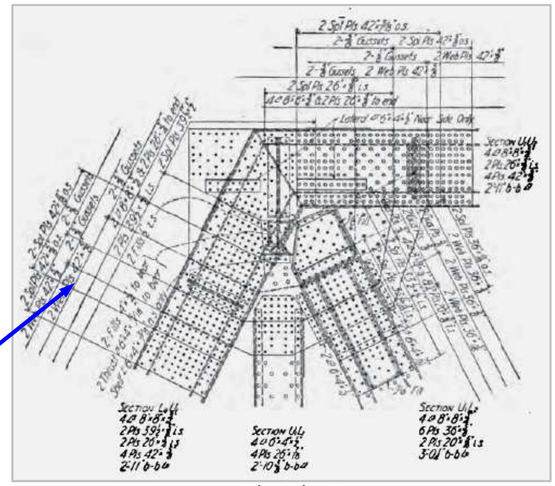
Photo from Wikipedia: http://en.wikipedia.org/wiki/Steel_Bridge

(a)

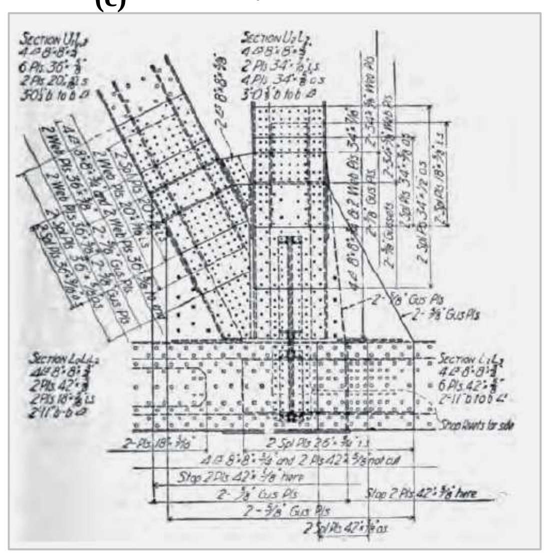


Sketch by Abolhassan Astaneh-Asl from a drawing in Ref: (Hool and Kinne, 1943)

(b)



(c) Hip Joint U_1



(d) Bottom Chord Joint L_2

The two gusset details above are from "Bridge Engineering, 1916 by J.A.L. Waddell.

Figure 2.16 The 1911 O.-W. R. R. & N. Company's Bridge over the Willamette River, Portland, Oregon, currently known as Steel Bridge, and its multi-plate gusset plates. The bridge was designed and detailed by the J.A.L. Waddell's bridge consulting firm (Currently Hardesty & Hanover, LLP) carrying capacity of the fractured , simply-supported girder. After the fracture of simply-supported girder, if there is a high degree of internal redundancy in the deck system and a presence of cross bracings between the fractured girder and the adjacent girders, the load on the collapsed girder might still be carried by the adjacent girders and the span might survive even with a total loss of one girder.

Like girders, understanding and establishing a degree of *structural redundancy* for trusses can also be relatively easy, and similar arguments are made for redundancy of continuous and simply-supported trusses..

2.3.c Load Path Redundancy

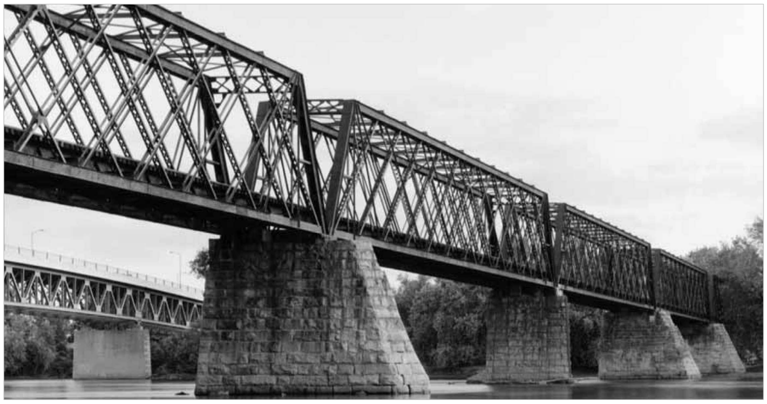
According to the NCHRP (2005), "Load-path redundancy is internal static indeterminacy arising from having three or more girders or redundant truss members. " An example of a bridge with *Load Path Redundancy* is a girder bridge with three or more girders, where if a girder fails the remaining two or more girders can take the load of the failed girder and transfer it to the substructure. If the girder bridge has only one or two girders, a failure of one girder quite likely will result in total collapse of the span.

In case of trusses, we can establish Load Path Redundancy in terms of the number of main trusses in the bridge and the number of redundant member in each truss. Similar to girder bridges, truss bridges with three or more main trusses have Load Path Redundancy while bridges with only two trusses do not. Unfortunately, truss bridges usually have only two longitudinal main trusses, therefore, such bridges do not have Load Path Redundancy as far as the number of trusses is considered.

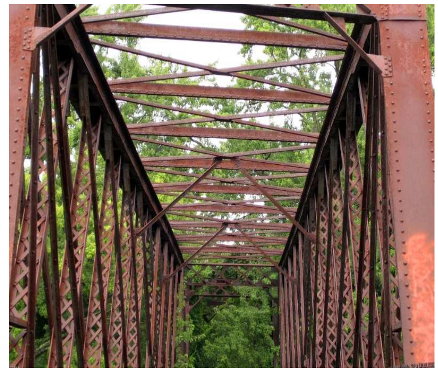
With regard to the number of redundant members in each truss, we can still find a certain level of Load Path Redundancy. However, the mere presence of redundant members in a truss does not mean that the truss has a Load Path Redundancy. First of all, the redundant members should be *primary* members of the global load path of the truss and not a local member such as a subdivision member of a subdivided truss. Secondly, the redundant members should have sufficient strength to carry the additional load transferred to them due to failure of any single member of the primary global load path. A good example of a truss with Load Path Redundancy in its web is a truss with "X" web diagonals instead of "/" diagonals. In an "X" diagonal configuration, if one diagonal fails, the other diagonal can take the load carried by the failed diagonal and transfer it to the substructure , provided that it has the extra strength to do so.

Examples of bridges with high degree of *Structural Redundancy* are shown in Figures 2.17 and 2.18. The B&M Railroad Bridge in Figure 2.17 has a high degree of indeterminacy and *load path redundancy* because of its overlapping diagonal members.

The 1888 bridge shown in Figure 2.18 was designed by George S. Morrison (1842-1903) and it was one of his many Whipple bridges built in 1880s and 1890s. Whipple trusses have a high degree of indeterminacy and load path redundancy in their webs. The superstructure of the 1888 bridge was replaced in 1916 by the structure shown in the photo on the right to enable the bridge carry heavier and faster trains. Like Whipple truss bridge that it replaced, the 1916 trusses also have structural redundancy in the web by having "X" web diagonals instead of "/" diagonals common in Warren and Pratt trusses.

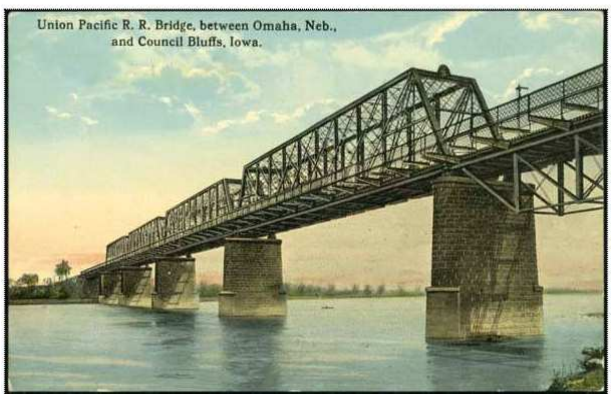


http://www.eecs.usma.edu/bridgecontest/pdfs/appenda.pdf



Above photo by: Ed Bacher athttp://1.bp.blogspot.com/

Figure 2.17. The Boston & Maine Railroad Bridge over Connecticut River, Northampton, MA



http://nebraskamemories.com/

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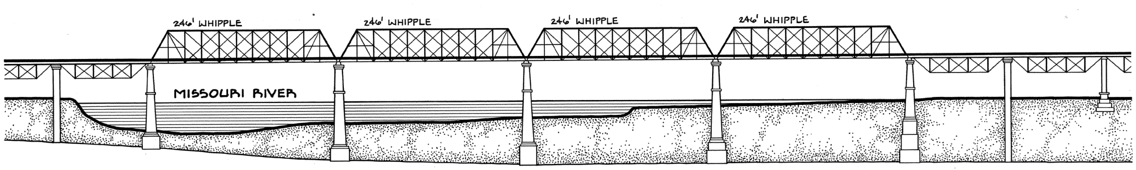


Sketch of 1888 Bridge from: http://memory.loc.gov/cgi-bin (The Library of Congress)

Figure 2.18. The 1888 (left) and the 1916 (right) Union Pacific R. R. Bridge, between Omaha, Neb., and Council Bluffs, Iowa. Both bridges had high degree of indeterminacy

One of the important and relatively recent steel truss bridges that has "X" web diagonals is the 1957 double deck highway/railroad Wuhan Yangtze River Bridge in China, Figure 2.19. The bridge was the first Yangtze River crossing making it a major transportation link between northern and southern China (travelchinaguide.com).

http://commons.wikimedia.org/

www.chinapage.com/bridge/shanghai/lupu/lupu.htm

Figure 2.19. The "X" diagonals of the 1957 First Wuhan Bridge over Yangtze River in China, a double-deck highway/railroad bridge creates high level of structural redundancy in the bridge.

It should be mentioned that out of the three redundancies in bridges; the Internal , the Structural and the Load Path Redundancies, the latter is the only one that the current bridge design and evaluation specifications (AASHTO, 2007) and (AASHTO, 2003) recommend to be considered while the two other redundancies, in a conservative move , are to be ignored. Therefore, due to the importance of Load Path Redundancy in design and evaluation of bridges, especially in truss bridges, I have provided some historical background and a discussion of Load Path Redundancy in steel truss bridges in the following section with a focus on the existing truss bridges.

2.3.d Historical Background on Redundancy in Steel Bridges

Early bridge design specifications published in 1800's, which were mostly for railroad bridges, and textbooks and manuals on bridge engineering at the time state that no redundant members should be used in steel bridge trusses. For example, John Alexander Low Waddell, in his *De Pontibus—A Pocket Book for Bridge Engineers* (Waddell, 1898), which became one of the most important bridge design manuals for decades, states: "All trusses shall be so designed as to admit of accurate calculation of all stresses, expecting only such unimportant cases of ambiguity as that involved by using two stiff diagonals in a middle panel" , see Figure 2.16(b) given earlier. Of course, the trusses at that time, being generally simply-supported, would have the force in the middle diagonal zero, since shear was zero at the mid-span. Having one or two diagonals at this middle panel would not make the truss indeterminate, since both diagonals had zero force. Waddell (1898) has a recommendation against making the trusses continuous over the points of support, allowing exception only for swing bridges and cantilever bridges where the bridge truss could be continuous over the supports. Such cantilever bridges were still determinate systems since the pin-and-hanger mechanism at the end of the cantilever arms to hang the suspended spans from, see Figure 2.3(a) given earlier in this chapter, made these bridges determinate system.

The issue of redundancy of trusses was the subject of sometimes heated debate during late 1800s in the bridge engineering community. J.A. L. Waddell, being one of the most famous and prominent bridge engineers of the day was on the side of avoiding redundancy and making trusses determinate. His main argument was that if trusses are indeterminate, with redundant members, the stresses in the members cannot be calculated in a precise manner and there would be *ambiguity* in stress calculation. The word *ambiguity* used by Waddell in his publications later found its way into the early editions of the AASHO Specifications. In his *De Pontibus* book (Waddell, 1898) as well as Bridge Engineering (Waddell, 1916); Waddell has a Chapter on *First Principles of Designing*. I have included Waddell's principles as Attachment to this report since in my view with the exception of two principles all of Waddell's principles are still amazingly valid as they were more than 110 years ago! The two Principles XIV (14) and XV (15) see below, which relate directly to redundancy, may not be as sound and valid today as they were more than a century ago only because of the advancement made in computational methods and the use of computers in analyzing indeterminate structures.

PRINCIPLE XIV.

The best modern practice in bridge-engineering does not countenance the building of structures having more than a single system of cancellation, except in lateral systems where the resulting ambiguity of stress distribution is of minor importance.

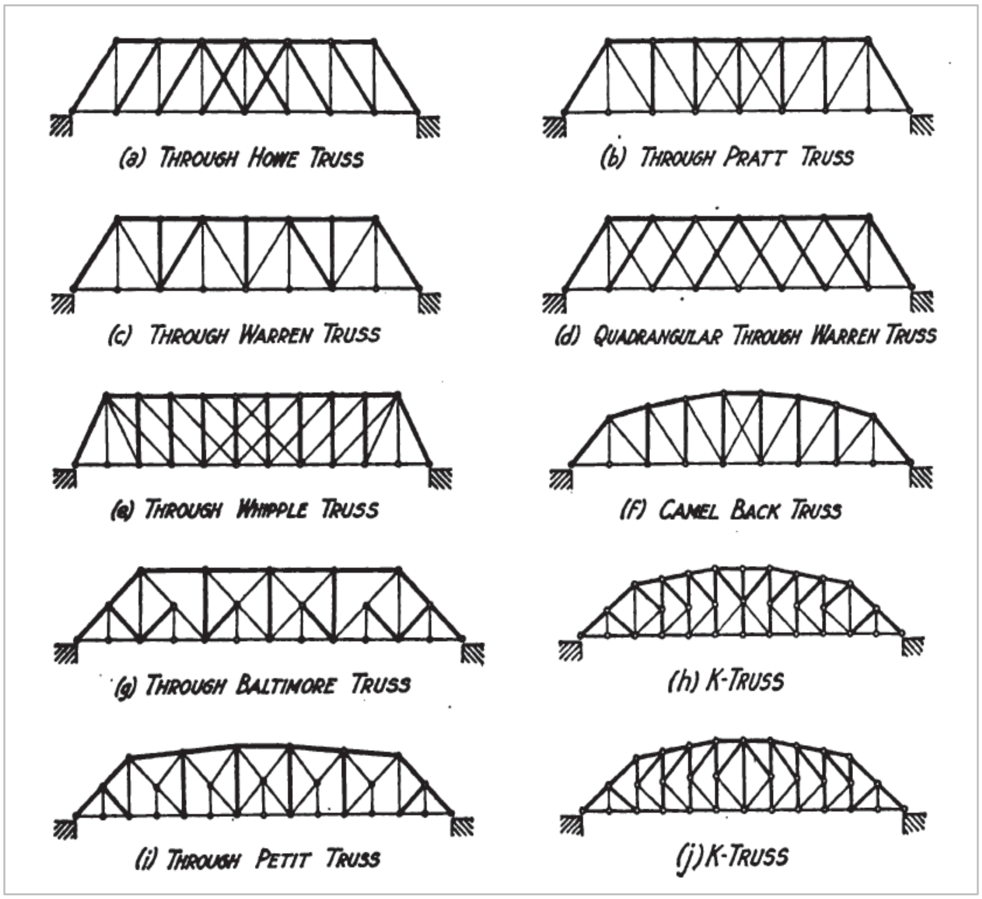
PRINCIPLE XV.

The employment of a redundant member in a truss or girder is never allowable under any circumstances, unless it be in the mid-panel of a span having an odd number of panels, in which case, for the sake of appearance, two stiff diagonals can be used.

Excerpts from Waddell (1898)

The strong wording of the above two principles by J.A.L. Waddell against the use of redundant members in bridge trusses, especially in Principle XV above, can also be found in his other writings of the time. It appears that there were certain truss types promoted by other bridge engineers that had redundant members, albeit only in the web portion of the trusses, and Waddell was a very strong and respectable force against the use of such redundant members. If we consider the analysis tools available to bridge engineers at the time and the type of members and connections that they were using in bridge trusses, his opposition to the use of redundant members makes every sense. His main concern was that the stresses (i.e. axial forces) in the members of an indeterminate truss could not be accurately and easily established considering the fact that analysis of indeterminate structures at the time was in its infancy and had to be done either by simplifying, and somewhat approximate assumptions or by using time consuming but more accurate methods based on using deflections. Another concern Waddell expresses in his writings was that the secondary stresses in indeterminate structures with redundant members will be larger and harder to establish accurately than the secondary stresses in determinate trusses.

During the period of late 19th and early 20th century when Waddell was opposing redundancy in trusses, some bridge engineers were developing, patenting and using steel truss configurations that were indeterminate. A good example of indeterminate trusses was the Whipple truss system shown in Figure 2.20(e) below among other types of common truss systems (Ketchum, 1920). Professor Milos Ketchum (1920) provides a numerical example of calculation of member forces for indeterminate Whipple trusses such as those shown in Figure 2.18 above. Albeit being approximate and on the safe side, the method can solve this indeterminate system using only equations of equilibrium. For more exact solution, the engineers would use the least work theorem also discussed in Ketchum (1920).



Drawings from Ketchum (1920)

Figure 2.20 Examples of truss configurations in riveted bridges of the 19th and 20th centuries

Considering the fact that Waddell and his fellow bridge engineers until 1960's had only slide rule to calculate forces and deflections and no easy way of solving multiple simultaneous equations, as we do today using computer-based structural analysis software, Waddell's concerns regarding indeterminate structures seem to be quite valid. However, with today's availability of structural analysis tools both primary stresses and forces as well as secondary stresses can be established accurately and it seems that the issue of redundancy in trusses needs to be revisited to avoid creating determinate and fracture critical truss structures. The issue becomes quite important if we consider blast resistance of bridges in this age of terrorist car bomb threats. Do we really want to construct determinate and fracture critical structures that if one member or one connection of the structure failed due to natural or man-made (blast or fire) hazards the entire span collapses?

One of the most important issues regarding design of trusses is the issue of redundancy in trusses. The first and second editions of the AASHO specifications (AASHO, 1931 and 1935) under Section 6-Structural Steel Design state that: "Structures shall be so designed as to avoid, as far as practicable, ambiguity in the determination of the stresses". In order to satisfy the statement, one had to avoid indeterminate trusses with redundant members and design trusses that can be easily and *accurately* analyzed to determine the stresses in the truss members. The statement does not appear in the 1941 edition (3rd Edition) of the AASHO and editions after that. However, as discussed in Chapter 2, this strong and clear statement against using indeterminate trusses, which in those days could not be accurately analyzed, may have been the reason why even now , the steel trusses are designed to be mostly determinate structures with a few exceptions such as cantilever and continuous trusses which have redundant members. Any determinate structure, such as non-redundant trusses, even though is easy to analyze by just using equations of equilibrium, because of being determinate is *fracture critical* as well. This means that, in these determinate and non-redundant fracture critical structures, failure of only one component of the superstructure, whether a member or a connection, in many cases, can result in total collapse of the span. This was the case in tragic collapse of the entire length of the I-35W steel truss bridge in Minneapolis in 2007 due to failure of gusset plates at one location (Astaneh-Asl, 2007), (NTSB, 2008).

The above statement regarding avoiding the ambiguity of stress calculation is still valid, but, today having computer based structural analysis software at our disposal we do not need to limit ourselves to designing structures such that they are determinate or have low degree of indeterminacy with only a few redundant members. In fact, to avoid catastrophic failure of determinate structures, specially in bridges, not only we should avoid designing determinate and by their nature, *fracture critical* bridge trusses, but in the interest of public safety we need to undertake a program of evaluating our inventory of steel truss bridges and find out how many of these structures are fracture critical needing retrofit or replacement to avoid their catastrophic and progressive failure due to failure of a member or a gusset plate.

In addition, the definition of redundancy divided into three categories of *Internal*, *Structural* and *Load Path* redundancies may need revisiting. It seems to me that what is important is preventing progressive and catastrophic collapse of bridge spans due to loss of a single member or connection as have occurred in several catastrophic collapses of bridges including the Yarmouth Bridge in 1845 with 79 deaths, the Ashtabula Bridge in 1876 with 83 deaths, the Tay Bridge in 1879 with 75 deaths, the Lundorff Bridge in 1945 with 28 U.S. soldiers killed, the Second Narrows Bridge in 1958 with 19 deaths, the Silver Bridge in 1967 with 46 deaths, the Seongsu Bridge in 1994 with 32 deaths and the 35W Bridge collapse in 2007 with 13 deaths.

Avoiding progressive and catastrophic collapse of bridge spans that can cause loss of lives and injuries to many is fulfillment of our primary design obligations providing safety to public as stated in all AASHTO Specifications since the 1st. Edition including the current 2007 edition:

> **2.5. DESIGN OBJECTIVES**
>
> **2.5.1 Safety**
>
> The primary responsibility of the Engineer shall be providing for the safety of the public.
>
> (Excerpts from AASHTO LRFD Bridge Design Specifications(2007), See footnote "*" below)

It must be noted that having high degree of redundancy, although helpful, does not necessarily guarantee that the bridge does not undergo *progressive collapse*. In order to prevent progressive collapse of a structure, including bridge trusses, we need to verify that if any element of a bridge such as a single member or a single connection fails, the load carried by the failed element can be transferred to the neighboring elements and those elements as well as all other elements on the load path, all the way to the ground, are capable of carrying the transferred load without failure. A practical way to verify this behavior in a truss is to conduct *progressive collapse analysis* and remove one member or one connection at a time and observe where the load of failed member or connection is transferred and whether or not the remaining structure is capable of carrying the load of the lost member or connection without a collapse. If the structure is capable of such transfer of load of the lost member, failure will remain local and progressive collapse will be prevented. Of course for this type of analysis factor of safety can be smaller than the regular design. For LRFD applications, the resistance factors (ϕ) values can be larger than the values for ordinary design. Progressive collapse analysis is done quite often for critical buildings, especially in cases of loss of a member or a connection due to blast effects of explosive attacks. There are also quite viable technologies that are developed for use in buildings to prevent their progressive collapse in the event of removal of a member by blast. Those can also be used to prevent progressive collapse of steel truss bridges in the event of loss of a single member or connection. One example of such technologies is the use of catenary cables in the floors of buildings to prevent collapse of floors in the event of loss of a column or a beam span due to blast. This technology can be easily and efficiently used in truss bridges. We tested the performance of the system at the University of California Berkeley which proved to be quite effective as well as economical. For more information, see Astaneh-Asl, et al. (2001a) and (2001b), and Tan and Astaneh-Asl (2003).

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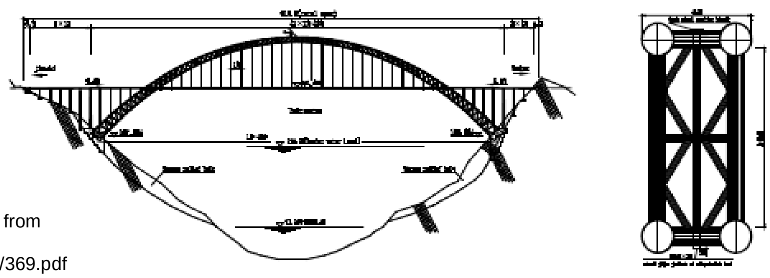
2.3.e A Modern Truss System with *Very High* Redundancy

One of the best examples of a truss bridge with very high degree of *Internal, Structural*, and Load Path Redundancy is the 2005 Wuxia Yangtze River Bridge, Figure 2.21, located in Wushan County, China upstream from the Three Gorges Dam. The bridge is currently the longest CFST (Concrete Filled Steel Tube) composite truss arch bridge in the world.



All photos and drawings from http://www.arch-bridges.cn/conf2008/pdf/369.pdf

(a) General Layout (b) Cross section of rib

Figure 2.21. The 2005 Wuxia Yangtze River Bridge in China is a modern half-through truss arch bridge with a very high degree of redundancy (Photo and drawing from Moi, 2008)

The arch ribs in this bridge are made of four concrete filled steel tubes (CFST) , as shown in Figure 2.21, from Mou (2008). The concrete-filled tubes have internal redundancy since steel and concrete share the compressive load in parallel. A loss of one element (i.e. steel or concrete) will result in shedding the load to the other parallel element. Having four tubes at the corners of the rib also provides internal redundancy since if one concrete –filled steel tube fails, the other three still can carry the load of the lost tube and the rib will not fail. In effect, the arch of this bridge is made of four parallel arch trusses, two on each side. This arrangement of course creates very high degree of redundancy and parallel load paths. There is “K” and “X” shaped lateral cross bracings between the concrete filled steel tubes in the rib. The bracing members are hollow steel tubes (Mou, 2008). In order for multiple member ribs to perform with high degree of redundancy, the elements should be connected to each other so the load of the failed element is transferred to the others. In this case, as Figure 2.21(b) above shows, the four corner elements of the arch truss ribs are properly connected to each other with multiple lateral bracing systems.

2.4. STRENGTH OF CONNECTIONS IN TRUSSES

Gusset plate connections of trusses have two distinct parts shown in Figure 2.22:

a. Connection of members to gusset plates consisting of rivets, bolts or welds with occasional use of lug angles to connect the member to gusset plate

b. Gusset plate itself

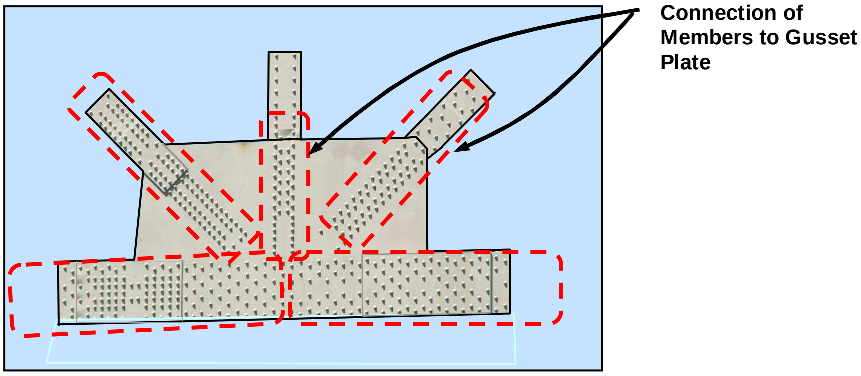


Figure 2.22. Connections of Member to Gusset Plate are shown with dashed lines

2.4.a Strength of Connections of Members to Gusset Plates

The early bridge engineering specifications and textbooks had provisions that would result in connections designed to be stronger than the members. If main members had an excess of cross section above that called for by the greatest combination of stresses, the entire connection detailing was to be proportioned to correspond to the utmost working capacity and not merely for the greatest total stress to which the member was subjected (Waddell, 1898). In calculating the working capacity Waddell (1898) warns not to forget to include what we call today the *shear lag effect* in calculating the effective net areas of tension members such as single angles, connected by some but not all their cross section. This is the “U” factor that today we use in design of axially loaded steel tension members for limit state of fracture of net area.

The above philosophy of design in a way being a *capacity design philosophy* later found its way to the AASHTO specifications, where as early as the 1924 Standard Specification for Steel Highway Bridges (AASHO 1924), Article 26 under Gusset Plates, in discussing the design of rivets connecting members to the gusset plates, states: “… rivets connecting each member shall be located, as nearly as practicable, symmetrically with the axis of the member. However, the full development of the member shall be given due consideration (AASHO 1924)”. The current AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) has essentially the same statement but uses word *fasteners* instead of *rivets* and states that: “The fasteners connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the full development of the elements of the member should be given consideration” (AASTHO, 2007). Therefore, still in design of connections of members to gusset plates, a consideration is given to design them for strength of the connected member and not for the forces in the member.

2.4.b Strength of Gusset Plate Itself

Let us now examine the very important issue of *Strength of Connection* and try to establish a rational method to be used in establishing forces that should be used in design of gusset plate connections. In other words, let us try to answer the question of what should we use as the member forces in design and evaluation of a gusset plate, *member forces due to the applied loads* or *member capacities?*

Since early days of the steel truss bridge design, the connections of trusses were designed to be at least as strong as the members connected to them. One of the early publications on design of steel bridges is the Manual of American Railway Association (AREA, 1921). Under *Strength of Connections* it states that:

> **Strength of Connections.**
> 57. Connections shall have a strength at least equal to that of the members connected, regardless of the computed stress. Connections shall be made, as nearly as practicable, symmetrical about the axis of the members.
> (Excerpt from AREA(1921))

J.A.L. Waddell in his first major book on bridge engineering: *De Pontibus* (Waddell, 1898) states that:

> In all main members having an excess of section above that called for by the greatest combination of stresses, the entire detailing is to be proportioned to correspond with the utmost working capacity of the member, and not merely for the greatest total stress to which it may be subjected. In this connection, though, the reduced capacity of single angles connected by one leg only must not be forgotten.
> (Excerpt from Waddell (1898))

The above statement is clearly implying a *capacity design* philosophy for the connections where connections are designed to develop the full strength of the connected members regardless of the actual applied loads.

A survey of the literature by the author on bridge design published over the last 120 years, which included bridge design specifications published in the United States, indicated that until the 1940's, the connections in steel bridges were required to be designed to develop 100% of the capacity of the member which in terms of Allowable Stress Design methods meant that the connections were designed to have allowable load carrying capacity equal to or greater than the allowable load carrying capacity of the connected members.

The early bridge specifications required that the connections be designed for full strength of the member. For example the AASHO (1927) specification under *Strength of Connections* states that: "Unless otherwise provided all connections shall be proportioned to develop not less than the full strength of the members connected." The 1931 AASHO specifications, which was the 1st official edition, has the same provision and adds that: "Connections shall be made symmetrical about the axis of the members in so far as practicable". The 2nd Edition of the AASHO Standard Specifications (AASHO, 1935) has also the same provisions requiring that the connections develop *full strength* of the members. However, in the 3rd Edition of the AASHO

Standard Specifications (1941) the provisions on *Strength of Connections* changed to:

> .-*Strength of Connections.*
>
> Except as otherwise provided herein, connections shall be designed for the average of the calculated stress and the strength of the member, but they shall be designed for not less than 75 per cent of the strength of the member…
>
> (Excerpts from AASHO Standard Specifications (1941) see footnote " 1" below)

It seems that in 1944, the very prudent concept of designing connections of trusses to develop *full strength* of the connected members is changed to the provision in the current AASHTO Specifications (AASHTO, 2007) that requires connections be designed for the larger of the average of the capacity of the member and the applied forces and the 75% of the capacity of the members. The actual provisions in the current AASHTO (2007) Specifications is:

> **6.13 CONNECTIONS AND SPLICES**
>
> **6.13.1 General**
>
> Except as specified otherwise, connections and splices for primary members shall be designed at the strength limit state for not less than the larger of:
>
> - The average of the flexural moment-induced stress, shear, or axial force due to the factored loadings at the point of splice or connection and the factored flexural, shear, or axial resistance of the member or element at the same point, or
>
> - 75 percent of the factored flexural, shear, or axial resistance of the member or element.
>
> (Excerpt from AASHTO, 2007, SECTION 6: STEEL STRUCTURES. See footnote "2" below.)

In Chapter 3, we will take the above provisions from the current AASHTO Specifications (AASHTO, 2007) and use it to establish the forces that should be used in design of gusset plates.

2.5. BACKGROUND ON METHODS USED IN DESIGN OF GUSSET PLATES

From the early days of design and construction of steel truss bridges in 19th century, there were design provisions on how to design gusset plates. Before designing the gusset plates, the designer has to design the connections of the members to the gusset plates. Since early truss bridges were riveted, due to relatively low strength of rivets, the connections of members to gusset plates had relatively large number of rivets since they were designed to develop the strength of the connected members. After designing this member to gusset plate connections the

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members were drawn to scale and then the *lay-out* of the gusset plate geometry was done in a way that the gusset plate would cover the riveted ends of all members in the node. The lay-out of the gusset plate would provide the dimensions of the gusset plates. The next and final step in design of gusset plates was to select the material of the gusset plate and then calculate its required thickness. The material used in gusset plates is discussed later in this chapter under "Material". The calculation of the thickness of gusset plate is discussed in the following.

Early riveted trusses of late 1800s and early 1900s had relatively short spans of less than 100 feet and relatively small members. Even with small members and small forces in them as mentioned earlier, still there were a considerable number of rivets needed at the end of each truss member to connect them to gusset plates. As a result, during the lay-out process, the gusset plates would end up being quite large and the stresses in them quite small. Therefore, gusset plates were quite often designed to have a minimum thickness of 3/8-1/2 inch. It appears that this minimum thickness must have evolved from intuitive or actual calculation of bearing stresses developed between the rivets and the gusset plates and limiting such stresses to allowable bearing stresses of carbon steel. Later, when higher strength steel as well as high strength rivets were manufactured and used in bridges, we find provisions for design of gusset plate thickness based on checking the bearing stress of rivets on the gusset plates (Ketchum, 1920).

During the early 1900s, as the more comprehensive bridge specifications were being developed and proposed, such as those by Waddell (1916), Ketchum (1920) and AASHO (1924), we see more specific provisions and guidelines regarding design of gusset plates. The AASHO Specifications (1927, 1931, and 1935) had the following provisions under the title of Gusset Plates: "Gusset or connecting plates shall be used for connecting all main members, except in pin-connected structures. In proportioning and detailing these plates the rivets connecting each member shall be located, as nearly as practicable, symmetrically with the axis of the member. However, the full development of the member shall be given due consideration. The gusset plates shall be of ample thickness to resist shear, direct stress, and flexure acting on the weakest or critical section of maximum stress. Reentrant cuts shall be avoided as far as practicable. '

The 1941, (3rd Ed.) of the AASHO specifications added the following statement to the end of the above paragraph:

> "If the unsupported edge of a gusset plate exceeds the following number of times its thickness, the edge shall be stiffened:
>
> 60 for carbon steel
>
> 50 for silicon steel
>
> 45 for nickel steel "
>
> (Excerpts from AASHO(1941) see footnote "*: below)

* "Excerpts from AASHTO Standard Specification (1941), Copyright 1941, by the American Association of State Highway and Transportation Officials, Washington, DC. Used by Permission. Document may be purchased from the AASHTO bookstore at 1-800-231-3475 or online at http://bookstore.transportation.org."

The above 1941 provisions appear to be the first time that AASHO recognizes the buckling of free edge of gusset plates and provides a limit on the length/thickness ratio for the free (unsupported) edges of gusset plates. Notice that the provisions at this time do not say that the *compression* edge of gusset plate should be checked for this, but, state the provision to be applied to any free (unsupported) edge of gusset plate regardless of whether that free edge is subjected to tension or compression. The current AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) has similar provisions but the numbers 60, 50 and 45 in the above box are given in the form of 2.06 $(E/F_y)^{1/2}$ to allow for various types of steel properties be plugged into the equation.

It is interesting that J.A.L. Waddell in his 1898 *De Pontibus* book (Waddell, 1898) makes a statement regarding *unsupported width of plates* that clearly, if applied to unsupported edges of gusset plates, could prevent edge buckling of gusset plates. The statement is: " The unsupported width of plates strained in compression, measuring between centre lines of rivets, shall not exceed thirty-two (32) times their thickness, .." . It should be mentioned that the main target of this statement apparently was the unsupported length of plates in built up compression members.

Later, in his 1916 *Bridge Engineering* book (Waddell, 1916, Page 519), J.A.L. Waddell provides a more detailed procedure for design of gusset plates. I have summarized Waddell's procedures and discussions in the following as a series of recommendations for design of gusset plates. For actual discussion given by J.A.L. Waddell, the reader is referred to *Bridge Engineering* by J.A.L. Waddell (1916). The recommendations which I have stated below in *Italic* are not verbatim but closely relay the essence of Waddell's procedures. These recommendations are followed by my discussions.

1. *Design the connection of each member to gusset plate to have allowable strength equal to or greater than the allowable strength of the connected members.*
   The connection of members to gusset plates were rivets in early bridges and after 1950's high strength A325 and A490 bolts replaced the rivets. This provision is clearly a *capacity design* provision *put in terms of Allowable Stress Design (ASD) terminology*.
2. *The rivets should be symmetric with respect to the centerline of the member.*
   By replacing *rivet* with *bolt* this recommendation is still valid today.
3. *Beveled end cuts for the members connected to gusset plates are objectionable.*
   Again, this is a very valid statement even today.
4. *Lug angles should be avoided.*
   In riveted structures, lug angles were used to connect all elements of the member to the gusset plates and to increase the effectiveness of the net section to 100%. By eliminating the lug angles and using an effective net section less than 100%, still the connection can be economical considering the saving in the weight of angle, holes and rivets or bolts on the angle.
5. *The members of trusses should not overlap each other unless they are perpendicular to each other. In triangular trusses with one vertical in the connection, the vertical was extended into the chord and the gusset plate was eliminated.*

This is still valid.

6. *In detailing the joint and establishing the dimensions of the gusset plates in the joint, first tentatively lay out the members and rivets in various connections.*
   By replacing *rivet* with *bolt* this recommendation is still valid today.

7. *For the lighter trusses, use two plates for joint. The plates can be either inside or outside the members.*
   Today, since most members are either rolled I-shapes or welded box sections, we use gusset plates outside the truss members.

8. *For heavier ones use four or more gusset plates.*
   Today, with availability of thicker plates, we can use only two plates, one on each side of the member.

9. *The gusset plate should have as few cuts as possible and plates with parallel edges are preferred to those with inclined edges.*
   This recommendation is still valid

10. *The gusset plate should be riveted to those members that will make erection easier. As a rule, shop-rivet the gusset plates to chord members and field-rivet to web members. Shop-rivet the end gusset plates to end posts.*
    By replacing *rivet* with *bolt* this recommendation is still valid today.

11. *Then, check (Waddell used word "test") the gusset plates for tearing out, crushing, bending, and direct stresses.*
    This statement establishes failure modes that gusset plates should be checked for. Today, these failure modes are in the form of shear fracture, block shear failure and fracture of Whitmore's section for tearing *out*, edge buckling and buckling of interior of gusset plate for *crushing*, and failure of critical sections of a gusset plate under combined axial force and bending moment for Waddell's *bending, and direct stress* failure mode on critical sections shown in Figure 2.23.

