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U.S. Department of Transportation
Federal Highway Administration

Seismic Retrofitting Guidelines for Complex Steel Truss Highway Bridges

By

Tom Ho, Roupen Donikian, Tim Ingham, Chuck Seim and Austin Pan

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Seismic Retrofitting Guidelines for Complex Steel Truss Highway Bridges

by

Tom Ho (Coordinating Author)[1], Roupen Donikian[1], Tim Ingham[1],
Chuck Seim[2] and Austin Pan[1]

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Contract Officer's Technical Representative: W. Phillip Yen, Ph.D., P.E. HRDI-7
Senior Research Structural Engineer/Seismic Research Program Manager
Federal Highway Administration

1 T.Y. Lin International, San Francisco, California
2 Consultant, San Francisco, California

MCEER
University at Buffalo, The State University of New York
Red Jacket Quadrangle, Buffalo, NY 14261
Phone: (716) 645-3391; Fax (716) 645-3399
E-mail: *mceer@buffalo.edu*; WWW Site: *http://mceer.buffalo.edu*

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised March 2002)

PREFACE

This manual, entitled *Seismic Retrofitting Guidelines for Complex Steel Truss Highway Bridges* (referred to as *Guidelines* in this document), collects and summarizes the state-of-the-practice, up to 2005, for retrofitting steel truss bridges on the highway system within the United States. These *Guidelines* are based on and are supplementary to the *Seismic Retrofitting Manual for Highway Structures Part 1: Bridges* (referred to in these *Guidelines* as the *Bridge Retrofitting Manual*), developed by the Multidisciplinary Center for Earthquake Engineering Research (MCEER), to be published by the Federal Highway Administration (FHWA) in 2006.

The *AASHTO Standard Specifications for Highway Bridges* generally apply to the design of new "ordinary" highway bridges with spans less than 500 ft and with a design life of 75 years. The *AASHTO Standard Specifications for Highway Bridges* also suggest that supplemental specifications may be required for the design of new highway bridges of "unusual types," or for the design of "long-span" bridges with spans longer than 500 ft. These two bridge classifications usually have design lives of 100 years or more.

Similarly, the seismic retrofit design of existing bridges can also be separated into "ordinary" highway bridges, "unusual" types of highway bridges, and "long-span" highway bridges with spans greater than 500 ft. These *Guidelines* function as a special supplemental document to the *Bridge Retrofitting Manual* for the seismic retrofit design of "unusual" types of highway steel trusses or for "long span" highway steel trusses, with spans longer than 500 ft.

The *Bridge Retrofitting Manual* focuses on the design of the seismic retrofitting of "ordinary" highway bridges. It is intended to be applicable nationwide for ordinary concrete substructures and for ordinary steel and concrete girder-type highway bridges.

The *Bridge Retrofitting Manual* divides ordinary highway bridges into two classifications, "essential" and "standard," by their expected performance after a seismic event. Essential bridges are all bridges that are expected to function after a seismic event by continuing to carry traffic. Standard bridges are all other ordinary highway bridges, which may sustain minor to serious damage after a seismic event (but preserve life safety) and may need extensive repair or replacement.

These *Guidelines* also define two classifications of highway steel trusses by their truss configurations: "seismically standard" trusses, which are "ordinary highway bridge" trusses; and "seismically complex" trusses, which are the "unusual" types of highway steel trusses and "long-span" highway steel trusses. All highway truss bridges that meet the classification of "essential bridges" in the *Bridge Retrofitting Manual* are automatically classified as "seismically complex" trusses in these *Guidelines.*

As in the *Bridge Retrofitting Manual,* a performance-based seismic retrofit philosophy is used in these *Guidelines* with performance criteria specified for two earthquake ground motions: a *lower level* earthquake with a mean return period of 100 years, and an *upper level* earthquake with a mean return period of 1,000 years. For the "seismically standard truss" classification, a higher performance requirement is specified for the *lower level* earthquake than for the *upper level* earthquake. For the “seismically complex truss” classification, a higher performance requirement is specified for both *lower level* and *upper level* earthquakes, because seismically complex trusses have special structural configurations that behave under seismic excitation in a complex manner; thus they require a higher standard of seismic retrofit.

These *Guidelines* are written primarily for practicing bridge design engineers who have some familiarity with the seismic retrofitting design of ordinary steel and concrete girder bridges. Experience in the applications of more advanced design techniques such as nonlinear analysis, soil-foundation-structure interaction, and experience with bridge construction methods are helpful in applying these *Guidelines*. U.S. customary units are used rather than SI units because they were used in the construction of most of the truss bridges that will require seismic retrofitting.

While these *Guidelines* cover all of the major aspects pertinent to the seismic retrofitting of steel truss bridges, its focus is on superstructure (including steel support towers) retrofit, and it does not include explicit guidelines for foundation retrofit. However