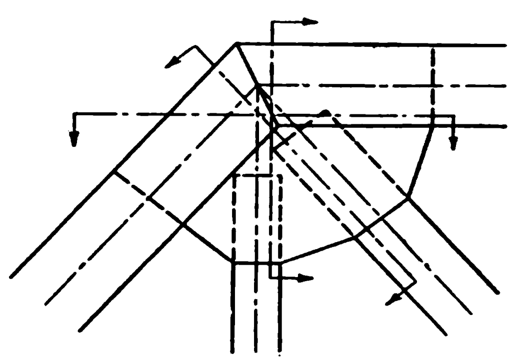


Fig. 22*mmm*. Sections for Testing Gusset-Plates at the Hip Joint.

From Waddell (1916)

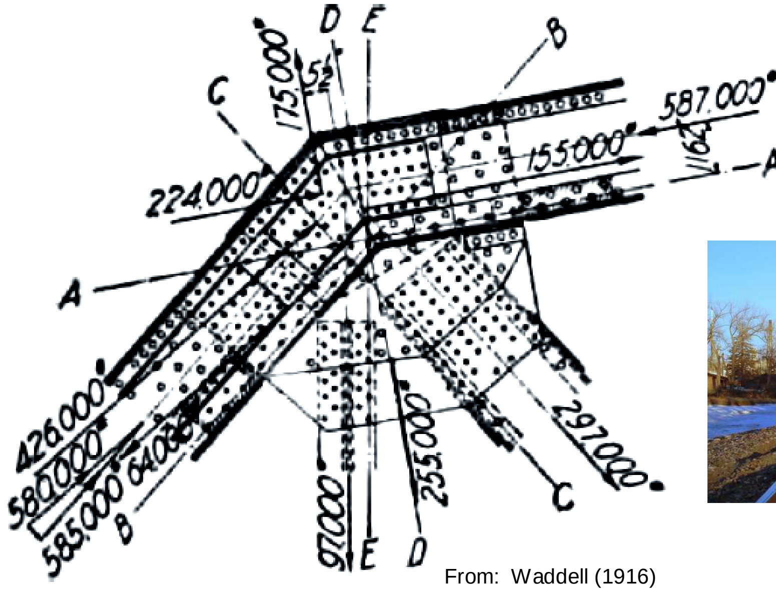
Figure 2.23. Critical sections of gusset plate to be checked (From: Waddell, 1916)

It is necessary to test the plates around the periphery of the rivet group as well as along the sides and on any intermediate row. Also, check sections through the rivets carried over to the edge of the gussets. The sections are tested for shear on surfaces parallel to the member and for tension and compression on those that are normal to the member as shown in figure below from Waddell (1916)
This is a recommendation to check shear failure of a critical section and failure modes of the connection of a member to the gusset plate. Today, this statement is as valid as in 1916 but we call the failure modes associated with connection of a member to the gusset plate fracture of Whitmore's section, block shear failure and buckling of gusset plate beyond the end of the members. It seems that Waddell was not considering the edge buckling of the gusset plate that today needs to be checked and is a failure mode for gusset plates.

12. *Checking the gusset plates for direct (i.e. axial) and bending stresses presents a somewhat uncertain problem due to irregularity in the gusset plates and in the geometry of the rivet groups and number of rivets in each group. The sections should be taken parallel to one of the intersecting members; and the various forces should be resolved normal and parallel to the sections as shown in the layout in the figure above (Figure 2.23). Using equilibrium establish direct force (axial) and bending moment on the selected sections. Calculate maximum normal stresses at extreme fibers of the gusset plate section due to axial force and bending moment and combine the stresses algebraically to establish maximum tensile and compressive stresses. Compare these maximum values to their corresponding allowable stresses in tension and compression.*
This statement and the statement No. 9 above appear to be early versions, if not the first versions, of the application of *Beam Theory* to critical sections of a gusset plate to establish maximum shear stresses due to direst shear and normal stresses due to combined effects of bending and axial force acting on the critical sections of a gusset plate. The current AASHTO Specifications (AASHTO, 2007) in essence has these two statements as the provisions for design of gusset plates. We will return to this statement in Chapter 3.

13. *In above check use the allowable capacity of the members as the applied loads and not the actual applied loads due to applied loads to the truss.* This is recommending designing the gusset plate such that the stresses created in the gusset plate by allowable load in the members are not exceeding the allowable stresses for the gusset plate. This is clearly a capacity design philosophy for gusset plates put in terms of allowable stress design method which was the method in bridge design until recently when the Load and Resistance Factor Design method was developed.

J.A.L. Waddell (1916), then, provides a numerical example and demonstrates application of his gusset plate design methodology to the hip joint of the side span of the Great Northern Railway Company's Bridge over the Yellowstone River, Figure 2.24 below.



From: Waddell (1916)

http://www.flickr.com/

Figure 2.24 A gusset plate designed by Waddell using his procedure of checking critical sections

In Figure 2.24, we see the critical sections of the gusset plate , sections AA, BB, CC, DD and EE, that J.A.L. Waddell has chosen to check. The critical sections AA, BB, CC and EE are parallel to one of the four members in the node as he had stated in his method and I paraphrased it into Statement 11 in the above list. The critical section DD in Figure 2.21 appears to be perpendicular to the top chord of truss at this node. The axial forces of the members acting on the gusset plates are also shown on the figure. These forces are the allowable capacity of rivet groups in shear for each member. The rivet groups of course were designed to have an allowable shear capacity at least equal to the allowable axial strength of the connected member. In other words the allowable strength of the gusset plate was at least equal to the allowable capacities of the rivet groups as well as truss members. This made the *connections of this truss stronger than the member.*

2.6. SECONDARY STRESSES IN TRUSSES

When truss members are not fully pin-connected to each other and gusset plates are used in the joints, due to fixity of the joints and bending created by the weight of the member secondary stresses can be present in the members as well as in the gusset plates. Traditionally, in simple trusses without subdivided panels, secondary stresses due to deformation of any member with length-to-width ratio greater than 10 were ignored (AASHO, 1927, 1931, 1935, 1941) and the specifications had provisions that members and their connections had to be proportioned to reduce secondary stresses to a minimum (AASHO, 1927, 1931, 1935). If secondary flexural stresses had to be considered in design, the early AASHTO Standard Specifications (AASHO, 1927, 1931, 1935) stated that : " In members designed for secondary stresses in combination with other stresses the specified allowable unit stress may be increased 30% but the sections shall be not less than required for primary stresses." The 1941 edition (3rd Ed.)of the AASHO Standard Specifications added the statement that: "If the secondary stresses exceed 4,000 pounds per square inch for tension members and 3,000 for compression members, the excess shall be treated as a primary stress." This addition, for the first time gave bridge engineers a precise level of stress below which the secondary stresses could be ignored. The current AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) has reverted to more general statement on secondary stresses in truss elements and states that:

> **6.14.2.3. Secondary Stresses**
>
> The design and details shall be such that secondary stresses will be as small as practicable. Stresses due to the dead load moment of the member shall be considered, as shall those caused by eccentricity of joints or working lines. Secondary stresses due to truss distortion or floor beam deflection need not be considered in any member whose width measured parallel to the plane of distortion is less than one-tenth of its length.
>
> (Excerpts from AASHTO LRFD Bridge Design Specifications(2007). See footnote '*' below.

The above statement needs to be applied both to members and gusset plates which support the members. This is due to the fact that the fixed end moment of the member, due to its dead load that creates secondary stresses in the member, also is present in the gusset plate (in the opposite direction) and creates secondary stresses in the gusset plate as well as in the member in general.

Prior to development of modern computer-based analysis software, trusses were analyzed with their joints assumed to be pin-connected even if there was no actual pin in the joint. The early trusses had actual pins and this assumption of joints being pin connections was quite accurate. However, in later trusses, when the pin was eliminated, due to rigidity of the gusset plate joints, fixed-end moments and shears were generated in the members that had to be resisted by the gusset plates. This was the time when the term *secondary* stresses entered the bridge engineering textbooks and specifications for design of trusses. The specifications (AASHO, 1927) defined the secondary stresses and when and whether they could be ignored or had to be included in design. With today's availability of computer analysis software, we can simply model the trusses with rigid joints and establish what bending moments and shear forces are generated in the members and include them in design of members as well as gusset plates.

In 1992 the question of: " how should we model a truss joint, pin or rigid? " came before the ASCE Committee on Design of Steel Building Structures of the Committee on Metals of the ASCE. The Committee after deliberations responded to the question in a paper (ASCE, 1992) as follows:

2.8 Secondary Stresses in Trusses

When is it necessary to analyze a truss as a frame, as opposed to as a pin-jointed system?

Response.

Secondary stresses in steel trusses may be neglected in most cases. However, it is important that "secondary stresses" be defined properly, and that the analysis and the design are consistent, as follows.

1. If the truss members are designed for the axial forces that would occur if the members were pin-connected, then the flexural stresses that would be indicated by a more refined analysis may be defined as secondary stresses and may be neglected, within limits, as suggested in the following.

2. If the axial forces for member design are obtained from an analysis that includes bending effects, flexural stresses cannot be dismissed as secondary stresses, since the bending moments may have reduced the axial forces given by the analysis. In this case, a designer who wishes to neglect flexural stresses must first judge whether (and by how much) the axial forces indicated by the analysis were affected by flexural effects, and then make appropriate corrections in the axial forces to be used for design.

(Excerpt from Ref: ASCE,(1992))

In a good design, secondary stresses should be reduced as much as possible. To reduce secondary stresses in the members as well as gusset plates the truss members should be concentric which means that the centroidal axes of all members in a node should pass through a single point called *work point*. The bridge design specifications since 1920s (AASHO, 1927) state that the joints in trusses should be concentric and if unavoidable eccentricity exists, the members and connections (e.g. gusset plates) should be designed to resist stresses due to eccentricity as well. This statement still is in the current AASHTO Specifications (AASHTO, 2007).

In addition, to reduce the secondary stresses, the riveted, bolted or welded connections of the truss members to gusset plates should be detailed such that the connection is symmetric with respect to the center of gravity of the member as much as possible. This requirement was in the early specifications for bridge design (Waddell, 1916) and has been in all AASHTO specifications since 1927 including the current AASHTO LRFD Bridge Design Specifications-(AASHTO, 2007). Waddell (1916) suggests the following to reduce the secondary stresses in truss bridges:

> The secondary stresses in riveted trusses are to be modified by lengthening and shortening the various truss members the amounts of their respective shortening and lengthening under dead load plus one-half the live-plus-impact load, drilling or reaming the chord splices while the chords are assembled in straight lines, then forcing the truss members into their proper positions for connection to each other before drilling or reaming the holes in the joints.

From Waddell (1916)

This provision is one of the very few items from Waddell's 1916 Specifications in his Bridge Engineering book (Waddell, 1916) that does not seem to be included in the AASHTO specifications.

2.7 ECCENTRIC CONNECTIONS

From early editions, bridge engineering books and the AASHO specifications have discouraged having eccentricity in the connections of trusses. The 1935, and 1941 AASHO Specifications state that: "Members including bracing, shall be so connected that their gravity axes will intersect in a point. Eccentric connections shall be avoided if practicable, but if unavoidable the members shall be so proportioned that the combined fiber stresses will not exceed the axial stress". Even though this provision only mentions members by name, but being given under the title of *Connections* it appears that it should be applied to connections such as gusset plate connections and the effect of eccentricities should be considered in design of gusset plates where the members are not intersecting at a single *work point*. The inclusion of effects of eccentricity in the connections is also mentioned in the current AASHTO provisions on Secondary Stresses (AASHTO, 2007).

2.8. MINIMUM THICKNESS OF STEEL (IN GUSSET PLATES)

From the early days of truss building, the thickness of steel (wrought iron in early days) used in gusset plates was required to be not less than 3/8 inch. Some textbooks such as Bresler and Lin (1960) were recommending that the thickness of gusset plate for light trusses to be 3/8 to ½ inch and for heavier trusses 5/8 to 7/8 inch. No definition is provided for *light trusses* and *heavier trusses* in this reference. There was also a requirement in the textbooks and specifications that when the material was expected to be exposed to severe corrosive conditions, either it had to be especially protected against corrosion or extra (sacrificial) thickness had to be added to the thickness established by design. This is still the case in the current AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) but the minimum thickness of steel is reduced to 5/16 inch from earlier value of 3/8 inch.

2.9 NET AREA IN THE MEMBERS AND IN THE GUSSET PLATES

Effective net area of angles connected by one leg is the net area of the connected leg plus 50% of the area of the unconnected leg. (AASHO, 1927, 1931, 1935, and 1941).

If double angles are connected to opposite sides of a gusset plate, the full net area should be considered as effective (AASHO, 1927, 1931, 1935, and 1941).

Effective area of double angles is 80% of net area of the member unless the ends connections and details are such that the angles are prevented from bending in both directions in which the full net area is used as effective area (AASHO, 1927, 1935, 1941)

In 1935 Edition of AASHO specifications, net section was more formally defined for all sections and the term s^2/4g was introduced in calculating the net section of staggered bolt lines. This provision has stayed in the later editions of the specification including in the current edition (AASHTO, 2007).

A new item in the 1941 edition is on lug angles which states: " Lug angles may be considered as effective in transmitting stress , provided they are connected with at least one-third more rivets than required by the stress to be carried by the lug angle" .

2.10. RIVETS, BOLTS AND WELDS IN GUSSET PLATE CONNECTIONS

2.10. a. Rivets and Riveting

To design riveted bridges, bridge engineers followed a number of rules. I feel the following two recommendations, paraphrased from Waddell (1898), are quite relevant to the evaluation of riveted gusset plates.

1. *Field riveting should be minimal, and if field rivets are necessary, they should be made such that they can be driven readily* (Waddell, 1898).

   This recommendation may be the reason why so often in riveted truss bridges one finds gusset plates riveted to some members and bolted to others. The rivets were done in the shop and the bolts were generally done in the field during the assembly and erection process although in many cases field-riveting was also done.

2. *Rivets are not to be used in direct tension* (Waddell, 1898).

   This is very relevant for gusset plates and other connections as well, in the sense that in evaluating the original design and intended capacity of a riveted gusset plate, one needs to recognize the fact that the above seems to be implemented in riveted bridges to the letter of the recommendation. The early specifications discouraged the use of rivets in tension. For example the AASHO Standard Specifications for Highway Bridges (AASHO, 1927, 1931, 1935, 1941, 1944, 1949, 1953, 1957, 1961) until 1965 edition state that: "Rivets in direst tension shall in general not be used. However, where so used their value shall be one-half that permitted for rivets in shear…" This provision does not appear in the 9th Edition of AASHO (1965) and the editions after that. Instead, the rivet provisions were replaced by more provisions for welds indicating a change in connectors of steel bridges from rivets and bolts to bolts and welds.

There were two types of rivets: shop and field rivets. The allowable shear stress for field rivets generally were 80 percent of the allowable stress for shop rivets. To ease erection process, the gusset plates were shop-riveted to the chords and then riveted in the field to the web members. The end bottom gusset plates were shop-riveted to the end posts. As a result, the web members generally had about 20% more rivets on them for the same cross sectional area as the chord members since rivets on them were assumed to have only 80% of the strength of the rivets installed in the shop on the chords and end posts. In evaluating the strength of riveted gusset plate and other connections today, the engineer should be aware of this fact and probably follow the same rule in assigning shear strength to field rivets equal to 80% of the strength of shop rivets as specified in the code. More on the strength of rivets can be found in AASHTO (2003) and Kulak, Fisher and Struik (2001).

In early bridge specifications, the rivets were recommended to be generally ¾ or $^7/_8$ inch in diameter. Rivets with $^5/_8$ inch diameter were allowed to be used only in 2.5 inches legs of angles and 6 or 7 inch flanges of beams or channels (AASHO, 1927, 1931, 1935, 1941). The 1931 edition of the AASHO specifications for the first time mentions 1 inch diameter rivets in addition to $^7/_8$, ¾ and $^5/_8$ inch which were mentioned in earlier editions.

The minimum spacing of rivets were specified to be not less than three times the diameter of the rivet and preferably the following values: 3 inches for $7/_8$ inch diameter, 2.5 for ¾ inch diameter and 2-1/4 inches for $5/_8$ inch diameter rivets. The maximum spacing of the rivets in the line of stress was specified not to exceed 6 inches or 16 times the thickness of the thinnest outside plate or angle leg (AASHO, 1927, through 1961).

The minimum edge distance for rivets were to be 1.5 times diameter of the rivet and whenever practicable this distance was recommended to be increased to 2 diameters. The edge distance was measured from the center of the hole to the edge of the plate or rolled section.

The first edition of AASHTO Standard Specifications (AASHO, 1931) had the limitations on the edge distance given in the following table. These limitations are still in the current AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) where bolt diameters 1-1/8, 1-1/4 and 1-3/8 inch are also added to the table with their corresponding minimum edge distances.

Minimum edge distance for rivets in AASHO Standard Specifications (1931):

Diameter of rivet	Minimum edge distance for sheared edge	Minimum edge distance for rolled or planed edge	Comments
1	1-3/4	1-1/2	Added in 1931 Ed.
$^7/_8$	1-1/2	1-1/4	Was in 1927 Ed.
¾	1-1/4	1-1/8	Was in 1927 Ed.
$^5/_8$	1-1/8	1	Added since 1927

In the early editions of the AASHO Standard Specifications (AASHO, 1927, 1931) the maximum edge distance from any edge was eight times the thickness of the thinnest outside plate, but not to exceed 5 inches. The current AASHTO Specifications (AASHTO, 2007) has exactly the same provision. This requirement is very important in design and evaluation of the gusset plate and it is meant to prevent local buckling between the two connectors as well as to prevent corrosion and rust packing under the outside plate. For information on rivets see Kulak, Fisher and Struik (2001). AASHTO (2003) also has much useful information on rivets, including strength values for rivets used in bridges.

2.10.b Bolts

The use of bolts was discouraged in early bridge specifications. For example the AASHO Specifications (1927, 1931, 1935, and 1941) state that: "Bolted connections shall not be used unless specifically authorized. Bolts shall be unfinished; turned as specified and meeting the requirements of Division IV; or an approved form of ribbed bolt". Division IV refers to the section of the specification where material properties are specified. The last part: "..or an approved form of ribbed bolt" appears for the first time in the 1941 edition of the AASHO. The ribbed bolts were bolts that had vertical ribs on the shank and a half-sphere head like a rivet. They were hammered into the hole and the ribs fill between the shank and the bolt hole preventing the slippage of the bolted connections. After development of high strength bolts, the production and use of ribbed bolts were discontinued and using high strength bolts prevented the slippage of bolt under service loads, which was the concern. However, there may be steel truss bridges designed and constructed during the 1940s and 50s that have ribbed bolts in their connections including their gusset plates. For information on bolts, including ribbed bolts, see Kulak, Fisher and Struik (2001).

2.10.c Welding

The 1941 (3rd) Edition of the AASHO, for the first time, included half page provisions under *Welding* . Primarily the provisions allowed welds to be used on low carbon steel and wrought iron where carbon and manganese contents did not exceed 0.25% and 1.0% respectively. A long list of components of steel bridges where welds were allowed to be used was provided followed by a statement that : "Welding is not permissible in main members or their connections where the failure of the weld would endanger the stability of the structure" (AASHO, 1941). It is interesting to note that under *welding* section of AASHO 1941 welding is allowed to be used in bracing members and their connections, but, a page later under *Bracings* it is stated that the " Bracing shall be composed of angles or other shapes and the connections shall be riveted." The later editions of the AASHTO Specifications (1965 and later) have extensive weld provisions. The most important issue in welded bridges is the fatigue and fracture. Since focus of this report is on the riveted and bolted gusset plates, for weld provisions and fatigue and fracture in welded connections the reader is referred to the latest AASHTO Specifications.

2.11. STAY PLATES IN LACED AND RIVETED MEMBERS

The early riveted truss bridges used laced members. Later, laced members were replaced with riveted, bolted and in recent years with welded perforated members. The laced built-up truss members had stay plates at their ends where they were connected to gusset plates. The dimensions of stay plates were governed by the specifications. In trusses, the stay plates were required “near the ends of the members as practicable, and should be extended well inside of gusset-plates” (Waddell, 1916, Page 505) as shown in Figure 2.25(a). Following this provision, in effect, laced members were riveted to the stay plates and the stay plates were riveted to the gusset plates. Therefore in evaluation of strength of gusset plates connecting laced members, failure modes of the stay plates also need to be considered.

When laced members were converted to perforated members, the ends of the members were not required to have stay plates at the location of the connection of the member to the gusset plates. However, it is a good practice not to have perforation in the end portion of the member that is inside the gusset plate box as shown on the right side member in Figure 2.22(b). Still, in some truss bridges, we find the perforation placed inside the gusset as in the member to the left of the gusset plate in Figure 2.25(b).

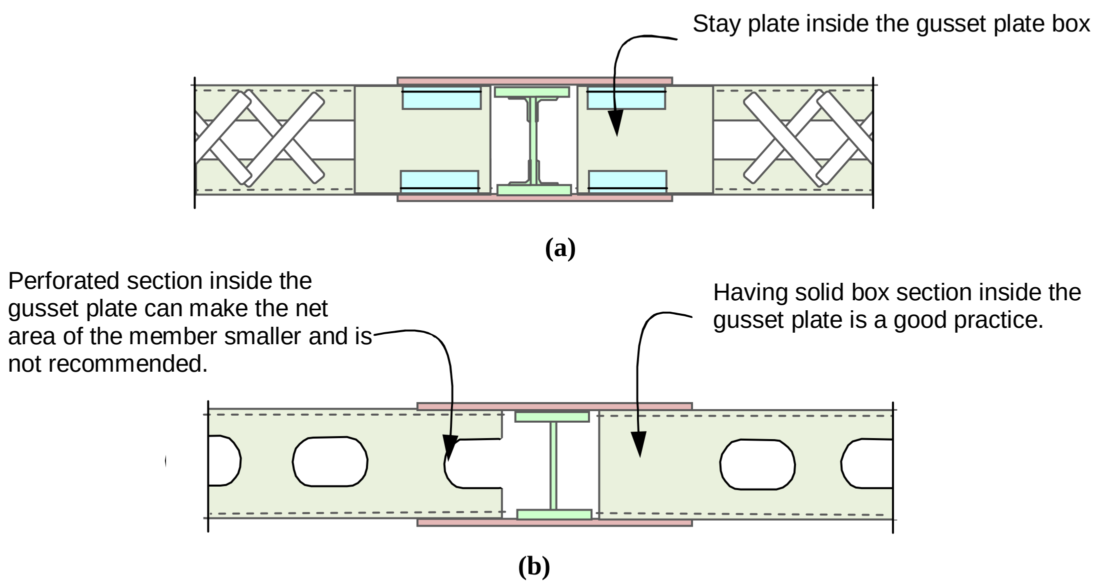


Figure 2.25. (a) Stay plates in a laced member; and (b) connection of ends of perforated members to gusset plates

2.12 SPLICES

Splices are usually used in trusses to connect the chord members. The web members and end posts are generally fabricated in one piece although in some old riveted bridges we find splices in the web members especially in truss web diagonals which naturally are longer than the web verticals. In the early specifications, and until recently in the AASHTO specifications the *splices* and *gusset plates* were two separate elements of the trusses located near each other. In the current edition of the AASHTO Specifications (AASHTO, 2007) splices are covered under connections but gusset plates still have their separate provisions. However as discussed below, in many cases, especially in later riveted trusses built after 1940’s as well as in most bolted trusses we find the splices placed inside the gusset plates and the gusset plates are used as splice plates as well. This can result in some difficulty and probably confusion in interpreting what provisions of the AASHTO Specifications apply to which parts of the gusset plate connection now that there is also a splice combined with it. For this reason the author feels that there is a need to include a discussion of splices in steel trusses and their design in this document which is primarily on gusset plates.

Until 1950's, the chord member splices were outside the gusset plate and on the side that had the smaller cross section for the chord. The chords in these trusses would pass continuously through the gusset plate. However, in some cases such as the top polygonal chords, "it is always necessary to splice these at the panel-points (Waddell, 1916)". Bresler and Lin (1960) suggest that using shingled splices result in more economical design. In shingled splices several overlapping plates are used and the plates are offset to transfer the load gradually from one side of the splice to the other side as shown in Figure 2.26.

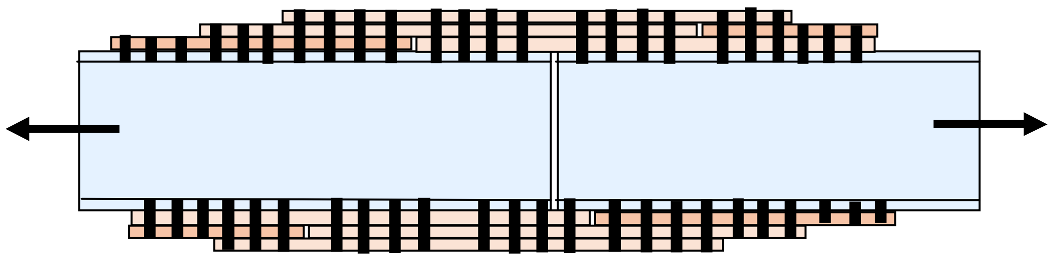


Figure 2.26. A "Shingled" splice

The 1927, 1931, 1935 AASHO state that: "Splices shall be located as close to panel points as possible and, in general, shall be on that side of the panel point which is subjected to the smaller stress." It appears that many truss bridges built after the 1940's have moved the splice in the chord members into the gusset plates and have used the gusset plates as splice plates.

The early rules for design of splices were that the splices for *compression* chord members had to be designed to have an allowable axial strength not less than sixty percent of the allowable axial strength of the smaller of the spliced members. The compression members of bridge trusses including the chord members are always required to have milled ends bearing against each other at the splices. For tension members the allowable axial strength of the splice, if any such splice was required, was to be at least equal to 100 percent of the allowable axial strength of the smaller of the spliced member (Waddell, 1916). Early bridge specifications such as the AASHO (1927, 1931, and 1935) stated that : "All splices, whether in tension or compression, shall be proportioned to develop the full strength of the member spliced and no allowance shall be made for milled ends of compression members". The splices of the top chords in deck trusses should be designed for the combined effects of axial load and bending due to direct application of dead load and live load to the top chord.

An important issue relevant to design and evaluation of gusset plates is the location of truss splices. Since trusses usually have a relatively long span, they cannot be assembled in whole and transported to the site. Historically, they are built in segments in the shop and then transported to the location and erected there. In many important riveted bridges of the 1800s and early 1900s, we find that the truss segments are riveted in the shop with the splice point to connect the segments of the truss to each other outside the gusset plate. An example is the 300-foot span trusses of the Bay Bridge shown in Figure 2.27. In this case, the splice point is outside the gusset plate (to the left of the gusset in the drawing). As a result of placing the splice outside the gusset plates, the gusset plates were fully riveted in the shop.

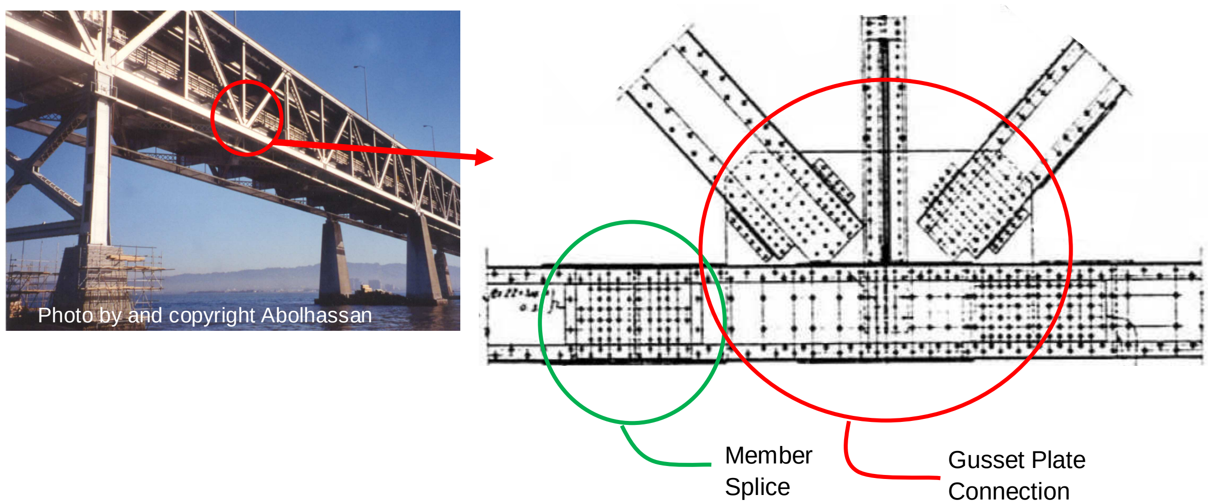


Figure 2.27. A joint from the 300-foot span trusses of the 1936 San Francisco–Oakland Bay Bridge with the member splice outside (to the left of) the gusset plate connection

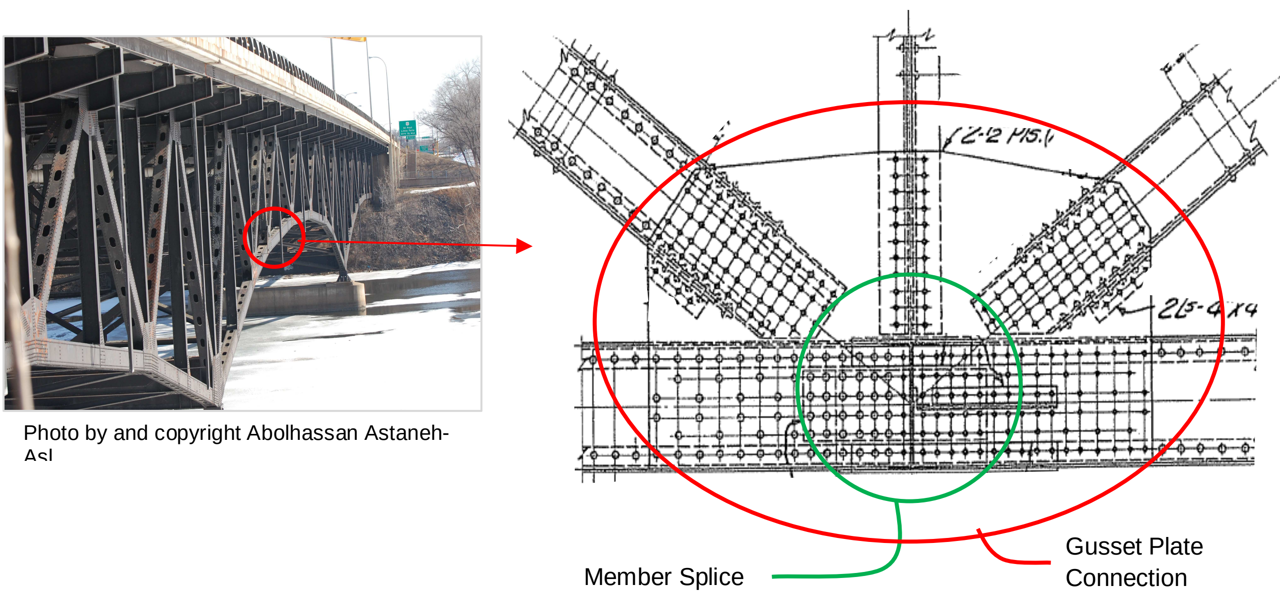


Photo by and copyright Abolhassan Astaneh-Asl

Figure 2.28. A joint from the 1957 De Soto Bridge in Minnesota (demolished in 2008 and is being replaced) with the member splice inside the gusset plate

However, in some bridges, mostly built during the mid and later years of the 1900s, we find that the splice and gusset plate connections are combined into one as shown in Figure

2.28. This is an important issue, since by placing the member splice inside the gusset plate connection, the gusset plates act as splice plates as well, being subjected to larger stresses than they would be if the chord splice were outside the gusset plate. I will revisit this important issue in Chapter 3 when I discuss the failure modes of the critical sections of a gusset plate under combined loads. In my search of trusses splice plate connections, I came across a gusset plate connection, shown in Figure 2.29, which the member splice was not totally outside the gusset nor in its inside at its center. But the chord splice was inside the gusset on the left side creating even more complex stress patterns in the gusset plates.

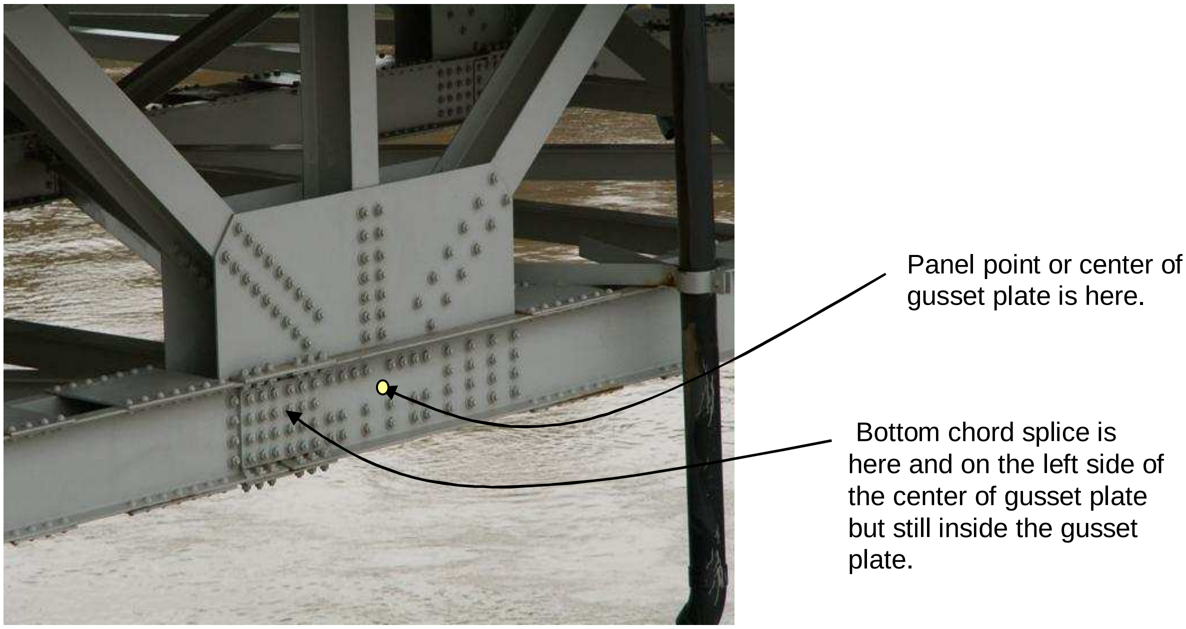


Figure 2.29. Bottom chord splice on one side of the panel point but inside the gusset plate

2.13 TYPES OF GUSSET PLATES

In this report, I have divided bridge gusset plates into two categories: riveted/bolted and welded/bolted. The riveted/bolted gusset plates are usually double gusset plates in the main trusses and towers and double or single gusset plates for wind bracings and other secondary members. The welded gusset plate connections in truss bridges in the United States are typically used in the lateral bracing system and secondary members and not in the main trusses. For main trusses and towers, riveted double gussets were used in the past, and in recent decades, mostly bolted double gussets are used. Since most gusset plates in the main trusses of bridges in the United States are bolted or riveted, the focus of this report is on the riveted and bolted gusset plates.

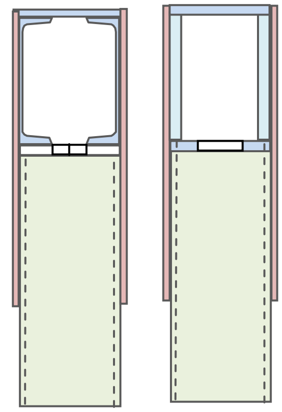
Riveted/bolted gusset plates are generally of two types, internal and external. In the internal gusset plates, which are found in many bridges built in the 1800s and the early 1900s, the main chord members are made of built-up members quite often consisting of two channels and a top cover plate, all connected to each other by rivets. The bottom flanges of the channels in these members usually were connected to each other by lacings. The two gusset plates in this case are connected to the webs of the two back-to-back channels having the channels outside the gusset plates as shown in Figure 2.30. Most bridges built prior to 1940's have internal gusset plates. The main reason for the member elements connected to the outside of gusset plates appear to have been to make the width of the member larger increasing its radius of gyration in out of plane of the truss. In many of trusses built between mid 19^{th} and mid 20^{th} centuries, only the compression members are connected to the outside of the gusset plates while the tension members are inside the gusset plates.

Photos: www.historicalbridge.org (with permission)

Figure 2.30. The 1937 riveted US-23 Bridge crossing the Ocqueoc River in Michigan has "Interior" gusset plates

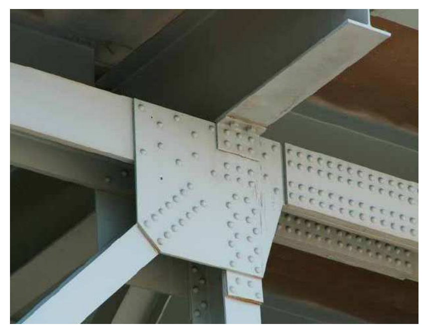
In the external gusset plates, as shown in Figure 2.31, two gusset plates are also used, one on either side of the main chord member sandwiching the chord member, which usually is a box or an I-section. Most bridges built after the 1940s have external gusset plates. In design, the same rules and concepts that are discussed in Chapter 3, are applied to both Internal and External gusset plates. However, in retrofitting these gusset plates , if they are found deficient, somewhat different concepts and configurations for each case should be used as discussed in the second part of this report (Astaneh-Asl and Tadros, 2009). Figure 2.32 shows another example of a bridge with external gusset plates.

Considering the rotational stiffness of the gusset plate joints, or *nodes*, in trusses, we divide the joints into two categories of *pin-connected* and *riveted*. The pin-connected truss joints, which had an actual pin at the truss panel points, were mostly used in the 1800s and to a limited extent in the early 1900s. Almost all of these pin-connected trusses had riveted members with the ends of the members reinforced with plates around the pin hole. These pin-connections did not have substantial gusset plates with some having field-bolted splices located outside the truss joints. Figure 2.33(a) shows an example of pin-connected hip joint. In pin-connected truss joints, all members at the joint were connected to a single pin located at the "work point," which is the point where centroidal axes of all members at the joint intersect.



Photos: www.historicalbridge.org (with permission)

Figure 2.31. The 1951 I-76 (Pennsylvania Turnpike) Bridge crossing the Alleghany River is a riveted bridge with "External" gusset plates



Photos: www.historicalbridge.org (with permission)

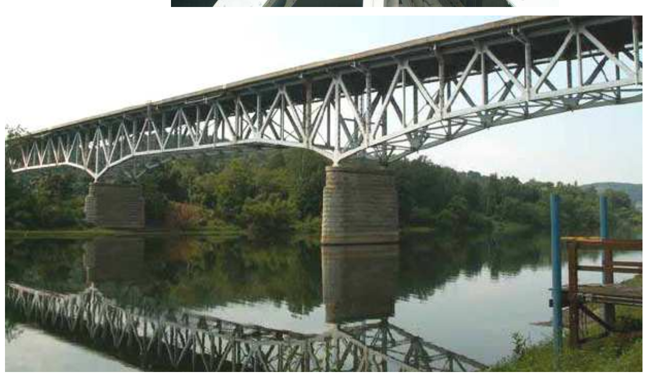
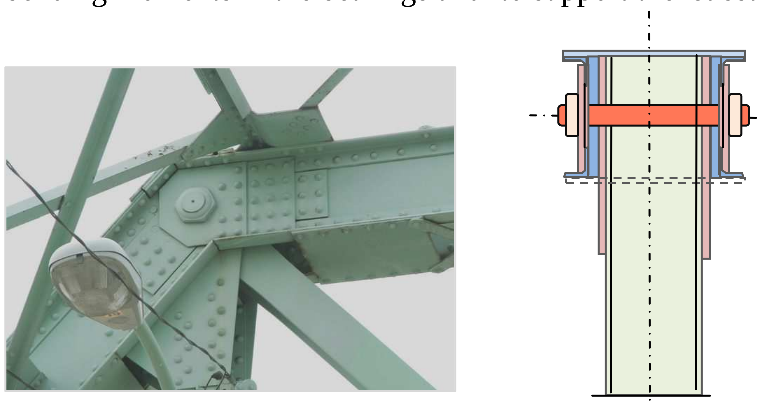


Figure 2.32. An example of a bridge with all gusset plate being "External" to the members

Later, apparently due to the cost of initial fabrication and lifetime maintenance of the pins as well as the fact that pin-connected trusses were more flexible than the riveted joints without a pin, the pins in the truss gusset plate connections were eliminated. Instead of using a gusset plate at the end of each member, a single gusset plate was used on either side of the panel point and all members were connected (i.e. riveted) to these gusset plates, Figure 2.33(b). In these connections, with no pin used in the connections, still the centroidal axis of the members passed through a single work point on the gusset plate. These connections which generally had two relatively sizeable gusset plates were referred to as *riveted* gusset plates to distinguish them from *pin-connected* gusset plates. The only places that actual pins were used in the trusses with *riveted* gusset plates were at the support nodes of the truss and at the end connections of the eye-bar members. It appears that during the 1950s and 1960s, the use of eye bars gradually discontinued and pins were only used in the bearings over the supports to eliminate the development of bending moments in the bearings and to support the substructure of the bridge.



(a) "Pin-Connected" Gusset Plate Connection

(b) "Riveted" Gusset Plate Connection

Figure 2.33 (a) *Pin-connected* and; (b) rigid or *Riveted* gusset plate connections

Obviously, the gusset plate joints in these more recent trusses, where a pin is no longer used, do not act as a true pin but behave in a somewhat rigid manner, introducing some bending moment into the members as well as into the connection of the member to the gusset plate as well as into the gusset plates themselves. However, as far as the design of truss members is concerned, these rigid joints are assumed to be simple and the trusses are analyzed as pin-connected trusses. After establishing the pure axial loads in the members of the truss based on the assumption of pin joints, depending on the length and weight of a truss member, the bending moment due to its self weight, is considered in design of the member. In Chapter 3, the issue of the analysis of trusses and assumptions made on the rigidity of the gusset plate connections is revisited and information on the current practice of truss analysis, which allows the use of either pin or rigid connections for the truss joints is provided.

Considering the importance of gusset plates in the global behavior of truss bridges, especially the gravity load carrying behavior of the bridge, gusset plates are divided into two categories: primary and secondary gusset plates. Primary gusset plates are those connecting the primary members of the bridge such as the members of the main trusses that are essential in carrying the gravity loads, Figure 2.34. Secondary gusset plates are those that connect the secondary non-gravity carrying members of the bridge, such as the lateral bracings, to each other and to the primary members, Figure 2.34.

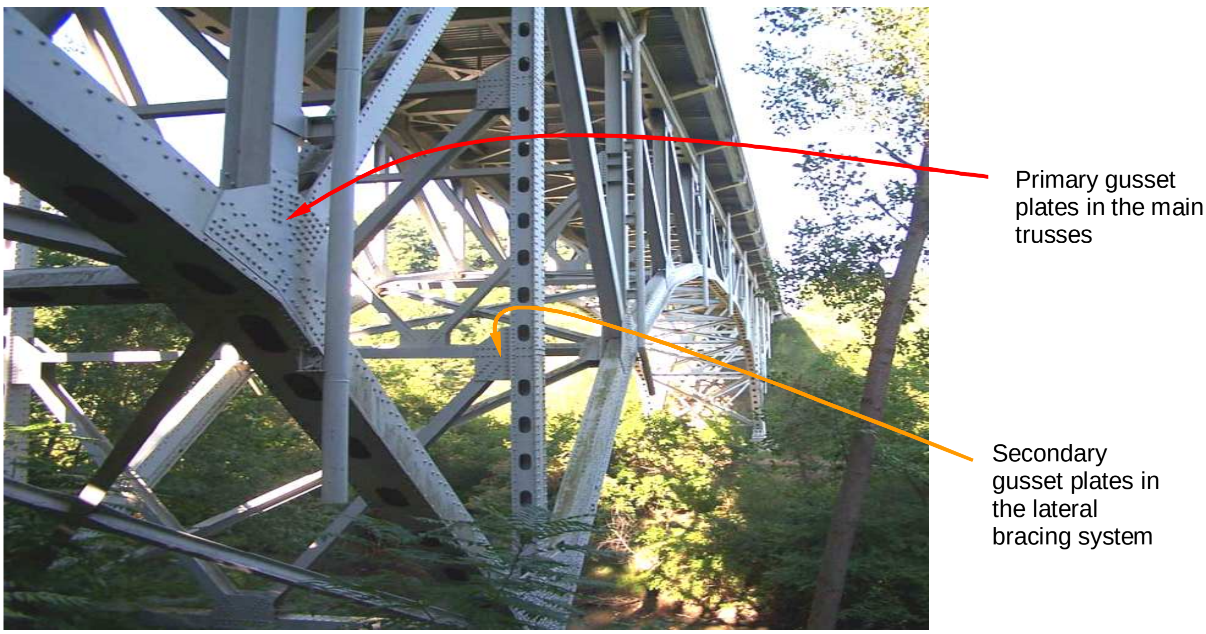


Photo: Ohio Department of Transportation, Ref: (ODOT 2007)

Figure 2.34. "Primary" gusset plates in main trusses and "secondary" gusset plates in the lateral bracing system of a truss bridge

Although all gusset plates of a bridge are designed following the same procedures, using the same load and resistance factors (or the same factor of safety in pre-LRFD bridges), the roles they play in preventing progressive and catastrophic collapse of the bridge are different. Since most truss bridges, even today, are designed using determinate (non-redundant) truss configurations, fracture failure of a primary gusset plate in the main gravity load carrying trusses can result in partial or full collapse of the entire span. Therefore, to inspect the gusset plates and to develop repair and retrofit measures for gusset plates of steel truss bridges, special attention should be paid to the primary gusset plates. This is due to the fact that a failure of a secondary gusset plate of the lateral bracing system is not believed to result in a progressive and catastrophic failure of a major part or the entire span of a truss bridge.

2.14 GUSSET PLATES IN MODERN WELDED/BOLTED TRUSS BRIDGES

Since the 1970s and the development of more ductile electrodes and steel as well as proper welding details more resistant to fatigue crack initiation and propagation bridge engineers, especially in Japan and Europe, are using welded gusset plates in modern continuous steel bridge trusses. Figure 2.35 shows two examples of modern continuous bridges in Japan and Europe. Most trusses in modern steel bridges in Japan, a highly seismic country, have welded/bolted gusset plate connections. A sketch of such connections is given in Figure 2.36. However, in the United States with only a few truss bridges built in the recent years, bolted connections are used. The only exception are relatively short welded trusses where most gusset plates are fillet-welded to truss members in the shop and only a few gusset plates at the location of field splices have bolts. Figure 2.37 shows an example of such bridges in which the welded segments of the truss are shop welded and then field bolted at the location of the gusset plate. Figure 2.37 shows a view of the 1998 Boydsville Pony Truss Bridge in Ohio with a span of 118 feet. The truss members are all rolled wide flange sections and the gusset plates are welded. In recent years in the US, welded gusset plates have been used in short span trusses , but in medium and long span trusses they are all bolted.

Photos: American Iron and Steel Institute publications

Figure 2.35. Continuous steel truss bridges: (a) in Europe and (b) in Japan

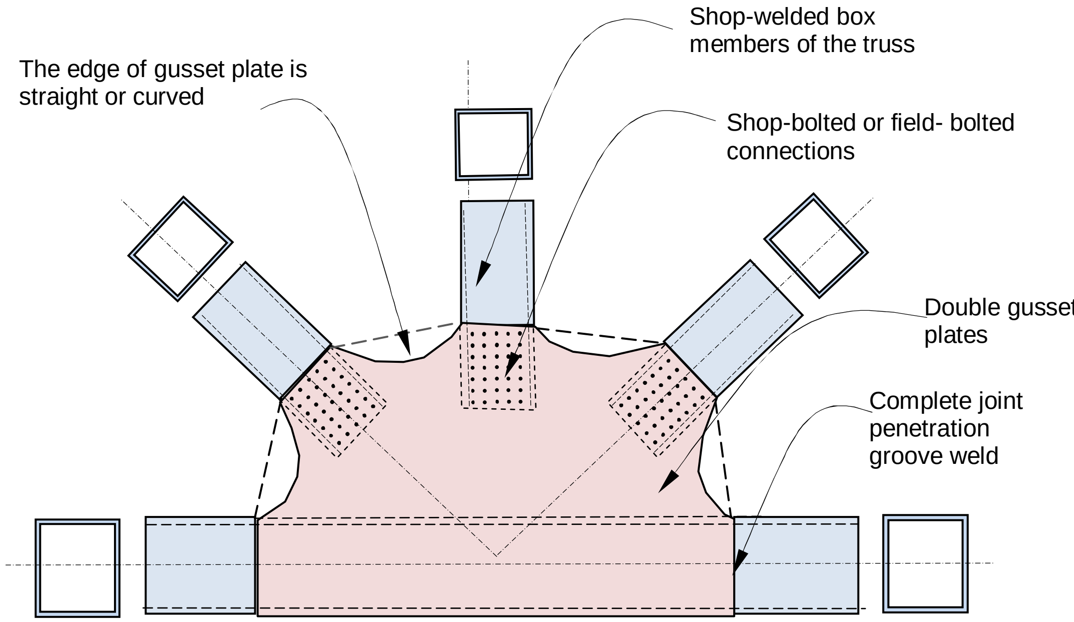
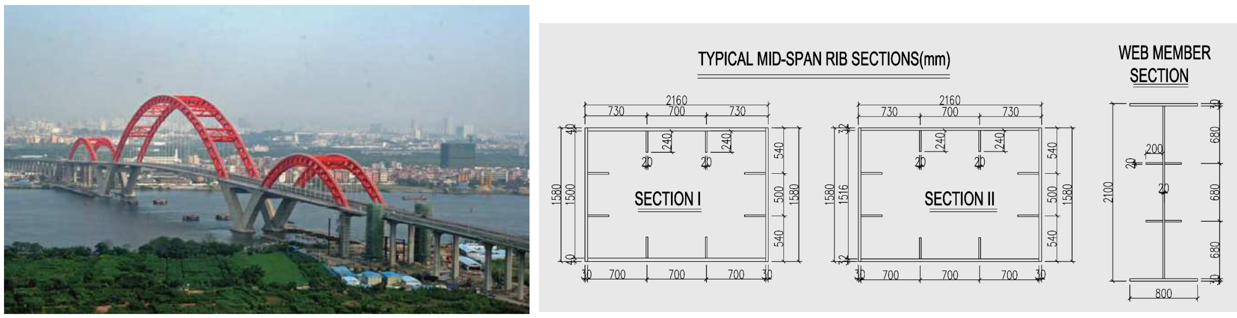


Figure 2.36. Shop-welded, field-bolted gusset plates frequently used in trusses of modern cable-stayed and suspension bridges in Japan

http://bridgehunter.com/oh/belmont/bh36218/
Photo taken by J.R. Manning in October 2007

Figure 2.37. The 1998 Boydsville Pony Truss, Belmont County, Ohio, a shop-welded truss bridge with field-bolted splices

Another modern truss bridge where welded/bolted gusset plates are used is the 2006 Xinguang Bridge over the Pearl River in the City of Guangzhou in China, Figure 2.38. The steel arch ribs in this bridge are trusses with single cell, longitudinally stiffened hollow steel boxes, and the web members are steel, longitudinally stiffened I-shapes, Figure 2.28(b) Chen (2008b) and the web members of trusses are stiffened I-shapes. According to Chen (2008b), in "integrated joint connections technique the chords of the arch ribs, as welded and bolted members, are connected in the joint position via an integrated joint plate which is formed by changing the height and thickness of the steel webs". This explanation seems to indicate that the connections in this bridge are somewhat similar to the sketch shown in Figure 2.36 above used in truss bridges in Japan with great success.



(a) (b)

All photos and drawings from:http://www.arch-bridges.cn/conf2008/pdf/439.pdf

Figure 2.38. The 2006 Xinguang Bridge over Pearl River in China has welded/bolted gusset plates integral with the members.

The www.structure.com/ web site has an entry on the Besos Yacht Port Footbridge in Barcelona, Spain, a 140-meter span Warren truss pedestrian bridge with all connections of the truss bridge being welded connections of members without using gusset plates, Figure 2.39. A paper by Aparicio and Ramos (2004) indicates that "… the absence of dynamic loads and the light live load allowed designing the trusses without using gusset plates." In the US ,although trusses with direct welding of members to each other without gusset plates, have been used in buildings, their use in bridge trusses are rare.

(Photos from www.structurae.com

Figure 2.39. The Besos Yacht Port Footbridge in Spain and a close-up of truss connections)

Figure 2.40 shows the 2005 Prospect Street Bridge in Niagara County, NY crossing Erie Canal. The bridge represents the current practice of truss bridge design and construction, where main compression cords are welded built up box sections and other members including tension chord and truss web members are rolled wide flanges. The gusset connections are all bolted. From the photos, it seems that all bolts in the main truss gussets, which carry live load, are slip-critical as required by the AASHTO (AASHTO, 2007).

Figure 2.40. The 2005 Prospect Street Bridge in New York, a typical modern steel truss bridge constructed in U.S. with bolted gusset plates

2.15 AESTHETIC OF TRUSS BRIDGES

Due to very strong geometric dominance of triangles in truss structures, they have not changed over the years aesthetically. Steel truss bridges lack the graceful curvature of catenary cables in suspension bridges and the compression arches in reinforced concrete ones. They are very much utilitarian structures which carry the load economically and efficiently..

The early truss bridges built during the 19th and early 20th centuries had many laced members for compression members and eyebars for tension members that gave them a certain lightness and see through quality. Some of these early truss bridges, especially in urban areas had ornamental guardrails and latticed portals which softened their sharp geometry giving them a decorative and artistic touch. Figure 2.41 shows an example of such bridge. As times passed specially during the mid and late 20th century, when a large number of bridges had to be built in the United States, there was a pressure on the bridge engineers to design bridges that could be built faster and cost less . The trend seems still to continue and with the exception of some cable supported bridges, and magnificent designs by Calatrava and a few others, there seems to be very few bridges, especially truss bridges that are considered aesthetically in par with other types of steel bridges such as cable-supported suspension, cable-stayed or arch bridges.

Photos taken Nov. 2005 by Jason and Birgit Smith

http://bridgehunter.com/mn/hennepin/merriam-street/

Figure 2.41. One span of the original 1887 Merriam Street Bridge in Minneapolis, MN with decorative ornaments

Figure 2.42 shows an elegant pedestrian truss bridge, the Python (also known as Dinosaur) Bridge in Amsterdam, the Netherlands. The bridge has high degree of redundancy in its web because of the latticed configuration of the web diagonals. Note that this bridge has no gusset plate and the stems of T-shape chord members are used to connect the web members to the chords.

http://members.virtualtourist.com/m/p/m/2d9988/

Figure 2.42 The pedestrian Python Bridge in Amsterdam, Netherlands

2.16 A TRUSS BRIDGE FOR "THE GREATEST GENERATION*"

A truss arch bridge, the 1918 Ludendorff Bridge at Remagen in Germany, Figure 2.43, became a landmark to the allied forces during the World War II to resist multiple attempts to destroy it by allied as well as German forces as the battle lines shifted. According to www.herrlichkeit.de during the period of September 1944 through February 1945 numerous bombing raids by the allied airplanes damaged this and other bridges on the Rhyne River. The bridge albeit sustaining damage survived the attacks. In early March of 1945 as allied forces were reaching the bridge, the Nazis attempted to destroy it by using explosive charges. The explosives failed to detonate the first time and in the second attempt, the bridge was lifted a bit, but fell back into its bearings. Since the exact locations of the charges are not known, one can only speculate that considering the suspension nature of the deck hanging from the truss arch, probably the charges were placed near the bearings to destroy the arch supports which may be the reason why the bridge bearings only slightly were uplifted and then settled down without the bridge collapsing into the Rhyne.

Following bombings and rocket fires finely collapsed the bridge though it had survived to save lives and affect the direction of the war.

With my keen interest to find out about this truss bridge and how it survived so much damage and why it collapsed, I came back almost empty handed. B.L. Dotson-Lewis (2001) has a quote from Ken Hechler, a WWII historian that on the day of collapse: "Eleven V-2z landed near the bridge, shaking it like an earthquake". It is possible that the initial bombardments and explosive charges had deteriorated the internal redundancy of the bridge and had damaged the structural redundancy of multi-support bridge and the final V-2 attacks and shaking of the super-structure caused its sudden collapse which did not have much of a load path redundancy.

Whatever the cause of the collapse, and regardless of the level of *Internal, Structural* and *Load Path Redundancy* ,the bridge had enough *resilience* to tolerate the damage and not enter the stage of *progressive and catastrophic collapse* until it had helped the "Greatest Generation" *(see the footnote) cross it and "shorten the war by enabling the Americans to encircle and trap 300,000 Germans east of the Rhine, thereby, causing the war to end earlier on May 8, 1945" (Ken Hechler in: Dotson-Lewis, 2001).

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* The term “The Greatest Generation” was coined by Tom Brokaw for the generation of Americans who grew up during the Great Depression, and then went on to fight in World War II.

http://www.bruecke-remagen.de/index_en.htm

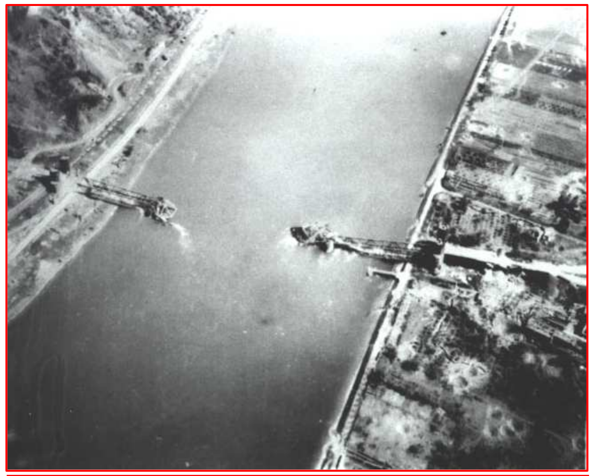


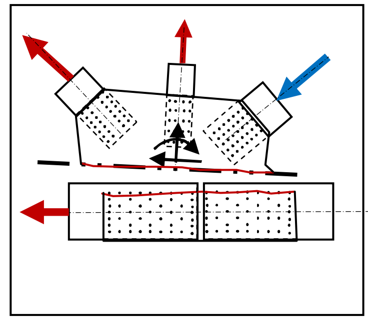
Photo: Courtesy of David Foster at:
http://www.appalachiacoal.com/WWII,%20Remagen%20Bridge,%20unpublished%20photos.html in:
"Memory of Paul (8th Air Force, 448th BG, 715th Sq.) and Frances Davis"
as he wishes his photos be dedicated.

http://de.wikipedia.org/wiki/Br%C3%BCcke_von_Remagen

http://de.wikipedia.org/wiki/Br%C3%BCcke_von_Remagen

Figure 2.43. The 1918 Ludendorff Bridge at Remagen, Germany, before (left) and after (right) its collapse during the WWII

3. Design of Gusset Plates in Steel Trusses



3.1 INTRODUCTION

This chapter first presents a discussion of the failure modes of steel trusses and then focuses on the failure modes and design of the gusset plates in steel trusses used in bridges based on LRFD methods. Since almost all bridge trusses in the United States have either riveted or bolted gusset plates the emphasis here is on trusses with riveted/bolted gusset plates. In recent years, welded gusset plates have been used in some short span bridge trusses in the United States. The information on welded gusset plates in this report is limited to the gusset plate details used in those short span trusses. In Japan, Europe and China welded gusset plates have been used in many steel truss bridges as well as in the stiffening trusses of cable-stayed and suspension bridges. The gusset plates of the Akashi-Kaikyo Suspension Bridge in Japan, currently the longest span bridge in the world, are shop-welded and field-bolted.

3.2 FAILURE MODES OF STEEL TRUSSES

Trusses, due to their triangular geometry, resist bending and shear primarily by axial strength and stiffness of their members. Behavior of a truss as a whole is determined by the collective behavior of its members and connections, that is, the gusset plates. Failure modes of a truss are the same as failure modes of its members and gusset plate connections. In addition to the failure of its members and/or connections, a truss can fail in a global buckling mode due to a lack of lateral bracing for its compression chord. Failure modes of the substructures are not included in this discussion which is focused on the steel truss superstructure of bridge.

The main failure modes of a steel truss are listed below and shown in Figure 3.1.

a. Global instability failure of the truss

b. Failure of truss members

c. Failure of truss connections such as gusset plates and splices and supports (i.e. bearings)

3.2.a Global Instability Failure of Trusses

In a typical truss, if sufficient lateral bracing is not provided at the truss nodes, the compression chord can buckle and even the tension chord can move out of plane due to instability of its nodes. Buckling of the compression chord, as shown in Figure 3.1(a), can occur when the lateral braces do not have the necessary strength or stiffness to keep the truss nodes, that is, the gusset plates, from moving out of plane in a lateral direction. In steel truss bridges, normally there are two parallel main trusses that carry the gravity load and that are connected to each other by the deck structure as well as by the lateral bracing system. The lateral bracing members have two main roles: (a) to act as web members of a horizontal truss to resist horizontal wind and seismic forces for which the chords of the main trusses act as chord members, and (b) to act as a lateral bracing system for main trusses and to brace the panel points (gusset plates) of these trusses from moving in the out-of-plane direction, thus preventing global buckling of the truss chords as shown in Figure 3.1(a).

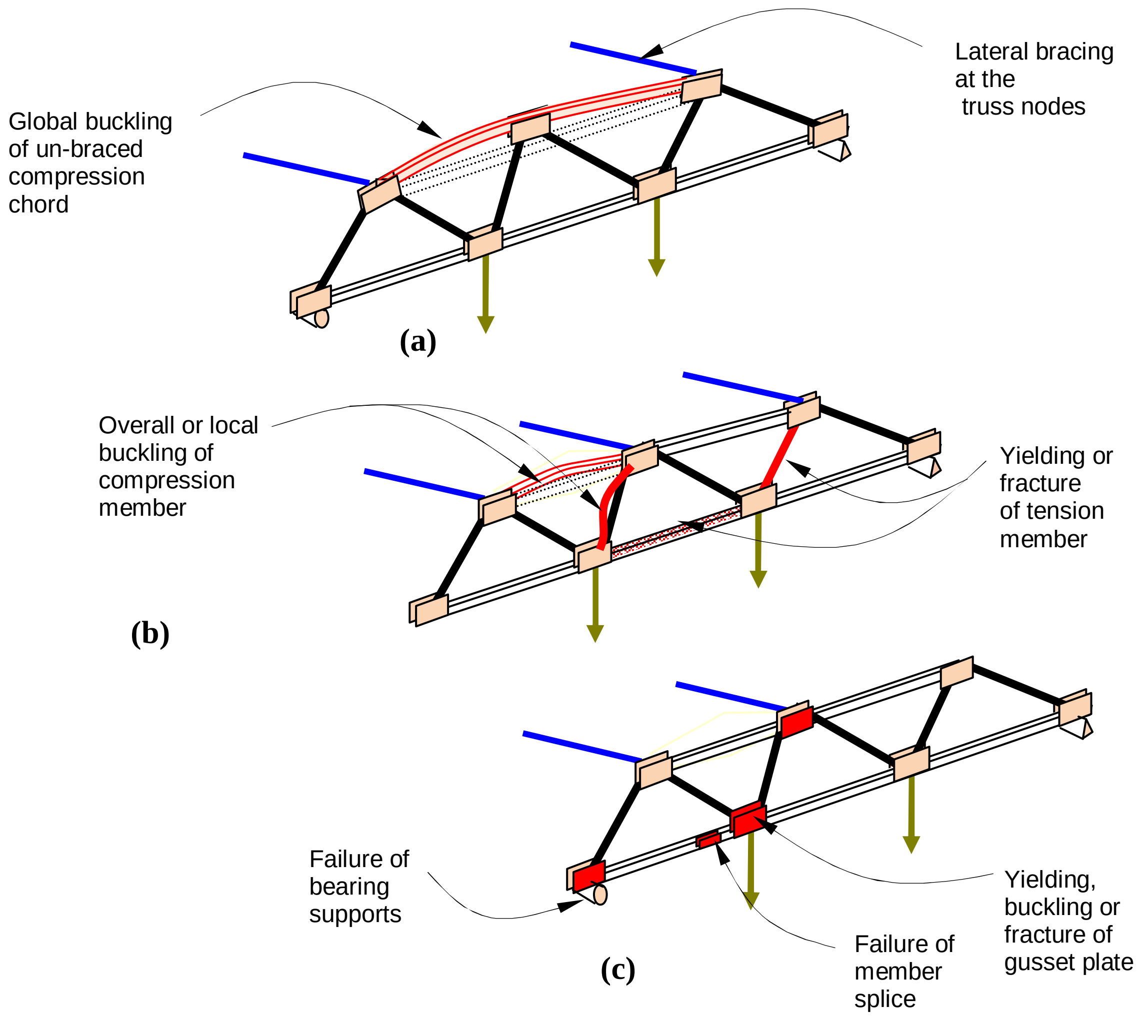


Figure 3.1. Failure modes of a typical truss: (a) global instability of truss system (b) compression or tension failure of members; and (c) compression or tension failure of gusset plate connections or failure of bearing supports

If the tension chord of a truss is not laterally braced at the joints, there is a possibility of out-of-plane movement of the un-braced tension chord. Some have called this behavior *"buckling"* of the tension chord although it might be more appropriate not to use the word *buckling* for a tension member out-of-plane deformation but rather to say truss *"web side sway* buckling" as used for the similar phenomenon in plate girders (AISC, 2005a). For more information on importance of bracing tension chords see Fisher (1983).

To better understand why the tension chord would move out of plane, let us consider the very simple deck truss shown in Figure 3.2(a). The figure also shows the axial forces in the members with the bottom chord ADC being in tension and the top chord ABC in compression. The deck and its transverse beams brace the truss joints at points A, B, and C against lateral movement. However, point D does not have a brace to prevent its lateral movement in direction out of plane of the truss. Notice that member BD is a compressive member. As the applied load P increases, the axial compression in member BD increases. Since member BD does not have significant bracing in out of plane direction at point D (see Figure 3.2(b)), then member BD acting as a column can buckle in a rigid body mode as shown in Figure 3.2(c), resulting in pushing the tension chord ADC out of plane of the truss by moving point D to point D'. This phenomenon was researched by Winter (1958) and later by Yura and his research associates (Yura 1995) for out-of-plane movement of the bottom tension flange of the plate girders. Yura also proposed equations to be used in the design of adequate lateral bracing to prevent this failure mode. The equations are now included in the AISC Specifications (AISC 2005a).

Normally, the lateral bracing designed to resist the horizontal wind or seismic forces is adequate to brace the main gravity load carrying trusses against the global buckling of compressive chord and side sway buckling of the web of the truss pushing the tension chord out of plane. However, the lateral bracing system should be such that it prevents lateral movement of every joint in the chords of the main trusses.

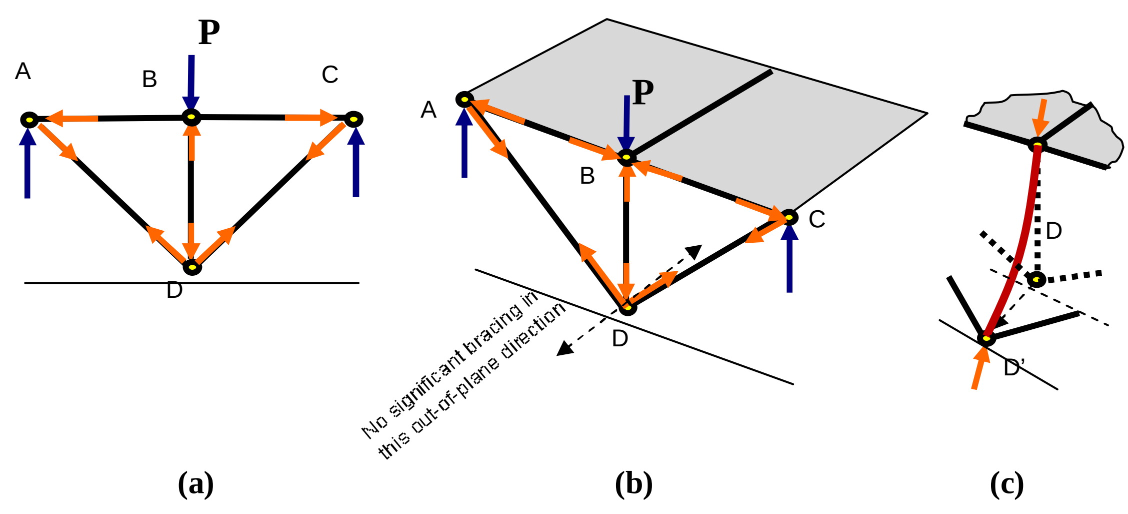


Figure 3.2. Out-of-plane movement of bottom tension chord

3.2.b Failure of Truss Members

Compression members of a truss bridge can fail in overall buckling or in local buckling modes. Members in tension can fail in yielding of the gross area or fracture of the net area. The ends of tension members can also fail in block shear failure mode. In addition to axial load in some cases truss members are also subjected to bending moments either due to direct application of loads between the ends of the member or due to rigidity of the gusset plate connections. Members of a truss subjected to combined loads may fail in a beam-column failure mode. All these failure modes are recognized by the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) which provide provisions to design the truss members considering the failure modes. Since the focus of this report is on gusset plates, the reader is referred to AASHTO Specifications and textbooks for design of tension, compression and beam-column truss members.

3.3.c Failure of Truss Connections and Supports

Failure of truss supports such as bearings and piers are outside the scope of this report while gusset plate connections of the bridge trusses are the main focus. The remainder of this chapter is on design and evaluation of gusset plates. Chapter 4 provides further notes on evaluation of gusset plates in existing bridges especially with regard to corrosion.

3.3 STEPS TO BE TAKEN IN DESIGN AND EVALUATION OF GUSSET PLATES

The following steps are taken in the design and evaluation of gusset plates in new and existing steel bridge trusses:

1. ***Establish forces that act on the gusset plate.*** These forces and the stresses that they develop in a component are the factored "Demand" in the LRFD method. The Demand forces are denoted in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) as $\sum \eta_i \gamma_i Q_i$ where η_i is load modifier factor relating to the ductility, redundancy, and operational importance of the component being designed and the bridge itself. The term γ_i is a statistically based load factor applied to force effects and Q_i represents the force effects (i.e. bending moment, shear, etc. in the member). The values of these parameters are given by the AASHTO Specifications (AASHTO, 2007) considering various failure modes, such as yielding, buckling and fracture and loads creating the force effects such as gravity, wind and earthquake loads. In Section 3.4, later in this chapter, a discussion of these parameters and their values are provided along with specific values of "Demand" (i.e. $\sum \eta_i \gamma_i Q_i$) to be used in design of gusset plates and their components.

2. ***Establish the strength of the gusset plate for all possible limit states.*** These strength values are considered the factored "Capacity" for each failure mode and denoted in

AASHTO Specifications (AASHTO, 2007) as $\phi\ R_n = R_r$ where ϕ is a statistically based resistance factor applied to the nominal resistance of the component being designed considering a specific failure mode. The term R_n is the nominal resistance of the component considering a specific failure mode. The term R_r is denoted by AASHTO as "factored resistance". The failure modes of gusset plates and their components are discussed later in this chapter in Section 3.7 where after discussing each failure mode equations to be used to establish "Capacity" (i.e. $\phi\ R_n$) for each failure mode are provided.

3. Satisfy the basic equation of design which is: Demand ≤ Capacity for each limit state. The AASHTO Specifications (AASHTO, 2007) gives the equation of design in LRFD format in the form of:

$$\sum \eta_i \gamma_i Q_i \leq \phi\ R_n = R_r \qquad \text{(AASHTO Eq.: 1.3.2.1-1)} \qquad (3.1)$$

Following sections provide more detailed information on how to perform each of the above steps.

3.4. ESTABLISHING THE "DEMAND" SIDE OF THE EQUATION OF DESIGN, ($\Sigma\eta_i\ \gamma_i\ Q_i$) FOR GUSSET PLATES

The term η_i in $\Sigma\eta_i\ \gamma_i\ Q_i$ is defined as:

$\eta_i = \eta_D \eta_R \eta_I$ = load modifier: a factor relating to ductility, redundancy and operational importance (AASHTO 2007). The terms in $\eta_i = \eta_D \eta_R \eta_I$ are defined as:

η_D = a factor relating to ductility, as specified in Article 1.3.3 of the current AASHTO Specifications (AASHTO, 2007). A discussion of this factor and the values to be use in design of gusset plates are given in Section 3.4.a below.

η_R = a factor relating to redundancy, as specified in Article 1.3.4 of the current AASHTO Specifications (AASHTO, 2007). A discussion of this factor and values of this factor to be used in design of gusset plates are given in Section 3.4.b below.

η_I = a factor relating to operational importance, as specified in Article 1.3.5 of the AASHTO Specifications (AASHTO, 2007). A discussion of this factor and values of this factor to be used in design of gusset plates are given in Section 3.4.c below.

3.4.a Ductility Factor η_D

The AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) has the following values for η_D:

> For the strength limit state:
>
> $\eta_D \geq$ 1.05 for nonductile components and connections
>
> $=$ 1.00 for conventional designs and details complying with these Specifications
>
> $\geq$ 0.95 for components and connections for which additional ductility-enhancing measures have been specified beyond those required by these Specifications
>
> For all other limit states:
>
> $\eta_D =$ 1.0
>
> (Excerpts from the AASHTO LRFD Specification (2007) . See Footnote "*" below.)

For non-ductile strength limit states (i.e. failure modes) of gusset plates involving fracture of steel, such as fracture of net section, block shear fracture failure, and failure of rivets, high strength bolts and welds, a value of η_D equal to 1.05 seems to be reasonable in design and evaluation of gusset plates. For ductile strength limit states (i.e. failure modes), such as yielding of the gross areas of the gusset plates, a value of η_D equal to 1.0 might be more appropriate. For "semi-ductile" strength limit states (i.e. failure modes) such as buckling of gusset plates or bearing failure of steel bearing against rivets and bolts, which are not brittle but do not demonstrate as much ductility as yielding and until more reliable research data on these limit states become available, using a value of η_D equal to 1.05 seems reasonable.

In summary, until more research on this item is conducted, the following values of ductility factors for gusset plates and their components are proposed:

1. For non-ductile strength limit states of fracture of net section; block shear failure; fracture of critical sections under combined effects of bending, axial load, and shear; buckling of gusset plates; and failure of welds, bolts and rivets:

   $\eta_D \geq 1.05$

   The semi-ductile bearing failure of gusset plate at rivet or bolt holes can also be put in this category.

2. For ductile strength limit states of yielding of gross or perforated

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sections under pure tension or compression as well as yielding of critical sections of the gusset plate under combined effects of bending moment, axial force and shear:

$\eta_D \geq 1.00$

3. For all other limit states:

$\eta_D \geq 1.00$

The AASHTO Specifications (AASHTO, 2007) in Article 1.3.3 on ductility states that "The structural system of a bridge shall be proportioned and detailed to ensure the development of significant and visible inelastic deformations at the strength and extreme event limit states before failure." To satisfy this provision in design of gusset plate connections, two approaches are given in the AASHTO (AASHTO, 2007) commentary to this article. The two approaches are:

> - Joints and connections are also ductile and can provide energy dissipation without loss of capacity.
>
> - Joints and connections that have sufficient excess strength so as to ensure that the inelastic response occurs at the locations designed to provide ductile energy absorbing response.
>
> (Excerpts from the AASHTO LRFD Specification (2007)
> (See footnote "*' below)

For gusset plates it appears that the first approach in the above list with significant inelasticity taking place in the gusset plate may not be quite appropriate. This is because of the relatively small volume of material in the gusset plates that yields compared to members, resulting in a relatively small amount of energy dissipation in the system. In addition, gusset plates have areas of stress concentration that can make them more vulnerable to fatigue than the members while making detection of fatigue cracks in gusset plates harder than those in the members. Furthermore, gusset plates in many cases located right under the expansion joints of the deck are more susceptible to corrosion and loss of section than the members; see Chapter 4 for more notes on corrosion. Also, gusset plates in steel bridge trusses are quite compact with very small areas of gusset plate not covered by the members. As a result, there is very small area of gusset plate between the members that can actually yield and elongate without constraint. Yielding and inelastic elongation of such small areas may not be large enough for observation during a routine visual inspection of the bridge leading to brittle and catastrophic failure of the bridge if the fractured gusset plate was a non-redundant component. See Section 3.7.b below for more on redundancy. Considering these issues, compared to members, gusset plates may not be as reliable in dissipating large amounts of energy.

The second approach in the above box, which results in designing gusset plates to be stronger than members, forcing inelasticity to occur in the members, appears to be more appropriate for design of gusset plates in bridge trusses. In this approach, the gusset plate and its components such as rivets, bolts, and welds remain essentially elastic while the members

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connected to it experience yielding and dissipate energy. This is the approach currently followed in ductile design of gusset plates in steel building structures (AISC 2005c) and (Astaneh-Asl, Cochran, and Sabelli, 2006) which is the concept of "Capacity" design applied to gusset plates by designing gusset plates to have larger capacity than the members to force the members yield while gusset plate remains "essentially elastic". Essentially elastic in this case means that some local yielding, especially around the holes and areas of high stress concentration is allowed to take place as long as the yielded areas are small and are surrounded by elastic areas.

3.4.b Redundancy Factor η_R

The AASHTO Specifications (AASHTO, 2007) has following values for η_R:

For the strength limit state:

$\eta_R \geq$ 1.05 for non-redundant members

$=$ 1.00 for conventional level of redundancy

$\geq$ 0.95 for exceptional level of redundancy

For all other limit states:

$\eta_R =$ 1.0

Excerpts from the AASHTO LRFD Specification (2007). See footnote "*" below)

The AASHTO Specifications (AASHTO, 2007) in Article 1.3.4 under Redundancy states that "Main elements and components whose failure is expected to cause the collapse of the bridge shall be designated as failure-critical and the associated structural system as non-redundant." Almost all gusset plates in main trusses and in steel braced towers are "primary" components whose failure is expected to cause the collapse of the bridge. The gusset plates in the lateral bracing system are "secondary" members. However, according to Section 6.7.5 of the AASHTO Specifications (AASHTO, 2007) if lateral bracing members are included in the structural model used to determine live load force effects they should be considered "primary" members. Today, using structural analysis software, in most cases, truss bridges are modeled as 3-dimensional structures with lateral bracings included in the analytical model. This results in lateral bracing system to be a primary system and gusset plates used in the primary system as well as in the bracings to be "primary" component. Therefore, it is suggested that the gusset plates in truss bridges, whether in the main trusses, the braced towers, or in the lateral bracing system, to be considered as *non-redundant and failure critical* components. To prevent the collapse of non-redundant truss systems, due to gusset plates being failure critical, we need to design these gusset plates to develop full strength of the members attached to them. More on what forces the gusset plates should be designed for and the merits of designing gusset plates for capacity of the members is given in Section 3.9 below when I discuss establishing values of Q_i forces.

Returning to the redundancy factor, η_i if ductile design procedure is followed, where the gusset plate is designed to develop full strength (capacity) of the members attached to it, the redundancy factor can be applied to the capacity of the members to increase or decrease the forces acting on non-redundant or exceptionally redundant members respectively according to the AASHTO provisions above.

In the absence of such special consideration given to provide ductility, the values of η_i for design of gusset plates and their components used in steel truss bridges is proposed to be equal to 1.05 as given by the AASHTO Specification (AASHTO, 2007) in the above box for non-redundant members.

3.4.c Operational Importance Factor η_I

The AASHTO Specification (AASHTO, 2007) has the following values for η

For the strength limit state:

$\eta_I \geq$ 1.05 for important bridges

$=$ 1.00 for typical bridges

$\geq$ 0.95 for relatively less important bridges

For all other limit states:

$\eta_I =$ 1.0

(Excerpts from the AASHTO LRFD Bridge Specification (2007))
(See footnote "*" below)

The Operational Importance factor is related to the importance of bridges and not any specific component characteristics. Therefore, the value of η_I for gusset plate design can be selected from the above AASHTO provisions.

If "ductile" design procedure is followed, where the gusset plate is designed to develop strength of the members attached to it, the Operational Importance factor can be applied to the capacity of the member to increase it for important bridges and to decrease it for relatively less

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important ones and use the factored capacity of the member as the force applied to the gusset plate in design and evaluation.

3.4.d Establishing value of Load Modifier, η_i, for Design of Gusset Plates

The load modifier, η_i, is equal to $\eta_D\,\eta_R\,\eta_I$. In the above three sections values of η_D, η_R and η_I were established. These values are summarized in Tables 3.1 with the last column providing the corresponding value of η_i to be used in Equation 3.1 above. Note that Table 3.1 is for bridges that are in the "Typical" category of the bridge importance as defined by the AASHTO Specifications (AASHTO, 2007) in Articles 1.3.5 and 3.10.3 and their commentaries. For "Important" and "Relatively Less Important" categories of bridge importance, the values of η_i given in the last column of Table 3.1 should be multiplied by 1.05 and 0.95 respectively.

Table 3.1. Values of η_D, η_R and η_I for "Typical" Category of Bridge Importance

Strength Limit State	η_D	η_R	η_I (a)	$\eta_i = \eta_D\,\eta_R\,\eta_I$
Yielding of gross area of steel	1.0	1.05	1.0	1.05
Fracture of net area of steel	1.05	1.05	1.0	1.10
Buckling of gusset plate	1.05	1.05	1.0	1.10
Block shear failure	1.05	1.05	1.0	1.10
Fracture of rivets, bolts or welds	1.05	1.05	1.0	1.10
Bearing failure of rivets or bolts	1.05	1.05	1.0	1.10

(a) Value of η_I equal to 1.0 corresponds to "Typical" bridges. For "Important" and "Relatively Less Important" categories of bridge importance, the values of η_I should be 1.05 and 0.95 respectively.

3.4.e Establishing value of Factored Force Effects, $\gamma_i Q_i$, for Design of Gusset Plates

The term $\gamma_i Q_i$ is the factored force effects used in design of bridge components after multiplied by η_i. For design of members the value of $\gamma_i Q_i$ is established by analyzing a structural model of the bridge subjected to the load factored combinations specified in the Section 3 of the AASHTO Specifications (AASHTO, 2007) under "Loads and Load Factors" and calculating member forces. Then, the member forces that are the result of application of the actual loads are used to design the members. For design of connections and splices in steel bridges, in addition to force effects resulting from the actual application of loads, the actual resistance of the connected member also needs to be considered. Section 6.13 of the AASHTO Specifications (AASHTO, 2007) under "Connections and Splices" states that:

6.13 CONNECTIONS AND SPLICES

6.13.1 General

Except as specified otherwise, connections and splices for primary members shall be designed at the strength limit state for not less than the larger of:

- The average of the flexural moment-induced stress, shear, or axial force due to the factored loadings at the point of splice or connection and the factored flexural, shear, or axial resistance of the member or element at the same point, or

- 75 percent of the factored flexural, shear, or axial resistance of the member or element.

(Excerpt from AASHTO, 2007, SECTION 6: STEEL STRUCTURES)
(see footnote "*' below)

Considering that gusset plates are connections of primary members in steel truss bridges, we can use the above AASHTO provisions to establish the forces and moments that should be used in design of gusset plates. If we represent the factored force effects present in the connection or splice by $\Sigma\eta_i\ \gamma_i\ Q_i$ and the factored resistance of the connected member or element by ϕR_n, the above AASHTO provisions can be written as:

$$(\phi R_n)_{conn.} \geq \ Larger\ of \left\{ \frac{(\Sigma\eta_i\gamma_i Q_i)_{Member} + \eta_i(\phi\ R_n)_{Member}}{2} \quad and \quad 0.75\ \ \eta_i(\phi\ R_n)_{Member} \right\} \tag{3.2}$$

The terms in $\sum \eta_i \gamma_i Q_i$ were defined earlier, as well as in Page 7-Notations. The term $(\phi\ R_n)_{Member}$ is the minimum factored resistance of the connected member considering its failure modes and $(\phi\ R_n)_{Conn.}$ is the factored resistance of gusset plate for the failure mode under consideration. The above equation needs to be applied to each failure mode of the connection or splice being designed separately and for each application, depending on the failure mode being ductile (such as yielding of gross area) or brittle (such as fracture of net area or failure of rivets, bolts or welds), appropriate values of ϕ, η, and γ, as discussed in Sections 3.4.a through 3.4.d of this chapter , are suggested to be used.

The author feels that considering the fact that most of the steel bridge trusses are determinate, as discussed at length in Chapter 2, and therefore they are *fracture critical* systems and considering the fact that establishing accurate stresses in the connections, especially in multi-

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member gusset plates of trusses, is very difficult if not impossible and with higher probability of fatigue cracks and corrosion developing in the connections, designing gusset plate connections in steel truss bridges for the applied loads or only for 75% of the capacity of the connected members, as the above AASHTO (2007) provision implies, will make the gusset plate connections weaker than the connected members. There seems to be ample reasons, first of which is the safety and prevention of progressive collapse of these fracture critical structures, that in the opinion of the author for design of gusset plate connections of determinate trusses we should follow the recommendations that were in the AASHTO Specifications until the 1935 Edition (AASHO, 1935) which were the same as the Waddell's recommendations discussed in Chapter 2 at length, and use 100% of the factored resistance of the connected members as the applied force to the gusset plate by that member and design the gusset plates to have a factored resistance larger than the applied force effect. The direction of the force in the member should be the same as the direction of force in the governing load case. The *full-strength design* recommendation can be written as:

$$(\phi R_n)_{Conn.} \geq \eta_i (\phi\ R_n)_{Member} \tag{3.3}$$

Values of η_i are those given in Section 3.7.d above. The parameter $(\phi\ R_n)_{Member}$ is the lowest factored resistance of the members attached to the gusset plate considering all strength limit states of the members which for truss bridge members are:

1. Yield on the gross section in tension
2. Fracture on the net section in tension
3. Overall buckling of compression members
4. Local buckling of compression areas

The above failure modes are shown in Figure 3.3 for a typical gusset plate connection. Notice that the *fatigue* failure mode is not included in the above list and needs to be considered separately for member design. However, for gusset plate design when we are establishing $(\phi\ R_n)_{Member}$ to be used as the applied force in design or evaluation of a gusset plate, we need to use the value of $(\phi\ R_n)_{Member}$ before it is reduced because of presence of fatigue.

Also notice that *block shear rupture* failure mode is not included in the above list as a member failure mode. The block shear rupture failure mode is considered appropriately to be a connection failure mode by the AASHTO Specifications (AASHTO, 2007) and is given under Section 6.13-Connections and Splices. I have also considered block shear rupture failure mode a strength limit state of connections and have discussed it in Section 3.6.1-Strength Limit States of Members to Gusset Plate Connections later in this report.

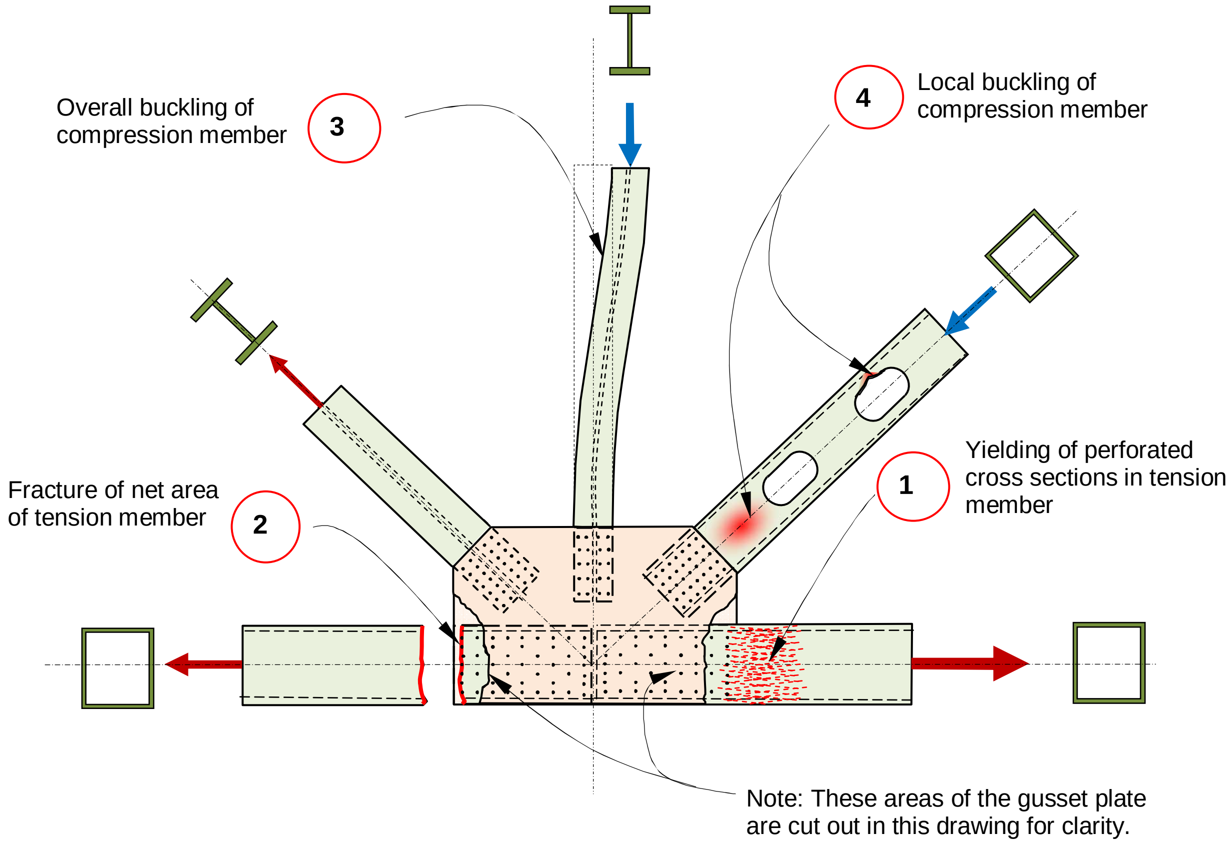

Figure 3.3. Failure modes of members just outside gusset plate

Fracture on the net section is listed in the above list as a strength limit state of the member. This is quite valid for riveted members where rivets are used throughout the member length to connect the member elements and laces to each other. As a result, net sections are present throughout the riveted and laced members. However, in welded truss members that are bolted to gusset plates, net sections are generally present at the end connections of the members. For these members, one might think that fracture on the net section should be considered a connection failure mode and not a member failure mode. Currently, the AASHTO Specifications (AASHTO, 2007), Section 6.8, considers both “Yield on the gross section” and “Fracture on the net section” to be the strength limit states for tension members.

Another item that could be included in the above list is *failure of a tension or compression member* to satisfy the slenderness requirements of the AASHTO Specifications (AASHTO, 2007). In design of new trusses, these limits are required to be satisfied. In evaluating an existing truss bridge, the slenderness requirements need to be checked, and most likely they will be satisfied since from the early days of steel truss bridge design in the 19th century, the slenderness requirements, very similar to those in current AASHTO Specification (AASHTO, 2007), were followed and included in the specifications and bridge engineering textbooks. In any event, in evaluating existing bridges for rating, the slenderness requirements need to be checked in case it was missed in the original design.

The AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) has design equations and provisions in Load and Resistance Factor Design (LRFD) format for all of the above failure modes. Since this report is focused on design of gusset plates, I refer the reader to the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) for these equations and do not repeat them here. Using the equations, the factored resistance $(\phi\ R_n)_{Member}$ of each member connected to the gusset plate can be established. Then these factored resistance values can be used in either in Equation 3.2 or 3.3 above to establish the "Demand" (i.e. force effects) on the gusset plate under design or evaluation.

3.5 ESTABLISHING THE "CAPACITY" SIDE OF THE EQUATION OF DESIGN, $(\phi R_n)_{Conn.}$ FOR GUSSET PLATES

In design of gusset plates, the *capacity* side of the basic equation of design given in the form of *Demand* ≤ *Capacity* is denoted as $(\phi R_n)_{conn}$ where ϕ is the resistance factor and R_n is the nominal resistance. The factored resistance $(\phi R_n)_{conn}$ is the smallest value established for the gusset plate considering all strength limit states of the gusset plate and its components. This section is devoted to establishing the term $(\phi R_n)_{conn}$ for gusset plates. First, a brief historical background on AASHTO provisions relevant to design of gusset plates and establishing their strength is given followed by a discussion of failure modes (i.e. strength limit states) of the gusset plates and their components such as rivets, bolts and welds. For each strength limit state, I have provided appropriate value of resistance factor ϕ as well as procedures and equations for establishing nominal resistance R_n.

3.5.a Historical Background on AASHTO Provisions Relevant to Design of Gusset Plates

Since the early days of the AASHO Specifications, the predecessor to the current AASHTO Specifications, there were provisions on design of gusset plates and how to establish their strength (i.e. resistance). The 1924 Standard Specifications for Steel Highway Bridges (AASHO 1924), which was adopted by the American Association of State Highway Officials (AASHO), included the following provisions relevant to design of gusset plates. This document appears to be one of the earliest documents on design of steel bridges and later was incorporated into the first AASHO Standard Bridge Specification published in 1931.

22. Strength of connections
Unless otherwise provided, all connections shall be proportioned to develop not less than full strength of the members connected.
No connection, except for lacing bars and handrails, shall contain less than three rivets.

23. Splices
Continuous compression members in riveted structures, such as chords and trestle posts, shall have milled ends and full contact bearing at the splices.
All splices, whether in tension or compression, shall be proportioned to develop the full strength of the members spliced and no allowance shall be made for milled ends of compression members.
Splices shall be located as close to panel points as possible and, in general, shall be on that side of the panel point which is subjected to the smaller stress.
The arrangement of the plates, angles, or other splice elements shall be such as to make proper provisions of the stresses in the component parts of the members spliced.
…..

26. Gusset plates
Gusset or connecting plates shall be used for connecting all main members, except in pin-connected structures. In proportioning and detailing these plates the rivets connecting each member shall be located, as nearly as practicable, symmetrically with the axis of the member. However, the full development of the member shall be given due consideration. The gusset plates shall be of ample thickness to resist shear, direct stress, and flexure acting on the weakest or critical section of maximum stress.
Reentrant cuts shall be avoided as far as possible.

27. Minimum thickness of metal
The minimum thickness of structural steel shall be 5/16 inch except for fillers and railings. However, gusset plates shall not be less than 3/8 inch in thickness.
Metals subjected to marked corrosive influence shall be increased in thickness.

(Excerpts from the Standard Specification for Steel Highway Bridges (AASHO 1924))
(See footnote "*" below)

Following items in the above provisions are quite important;

1. The connections were required to be designed for the capacity of the members and not for the forces in the member, "unless otherwise provided" as stated in Article 22 above. I was not able to establish from the literature what this term *unless otherwise provided* meant to the designer and what these other provisions were. However, it is clear that the notion of "capacity design" was well-emphasized in the very early steel bridge design specifications. The provision of connections being as strong as the members may have been the reason why so many riveted truss bridges have done so well in the past when subjected to the extreme forces of earthquakes or winds for which they were not designed. Alas, this provision was diluted to some extent in the later releases of the AASHTO Specifications (AASHTO, 2007) as discussed below.

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2. The issue of splices in members of steel bridges, especially in relatively long span trusses, is important. In the case of splices, it is clear that according to the above Article 23 of AASHO 1924, they had to be designed for the full capacity of the members being spliced even for splices of continuous compression chord members with the ends of the chord members milled and in full contact. Although the provision does not specifically prohibit having a chord spliced inside the gusset plate, by emphasizing the location of the splice being near the gusset plate, the provision implies that the splices for the chord members were not to be located inside the gusset plate connections. A study of drawings of a number of all riveted bridges built during the 1920s, '30s, and '40s indicates that the designers have indeed followed this provision and had the splices outside the gusset plate panel point. Figure 3.4(a) shows a bottom chord panel point with the chord splice outside the gusset plate area. However, as discussed in Chapter 2, in some truss bridges, including the collapsed I-35W bridge in Minnesota, the chord splices are inside the gusset plate panel point as shown in Figure 3.4(b). When a chord member splice is placed inside the gusset plate connection, the stresses in the gusset plate can be much larger than in a case where the chord member is continuously passing through the gusset plates, which results in relatively small forces transmitted to the gusset plate.

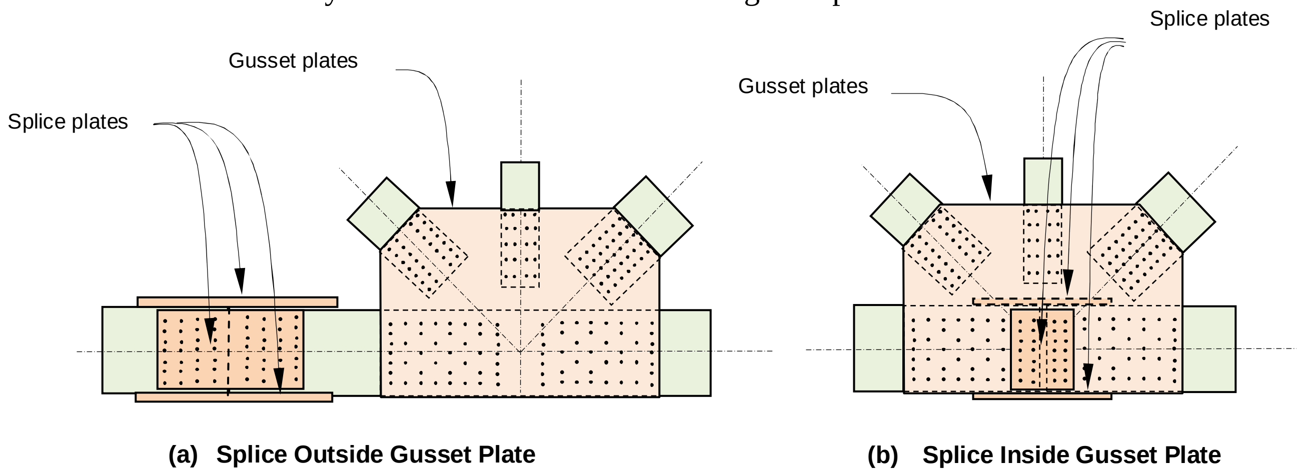


Figure 3.4. (a) Chord splice outside gusset plate and; (b) chord splice inside gusset plate.

3. The AASHO 1924 Article 26, given above, specifies that "the full development of the member shall be given due consideration" (AASHO 1924). Although giving "due consideration" is not as strong as a requirement for design of rivets and plates in the gusset plate connections for capacity of the members, it is as close as one can get to such capacity design. As for the thickness of gusset plates, the AASHO 1924 specifies a minimum thickness of 3/8 inch. In the literature ½ inch is mentioned as the minimum. Bresler and Lin (1960) recommend a minimum gusset plate thickness of 3/8 to ½ inch for light trusses and 5/8 to 7/8 inch for heavier ones without specifying how one can identify a truss as 'light" or 'heavier".

3.5.b Current AASHTO Provisions Relevant to Design of Gusset Plates

Let us now examine the provisions of the most current AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) relevant to design of gusset plates, which includes:

6.14.2.8. Gusset Plates	**6.14.2.8 [Commentary]**
The provisions of Article 6.13.4 and 6.13.5 shall apply, as applicable. Gusset or connection plates should be used for connecting main members, except where the members are pin-connected. The fasteners connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the full development of the elements of the member should be given consideration. Reentrant cuts, except curves made for appearance, should be avoided as far as possible.	Gusset plates may be designed for shear, bending, and axial force effects by the conventional "method-of-Sections" procedures or by continuum methods.
The maximum stress from combined factored flexural and axial loads shall not exceed $\phi_y F_y$ based on the gross area.	Plastic shape factors or other parameters that imply plastification of the cross section should not be used.
The maximum shear stress on a section due to the factored loads shall be $\phi_v F_y/\sqrt{3}$ for uniform shear and $\phi_v 0.74\ F_y/\sqrt{3}$ for flexural shear computed as the factored shear force divided by the shear area. If the length of unsupported edge of a gusset plate exceeds 2.06 $(E/F_y)^{1/2}$ times its thickness, the edge shall be stiffened. Stiffened and unstiffened gusset edges shall be investigated as idealized column sections.	

(Excerpts from the AASHTO LRFD Bridge Design Specifications (2007), See footnote "*" below.)

A few observations on the above provisions:

1. The Article 6.13.4 in the above box is on the *limit state of block shear rupture* indicating that this limit state should be considered in design of gusset plate connections.
2. The Article 6.13.5 in the above box is on the design of elements of gusset plate connections, including the gusset plate itself, for *limit states of tension or shear failure.*
3. In design of member to gusset plate connections, the current AASHTO specifications has almost exactly the same provision as it was in the AASHO Specifications in 1924 given earlier, which state that "For fasteners connecting each member…the full development of the elements of the member should be given consideration."

In the above provisions the use of curved edges for appearance is allowed. As shown in Chapter 2, this is done in some bridges in the past including several bascule bridges in Chicago and some cantilever bridges in Michigan, Ohio and elsewhere. It is not clear what effect the curved edge of gusset plate would have on the behavior of a gusset plate especially edge buckling. More information on edge buckling of curved gusset plates is given in Section 3.7.e later in this chapter.

4. In the above box, there are statements on how to establish factored resistance of a gusset plate under the combined effects of bending, axial, and shear stresses. It appears that the intent of the provision is that the maximum normal stress due to the addition of bending and axial normal stresses at any point within the gusset plate should not exceed the yield stress. The Commentary to these provisions, (see right side of the above box), states two methods: the Method of Sections and the continuum method (Finite Element Analysis) may be used to establish the normal and shear stresses in the gusset plate. Since the Commentary to these provisions also states that "Plastic shape factors or other parameters that imply plastification of the cross section should not be used" in establishing applied stresses, we need to consider the equations based on elastic behavior. For example, in establishing the maximum bending stresses using "Method-of-Section", after establishing bending moment on a particular critical section of the gusset plate, we need to use the equation: $\sigma_b = M / S_x$, to establish the normal stress due to bending, where M is the bending moment acting on the critical cross section and S_x is the elastic section modulus of the cross section. More information on combined stresses in the gusset plate is given in Section 3.7.f later in this Chapter.

5. Separate from the normal stresses. The maximum shear stress at any point within the gusset plate should not exceed $\phi_v F_y/\sqrt{3}$, which is the shear yield stress according to the Von Mises criteria. In addition to this limit, in the above box there is another limit for shear stresses in the gusset plate. This limit is for the maximum uniform shear stress on a critical section of the gusset plate not to exceed a value of $(\phi_v)(0.74\,F_y/\sqrt{3})$. The uniform shear stress is calculated by dividing the shear acting on the critical cross section of the gusset plate under consideration by the shear area of the cross section. It is not specifically stated here if the shear area referred to here is the gross area or the net section area that is subjected to shear. However, from the fact that the limit of $(\phi_v)(0.74\,F_y/\sqrt{3})$ is related to yield stress F_y, the shear area here can be interpreted to be the gross shear area. Otherwise, if the net area was intended to be the shear area, the ultimate strength, F_u of the steel would have been used instead of F_y.

   It should be realized that in bolted and riveted gusset plates, the net shear passing through the bolt or rivet rows should also be checked for the limit state of shear fracture on the net area. More information on this limit state is provided in Section 3.7.g later in this Chapter.

6. In the above box, edge buckling of the gusset plate is considered a failure mode and the limit of $2.06\,(E/F_y)^{1/2}$ is provided to prevent this failure mode. However, buckling of the inner areas of the gusset plate itself is not addressed in these provisions. In Section 3.7.f. of this report this limit state of failure of gusset plates is

discussed.

7. Another important limit state of gusset plate that is not in the above box is a failure of gusset plate in the "Whitmore's Tension Area" of the gusset plate just beyond the end of tension members. These areas of gusset plate can fail in tension yielding of gross area or tension fracture of net area. This item is discussed in Section 3.7.b later where equations to establish the resistance of gusset plates based on the failure of Whitmore's tension areas are provided.

3.5.c Material of Gusset Plates

The material of gusset plate should be selected following Article 6.4 of the AASHTO LRFD Bridge Design Specification (AASHTO, 2007). There are no restrictions on the use of any of the steels specified by AASHTO Specifications (AASHTO, 2007) in gusset plates. However in using very high strength steel such as AASHTO M 270 Grades 100/100W (ASTM A 709 Grades 100/100W), due to closeness of the specified yield stress and maximum tensile strength, it is possible that a brittle fracture on the net section in the bolted and riveted gusset plates will become the governing failure mode over yielding of the gross section. The brittle net section fracture being the governing failure mode will not result in ductile and desirable behavior of the gusset plate. Steels with 36 and 50 ksi specified minimum yield strength are frequently and successfully used in gusset plates. If weathering steel is used for gusset plates, weathering grade bolts, nuts, and washers should also be used for the joining components.

The use of High Performance Steel (HPS) plates in new gusset plates as well as in retrofit of existing gusset plates is recommended. Following table from FHWA(2002) provides basic mechanical properties of HPS. More information on chemical, fatigue and fracture properties of HPS can be found in FHWA (2002) and AASHTO Specifications (AASHTO, 2007).

Table 3.1* - Mechanical Properties for High Performance Steel Plates

	HPS 50W **Up to 4" As-Rolled**	**HPS 70W** **Up 4" (Q&T). 2" (TMCP)**
Yield Strength, *Fy*, ksi (MPa) min.	50 (345)	70 (485)
Ultimate Tensile Strength, Fuksi (MPa)	70 (485)	85-110 (585-760)
CVN, minimum Longitudinal orientation	25 ft.-lbs. (41 J)@ 10°F (-12°C)	30 ft.-lbs. (48 J)@ -10°F (-23°C)

*(Above table is Table 2.2.1. from: http://www.fhwa.dot.gov/bridge/guide02.htm#2

In evaluating the existing gusset plates, material properties of steel used in the members as well as the gusset plates are needed. The best source of information for material properties is the shop drawings (if available) and the original construction plans. Previous editions of the AASHTO Specifications can also be helpful to establish the type of steel used in the bridge. Normally, the construction plans state which edition of the AASHTO Specifications (for highway bridges), or AREA Specifications (for railroad bridges) were used. In many cases, allowable stresses are also given on the construction plans. Knowing the allowable stresses and the specific edition of the Specifications used in the design of a bridge, we can use the Specifications to find out what type of steel in that particular edition would correspond to the given allowable stresses and establish the type of steel. Then by referring to the literature, we can establish the mechanical properties of the steel used in an existing steel bridge. If all fails and the mechanical properties cannot be established reliably, non-destructive tests, such as hardness tests, and destructive tests, such as tension coupon tests can be performed using members and gusset plates. The destructive tests that require cutting coupons from the bridge are the last resort and should be done with utmost care and after conducting necessary stress analysis to ensure that removing the portion of a member or a gusset plate will not affect the structural safety or stability of the component or the bridge.

References by Hassett (2003), Hatfield (2001) and Vermes (2007) provide useful information on mechanical properties of steel used in historical steel structures. Table 3.2 below is from Vermes (2007) and Tables 3.3, 3.4 and 3.5 are from Hatfield (2001).

Table 3.2. Properties of Steel Used in Riveted Bridges *

	Carbon Steel	**Silicon Steel**	**Nickel Steel**
Yield Strength**, F_y, in ksi	33 to 36	48 to 50	48 to 50
Ultimate Tensile Strength, F_u in ksi	60 to 70	80 to 95	85 to 100
Allowable Stress in tension**, F_a , in ksi	16 to 18	24 to 27	27 to 33

* Above Table is based on a Table in Ref: (Vermes, 2007) at http://www.otecohio.org

** Documented values 1910 to 1960.

Table 3.3 . Properties of Wrought Iron Used in Riveted Bridges *

Table 2. Tensile strengths of wrought iron and factors of safety for tension fracture.

Source	Year	Grade of Steel	Yield stress, minimum, ksi (MPA)	Ultimate stress, minimum, ksi (MPA)	Allowable stress ksi (MPA)	Factor of safety for fracture
Carnegie Kloman & Co.[7]	1873	wrought iron			14 (97)	3
Waddell[15]	1883	iron	26 (179)	50 (345)	8 to 12.5 (55 to 86)#	4.0 to 6.2#
Phoenix Iron Co.[7]	1885				12 (83)	
IATM[11]	1900	refined iron	25 (172)	48 (331)		
		test iron class A	25 (172)	48 (331)		
		test iron class B	25 (172)	50 (345)		
		stay-bolt iron	25 (172)	46 (317)		
Waddell[16]	1901	wrought iron	26 (179)	50 (345)	13 (90)	3.8
AASHTO[3]		wrought iron			14.6 (101)*	

* for inventory rating # depending on service class and influence area

Above Table is from Ref: (Hatfield, 2001) at http://www.jflf.org/pdfs/wi301/historicbridges.pdf (printed with permission)

Table 3.4 . Properties of Wrought Iron Used in Riveted Bridges *

Table 2. Tensile strengths of wrought iron and factors of safety for tension fracture.

Source	Year	Grade of Steel	Yield stress, minimum, ksi (MPA)	Ultimate stress, minimum, ksi (MPA)	Allowable stress ksi (MPA)	Factor of safety for fracture
Carnegie Kloman & Co.[7]	1873	wrought iron			14 (97)	3
Waddell[15]	1883	iron	26 (179)	50 (345)	8 to 12.5 (55 to 86)#	4.0 to 6.2#
Phoenix Iron Co.[7]	1885				12 (83)	
IATM[11]	1900	refined iron	25 (172)	48 (331)		
		test iron class A	25 (172)	48 (331)		
		test iron class B	25 (172)	50 (345)		
		stay-bolt iron	25 (172)	46 (317)		
Waddell[16]	1901	wrought iron	26 (179)	50 (345)	13 (90)	3.8
AASHTO[3]		wrought iron			14.6 (101)*	

* for inventory rating # depending on service class and influence area

* Above Table is from Ref: (Hatfield, 2001) at http://www.jflf.org/pdfs/wi301/historicbridges.pdf (printed with permission)

Table 3.5 . Properties of Steel Used in Riveted Bridges *

Table 1. Tensile strengths of steel and factors of safety for tension fracture at net section.

Source	Year	Grade of Steel	Yield stress, minimum, ksi (MPA)	Ultimate stress, minimum, ksi (MPA)	Allowable stress on net section, ksi (MPA)	Factor of safety for fracture
Pottsville Iron & Steel Co.[7]	1887				15.6 (108)	
Carnegie Phipps & Co.[7]	1889-1893	for bridges			12.5 (86)	
IATM[10]	1900	medium	35 (241)	60 (414)		
Waddell[16]	1901	medium	35 (241)	60 (414)	16 (110) 18 (124)	3.8 3.3
Burr and Falk[4]	1901					3.5 to 6.0@
Copper[12]	1909	medium			10 to 25 (69 to 720)#	2.4 to 6.0#
Michigan[13]	1910	medium	30 (207)	60 (414)	15 (103)	4.0
Bethlehem Steel Co.[7]	1907-11	moving loads			12.5 (86)	
Waddell[17]	1916	medium	35 (241)	60 (414)	16 (110)	3.8
Ketchum[12]	1920	medium			16 (110)	
AASHTO[3]	pre 1905 1905-36		26 (179) 30 (207)	52 (358) 60 (414)	26 (179)* 30 (207)*	2.0* 2.0*
AASHTO[1]	current	ASTM A36	36 (248)	58 (400)	29 (200)*	2.0*

* for inventory rating, less than 100,000 load cycles
@ depending on span # depending on type of load, including impact factor

* Above Table is from Ref: (Hatfield, 2001) at http://www.jflf.org/pdfs/wi301/historicbridges.pdf

3.6 VALUES OF ϕ_I AND R_N FOR GUSSET PLATE CONNECTIONS

This section focuses on strength limit states of gusset plate connections and provides information on how to establish values of ϕ_i and R_n for each strength limit state.

Failure modes of connection of a member to a gusset plate are:

1. Shear failure of the bolts or rivets and failure of welds (in welded connections)

2. Bearing failure of the rivets or bolts bearing against the member or gusset plate
3. Edge distance or end distance failure of gusset plate or member elements in riveted or bolted connections
4. Rivet or bolt spacing failure
5. Block shear rupture of member

The above failure modes are shown in Figure 3.5. The AASHTO LRFD Bridge Design Specifications (AASHTO 2007) has design equations and provisions in Load and Resistance Factor Design (LRFD) format for all of the above failure modes.

Photo: www.historicalbridges.org

Photo by A. Astaneh-Asl

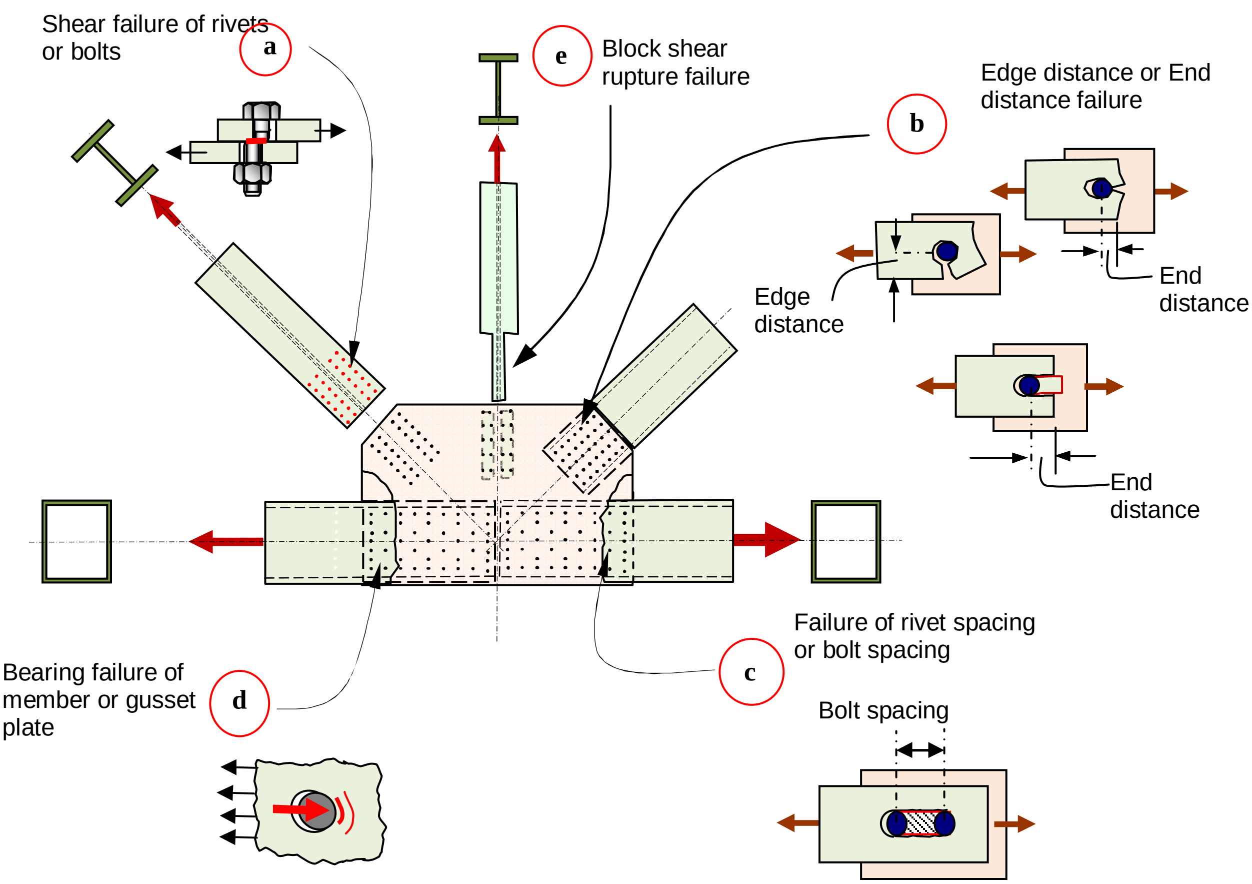


Figure 3.5. Failure modes of connection of the members to the gusset plates

In designing the connection of a member to the gusset plate, all of the above failure modes need to be considered and the connection should be designed using the AASHTO (2007) equation of design given by Equation 3.1 earlier. In this case, left side of equation, $\sum \eta_i \gamma_i Q_i$ should be taken as equal to the reduced capacity of the member, $(\phi\ R_n)_{Member}$ established. The right side of the above equation of design is the factored (reduced) capacity established for each failure modes 1 and 2 above for the connection of member to gusset plate following the corresponding AASHTO equations as given below. The 3rd and 4th failure modes in the above list are prevented by satisfying the limits of edge distance and bolt spacing given by AASHTO (2007) for member ends.

In the following sections, each of the limit states listed above for connection of members to gusset plates is discussed and design equations for calculating values of $\phi\ R_n$, the factored resistance, for these limit states are provided. The design equations are in the Load and Resistance Factor Design (LRFD) format and consistent with the current AASHTO LRFD Bridge Design Specifications (AASHTO, 2007).

3.6.a Shear Failure of the Rivets or Bolts

Figure 3.6 shows stress-strain curves for material of the rivets and bolts (Kulak, Fisher, and Struik 2001). The A325 bolt shows larger material ductility than the A490 bolt. Tests (Kulak et al. 2001) indicate that the shear strength of a bolt is 8–13% higher if shear is due to compression in the connected members. The A325 shows more ductility as a connector than the A490 bolt. Also, notice that a portion of the shear deformation in bolted connections can be due to the slippage of the plates, which depends on the gap between the bolt hole and the bolt shank. Rivets, not being a high strength material, show less strength but more ductility than both the A325 and A490 high strength bolts.

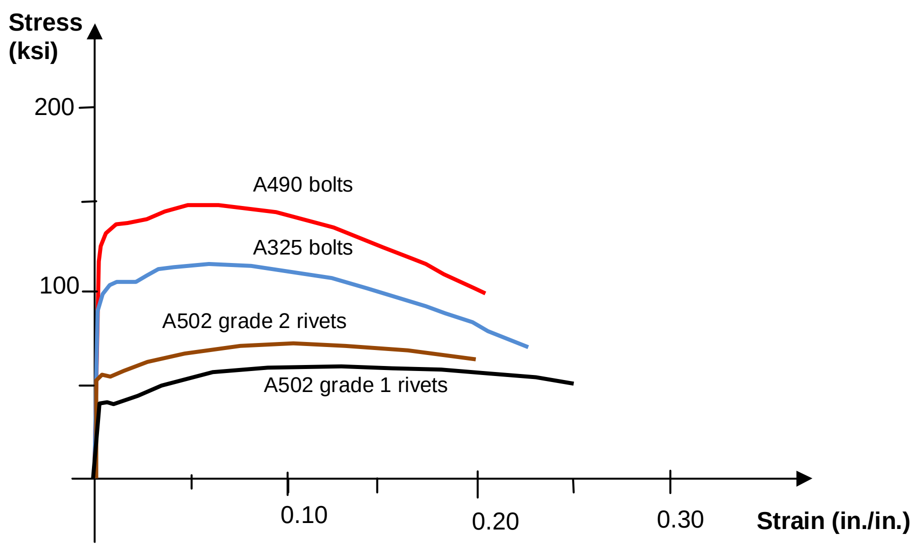


Figure 3.6. Schematic behavior of bolt assemblies subjected to shear
(Source: Kulak, Fisher, and Struik 2001)

The factored resistance $R_r = \phi R_n$ for this limit state is given as:

$$R_{r-Bolt} = N \phi_{bs} R_{nbs} \tag{3.4}$$

Where;

R_{r-Bolt} = factored shear resistance of bolt group

N = number of bolts in the group

ϕ_{bs} = resistance factor for bolt shear equal to 0.80 (as per AASHTO (2007) Article 6.5.4.2.)

R_{nbs} = nominal shear strength of a single bolt in the group connecting the member to the gusset plate (see Article 6.13.2.7 of AASHTO (2007)).

For riveted connections similar equations are used but in calculating value of R_n as per article 6.13.2.7, instead of shear strength of bolt, shear strength of rivet material should be used. Reference Kulak et al (2001) is a very good source for information on behavior and design of riveted connections.

3.6.b Bearing Failure at the Bolt Holes

When bolts are subjected to shear, depending on the amount of pre-tensioning, at one point the bolts will slip and will bear against the wall of the hole. Until slippage occurs, transfer of shear is through friction at the faying surfaces. After slippage, the primary shear transfer mechanism is by bearing compression of the bolt against the wall of the hole as shown in Figure 3.7. After slippage is complete and the bolt is bearing against the wall, the behavior of the joint depends on the relative strength of the bolt and the connected part. For typical civil engineering applications, the bolts are usually high strength and the connected part in some occasions is high strength (F_y = 70–90 ksi) and more often moderate strength (F_y = 50–65 ksi) or low strength (F_y = 36 ksi). In all of these applications, high strength bolts are stronger than the connected part. In these cases, when the bolt is bearing against the wall of the hole, the bulk of yielding will occur in the connected part while the bolt remains essentially elastic.

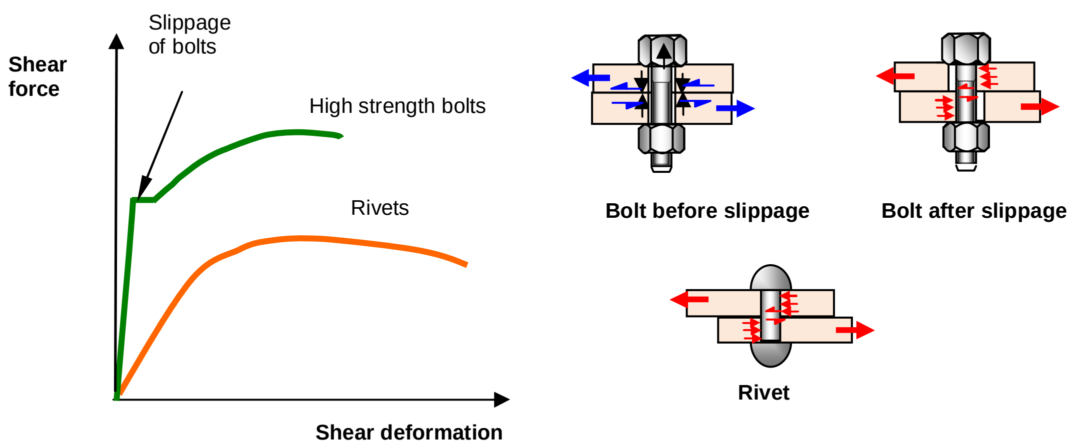


Figure 3.7. Schematic behavior of bolts and rivets subjected to shear

In rivets, there is no gap between the rivet and the hole. As a result, rivets are in bearing from the start of the application of the shear load. In riveted joints in existing bridges, the yield strengths of the rivets and the connected parts are usually close. In addition, other than relatively small pre-tensioning developed in the rivets due to cooling and shrinkage during the riveting process, riveted joints do not have significant friction forces in their faying surfaces. Behavior of a typical rivet in shear is shown in Figure 3.7 as well.

The design equation for failure mode of bearing failure of a bolt pressing against the material of the member in the bolt hole is similar to Equation 3.4, which was given for bolt shear failure:

$$\eta_i (\phi\ R_n)_{Member} \leq \phi_{br} R_{nbr} \tag{3.5}$$

Where,

η_i and $(\phi\ R_n)_{Member}$ are the same as discussed in Section 3.7.a above and values of ϕ_{br} and R_{br} are defined as:

ϕ_{br} = Resistance factor for bearing failure of member due to bolt pressing against it equal to 0.80 as per AASHTO (2007) Article 6.5.4.2.

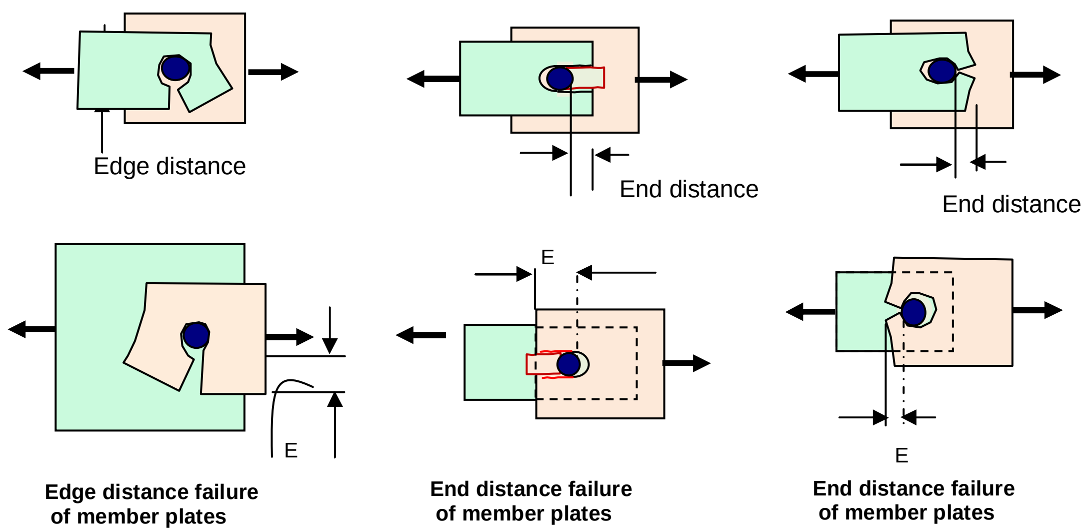
R_{nbr} = Nominal bearing strength of bolt group given in Article 6.13.2.9 of AASHTO (2007).

The reader is referred to Article 6.13.2.6.2 of the AASHTO Specification for these provisions.

3.6.c. Minimum Edge Distance and End Distance on the Member

Figure 3.8 shows edge distance and end distance failure modes. The AASHTO LRFD Bridge Design Specification (AASHTO, 2007) Article 6.13.2.6 provides a table as well as information on minimum edge distances and end distances for "sheared" and "rolled" edges to prevent these failure modes from occurring.

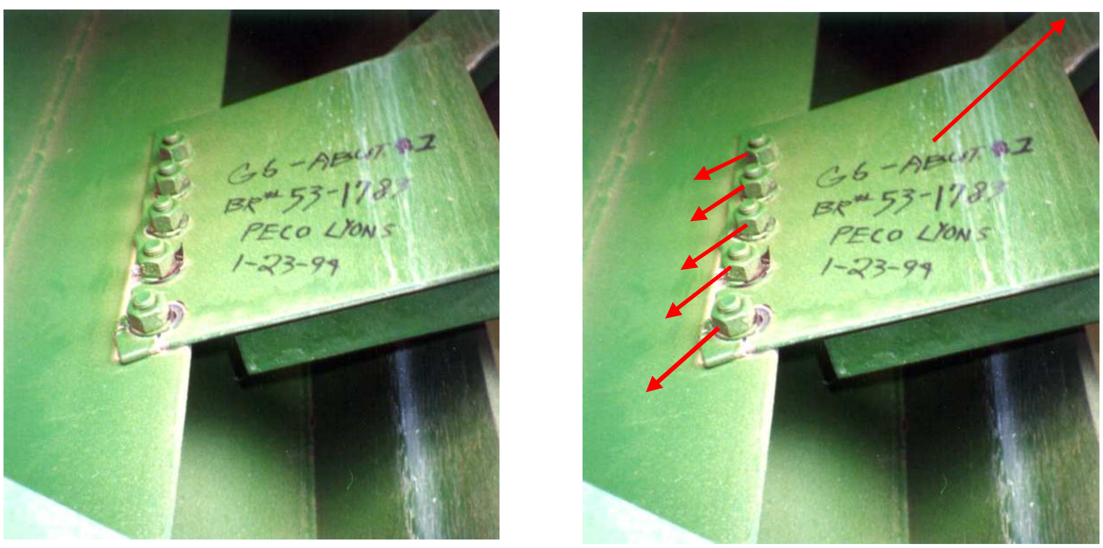
In design of steel bridges, the minimum edge distance is as given by AASHTO (2007). However, since the early days of riveting, it has been recommended that "wherever practicable this distance shall be at least two (2) diameters of the rivet" (Waddell, 1898). Waddell's prudent recommendation seems to be still valid not only for riveted but also for more modern bolted connections such as gusset plates. Figure 3.9 shows the edge and end distances of a gusset plate in a plate girder bridge failure that occurred during the 1994 Northridge earthquake in California.



Fracture of gusset plate

Probable forces acting on the gusset plate causing edge distance fracture

Figure 3.8. Edge distance and end distance failures in a gusset plate of cross bracing



Photos by A. Astaneh-Asl

Figure 3.9. Edge distance and end distance failures in the member connected to the Gusset plate

3.6.d Minimum Bolt Spacing

One of the failure modes of the bolts subjected to shear is failure of the bolt spacing. To prevent failure of the fastener spacing, the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) states that the center-to-center distance of the standard bolts holes should not be less than three times the nominal diameter of the bolt. For slotted or oversized holes, the minimum clear distance between the edges of the adjacent holes parallel and perpendicular to the direction of the force should not be less than two times the diameter of the bolt. The AASHTO LRFD Bridge Design Specification (AASHTO, 2007) has provisions on maximum spacing of bolts that are adjacent to the free edge and act as “sealing” bolts to prevent penetration of moisture in joints.

3.6.e Block Shear Rupture

Block shear rupture failure mode can occur at the end of members. The AASHTO LRFD Specification (AASHTO, 2007) in Article 6.13.4 requires that all tension connections that include gusset plates be investigated for block shear failure. Block shear failure is considered a non-ductile failure mode. This failure mode is discussed further in Section 3.7.c later in this chapter for gusset plates where equations of design for block shear rupture limit state are given.

3.7 VALUES OF ϕ_i AND R_n FOR GUSSET PLATE ITSELF

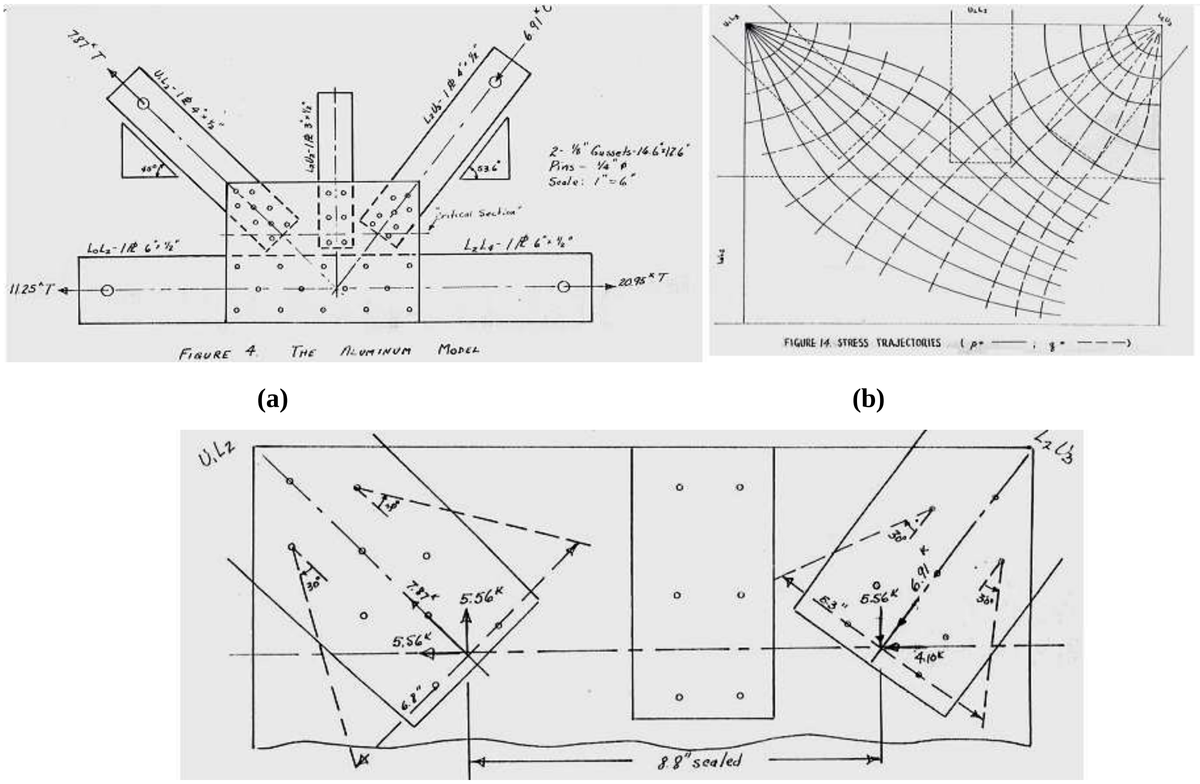
Values of ϕ_i and R_n should be established for each failure mode (or limit state) of gusset plates. Based on the information on the behavior of gusset plates, failure modes (or strength limit states) of the gusset plate can be listed as:

1. Yielding of the Whitmore gross area at the termination areas of the tension members
2. Fracture of Whitmore net section at the termination areas of the tension members
3. Block shear failure of the gusset plate where tension members are attached to it
4. Buckling of the gusset plate in compression
5. Buckling of the “free” edges of the gusset plate
6. Yielding of critical gross areas of the gusset plate under combined bending moment, shear, and axial force
7. Fracture of critical net sections of the gusset plate under combined bending moment, shear, and axial force

The above limit states (or failure modes) are discussed in the following sections and corresponding equations for establishing their factored resistance $\phi_i R_n$ in LRFD format are provided.

3.7.a Yielding of a Whitmore Section in the Gusset Plate

R. E. Whitmore (1950) conducted tests of aluminum gusset plate specimens, Figure 3.10(a), and established stress trajectories inside the gusset plates, Figure 3.10(b). He proposed that the stress distribution within the gusset plate be assumed to be under an angle of distribution equal to 30 degrees; see Figure 3.10(c). This simplification of stress distribution under a single angle was not new, and many decades before Whitmore’s tests, bridge engineers had used the concept with an angle of distribution of 22–25 degrees. However, Whitmore's contribution was very valuable in the sense that he for the first time used a metallic (aluminum) gusset plate and strain gages to actually measure the stresses. By using such a proper scientific method, his suggestion of using a30-degree angle for the distribution of stress inside the gusset plate gained more acceptability and resulted in this method being associated with his name and used today in design of gusset plates.

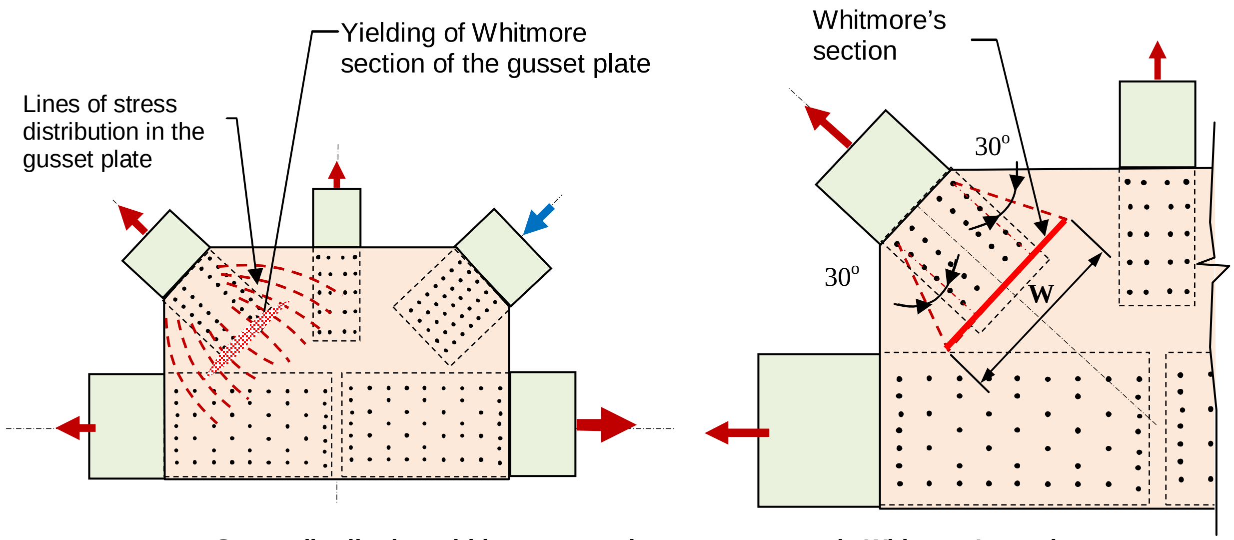


(a) (b)

(c)

Figure 3.10. Whitmore's specimen, trajectories of stress in the specimen, and Whitmore's suggestion of 30-degree angles for stress distribution (Graphics from Whitmore 1950)

Yielding of gusset plates can occur due to direct tension or compression, bending moment, shear, and the combination of those. Yielding due to direct tension or compression can occur within the Whitmore's effective width area due to "direct" stresses on the gross area of the Whitmore's section, Figure 3.11.



a. **Stress distribution within a gusset plate** **b. Whitmore's section**

Figure 3.11. Yielding of the Whitmore section of a gusset plate

The Whitmore section, as shown in Figure 3.11, is established by drawing two 30-degree lines from the center of the last bolts to intersect a line passing through the first bolt line and perpendicular to the axis of the member. The area of the gusset plate between the two 30-degree lines is considered the gross area of Whitmore's section. Yielding of the Whitmore section of a gusset plate is the most desirable failure mode. The following equation can be used to check this limit state of strength:

$$\eta_i (\phi\ R_n)_{Member} \le \phi_y R_{nW} \tag{3.6}$$

Where,

η_i and $(\phi\ R_n)_{Member}$ are the same as discussed in Section 3.4 above and values of ϕ_y, and R_{nW} are defined as:

ϕ_y = Resistance factor for yielding of steel equal to 0.95 (as per AASHTO (AASHTO, 2007) Article 6.5.4.2.)

R_{nW} = Nominal yield strength of a Whitmore section of the gusset under direct axial tension or compression given by:

$$R_{nW} = A_{gW} F_y \tag{3.7}$$

A_{gW} = Gross area of gusset plate at the Whitmore section $= (W)(t_g)$.

W = Width of the Whitmore section (see Figure 3.11).

t_g = Thickness of the gusset plate.

F_{yg} = Specified minimum yield strength of the gusset plate.

3.7.b Fracture of a Whitmore Section in the Gusset Plate

The “net area” of the Whitmore section can fracture if the member force is a tension force as shown in Figure 3.12. Fracture of the Whitmore section of a gusset plate is a non-ductile failure mode. This failure mode is observed in laboratory tests and reported in the literature (Bjorhovde and Chakrabarti 1985). Currently, in the design of gusset plates for braced frames in buildings, this failure mode is not generally considered, probably due to the fact that with proportionality of connections in the modern building gusset plates, fracture of the Whitmore section will not occur prior to block shear failure. However, in riveted and bolted bridge gusset plates, especially with angles connecting the members to the gussets in riveted bridges, there is a possibility that the net section of the gusset plate through the last bolt line could become quite weak and this failure mode becomes governing.

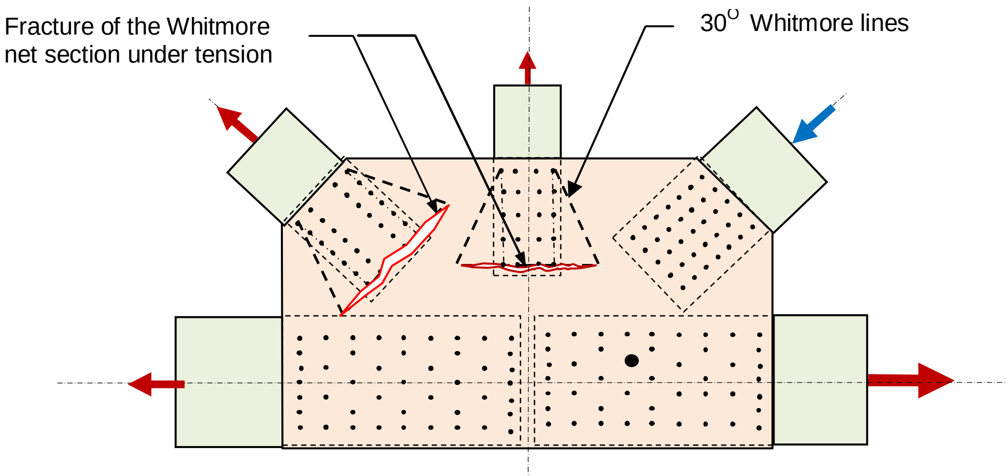


Figure 3.12. Tension fracture of the Whitmore areas of a gusset plate

The following equation can be used to check this limit state of strength:

$$\eta_i(\phi\ R_n)_{Member} \le \phi_u R_{nWu} \tag{3.8}$$

Where $\eta_i = \eta_D\,\eta_R\,\eta_I$ and $(\phi\ R_n)_{Member}$ are the same as discussed earlier in Section 3.4 for non-ductile failure modes and values of ϕ_u and R_{nWu} are defined below:

ϕ_u = Resistance factor for tension fracture in net section equal to 0.80 (as per AASHTO (AASHTO, 2007) Article 6.5.4.2.)

= Nominal ultimate strength of a Whitmore section of the gusset plate under direct axial tension given by:

$$R_{nWu} = A_{nW}F_{ug} \tag{3.9}$$

A_{nW} = Net section of the gusset plate at Whitmore’s section (in.^{2}). The net section is calculated following the procedures to establish net section of tension members as discussed in Article 6.8.2 Tensile Resistance of the AASHTO Specification (AASHTO, 2007).

To calculate the net area of riveted components, the diameter of rivet holes is assumed equal to the nominal diameter of the rivet plus 1/16 inch. This is based on the riveting practices as reported by J. A. L. Waddell (1898). Rivet holes were made either by drilling or by punching and reaming. If drilling was used, usually for plates having a thickness greater than the diameter of the rivet, the hole diameter was 1/16 inch greater than the diameter of the rivet. If punching and reaming was used, first a hole with a diameter of 1/8 inch less than the diameter of the rivet was punched. Then, the pieces were assembled and bolted to each other and the holes were reamed by twist-drilling to a diameter of 1/16 inch greater than the diameter of the rivets. As Waddell (1898) explains it, the reaming was done to make sure that all holes match, but also it "ensures the removal of most, if not all, incipient cracks started by the process of punching" (Waddell 1898).

3.7.c Block Shear Failure of the Gusset Plate

Block shear failure can occur in a gusset plate where tension members are attached to it as shown in Figure 3.13.

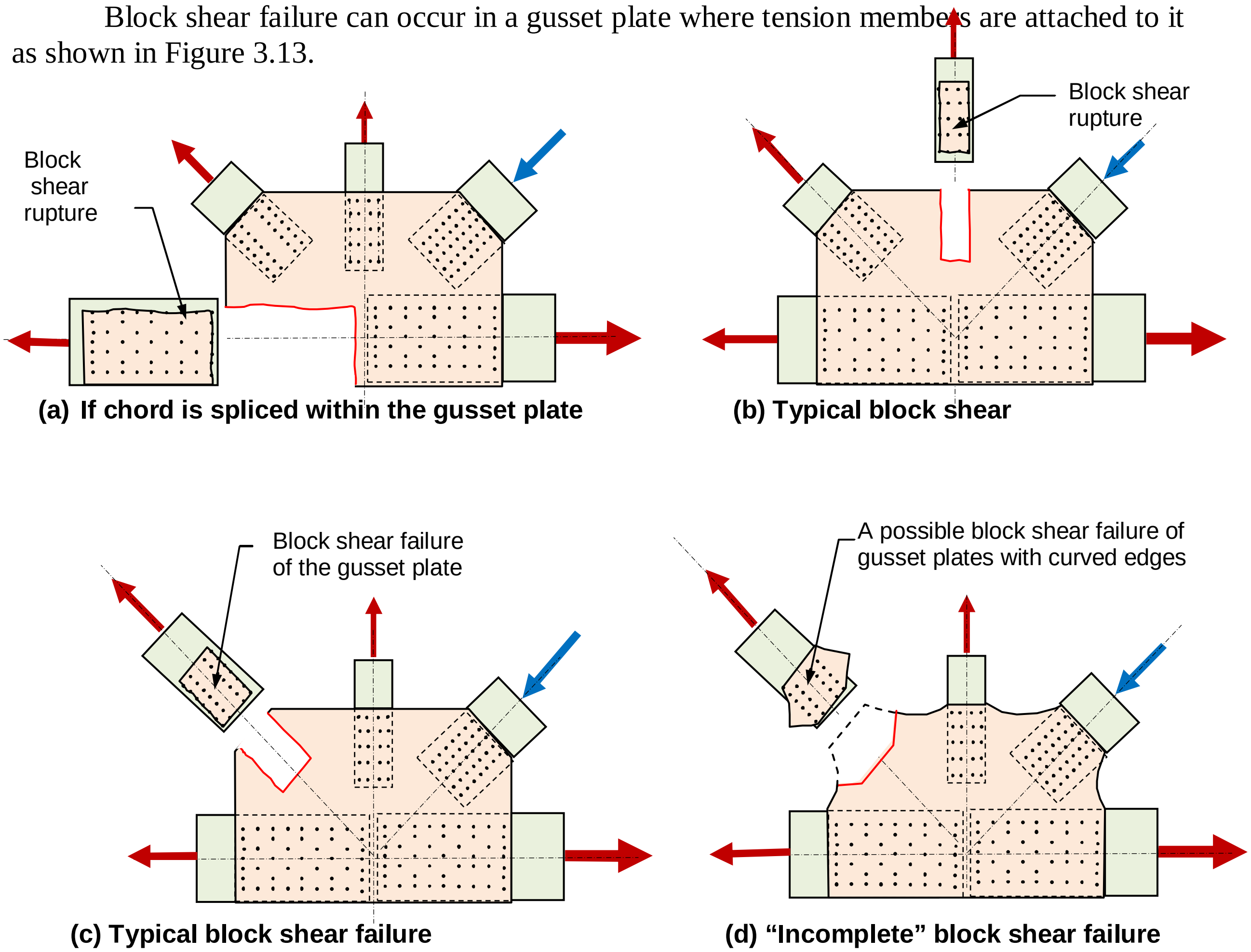


Figure 3.13. Block shear failure of gusset plates

Block shear failure of gusset plates has been studied by a number of investigators including Williams (1986), Williams and Richard (1986), Richards (1986), Astaneh-Asl (1998), and Kulak and Grondin (2001).

Williams (1986), based on finite element analyses of block shear failure of gusset plate tests of the University of Alberta, concluded that the capacity of the gusset plate in block shear can be predicted by Whitmore's method. Richard and Williams (1986) proposed the following Equation 3.10 for block shear failure capacity of the gusset plates:

$$R_{nbs} = 0.58 F_y A_{vg} + F_u A_{tg} \qquad \text{(Williams and Richard 1986)} \qquad (3.10)$$

Where, F_y and F_u are yield stress and ultimate strength of the steel, and A_{vg} and A_{tg} are gross areas in shear and tension equal to (L)(t) and (s)(t) respectively. Parameter t is the thickness of the gusset plate and L and s are shown in Figure 3.14.

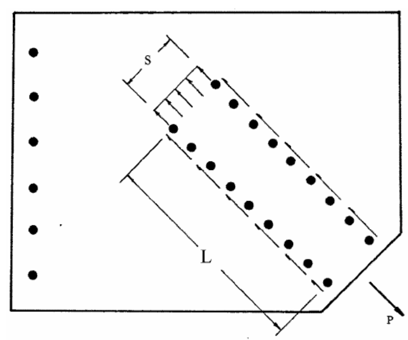


Figure 3.14. Definition of L and s (from Williams 1986)

Figure 3.15 shows block shear failure of bolted gusset plates. The tests were done on single gusset plates (Astaneh-Asl 1998); however, similar failure mode is expected for double gusset plates used in bridges.

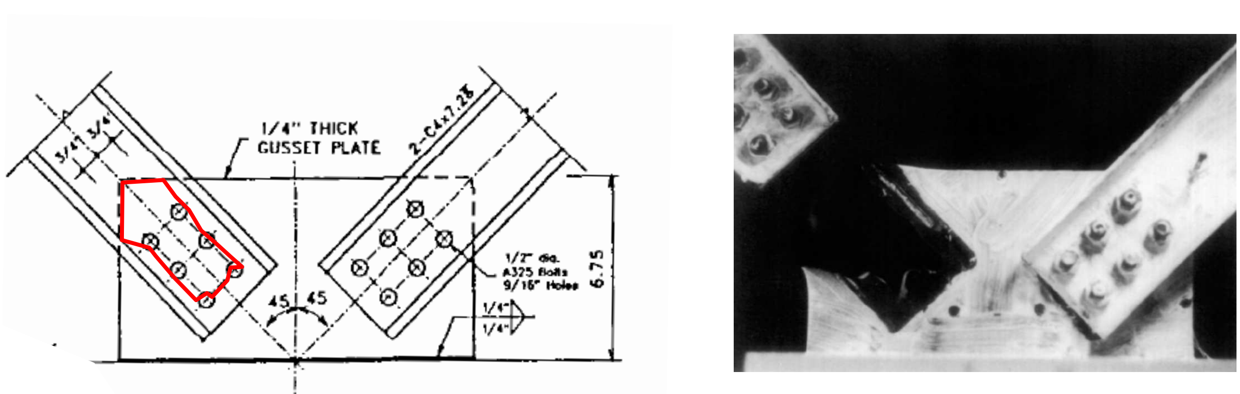


Figure 3.15. Block shear failure of gusset plates (Astaneh-Asl 1998)

In my Steel TIPS on seismic design of building (Astaneh-Asl 1998), reporting on tests of gusset plates, I suggested the following equations for block shear failure of the gusset plates. The equations are a modified version of the block shear equations given at the time by the AISC LRFD Specifications (AISC 1994). The equations are given with AASHTO notations to avoid confusion.

If $A_{nt} \geq 0.6\, A_{vn}$:

$$R_{nbs} = 0.6\, R_y F_y A_{vg} + F_u A_{tn} \qquad \text{(Astaneh-Asl 1998)} \qquad (3.11a)$$

Otherwise:

$$R_{nbs} = 0.6\, F_u A_{vn} + R_y F_y A_{tg} \qquad \text{(Astaneh-Asl 1998)} \qquad (3.11b)$$

where

R_{nbs} = Nominal block shear resistance.

A_{vg} = Gross area along the plane resisting shear stress

A_{tn} = Net area along the plane resisting tension stress

A_{vn} = Net area along the plane resisting shear stress

A_{tg} = Gross area along the plane resisting tension stress

The above equations are derived with each corresponding to a specific failure mode as shown in Figure 3. 16. Equation 3.11a corresponds to a case where failure occurs due to yielding of gross areas in shear and fracture of net area in tension, Figure 3.16(a). Equation 3.11b corresponds to a case where net area in shear is fracturing, while gross area in tension is yielding, as shown in Figure 3.16(b).

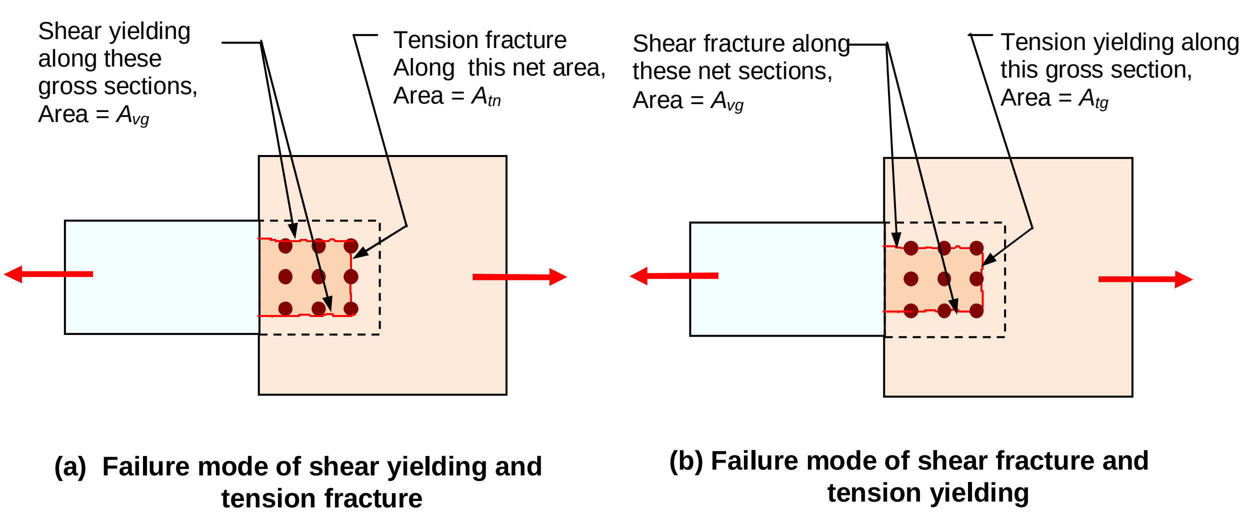


Figure 3.16. Two failure modes for block shear failure according to AASHTO (2007)

In the AASHTO Specifications, the ductility factor of a bridge is not on the capacity side of the design equations but has been moved to the load side of the equation. As a result, in design of gusset plates for bridges, there is no need for R_y in the block shear equations and the equations convert to what is currently in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) as discussed below. In addition, in conducting tests reported in the gusset plate Steel TIPS (Astaneh-Asl 1998), the author observed that in some cases, where a gusset plate is relatively narrow and long, and in cases where the edge is curved, "incomplete" block shear failure could occur as shown in Figure 3.13 for a gusset plate with curved edges and in Figure 3.15 for one of the test specimens.

Block shear failure mode is a relatively recent addition to the list of failure modes in steel structures and was introduced to the design specifications in the 1980s. Since then, as the research on block shear failure has continued, the equations to establish block shear failure capacity are continuously refined and modified to reflect the new research findings. One of the most recent proposals came from Kulak and Grondin (2001) where, after reviewing block shear equations used in various codes, they proposed the following single equation to calculate block shear capacity of gusset plates and angles and recommended more research in this area.

The Kulak-Grondin equation is given below using the AASHTO notations for parameters involved:

$$R_{nbs} = 0.6\, F_y A_{vg} + F_u A_{tn} \qquad \text{(Kulak and Grondin 2001)} \tag{3.12}$$

The above equation is almost identical to the preceding Equation 3.10 by Williams and Richard (1986), with the only difference being that in the above equation the tension area is the net area and in the Williams-Richard equation the tension area is the gross area.

The AASHTO LRFD Specification (AASHTO, 2007) in Article 6.13.4 requires that all tension connections that include gusset plates be investigated for block shear failure. Block shear failure is considered a non-ductile failure mode. The following equation can be used to check this limit state of strength:

$$\eta_i (\phi\ R_n)_{Member} \le \phi_{bs} R_{nbs} \tag{3.13}$$

Where $\eta_i = \eta_D\, \eta_R\, \eta_I$ and $(\phi\ R_n)_{Member}$ are the same as discussed in Section 3.7.a earlier and values of ϕ_{bs} and R_{nbs} for block shear failure are defined below:

ϕ_{bs} = Resistance factor for block shear failure equal to 0.80 as per AASHTO (2007) Article 6.5.4.2.

R_{nbs} = Nominal block shear strength of the gusset plate due to axial tension in the member and given in Article 6.13.4 of the AASHTO LRFD Specifications (AASHTO, 2007) as follows:

- If $A_{tn} \ge 0.58\, A_{vn}$, then:

$$R_r = \phi_{bs}\,(0.58 F_y A_{vg} + F_u A_{tn}) \qquad \text{(AASHTO 6.13.4-1)} \tag{3.14a}$$

- Otherwise:

$$R_r = \phi_{bs}\,(0.58F_u A_{vn} + F_y A_{tg})$$ (AASHTO 6.13.4-2) (3.14b)

Notice that in some bolted and riveted truss bridges, rivets or bolts connecting the truss members to the gusset plate are staggered. Figure 3.17 shows an example of such connection. In these cases, in order to establish the net areas in tension and shear (i.e. A_{tn} *and* A_{vn}) in the above equations we need to follow the AASHTO provision given in section 6.13.4. According to these provisions if there are staggered bolt or rivets within the tension area of the shear block, for each staggered bolt or rivet line we need to add the term $(s^2/4g)(t)$ as we do in calculating net section of the tension members with staggered bolts or rivets. If the bolts on the shear lines of the shear block are staggered, then AASHTO (2007) , Section 6.13.4 states that : "for net sections carrying shear stress, the effective diameter of holes centered within two diameters of the cut shall be deducted. Holes further removed may be disregarded". Applying this provision to the connection shown in Figure 3.16 indicates that in calculating net section in shear for this connection, the second row of rivet holes also need to be deducted from the net section in shear since the distance from the second rivet line to the first rivet line is less than twice the diameter. A numerical example of application of this provision can be found in Example A6, Page A-121 of Ref.: (AASHTO, 2003).

Photo from www.historicbridges.org/
(With permission)

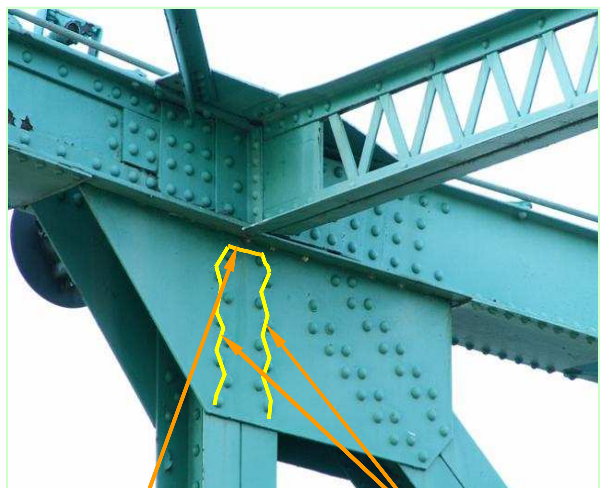


Figure 3.17. Gusset plate with staggered holes (left) and fracture lines for block shear failure

Another important issue regarding block shear failure is that in gusset plates where bolt group is too close to the edge of the gusset plate or the edge of gusset plate is curved the author suggests that a multi-linear block shear failure pattern, as shown in Figure 3.18, also be checked. To check the strength of multi-linear block shear failure there is a method in the "Code for

Design of Steel Structures “ in China (CMC, 2003) and (Shi, 2005). Considering the information in AASHTO LRFD Bridge Design Specifications (2007), the author suggests (Astaneh-Asl, 2010), the use of rules regarding “staggered” holes in net area of tension members and addition of not $S^2/4g$ term in block shear failure cases with inclined “staggered” surfaces that are not parallel or perpendicular to the line of action of tension.

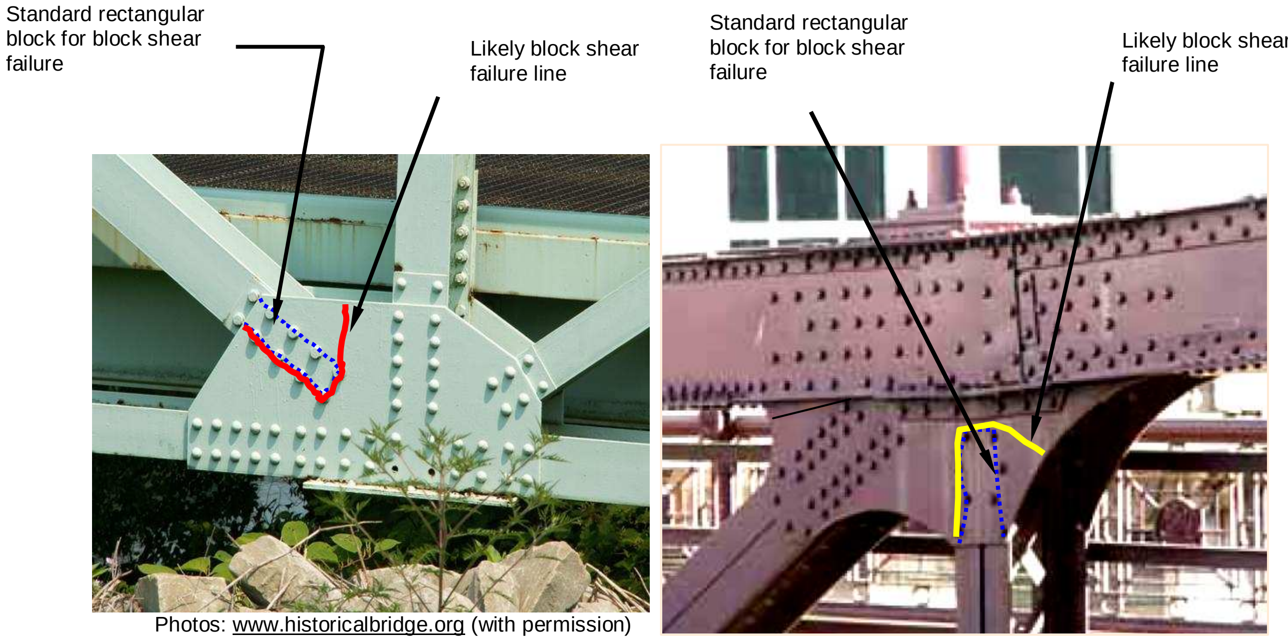


Figure 3.18. Multi-linear block shear failure in curved gusset plates and when outside rivet or bolt lines are too close to the edge of gusset plate

Figure 3.19 shows a typical case of gusset plate with block shear failure surfaces inclined. And the “block” in the block shear failure not a well-defined rectangular block. Information on actual behavior of irregular blocks in block shear failure is almost non-existence. As mentioned earlier, the author feels a rational approach to the problem can be to apply the concept of using $s^2/4g$ that was used earlier in staggered net sections for staggered blocks. Using this concept, the equation of design for checking failure mode of block shear failure for blocks with staggered boundaries is the same as for regular blocks as was given earlier by Equations 3.13 and 3.14 (a, b) but with definition of A_{nt}, the net area in tension equal to [A_g - N (d_h+1/16") t+ $\Sigma(s^2/4g)\ t$].

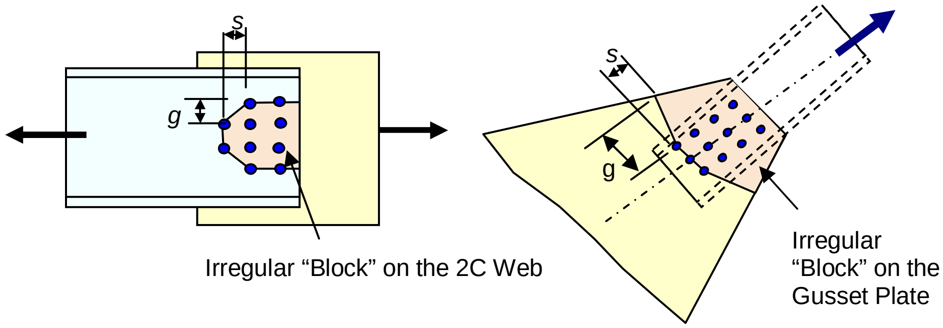
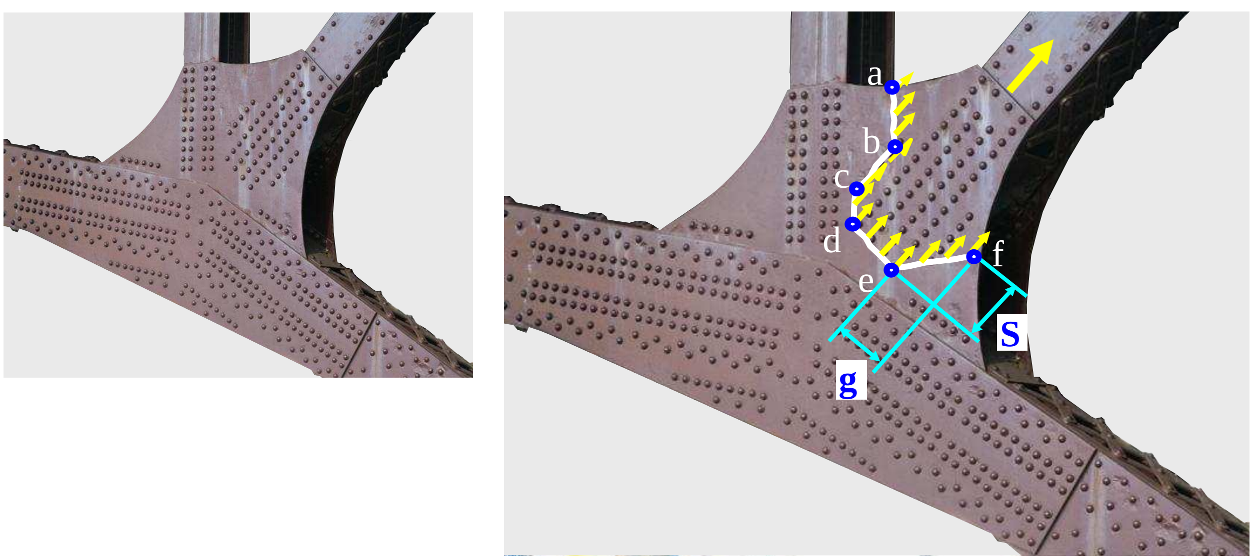


Figure 3.19. Block Shear Failure Where Irregular Blocks Form

Let us consider block shear fracture line shown in Figure 3.20 and establish its nominal resistance R_{n}. The block shear failure line for this case is irregular and possibly can develop along the line *abcdef* in Figure 3.20. Assuming that the line *abcdef* is the block shear failure line, we can establish the segments of this line that are parallel to the applied tension, such as the segment *bc* in Figure 3.20, as the shear areas, and the segments that are perpendicular to the applied tension ,such as the segment *de* in Figure 3.20, as the tension areas. The inclined areas such as *ab*, *cd* and *ef* can be considered the inclined areas similar to those in a net section where bolts or rivets are staggered. These inclined areas are also considered to be in tension but in calculating their net area we need to add the term $(s^2/4g)t$ for each inclined area. The dimensions *s* and *g* are shown in Figure 3.20 for one inclined segment. Dimension *s* is measured parallel to the applied tension force and dimension *g* is perpendicular to the applied tension force.



Original photos from www.historicalbridges.org (with permission)

Figure 3.20. Irregular block shear fracture line and dimensions "*S*" and "*g*" for inclined fracture lines

3.7.d Buckling of Gusset Plate

Buckling of gusset plates can occur over the areas near the edge, which I will call "edge-buckling," or it can occur in the interior areas of the gusset plate, which I will denote as "buckling of gusset plate." Depending on how many of the diagonals and verticals that are connected to the gusset plate are in compression, the buckling region can be small or quite extensive as shown in Figure 3.21 In Figure 3.21(a) only one web member of the truss, the right side diagonal is in compression, which subjects relatively small inner areas of the gusset plate to compression. In Figure 3.21(b), in addition to a diagonal, the vertical member at the panel point is also in compression, which subjects larger inner areas of the gusset plate to compression.

The current AASHTO Specifications (AASHTO 2007) has provisions for edge buckling, but it does not have specific provisions nor recommendations for treatment of buckling within the inner areas of the gusset plate. Due to a lack of sufficient research on buckling of gusset plates, it is not clear if edge buckling and buckling of the inner areas of the gusset plate are interrelated or even are manifestations of a single phenomenon. There are research results (Chambers and Ernst 2005) that show that if edge buckling is prevented by adding edge stiffener, the buckling capacity of the inner parts of the gusset plate also increases. Furthermore, Sheng, Yam, and Iu (2002) show that if the inner compressive areas of the gusset plate are stiffened and buckling of inner areas of the gusset plate prevented, the edge buckling is also delayed. In this design-oriented report, I am treating these two phenomena as two separate limit states with buckling of the gusset plate covered in this section and edge buckling discussed in the next section.

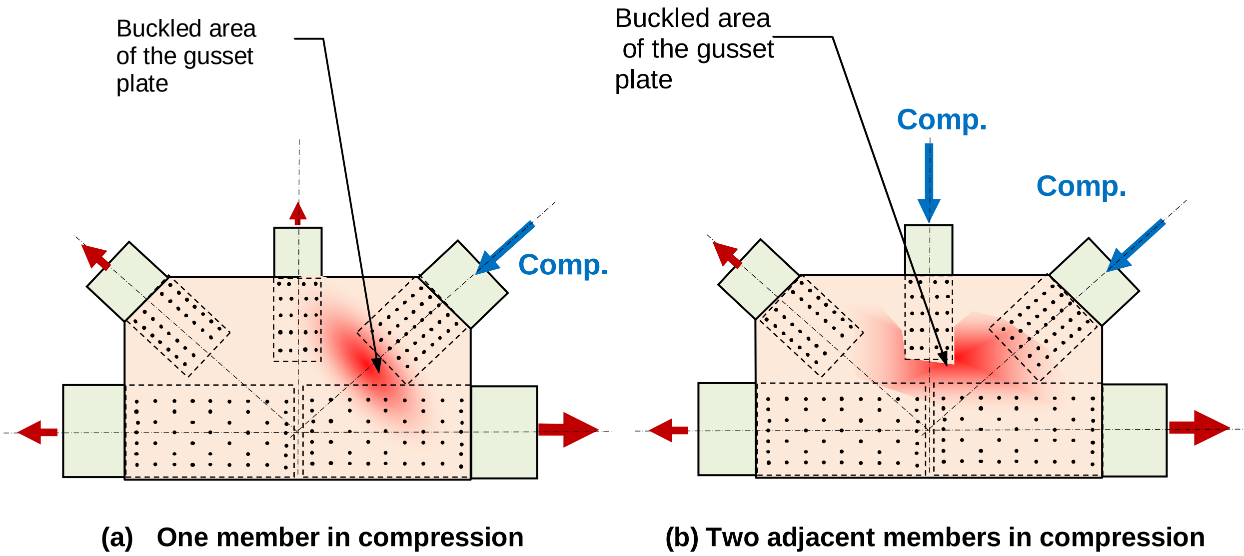
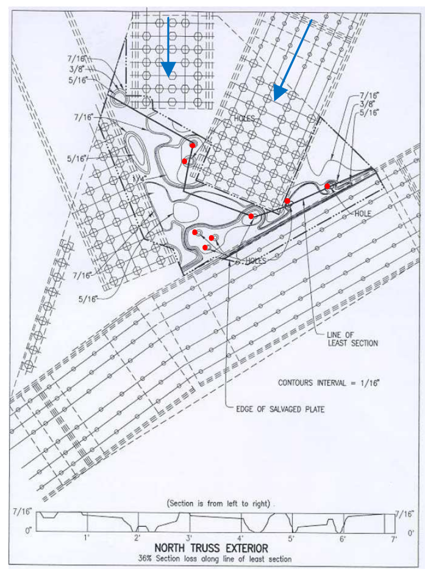


Figure 3.21. Buckling of compression areas of the gusset plate

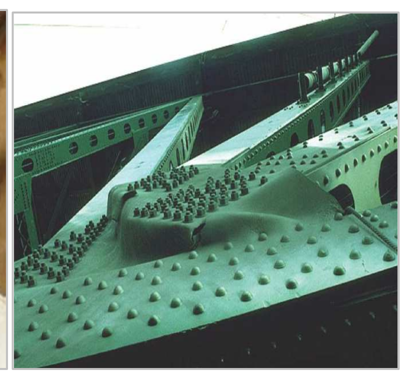
Buckling of gusset plates has been studied by a number of investigators, among them, Yamamoto, Akiyama, and Okumura (1988), Cheng, Rabinovich, and Yam (1993), and Astaneh-Asl (1998). An excellent summary of these and other projects on gusset plates and their findings is provided by Chambers and Ernst (2005). The studies included laboratory tests as well as finite element analyses of gusset plates subjected to compression by one or more members bolted to the gusset plate. Although the tests and even the analytical studies were primarily conducted on single gusset plates used in braced frames in buildings and some tests were cyclic tests for seismic applications, many of the findings and recommendations of these studies are equally applicable to double gusset plate connections of bridge trusses subjected to bridge loads.

One of the most important and dramatic cases of gusset plate buckling occurred in Ohio in 1996, when a gusset plate in a deck truss bridge on the I-90 freeway crossing over the Grand River suddenly buckled. Figure 3.22 from ODOT (2008) shows a view of the bridge and details of buckled gusset plates showing a very visible buckling of gusset plates. The buckled gusset plate was removed and replaced and the bridge was returned to service (Palmer, 1996). The Federal Highway Administration, after analysis of the failed gusset plates, concluded that "... the design thickness of the original gusset plate was marginal, at best, and its load carrying capacity was further exacerbated by the loss of thickness due to corrosion" (Palmer 1996).

The I-90 Bridge in Ohio



Contours of constant thicknesses



All photos courtesy of the Ohio Department of Transportation

Photo from: (Albert, 2008)

Buckled gusset

Buckled gussets

Buckled and torn gusset

Figure 3. 22. The I-90 Bridge over Grand River in Ohio and buckling of its gusset plates (ODOT 2008)

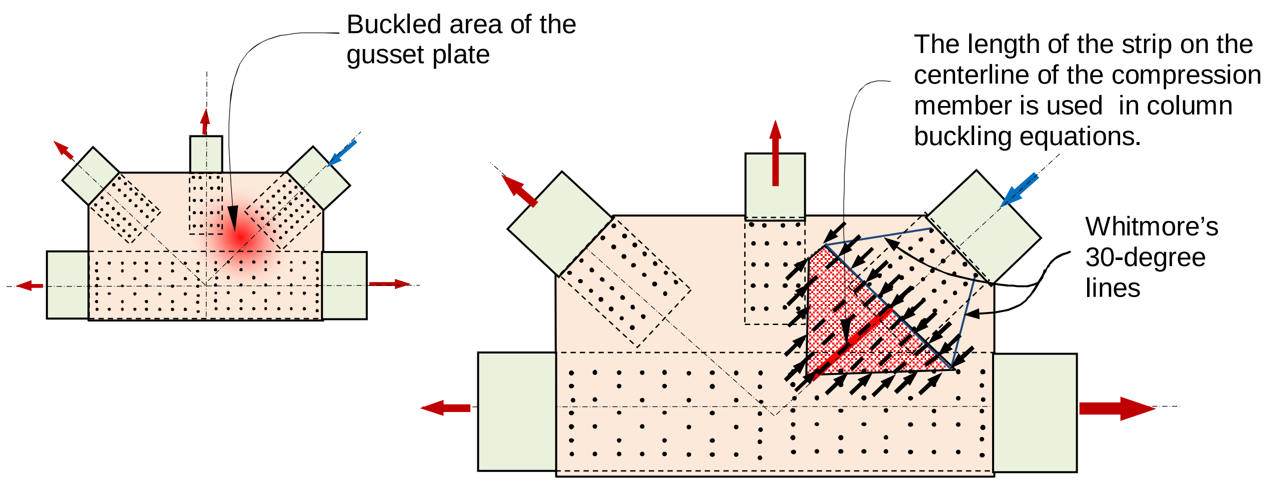


Figure 3.23. Buckling of compression areas of the gusset plate when only one web member is in compression

To establish buckling capacity of a gusset plate subjected to direct compression, Whitmore's effective width can be used (Astaneh-Asl 1998, Doswell 2006) as shown in Figure 3.23. The following equation is suggested for a limit state of the buckling of the gusset plate. This failure mode is considered a non-ductile failure mode.

$$\eta_i (\phi\ R_n)_{Member} \leq \phi_c R_{nc} \tag{3.15}$$

Where $\eta_i = \eta_D \eta_R \eta_I$ and $(\phi\ R_n)_{Member}$ are the same as discussed in Section 3.7.a earlier, and values of ϕ_c and R_{nc} for buckling of gusset plate are defined below:

ϕ_c = Resistance factor for buckling of gusset plate equal to 0.90. Notice that AASHTO (2007) does not have a specific resistance factor for buckling of gusset plates. It provides a resistance factor of 0.90 for "axial compression" in steel in Article 6.5.4.2.

R_{nc} = Nominal capacity of gusset considering buckling failure mode. Value of R_{nc} is established using column buckling equations of Article 6.9.4 of the AASHTO LRFD Specifications (AASHTO, 2007) on design of compression steel members.

To apply column buckling equations to gusset plates, designers need to establish an effective length factor, *K*, for the gusset plate. Doswell (2006) suggests a *K* value of 0.75. For gusset plates in steel trusses, larger *K* values may be more appropriate. When a gusset plate buckles, there is a possibility of gusset plate buckling out of plane. This appears to be the case for buckling of the gusset plates in the I-90 bridge as shown in Figure 3.22 above. The author suggests the use of an effective length factor, *K*, of 1.0 for gusset plate strips to incorporate this out-of-plane buckling of the gusset plate, which results in increasing the effective length factor.

Dowsell and Barber (2004) also suggest a *K* value of 1.0 for gusset plate buckling length.

In order to establish the length of the column for buckling equations, the Whitmore section of the gusset plate is divided into one-inch-wide strips and each strip is considered an independent column with a rectangular section with dimensions of one inch and a thickness equal to the thickness of the gusset plate. The length of the strip located on the centerline of the compression member, as shown in Figure 3.23, is suggested as the column length for all strips. This assumption of course is somewhat conservative since shorter strips will resist larger forces. But the assumption is reasonable since the strips are not actually separate columns next to each other but rather are part of a continuous plate with stresses being transferred from one strip to the next as the loading increases.

If two adjacent members connected to the gusset plate are under compression, the compressive stresses that each creates in the gusset plate are added together, creating larger compressive stresses in the gusset plate than the individual members would create. As a result, the buckling resistance of the gusset plate should be checked against this larger stress.

Let us consider the gusset plate in Figure 3.24 here two members, the right side diagonal and the vertical member, are in compression. The forces applied to the gusset plate are assumed to be distributed inside the gusset plate according to Whitmore's method and under 30-degree angles, a reasonable assumption for design purposes. Of course the actual state of stress in the gusset plate is more complex, and elements inside the gusset plate develop shear as well as normal stresses. Let us take the triangular element "A," located in the area of the gusset plate

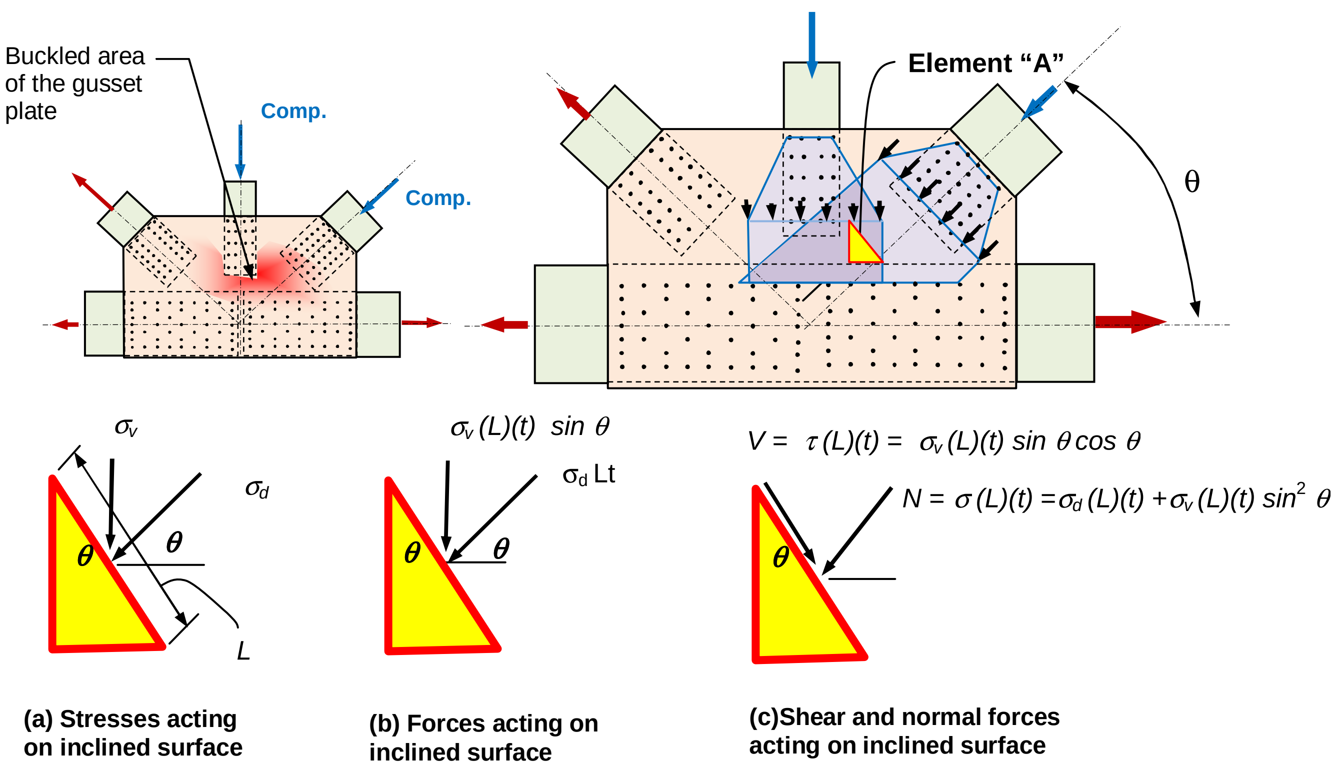


Figure 3.24 Buckling of compression areas of the gusset plate when two adjacent truss members are in compression

where the compression Whitmore areas of two members overlap. The stresses acting on this element are σ_d and σ_v, Figure 3.24(a), created by diagonal and vertical members within the Whitmore area of each member. These stresses acting on the inclined section are then used to calculate the forces that are acting on the inclined member, Figure 3.24(b). Finally these forces are used to calculate shear and normal stresses on the inclined member surface of the element, Figure 3.24(c).

As Figure 3.23(c) shows, the normal and shear forces acting on the inclined surface are:

$$V = \tau (L)(t) = \sigma_v (L)(t) \sin \theta \cos \theta \qquad (3.14)$$

$$N = \sigma (L)(t) = \sigma_d (L)(t) + \sigma_v (L)(t) \sin^2 \theta \qquad (3.16)$$

Therefore, the normal and shear stresses acting on the inclined section are:

$$\tau = \sigma_v \sin \theta \cos \qquad (3.17)$$

$$\sigma = \sigma_d + \sigma_v \sin^2 \theta \qquad (3.18)$$

The normal stress σ given by Equation 3.18 above is suggested as the applied stress in this case instead of just σ_d, which would be used if only the diagonal member was under compression. Notice that this equation ignores the shear stress, τ, which is also acting on the inclined section of the strip. To be conservative, the normal and shear stresses acting on the inclined section could be combined using the von Mises criterion, as given below, and the von Mises stress could be used as the applied normal stress to be compared to the critical buckling stress of the strip.

$$\sigma_{vM} = \sqrt{\sigma^2 + 3\tau^2} \qquad (3.19)$$

For definitions of terms, see “Notations” on Page 7.

The author has applied the above method successfully to a large number of gusset plate designs. Figures 3.25 shows an example of such application. In this example, shear stresses within the inner parts of gusset plate were quite high while normal stresses were relatively low.

Figure 3.25 shows a gusset plate with two diagonal members bolted to it. The gusset plate was analyzed using three methods: (1) Finite element Method, (2) the above method proposed by the author; and (3) the Beam Theory method. In applying Beam Theory method, critical sections of the gusset plate are considered to be beam sections and the normal stresses are established by adding normal stresses due to bending moment *M* and axial force *N*. More information and a discussion of Beam Theory are provided in Section 3.7.f later in this report.

Figure 3.25 shows von Mises stresses established at each nodal point by analyzing the gusset plate using the above-mentioned three methods. To establish von Mises stresses shown on Figure 3.24, first normal stress σ and shear stress τ was established for each node point on the gusset plate, then these stresses were combined using Equation 3.19 above. As Figure 3.25 shows the results of analysis using Finite Element Methods (FEM) and author (A. Astaneh) ‘s proposed method are sufficiently close for design office applications while the results obtained by applying the Beam Theory are quite off.

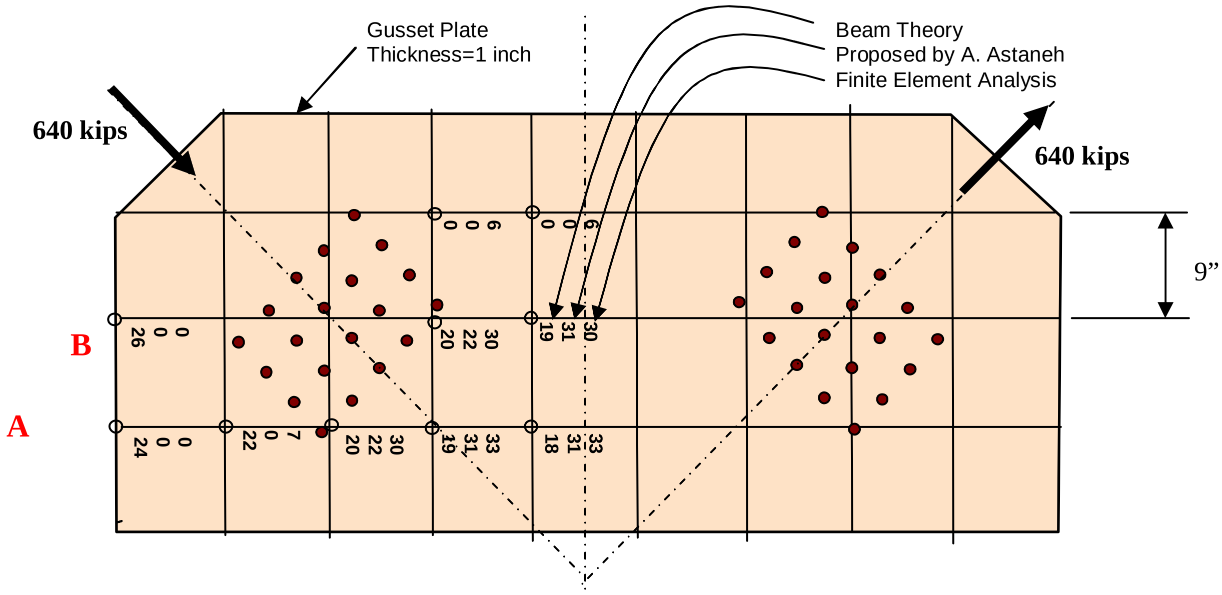


Figure 3.25 von Mises stresses in a gusset plate using three methods: FEM, Astaneh’s proposed simple method and the Beam Theory method

The stresses established by the Beam Theory, (i.e. the number in the third line for each node), are quite unrealistically high at the edges of the gusset plate, see Node “A” in Figure 3.25, while they are small in the inner nodal points at the center of the gusset plate, see numbers for the Node “B”. For example considering Node “B” in Figure 3.25 which is one of the most highly stressed points on this gusset plate, the von Mises stresses for this point are 33.0 ksi (by FEM), 31.0 ksi by Astaneh’s method , and only 19.0 ksi by Beam Theory.

Figure 3.26 shows a gusset plate similar to those used in the lateral bracing system. Unlike the previous case where shear stresses on critical sections were high, in this example normal stresses are quite high while shear stresses are relatively low. Similar to previous case shown in Figure 3.25, in this case also the gusset plate was analyzed using the three methods: (1) Finite element Method, (2) the above simple method proposed by the author (A. Astaneh-Asl); and (3) the Beam Theory method. Figure 3.26 shows von Mises stresses established at each nodal point by analyzing the gusset plate using the above-mentioned three methods. Again as Figure 3.26 shows the von Mises stresses established using FEM and the author’s proposed method are sufficiently close for design office applications while the results obtained by applying the Beam Theory are quite off. Again, the von Mises stresses established by application of Beam Theory method, the number on the third line for each nodal point, are quite low compared to more realistic stresses predicted by FEM (the number on the first line) and Authors (Astaneh’s) proposed method which is the number on the second line. For example considering Node “C” in Figure 3.26 which is one of the most highly stressed points on this gusset plate, the von Mises stresses for this point are 19.2 ksi by FEM, 19.6 ksi by Astaneh’s method, and 6.6 ksi by Beam Theory.

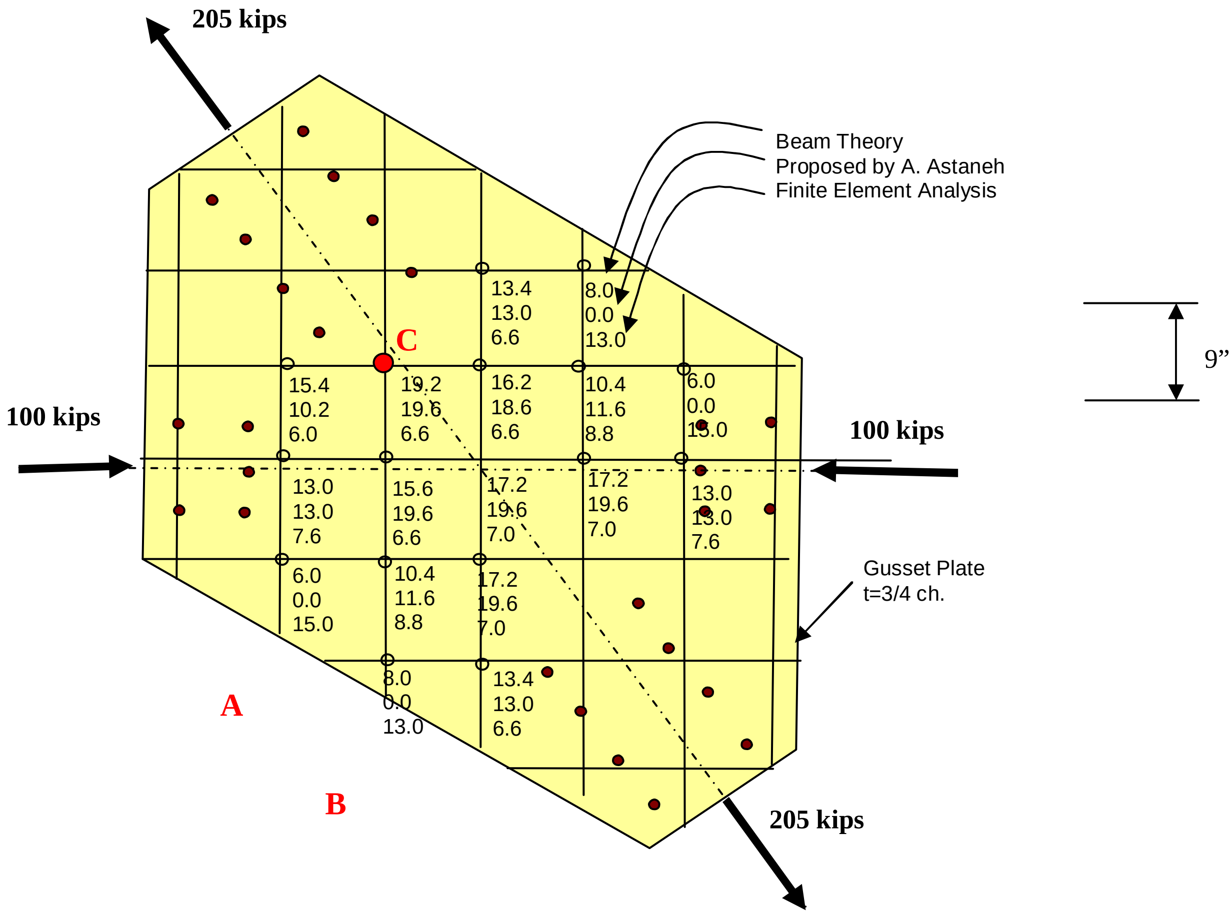


Figure 3.26 von Mises stresses in a bracing system gusset plate using three methods: FEM, Astaneh’s proposed simple method and the Beam Theory method

3.7.e Buckling of Free Edges of the Gusset Plates

When a compression member attached to the gusset plate applies compression to the gusset, the free edges of the gusset plate can buckle under the compressive stresses as shown in Figure 3.27. As discussed in the previous section and based on the available research, most of which is summarized by Chambers and Ernst (2005), it is not clear to me whether the edge buckling of the gusset plates is a failure mode separate from buckling of the inner parts of the plate and whether or not designing a gusset plate for the buckling limit state will prevent its edge buckling. Therefore, these two buckling modes are treated separately and both limit states are checked. This section focuses on the edge buckling.

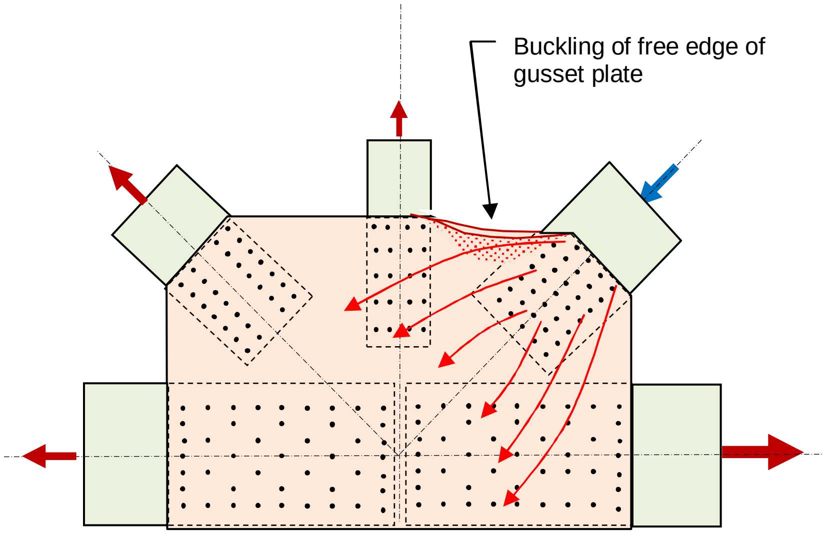


Figure 3.27. Edge-buckling of the gusset plate

A case that looks like an edge buckling was recently (March 2008) discovered in the gusset plates of the DeSoto Bridge in Minnesota during an inspection of the bridge by the Minnesota Department of Transportation. A photo of the bridge and a buckled gusset plate are shown in Figure 3.28.

Photo: A. Astaneh-Asl

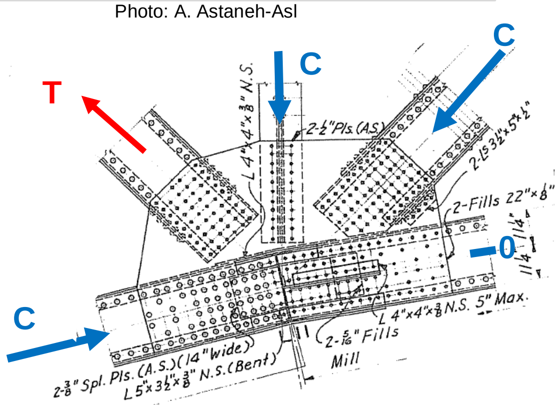


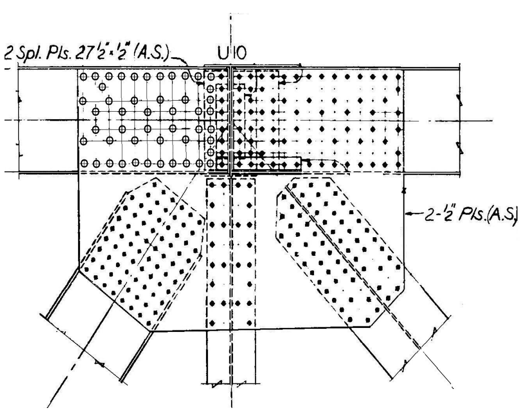
Photo by Mn/DOT

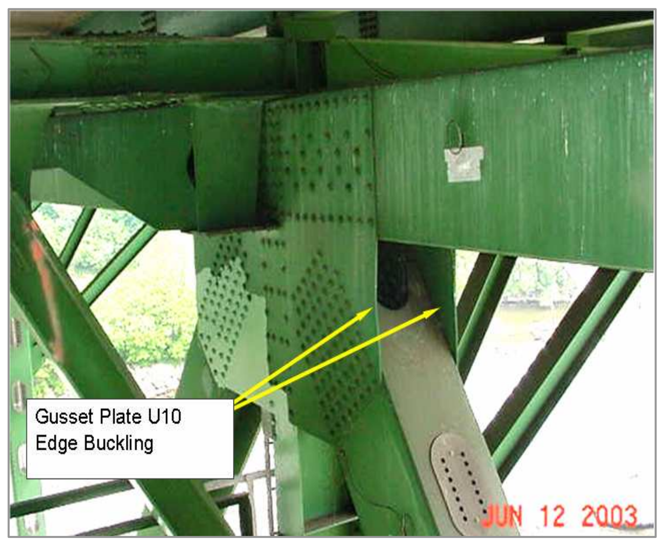
Figure 3.28. The 1959 DeSoto Bridge (demolished in 2008 and is being replaced) and one of its gusset plates, at Location L11', which had edge buckling.

Another actual case of gusset plate buckling appears to be the case of I-35W Bridge in Minnesota. Figure 3.29 shows the photos of the gusset plates of the bridge taken in 2003 and released by the National Transportation Safety Board in 2008. From the photos, it is not clear whether this buckling is an edge buckling or buckling of the inner regions of the gusset plate combined with or initiating edge buckling. Placing this item here in the section on edge buckling does not necessarily mean this is an edge buckling.

Edge buckling of gusset plates is a failure mode long known in steel bridge engineering community. The 1921 Manual of the American Railway Engineering (AREA, 1921) states that: "If the unsupported length of the inclined edge of the gusset plate exceeds 18 inches, the gusset plate shall have one or two stiffening angles riveted along its edge" (AREA, 1921, P759). This limit of 18 inches on the free edge of gusset plate, gradually evolved into a limitation placed on the length to thickness ratio of the free edge. With the development and use of steels with a variety of strength (yield point), the limit of length to thickness ratio for the free edge was related to the yield stress as well. The current AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) states that the free length to thickness ratio of the free edge of gusset plates should not exceed 2.06 times square root of yield stress of steel (in ksi). Later in this chapter, I return to this subject and the equation.

Photo: John A Weeks Jr.





(Photo from URS(2003)

Figure 3.29. The 1968 I-35W Bridge and its buckled gusset plates, at Location U10.

Another case of problems with edge buckling is the case of John A. Blatnik Bridge in Minnesota carrying Interstate 535 over Mississippi, Figure 3.30. In May of 2008 calculations and field inspections done by the Minnesota Department of Transportation indicated that the “gusset plates in eight locations on the bridge did not meet the bridge’s load requirement.” (MnDoT, 2008). The bridge opened to traffic in December 1961 and has a continuous steel arch truss as main span. Figure 3.30 also shows location of gusset plates that were strengthened as well as one of the strengthened gusset plates. The strengthening consisted of adding edge stiffeners (angles) and adding angles to the end of the diagonal compression member to prevent buckling of the corroded gusset plate as shown in Figure 3.30.

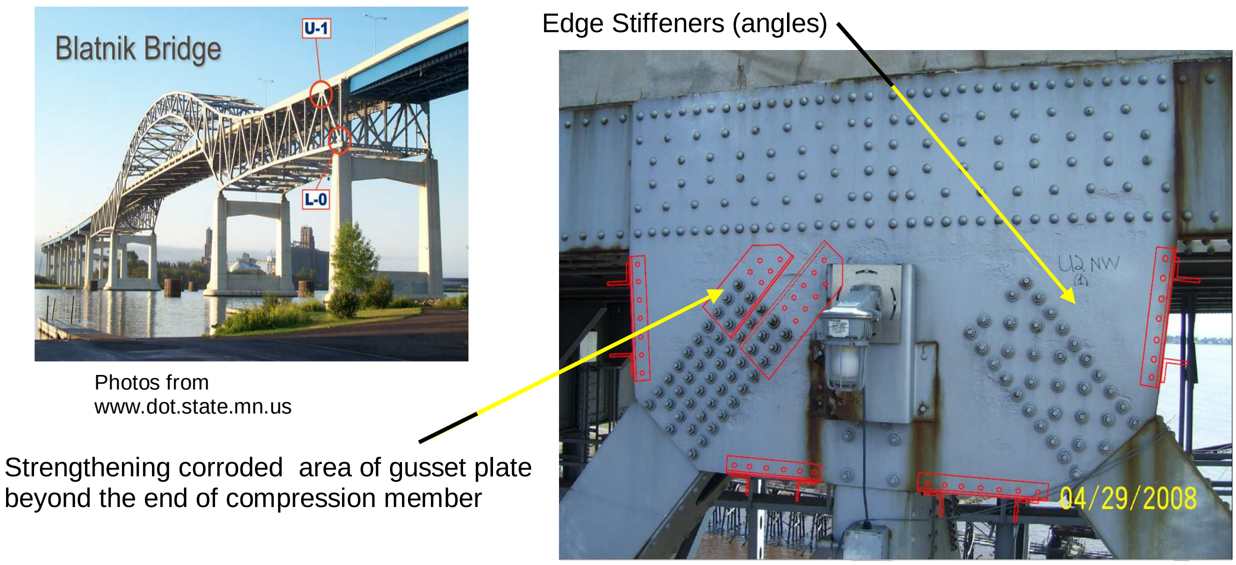


Figure 3.30. Blatnik Bridge with locations of strengthened gusset plates

The AASHTO LRFD Bridge Design Specification (AASHTO, 2007) has a limit on the maximum unsupported length of a gusset plate to be less than or equal to 2.06 times the thickness of the gusset plate. This can be written as:

$$\frac{L_{fg}}{t_g} \leq 2.06\sqrt{\frac{E}{F_y}} \tag{3.20}$$

For definitions of terms, see “Notations” on Page 7.

V. L. Brown (1988) reported the results of compressive gusset plate tests and analyses of edge buckling. The study recommended a formula to prevent edge buckling prior to gusset yielding. It can be expressed as:

$$\frac{L_{fg}}{t_g} \leq 0.83\sqrt{\frac{E}{F_y}} \tag{3.21}$$

Yamamoto, Akiyama, and Okumura (1988) also proposed a very similar equation, but in their equation the length L_{fg} is the vertical free length of the gusset plate.

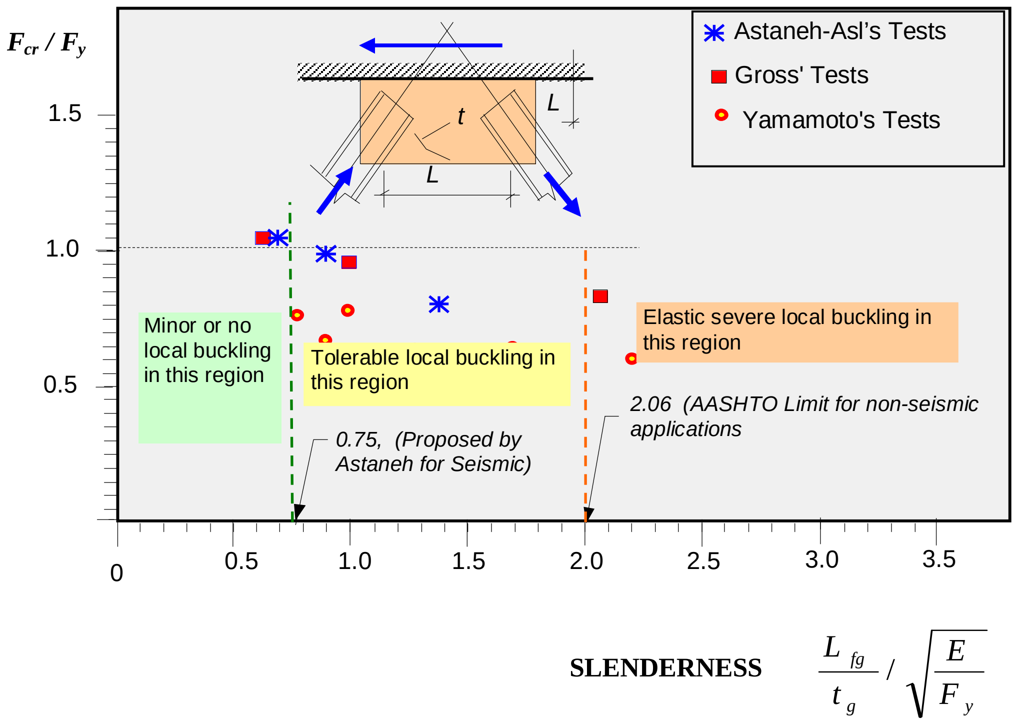


Figure 3.31. Variation of buckling stress in the gusset plate with slenderness parameter (Astaneh-Asl 1998)

Based on experience with performance of gusset plates in bridges, the above criterion seems adequate to prevent elastic buckling of free edges of relatively thin gusset plates subjected to monotonic loads. However, for gusset plates under a large cyclic push-pull load, which can occur in stiffening trusses of cable-stayed or suspension bridges during a major earthquake, the edge buckling can occur even when the above criterion is satisfied. The available test data on edge buckling of gusset plates under cyclic loading are limited. The test results found in the literature are plotted in Figure 3.31. As the figure indicates, for values of L_{fg}/t_g greater than $0.75\sqrt{E/F_y}$, the critical buckling stress, F_{cr} / F_{max}, is reduced significantly.

Using these results, I proposed the following criterion (Astaneh-Asl 1998). The criterion is formulated to prevent cyclic buckling of the free edge of the gusset plates prior to the gusset plates reaching their maximum compression capacity.

$$\frac{L_{fg}}{t_g} \leq 0.75\sqrt{\frac{E}{F_y}} \tag{3.22}$$

3.7.f Yielding of Critical Sections of Gusset Plates under Combined Stresses

The critical sections of gusset plates can yield under a combination of axial load, bending, and shear as shown in Figure 3.32. Currently, there is no single rule on how these loads should be combined. In past designs of gusset plates, the critical sections are treated as a beam subjected to bending moment , shear, and axial load and then, as with a beam, the maximum normal stresses due to the combination of axial load and bending moment are compared to the maximum allowable normal stresses. In addition, the stresses due to shear are separately compared to maximum allowable shear stress. However, as S. C. Goel (1986) points out, the stress distribution in the critical sections of gusset plates subjected to combined loads is not similar to stress distribution in beams where normal stresses are usually maximum at extreme fibers away from neutral axis while shear stresses are maximum on the neutral axis. This issue has been known in bridge engineering community for some time.

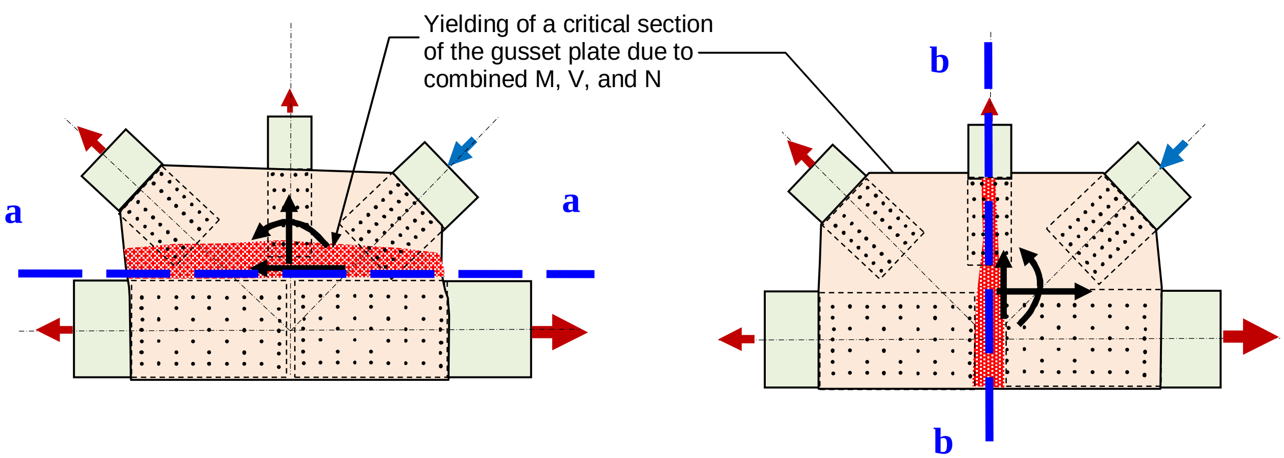


Figure 3.32. Yielding of critical sections "a-a" and "b-b' of gusset plate under combined effects of bending moment and shear and axial forces

One of the objectives of Whitmore's tests (Whitmore 1952) was to establish the actual stresses in the gusset plates and compare the stresses to those obtained using beam theory. He concluded that although the maximum stresses were predicted by beam theory reasonably well, the location where the maximum stresses occurred was not predicted by beam theory accurately. Failure of critical sections of gusset plates has been known since early days of design of steel truss bridges.

As early as late 1800s , the bridge engineering textbooks recommended that thickness of gusset plate be selected based on application of beam theory. In this method, gusset plates were assumed to be beams. Then by using free body diagrams, bending moment, axial and shear forces acting on critical sections of gusset plates were established. Finally, using bending moment and axial and shear forces acting on each critical section, normal and shear stresses were calculated and compared to the allowable stresses. Normal stresses were established by linearly adding normal stresses due to axial force and bending using following interaction equation. Shear stresses were calculated by simply dividing the shear force acting on a critical section of gusset plate by the length of the section times its thickness. In other words shear stresses were "uniform" stresses.

$$\sigma_{\max} = \frac{N}{Lt} + \frac{Mc}{I} \le \sigma_{allow} \qquad \text{(normal stress)} \qquad (3.23)$$

$$\tau_{\max} = \frac{V}{Lt} \le \tau_{allow} \qquad \text{(shear stress)} \qquad (3.24)$$

Where,

σ_{max} = Maximum normal stress acting on the critical section of gusset plate

N = Axial force acting on the critical section of gusset plate

L = Length of critical section

t = Thickness of critical section of gusset plate which is the thickness of gusset plate

M = Bending moment acting on the critical section of gusset plate

c = Distance from extreme fiber of gusset plate to its neutral axis = $t/2$

I = Moment of inertia of critical section = $tL^3/12$

σ_{allow} = Allowable normal stress for the material of gusset plate

V = Shear force acting on the critical section of gusset plate

τ_{allow} = Allowable shear stress for the material of gusset plate

The provisions of the current AASHTO Specifications (AASHTO, 2007) are based on the above concept of using beam. The equations are in LRFD format but basically the same as the above equations. Brockenbrough and Merritt (1999) and most other references recommend the application of beam theory to critical sections of the gusset plate. However, Blodgett (1966) suggested using a maximum of principle stresses in the critical section and compare it to yield stress divided by a factor of safety. I recommended (Astaneh-Asl 1998), the use of an interaction equation given later in this section by Equation 3.27, which was derived using the normal stress and shear stress interaction equation suggested by Neal (1977).

The above methods were reviewed by Thornton (2002), who concluded: "… The use of the von-Mises criterion, which is technically correct, is not justified in the design of gusset plates. Blodgett (1966) uses principal stresses, but not the von-Mises combined or effective stress. Blodgett's approach is typically the method used in machine design where fatigue is a consideration. It is not typical in U.S. standard practice in gusset design. If it is nevertheless decided to use combined stresses, the interaction equation from Neal (1977) (also Astaneh [1998]) using *M*, *N*, and *V* is the most reasonable approach, but as stated above, it will probably never control the thickness of a gusset plate (compared to the thickness derived from Whitmore's method)." The last sentence in Thornton's conclusion is plausible only if one is looking at single-member gusset plate design such as the gusset plates in the braced frames of buildings. However, for the multiple-member gusset plates of bridge trusses, where failure of Whitmore's section is independent of the failure of the critical section under combined stresses, it does not apply.

One of the methods used in the past, albeit very rarely, for design of a gusset plate is the maximum principal stress method. In this method, which is based on the elastic behavior of the gusset plate only, the critical section of the gusset plate is assumed to behave as a beam, and the stresses due to shear, axial force, and bending are calculated for a number of critical elements on the critical section of the gusset plate. Then by using Equation 3.25 (Timoshenko, 1955) principal stress for each critical element is calculated. The maximum principal stress is the larger of principal stresses acting on critical elements of a critical section of the gusset plate.

$$\sigma_{1,2} = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau^2} \tag{3.25}$$

For definition of terms see Notations on Page 7.

Selection of critical elements in the above-mentioned maximum principal stress method is very important and needs to be done correctly. Some (Holt and Hartman, 2008) consider two elements on the critical section of the gusset plate to be the critical elements for calculation of principal stresses. These two critical elements shown in Figure 3.33, are: (a) one element, Element “A” in Figure 3.33, on the neutral axis, where shear stress is maximum but normal stress due to bending is zero, and; (b) another element, Element “B” in Figure 3.33, on the extreme fiber (i.e. edge) of the gusset plate where bending stresses are maximum but shear stress is zero. The larger value of principal stress calculated for these two elements is then selected as the “maximum” principal stress acting on the critical section of the gusset plate. However, this

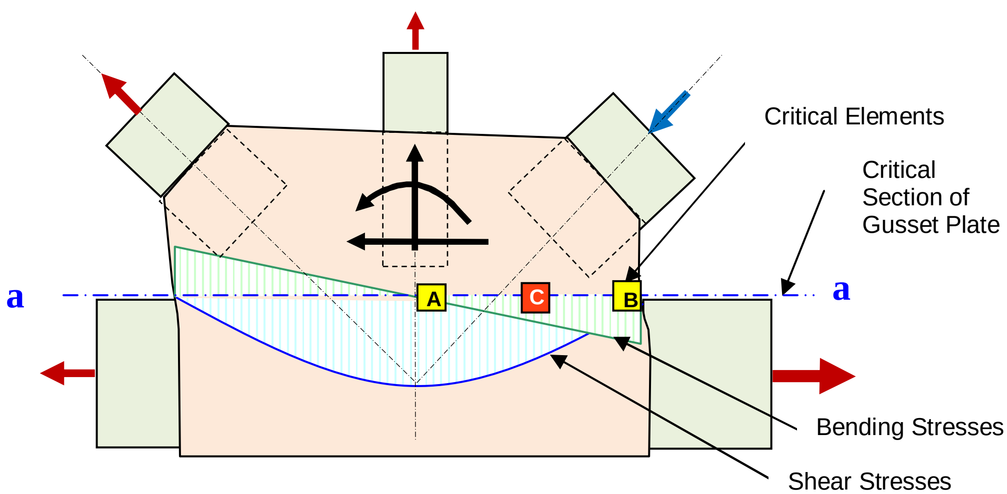


Figure 3.33. Elements to be considered in calculating maximum principal stress.

S. C. Goel (1986) provided a discussion of using the maximum principle stress criterion in design of gusset plates subjected to combined shear and tension forces and showed that by using a maximum principal stress criterion “increasingly un-conservative results are obtained as the tension stress becomes smaller.” He stated that the use of maximum principal stress theory can be justified in design situations where the failure mode is governed by a brittle fracture, such as fatigue loading (Goel 1986). He also suggested an empirical equation for interaction of shear and axial tensile stresses on the critical section of a gusset plate in the form of:

$$\frac{f_t}{F_t}+\left(\frac{f_v}{F_v}\right)^2 \leq 1.0 \tag{3.26}$$

Where:

ϕ = Resistance factor for yielding equal to 0.90.

f_t = Applied tensile stress on the critical section (un-factored)

F_t = Allowable tensile stress on the critical section

f_v = Applied shear stress on the critical section (un-factored)

F_v = Allowable shear stress on the critical section

Based on available information and research results, in 1988, I suggested the use of Equation 3.27 below (Astaneh-Asl 1988). The equation is in LRFD format, for design of critical sections of gusset plates in the braced frames of buildings, primarily for seismic applications and ductile design of connections. The justification for this interaction equation lies in the fact that since the cross sections of the gusset plates are normally rectangular, the curve for interaction of the axial load, *N*, and the bending moment, *M*, is a parabola. However, the interaction of shear, *V*, and the bending moment, *M*, is closer to a 4th-degree polynomial (Neal 1977). In addition, Goel's proposed Equation 3.26 above, also gives a parabolic interaction equation for combination of shear and tension. Combining these interaction equations, I suggest the following equation for interaction of axial force, *N*, bending moment, *M*, and shear, *V*.

With regard to bridge gusset plates, as discussed below, plastification of gusset plates in bridges is not allowed by AASHTO Specifications (AASHTO, 2007). However, for trusses that will experience large seismic effects, such as the stiffening trusses of suspension bridges, it appears that there need to be dual design criteria. One criterion, which should allow plastification of the gusset plate, such as given by Equation 3.27 below, should be used for seismic design; the other criterion, such as that given in the current AASHTO Specifications and also discussed later in this section, which does not allow for gusset plate plastification, should be used for design of gusset plates for loads other than the seismic.

$$\left(\frac{N}{\phi N_y}\right)^{2.0}+\left(\frac{M}{\phi M_P}\right)^{1.0}+\left(\phi\,\frac{V}{V_y}\right)^{4.0} \leq 1.0 \tag{3.27}$$

Where;

ϕ = Resistance factor for yielding equal to 0.90.

N = Normal force (axial force) acting on the critical section of gusset plate under consideration calculated using the load in the members equal to $\eta_i\,(\phi\, R_n)_{Member}$. The values of $\eta_i\,(\phi\, R_n)_{Member}$ are the same as established in Section 3.8.a earlier.

M = Bending moment acting on the critical section of the gusset plate under consideration calculated using the load in the members equal to $\eta_i\,(\phi\,R_n)_{Member}$. The values of $\eta_i\,(\phi\,R_n)_{Member}$ are the same as established in section 3.8.a earlier.

V = Shear force acting on the critical section of gusset plate under consideration calculated using the load in the members equal to $\eta_i\,(\phi\,R_n)_{Member}$. The values of $\eta_i\,(\phi\,R_n)_{Member}$ are the same as established in section 3.8.a earlier.

N_y = Yield capacity of critical section of gusset plate under consideration = A_gF_y.

M_p = Plastic moment capacity of critical section of gusset plate under consideration = Z_xF_y.

V_y = Shear yield capacity of the critical section of gusset plate under consideration = $A_g(0.58F_y)$.

F_y = Minimum specified yield strength of gusset plate.

The above interaction equation is based on the assumption that the gusset plate in the compression zone of the critical section will not buckle until it yields. In other words, if the critical area of the gusset plate is free of members over a relatively long length, then this free length can buckle before it reaches yield in compression. For buckling of gusset plates, see Section 3.4.d earlier in this chapter.

The current AASHTO LRFD Specification (AASHTO, 2007) in Article 6.2.8 states: “The maximum stress from combined factored flexural and axial loads shall not exceed ϕ_yF_y based on the gross area. The maximum shear stress on a section due to the factored loads shall be $\phi_vF_u/\sqrt{3}$ for uniform shear and $\phi_v0.74\,F_u/\sqrt{3}$ for flexural shear computed as the factored shear force divided by the shear area.” Following this article, the design of gusset plates is done by checking the normal stresses due to combination of factored axial load and bending moment against reduced yield stress (that is, ϕ_yF_y) and separately checking shear stress due to factored shear force against reduced shear fracture stress (that is, $\phi_vF_u/\sqrt{3}$ for uniform shear stress and $\phi_v0.74\,F_u/\sqrt{3}$ for flexural stress.) The AASHTO Specification does not provide a specific equation for combination of normal stresses due to axial force and bending. However, since the commentary to this section states that “Plastic shape factors or other parameters that imply plastification of the cross section should not be used,” the combination of normal stresses due to axial force and bending moment needs to be linear. Furthermore, these stresses need to be calculated using elastic properties of the cross sections.

As mentioned earlier, the author has developed a simple method that can be used to establish Von Mises stresses in the gusset plates without a need to conduct time consuming finite element analysis. The method was explained in detail earlier in Section 3.7.d of this chapter. The results of applying this simple method provides von Mises stresses that are very close to those predicted by more elaborate and time consuming finite element analysis as was shown in Figures 3.25 and 3.26 earlier in this chapter. The method proposed by the author (A. Astaneh-Asl) has following simple steps.

Steps in Calculating von Mises Stresses in the Gusset Plates using Astaneh's Method:

Step 1. Have a drawing of the gusset plate by scale with members attached to it shown on it, Figure 3.34(a). For each member establish the axial force to be used in design or evaluation of gusset plate. These are the "Demand" forces of each member, $\Sigma\eta_i\gamma_i\,{}_{Qi}$, discussed in Section 3.4 earlier in this chapter.

Step 2. For each member connected to the gusset plate, draw 30^o-degree (Whitmore's) lines from the last bolt (or rivet) to intersect with the first bolt (or rivet) line. This will establish Whitmore's width W_i for each member as shown in Figure 3.34(b).

Step 3. For each member, from the ends of Whitmore's width, draw a line parallel to the axis of the member as shown in Figure 3.34(c). These two lines will show the stressed area of gusset plate for that member. The stress on any element inside this area is only normal stress which is calculated as equal to the axial force in the member divided by Whitmore area (i.e. $\sigma = N/Wt$)

Step 4. After establishing these normal stresses in Whitmore areas of each member, select any critical element and establish normal stresses acting on it by each member force, Figure 3.34(d).

Step 5. By using Equations given in Section 3.7.d earlier, calculate von Mises stresses on the selected critical element.

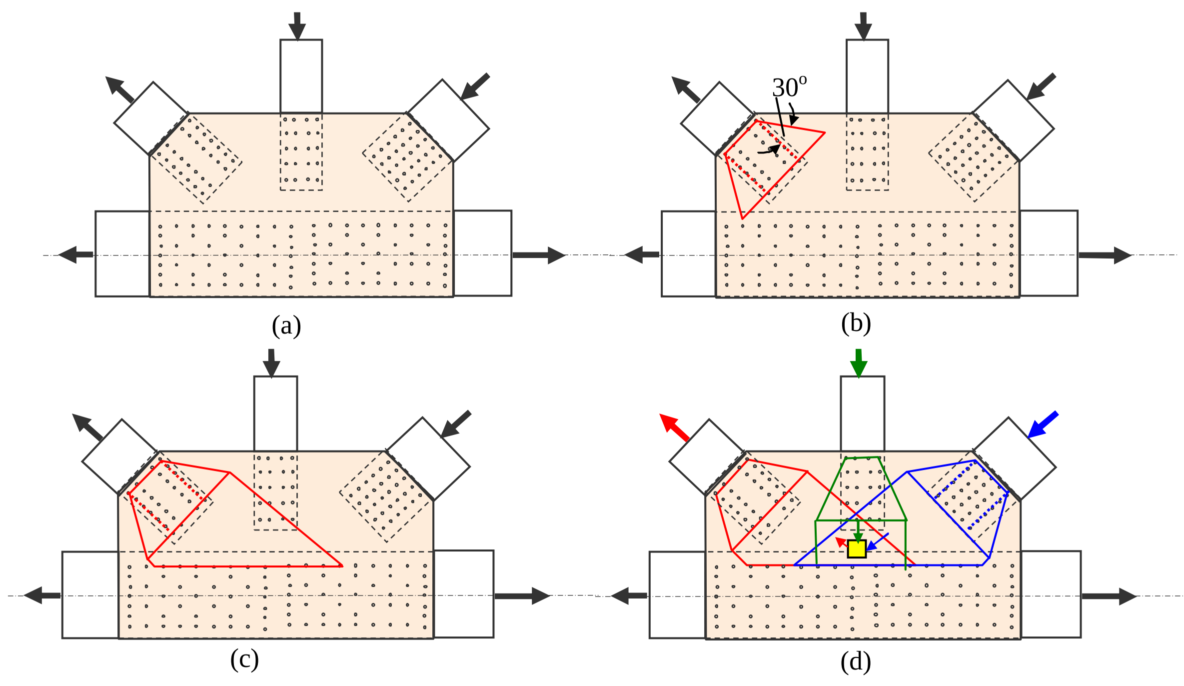


Figure 3.34. Application of Astaneh's Simple Method to a gusset plate to calculate von Mises stresses in it.

b

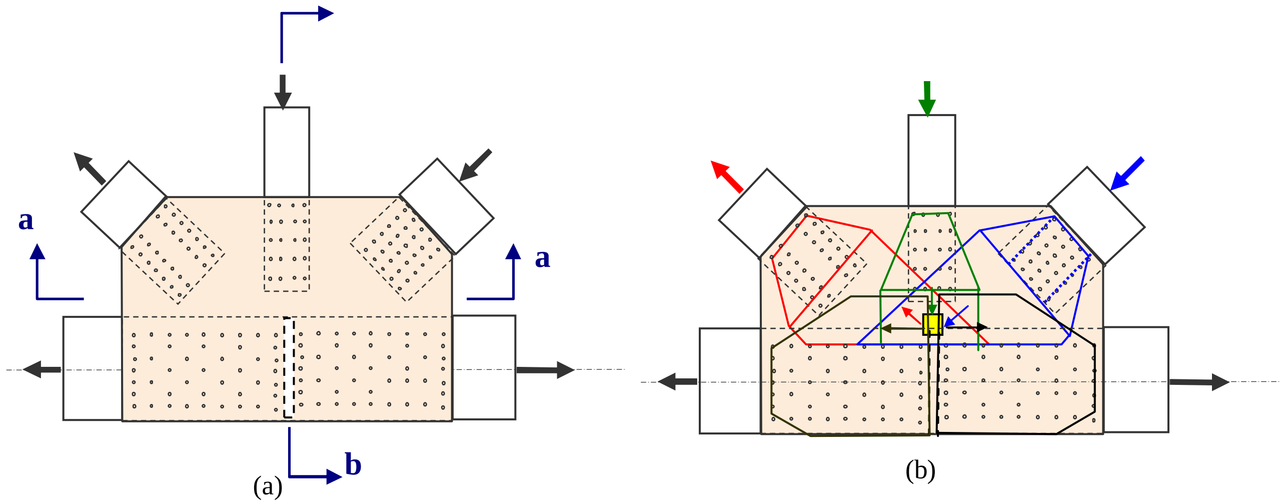


Figure 3.35. Application of Astaneh's Simple Method to a gusset plate with chord member splice located within the gusset plate.

Note that the gusset plate shown in Figure 3.34 has a continuous chord member and as a result the forces in the chord pass through the chord member and are not transferred to the gusset plate. In this case we only needed to consider the critical section "a-a" in the gusset plate as shown in Figure 3.34(a). If the chord members were spliced as shown in Figure 3.35 at the location of the gusset plate, then we had to consider both critical sections "a-a" and "b-b", Figure 3.35(a) and have Whitmore's areas established for all five members attached to the gusset plate, see Figure 3.35(b).

3.7.g Gusset Plates with Irregular Geometry

Occasionally, gusset plates, especially in the lateral bracing system, do not have the regular geometry shown in the previous sections. To check the stresses in the gusset plates following any of the methods discussed above, the first step is to establish critical sections of the gusset plates. For regular sections we identified them as sections "a-a" and "b-b". For irregular gusset plates, if Beam Theory or Interaction equations given earlier are used, one has to establish, based on intuition and understanding of stress distribution in the gusset plates, several potentially critical sections and then using free-body diagrams of various parts of the gusset plate establish normal force N, bending moment M, and shear force V on each of these critical sections.

If the Finite Element methods are used or the Astaneh's Simple Method, as discussed earlier is used, then one can establish von Mises stresses at several potentially critical nodes and limit the von Mises stress to yield stress.

3.7.h Fracture of Critical Sections of Gusset Plates

The "net area" of critical sections of gusset plates can fracture under a combination of axial tension load, bending moment, and shear, as shown in Figure 3.36. Notice that the failure mode on Figure 3.35(b) is not likely to occur if the chord member is continuous within the gusset plate.

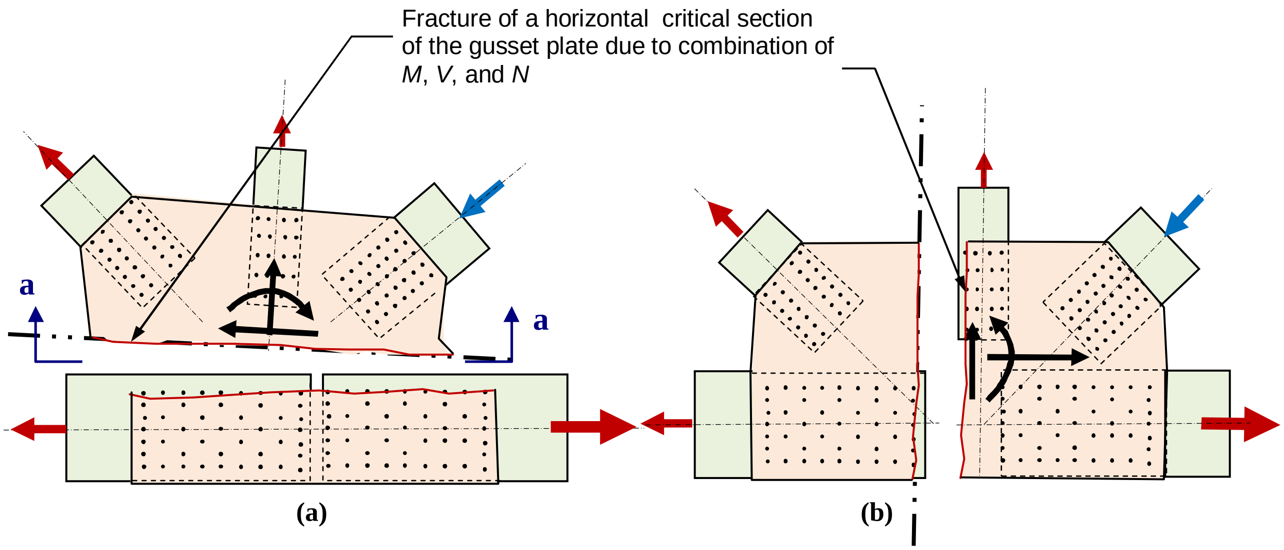


Figure 3.36. Fracture failure of critical sections of the gusset plate under the combined effects of bending moment and shear and axial forces

Based on the discussion in the previous section and similar to yielding of critical sections of gusset plates under combined loads, for fracture of critical sections I suggest the use of Equation 3.28 below.

$$\left(\frac{N}{\phi_u N_u}\right)^{2.0} + \left(\frac{M}{\phi_u M_u}\right)^{1.0} + \left(\frac{V}{\phi_u V_u}\right)^{4.0} \leq 1.0 \qquad (3.28)$$

Where;

ϕ = Resistance factor for yielding equal to 0.90.

N = Normal force (axial force) acting on the critical section of gusset plate under consideration calculated using the load in the members equal to $\eta_i\,(\phi\,R_n)_{Member}$. The values of $\eta_i\,(\phi\,R_n)_{Member}$ are the same as established earlier in Section 3.8.a.

M = Bending moment acting on the critical section of the gusset plate under consideration calculated using the load in the members equal to $\eta_i\,(\phi\,R_n)_{Member}$. The values of $\eta_i\,(\phi\,R_n)_{Member}$ are the same as established earlier in Section 3.8.a.

V = Shear force acting on the critical section of the gusset plate under consideration calculated using the load in the members equal to $\eta_i\,(\phi\,R_n)_{Member}$. The values of $\eta_i\,(\phi\,R_n)_{Member}$ are the same as established earlier in Section 3.8.a.

N_u = Tensile fracture strength of net area of critical section of gusset plate under consideration = A_nF_u.

M_u = Ultimate moment capacity of critical section of gusset plate under consideration= $Z_{x\text{-}net}F_u$.

V_u = Ultimate shear capacity of the net section of the critical section of the gusset plate under consideration = $(A_n)(0.58F_u)$.

$Z_{x\text{-}net}$ = Section modulus of the net area of the critical section of the gusset plate under consideration.

Again, similar to what was suggested for yielding of critical sections, when gusset plates do not have the regular geometry shown in previous sections, by using free body diagrams, all possible critical net sections of the gusset plate should be identified and Equation 3.28 applied to these sections.

3.8. FATIGUE LIMIT STATE

Gusset plates, especially welded gusset plates, should be investigated for a possibility of fatigue failures. The study of fatigue and design to prevent fatigue failures is a major field by itself, and the limited pages of this report do not allow for the treatment that this important subject, especially in welded gusset plates deserves. The reader is referred to the AASHTO LRFD Bridge Design Specification (AASHTO, 2007) for treatment of fatigue.

4. Notes on Evaluation of Existing Gusset Plates

4.1 INTRODUCTION

The main objective of this chapter is to provide a brief discussion of issues that are important in evaluation of gusset plates in existing truss bridges. This chapter by no means tries to provide comprehensive guidelines or procedures on how to inspect a gusset plate; rather it is hoped that the notes and tips here will be somewhat useful to bridge inspectors who, in the aftermath of the I-35W tragedy, are asked to inspect gusset plates in steel truss bridges.

To evaluate an existing bridge in the United States to establish its actual physical condition, maintenance needs and load capacity, the procedures and policies in the Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges (AASHTO, 2003) is followed. The Manual contains an introduction and background to evaluation and load rating of bridges (Section 1), followed by sections on keeping file records of the existing bridges (Section 2), the types and frequency of bridge inspections (Section 3), effective bridge management system (Section 4), various test methods (Section 5), use of LRFD in evaluation of bridges (Section 6), evaluation of existing bridges for fatigue (Section 7), bridge load test procedures (Section 8), and special topics such as evaluation of masonry bridges (Section 9). The Manual has nine appendices each providing a numerical case study of a specific type of bridge. Appendix 6 is on “Through Pratt Truss Bridge, Design Load Check of Selected Truss Members”. This appendix along with two other appendices on steel bridges can be particularly useful.

4.2 NOTES ON EVALUATION OF GUSSET PLATES

Evaluation of gusset plates and their components should be done following the provisions of the AASHTO LRFR (2003). The following, discusses some of the provisions directly applicable to evaluation of gusset plates., The items given here are for information only. In evaluation of an actual bridge and its components such as gusset plates, the AASHTO LRFR should be used.

4.3 AASHTO LRFR MANUAL ITEMS RELEVANT TO EVALUATION OF GUSSET PLATES

Section 4.8.3.6 of the AASHTO LRFR(2003) , which is on trusses, states that:

> " Check rivets and bolts to see that none are loose, worn, or sheared.
> ….
> All splice points should be checked for soundness in the shear connections. All bolts should be checked to see that they are tight and in good condition.
> ….
>
> (Excerpts from the AASHTO LRFD Bridge Design Specifications (2007), See footnote "*" below)

Following are observations on the above provisions.

1. Checking rivets and bolts on the gusset plate is extremely important as emphasized in the above box.

2. As discussed in Chapters 2 and 3, until early 1900s the splices of chord members were placed out side the gusset plate. During the 1930s-1950s gradually the splice for the chord members were moved into the gusset plate, making the gusset plate act as a splice plate as well. In bridges built after 1960's seldom chord members have a splice out of the gusset plates. Instead, gusset plates in these truss bridges act as a traditional gusset plate to transfer forces from web members of the truss to the chords as well as act as splice plate for the chord members. In evaluating truss bridges and in load rating them, evaluating strength of gusset plates is as important as the members for two reasons:

   First , although early riveted truss bridges may have had gusset plates in some cases that were stronger than the members, in most cases of steel truss bridges, gusset plates either have a strength equal to the members or most likely less than the member. This is due to the fact that AASHTO , even today does not require gusset plates to be designed stronger than the members connected to it.

   The second reason for gusset plates being as important as the members in any evaluation program is that the state of stresses and strains in a gusset plate is more complex than the state of stress in a member which adds to further uncertainty in the actual performance of gusset plates. There are the uncertainties regarding the uniformity of distribution of forces to various connecters such as rivets and bolts in a group. Also, there are stress concentration points and strain risers in the gusset plates and in the vicinity of irregularities and around the holes. As J.A.L. Waddell's 8th Principle (See Appendix) states: "The Strength of a structure is measured by the strength of its weakest part". In case of truss bridges, the "weakest part", in many cases is the gusset plate connection.

A report by Prucz and Kulicki (1998) discusses effects of corrosion section loss in steel bridges and provides valuable information on corrosion. An earlier report by Kulicki et al (1990) focuses on *evaluation* of steel bridge superstructures and provides practical field and office guidelines that can be used in evaluating the effects of corrosion on existing steel bridges including truss bridges. The AASHTO LRFR (2003) in Section 4.8.3.10 discusses issues related to inspection and evaluation of "Rivets, Bolts, and Welded Connections" and provides information on inspection of high strength friction type as well as bearing type bolts. In addition, it calls for inspection of rivets, especially those in tension to be hammer sounded for the presence of distress or movement. Also, this section states that the heads of rivets be inspected for severe losses which can occur due to corrosion. The issue of corrosion in gusset plates is an important issue and need to be included in any inspection and evaluation program since experience with steel truss bridges indicate that connections of these bridges especially gusset plates and bearing supports, in general are more susceptible to corrosion than the typical members. The following section provides some notes and information on corrosion in gusset plates.

4.4 NOTES ON CORROSION EFFECTS

Corrosion of highly stressed areas of gusset plates and loss of thickness in these areas can result in removal of part of the factor of safety of the original design and cause the gusset plate to serve with a smaller margin of safety than the bridge design specifications allow. Such a small margin of safety can easily be exceeded if any additional effect, such as temperature effects, additional construction load, heavy permit-loads, or overlay, is placed on the bridge, resulting in failure of the gusset plate. Before inspecting gusset plates in a bridge, the inspector should review the drawings and especially the past inspection reports. Any mention of corrosion in the members or gusset plates should be considered important requiring further field investigation as to the status of the corrosion that was reported in the past. Sometimes, past corrosions are cleaned and the corroded area painted over, with no mention of corrosion in the inspection reports. However, the gusset plate is still thinner than the original thickness. It is important in inspecting gusset plates for corrosion that not only the current visible corrosion is reported but also any visual evidence of loss of thickness of the gusset plate, even if it is painted over, is reported, and depending on the severity of the loss of thickness, it may be necessary to measure and map the areas where the gusset plate has thinned due to corrosion.

Gusset plates that are directly under the expansion joints that drain water over the gusset plates are susceptible to corrosion. This is especially true in cold regions where corrosive deicing may have been used on the bridge. Gusset plates that are at the support panel points, such as shown in Figure 4.1 or abutment end panel points sometimes show more corrosion than the other gusset plates. Also, gusset plates on the lower chord, probably because of being exposed to more rain water than the upper chord gusset plates, in general show more corrosion than the gusset plates on the upper chords which are in many cases under the cantilevered deck portion. Gusset plates subjected to drainage coming through the bridge joints can have severe corrosion, which in turn can result in loss of thickness of the gusset plate and its strength. The corrosion can also cause build-up of the corroded material in the space between the gusset plate and the member, pushing the gusset plate outward. This outward deformation or "dishing" of the gusset plate due to packing of corroded material was established to be the cause of buckling of a non-redundant gusset plate of I-90 Bridge in Ohio in 1996 (ODOT 2007).

The corrosion of the gusset plate due to drainage from the joints above it, particularly in cold regions, where deicing chemicals are used on the deck, can be a serious problem and needs to be prevented. In such situations, the engineer may choose to round up the calculated thickness of the gusset plate 1/8 or 3/16 inch to account for the future loss of thickness due to corrosion.

Inspection of steel truss bridges with gusset plates should include conditions of the other elements of the bridge such as lateral bracings and their gusset plates that are connected to the gusset plate under inspection.

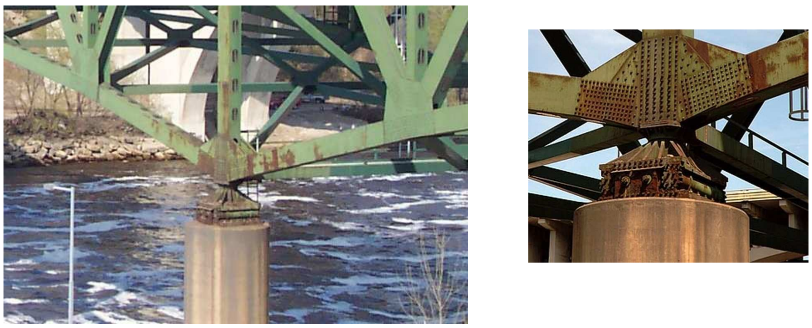
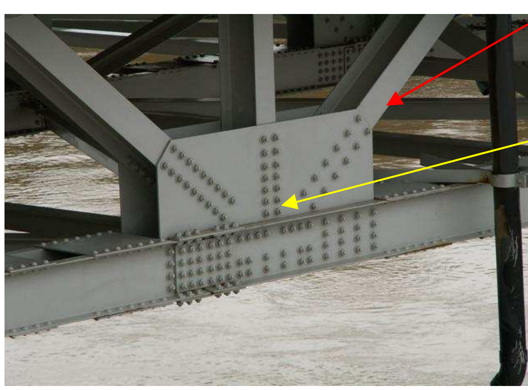


Photo by John Weeks

Figure 4.1 An example of corrosion of gusset plate right above the support being more serious than the corrosion of other gusset plates

Figures 4.2 through 4.18 show annotated photographs of gusset plates in various state of corrosion. In viewing these photographs, the reader is reminded that the photos are given here to make notes on them regarding corrosion. They do not represent the state of steel gusset plates in bridges in the U.S. I have been involved in studying gusset plates in bridge trusses for the last 20 years, and over the last few months, in search of photos for this report, I have searched steel bridges on the Internet; after reviewing hundreds of photos, I could only find these few showing gusset plate corrosion to include here for illustrative purposes. Almost the majority of the gusset plate photos were similar to Figure 4.1 with no sign of past or present corrosion visible on them, which indicates their routine maintenance by the State Departments of Transportation.



A typical gusset plate connection, where there is no visible corrosion.

This area is where corrosion can be the worse since water can be accumulated here. It is better to have this line sealed.

Photos: www.historicalbridge.org (with permission)

Figure 4.2 A gusset plate with no visible corrosion

Notice the caulking that is used to seal the contact areas to reduce water seepage into the faying surface causing corrosion

Photos from: http://www.historical bridges.com (with permission)

Figure 4.3 caulking of gusset plate to prevent water seepage into the faying surface

Birds sometimes nest in the gusset plate areas of the truss bridges or inside perforated members. Bird nests and bird droppings can cause corrosion on the bridge.

Photos from: http://www.historical bridges.com (with permission)

Figure 4.4 Birds sometimes find the confined areas of the gusset plates a perfect location for their nest

This is an example of corrosion in the critical areas of the gusset plate.

Corrosion in the member

These inner areas of gusset plate are susceptible to corrosion. A typical gusset plate connection, where there is no visible corrosion.

Photos: www.historicalbridge.org (with permission)

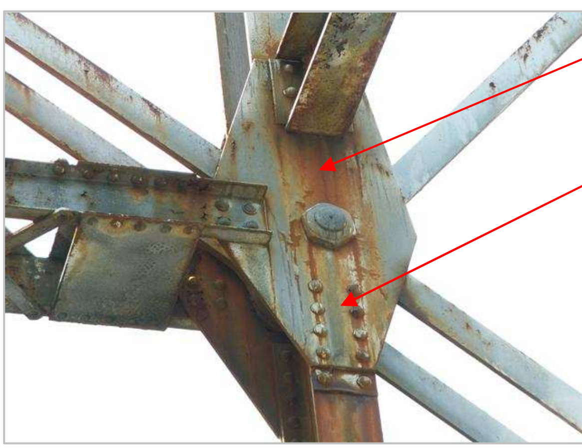
Figure 4.5 A gusset plate with relatively minor corrosion

Corrosion in the gusset plate indicates that areas of high stress are corroded more. This can be a case of stress-corrosion. It is also possible that due to elastic deformations in highly stressed areas of gusset plate, the old paint, being brittle, has peeled off and has exposed bare steel to the rain water causing the corrosion.

Figure 4.6 Moderate corrosion in gusset plates in the highly stressed areas

The corners here, at the intersection of members and gusset plates are very susceptible to corrosion and should be inspected carefully. The accumulated debris, if any, should be removed and by using wire-brush the surface be cleaned and extent of corrosion be established.
s that areas of high stress are corroded more. This can be a case of stress-corrosion.

Figure 4.7 Moderate corrosion in gusset plates in the highly stressed areas



Corrosion in the gusset plate perhaps with more serious corrosion on the back side (interior surface) as well as on the pin and its shaft need to be investigated.

The vertical corrosion lines along the block shear areas of the rivets need to be investigated for loss of thickness.

Figure 4.8 Corrosion at pin connection, one of the critical areas of truss bridges

In deck trusses, top chord gusset plates located right below the deck expansion joints are quite vulnerable to corrosion due to drainage .

Serious corrosion and loss of gusset thickness.

Photos from 2004 Annual Bridge Inspection report on I-35W by Mn/DOT

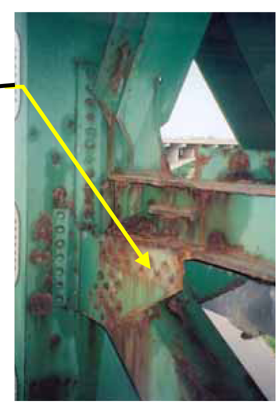
Figure 4.9. Corrosion in top chord and transverse truss joints of I-35W

Photos from: http://bridgehunter.com

As part of bridge retrofit, these drainage systems were added. This is an excellent example of corrosion protection of gusset plates under expansion joints. The solution of course is very good in draining the water away from truss members, but, the presence of so many vertical plastic pipes in front of the steel truss does not add to the aesthetic of the bridge.

Figure 4.10 The 1962 Lewis and Clark Viaduct carrying I-70 over Kansas River in Kansas with its extensive drainage system for the deck

This is an example of corrosion in the connections of sway frames.



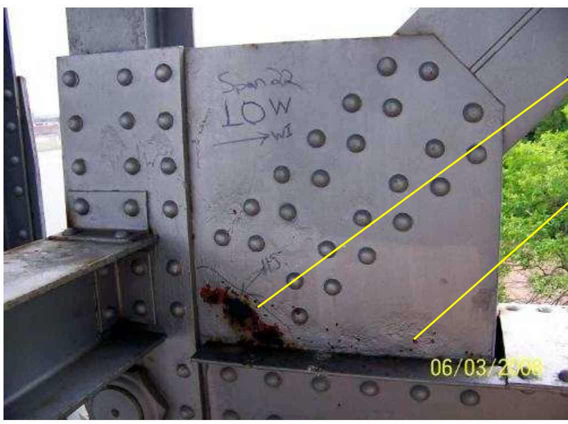
Photos from 2005 Annual Bridge Inspection report on I-35W by Mn/DOT

Figure 4.11 Serious corrosion in gusset plates and members

This is an example of corrosion in the member and critical area of the gusset plate. Probably some loss of thickness has occurred here.

Photos:
www.historicalbridge.org

Figure 4.12 Serious corrosion in gusset plates and members



This is an example of serious corrosion and loss of thickness of gusset plate.

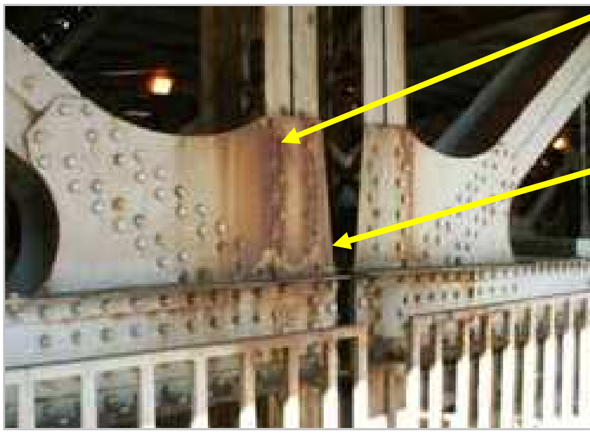
At this location of gusset plate, past corrosion has resulted in loss of thickness. It appears that the corroded area is painted over. It is very important to carefully and from close-up inspect the gusset plates and discover old corroded areas that might have been painted over.

Figure 4.13 Serious corrosion in gusset plates and probably in the pin

This is an example of corrosion in the member and critical areas of the gusset plate.

Photos: www.historicalbridge.org

Figure 4.14 Serious corrosion in gusset plates and probably in the pin



This is an example of corrosion in the critical section of the gusset plate. Its severity and loss of thickness of gusset plate needs to be investigated.

This area of gusset plate also can have serious corrosion and loss of thickness.

The source of corrosion here seems to be drainage of deicing water through the expansion joint in above. The situation needs to be corrected. See figure below for drainage system

Figure 4.15 Corrosion due to drainage of water from expansion joint

Packing of rust between the steel plates in riveted built-up members. Such corrosions can also occur in the space between the gusset plate and the steel members.

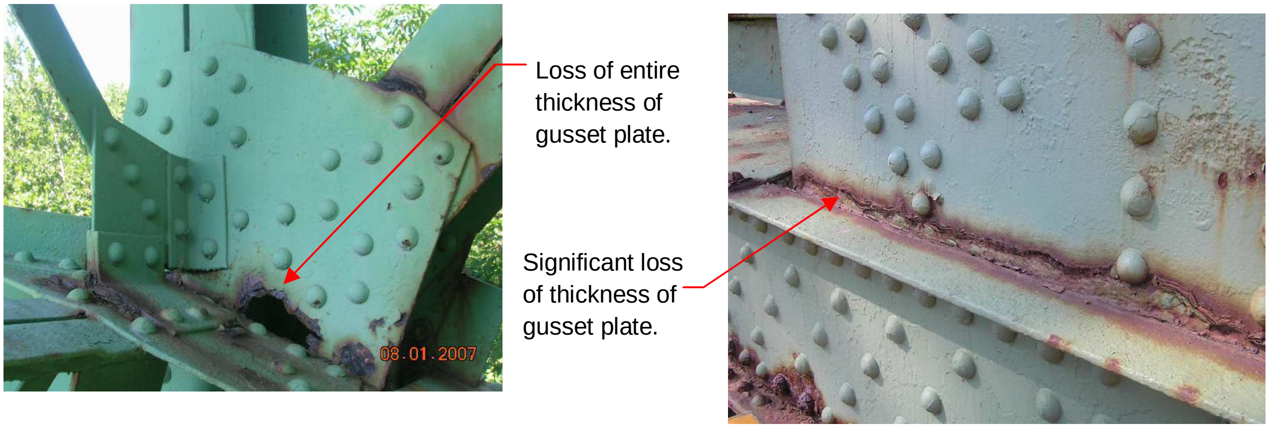
Photo: HW23 Bridge Special Inspection report by Mn/DOT(2007)

Figure 4.16. Pack rust in a riveted member

Pack rust can be a major problem in gusset plates. The force of expansion of iron oxide can bend the steel piece breaking rivets or bolts.

Photos from: http://corrosion-doctors.org/

Figure 4.17 caulking of gusset plate to prevent water seepage into the faying surface



Photos courtesy of Wahid Albert (Albert, 2008) at https://www.nysdot.gov/

Figure 4.18. Very serious condition of corrosion in gusset plates

I will close this chapter with the case of corrosion of the 1923 Devil's Elbow Bridge carrying the Old Route 66 over the Big Piney River in Missouri. Figure 4.19 shows two views of the bridge which appears to have some corrosion. However, the actual extent of corrosion of steel elements and deterioration of concrete deck and substructure elements of bridge becomes only clear by closer look at this bridge. I found condition of this bridge in the article: "The Historical Devil's Elbow Bridge Is In Need Of Repair-Rusting Steel and Crumbling Concrete Threatens This Landmark" by Conor Watkins posted at the http://www.rollanet.org/~conorw/cwome/article51&52combined.htm. The photographs and information given by Conner Watkins (2006) in this article shows a bridge that has extremely serious level of corrosion in steel superstructure including its gusset plates as well as very serious deterioration, cracking and spalling in the concrete deck and the substructure of the bridge, yet, the bridge is open to traffic, (although very light traffic). Figure 4.20 shows a few of the photographs from Connor Watkins' article with my notes added to each photograph.

Photos and information from: http://bridgehunter.com/mo/pulaski/piney-truss/

Figure 4.19. The 1923 Devil's Elbow Bridge over Old Route 66 in Missouri

The site bridgehunter.com states that this bridge has a rating of 4 out of 9 and is appraised as *structurally deficient*. The deteriorated condition of this bridge, although may not represent the conditions of a large percentage of structurally deficient bridges, shows that how badly some bridges and other components of the infrastructure are deteriorated and are in need of repair, retrofit or replacement. It is hoped that the current plans for allocating billions of dollars to upgrade the infrastructure as a component of the economical stimulus package of the Federal government results in allocating sufficient amount of funding to inspection, evaluation, repair, retrofit and replacement of the structurally deficient components of the infrastructure and in particular to deteriorated fracture critical bridges that can potentially be public safety hazards, yet are used by the public while continue to deteriorate further.

A view of the bridge in its breathtaking surrounding

Deterioration of the concrete deck

Corrosion in the bearing supports

Deterioration of concrete pier and confinement mechanism added to the top.

Large areas of the gusset plates for the lateral bracing system have been corroded away and torn apart

Very serious corrosion of the gusset plates, bottom chord, end post end beam and floor stringers at one of the abutments

Another view of the same support as shown in the above photo with very serious corrosion

Photos by Connor Watkins from his article athttp://www.rollanet.org/~conorw/cwome/article51&52combined.htm

Figure 4.20. Close-up photos of the 1923 Devil's Elbow Bridge showing its severe deterioration

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APPENDIX:
J. A. L. Waddell's First Principles of Designing

This appendix provides an abbreviated version of Chapter II of the *De Pontibus* (Waddell, 1898) by John Alexander Low Waddell, the father of modern bridge engineering. In the following only the opening paragraphs of the chapter are given followed by the statements of his 42 principles. In the actual book each statement of principle is followed " by remarks of an explanatory nature giving its *raison d'être* or application".. as J.A.L. Waddell explains.

A full electronic version of the De Pontibus book by J.A.L. Waddell can be downloaded from the http://books.google.com

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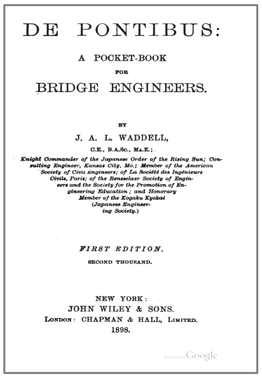
DE PONTIBUS:

A POCKET-BOOK

FOR

BRIDGE ENGINEERS.

BY

J. A. L. WADDELL,

C.E., B.A.Sc., MA.E.;

Knight Commander of the Japanese Order of the Rising Sun; Consulting Engineer, Kansas City, Mo.; Member of the American Society of Civil Engineers; of La Société des Ingénieurs Civils, Paris; of the Rensselaer Society of Engineers and the Society for the Promotion of Engineering Education; and Honorary Member of the Kogaku Kyokai (Japanese Engineering Society.)

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J. A. L. Waddell.

CHAPTER II.

FIRST PRINCIPLES OF DESIGNING.

BOTH the student and the practitioner in bridge-designing will do well to recognize and bear constantly in mind certain first principles of design; and to enable them to do so, the author offers the following, which he considers will cover the essential fundamental principles that should govern the designing of all structural metal-work. Most of these will be repeated in the "General Specifications" given in Chapter XIV. under the heading "General Principles in Designing all Structures," for the reason that the said specifications would be incomplete without them.

The reason for this special chapter being devoted exclusively to these general principles is that the subject is of the utmost importance, and needs much more elaboration than could properly be given it in specifications. On this account the statement of each principle herein will be followed by remarks of an explanatory nature giving its *raison d'être* or application. It is to be noticed that the numbering does not agree with that of the "General Principles" in Chapter XIV.

The attention of the reader is called to the fact that this chapter is by far the most important one in the book, in that it contains in a concentrated form the most important conclusions drawn from the author's entire experience in his chosen specialty. The principles given have been established mainly by observation of the mistakes of others, and in a few cases, it must be confessed, by those of his own.

Few designers care to make public their errors for fear of the result being to their disadvantage; nevertheless far more is learned from the mistakes of construction than is learned in any other way.

The author would therefore suggest to the reader that he peruse this chapter carefully more than once before proceeding to the next.

Principle I.

Simplicity is one of the highest attributes of good designing.

Principle II.

"The easiest way's the best."

Principle III.

The systemization of all that one does in connection with his professional work is one of the most important steps that can be taken towards the attainment of success.

Principle IV.

There is an inherent sense of fitness in the mind of a well-trained and well-balanced metal-work designer, which sense of fitness is of the greatest importance in all that he does.

Principle V.

There are no bridge specifications yet written, and there probably never will be any, which will enable an engineer to make a complete design for an important bridge without using his judgment to settle many points which the specifications do not properly cover; or as Mr. Theodore Cooper puts it: "The most perfect system of rules to insure success must be interpreted upon the broad grounds of professional intelligence and common sense."

Principle VI.

In every detail of bridge-designing the principles of true economy must be applied by every one who desires to be a successful bridge engineer.

Principle VII.

In bridge-designing rigidity is quite as important an element as is mere strength.

Principle VIII.

The strength of a structure is measured by the strength of its weakest part.

Principle IX.

In bridge-designing provision must always be made for the effect of impact, either by increasing the calculated total stresses by a varying percentage of the live-load stresses, or by decreasing the intensities of working stresses below those allowed for statically applied loads.

Principle X.

In making the general layout of any structure, due attention should be given to the architectural effect, even if the result be to increase the cost somewhat.

Principle XI.

For the sake of uniformity, and to conform to the unwritten laws of fitness, it is often necessary in bridge-designing to employ metal which is not really needed for either strength or rigidity.

Principle XII.

Before starting a design, one should obtain complete data for same.

Principle XIII.

A skew-bridge is a structure the building of which should always be avoided when it is practicable.

Principle XIV.

The best modern practice in bridge-engineering does not countenance the building of structures having more than a single system of cancellation, except in lateral systems where the resulting ambiguity of stress distribution is of minor importance.

Principle XV.

The employment of a redundant member in a truss or girder is never allowable under any circumstances, unless it be in the mid-panel of a span having an odd number of panels, in which case, for the sake of appearance, two stiff diagonals can be used.

Principle XVI.

The use of a curved strut or tie in bridge-designing for the sake of appearance (or for any other reason) is an abomination that cannot for an instant be tolerated by a good designer.

Principle XVII.

In all structural metal-work, excepting only the machinery for operating movable parts, no torsion on any member should be allowed if it can possibly be avoided; otherwise, the greatest care must be taken to provide ample strength and rigidity for every portion of the structure affected by such torsion.

Principle XVIII.

The gravity axes of all the main members of trusses and lateral systems coming together at any apex of a truss or girder should intersect in a point whenever such an arrangement is practicable; otherwise the greatest care must be employed to insure that all the induced stresses and bending moments caused by the eccentricity be properly provided for.

Principle XIX.

Truss members and portions of truss members should always be arranged in pairs symmetrically about the plane of the truss, except in the case of single members, the axes of which lie in said plane of truss.

Principle XX.

In proportioning main members of bridges, symmetry of section about two principal planes at right angles to each other is a desideratum to be attained whenever practicable.

Principle XXI.

In both tension and compression members the centre line of applied stress must invariably coincide with the axial right line passing through the centres of gravity of all cross-sections of the member taken at right angles thereto.

Principle XXII.

The principle of symmetry in designing must be carried even into the riveting; and groups of rivets must be made to balance about central lines and central planes to as great an extent as is practicable.

Principle XXIII.

In proportioning members of bridges to meet stresses and combinations of stresses it is important to consider duly the quality, frequency, and probability of the action of said stresses or combinations of stresses.

Principle XXIV.

In all main members having an excess of section above that called for by the greatest combination of stresses, the entire detailing should be proportioned to correspond with the utmost working capacity of the member, and not merely for the greatest total stress to which it may be subjected. In this connection, though, the reduced capacity of single angles connected by one leg only must not be forgotten.

Principle XXV.

In every bridge and trestle adequate provision must be made for the contraction and expansion of the metal.

Principle XXVI.

No matter how great its weight may be, every ordinary fixed span should be anchored effectively to its supports at each bearing on same.

Principle XXVII.

The bridge-designer should never forget that it is essential throughout every design to provide adequate clearance for packing, and to leave ample room for assembling members in confined spaces.

Principle XXVIII.

Although for various reasons engineers are agreed that field-riveting should be reduced to a minimum, such an opinion should not be allowed to militate against the employment of rigid lateral systems. All designs should be arranged so that the field-rivets can be driven readily.

Principle XXIX.

Rivets should not be used in direct tension.

Principle XXX.

For members of any importance two rivets do not make an adequate connection.

Principle XXXI.

Designs must invariably be made so that all metal-work after erection shall be accessible to the paint-brush, except, of course, those surfaces which are in close contact either with each other or with the masonry.

Principle XXXII.

In multiple-track structures, if any bracing-frames be used between panel points to connect the longitudinal girders of adjoining tracks, they must be designed without diagonals, in order to prevent the transference of any appreciable portion of the live load from one pair of girders to any other pair of same

Principle XXXIII.

In bridges, trestles, and elevated railroads, the thrust from braked trains and the traction should be carried from the stringers or longitudinal girders to the posts or columns without producing any horizontal bending moment on the cross-girders.

Principle XXXV.

Every column that acts as a beam also should have solid webs at right angles to each other, as no reliance can be placed on lacing to carry a transverse load down the column.

Principle XXXIV.

In trestles and elevated railroads the columns should be carried up to the tops of the cross-girders or longitudinal girders and be effectively riveted thereto.

Principle XXXVI.

In trestles and elevated railroads every column should be anchored so firmly to its pedestal that failure by overturning or rupture could not occur in the neighborhood of the foot if the bent were tested to destruction.

Principle XXXVII.

All pedestals for trestles, viaducts, and elevated railroads should be raised to such an elevation as to prevent the accumulation of dirt and moisture about the column feet, and all boxed spaces in the latter should be filled with extra-rich Portland-cement concrete.

Principle XXXVIII.

In designing short members of open-webbed, riveted work, it is better to increase the sectional area of the piece from ten to twenty-five per cent than to try to develop the theoretical strength by using supplementary angles at the ends to connect to the plates.

Principle XXXIX.

Star-struts formed of two angles with occasional short pieces of angle or plate for staying same do not make satisfactory members. Better results are obtained by placing the angles in the form of a T.

Principle XL.

In making estimates of weights of metal the computer should always be liberal in allowing for the weight of details.

Principle XLI.

In general details must always be proportioned to resist every direct and indirect stress that may ever come upon them under any possible condition, without subjecting any portion of their material to a stress greater than the legitimate corresponding working stress.

Principle XLII.

There is but one correct method of checking thoroughly the entire detailing of a finished design for a structure, viz.: "Follow each stress given on the stress-diagram from its point of application on one main member until it is transferred completely to either other main members or to the substructure, and see that each plate, pin, rivet, or other detail by which it travels has sufficient strength in every particular to resist properly the stress that it thus carries; check also the sizes of such parts as stay-plates and lacing, which are not affected by the stresses given on the diagram, and see that said sizes are in conformity with the best modern practice."

About the Author:

Abolhassan Astaneh-Asl, Ph.D., P.E., is a professor of structural engineering at the University of California, Berkeley. He is the winner of the 1998 T. R. Higgins Lectureship Award of the American Institute of Steel Construction. From 1968 to 1978 he was a structural engineer and construction manager in Iran. During the period 1978–1982, he completed his M.S. and Ph.D. in structural engineering, both at the University of Michigan in Ann Arbor. During 1982-1986, he was on the faculty of the University of Oklahoma, Norman and since 1986 he has been a faculty member of structural engineering at the University of California, Berkeley, involved in teaching, research, and design of both building and bridge structures.

He has conducted several major projects on seismic design and retrofit of steel long span cable supported and truss bridges including the Golden Gate, the San Francisco Oakland Bay Bridge, the Carquinez , the Richmond San Rafael, the Hayward San Mateo, the Rama-8 and the Auckland Harbour Bridges and a number of short span girder bridges. In addition, he has investigated failure of the San Francisco Oakland Bay Bridge, the I-35W Bridge and seismic damage to major bridges in Japan including the Akashi-Kaikyo, the Higashi Kobe and the Nishinomia Bridges.

Since 1979, he has studied and tested gusset plates and has developed design guidelines that are now included in the national design specifications. Since 1986 he has studied behavior of steel truss bridges and has developed guidelines for their seismic design and evaluation.

Since1995, he has been studying behavior of steel and composite structures, both buildings and bridges, subjected to blast and impact loads and has been involved in research and development of technologies and design concepts to reduce blast damage and to prevent progressive collapse of steel and composite building and bridge structures subjected to terrorist blast (car bombs) or impact (planes and rockets) attacks.

After the September 11, 2001, tragic terrorist attacks on the World Trade Center and the collapse of the towers, armed with a research grant from the National Science Foundation, he conducted a reconnaissance investigation of the collapse and collected perishable data. As an expert, he later testified before the Committee on Science of the House of Representative of the U.S. Congress on his findings regarding the collapse of the World Trade Center towers. His most recent work on buildings includes development of rational procedures for seismic design of steel and composite shear walls as well as seismic design guidelines for connections such as gusset plates and base plates..

His current research includes studies of blast effects on steel and composite bridge and building structures and development of technologies to prevent their progressive collapse , seismic behavior and design of steel and composite structures, failure analysis of collapsed structures including the I-35W bridge and the Mac Arthur Maze and research on earthquake hazard reduction in the Middle East.

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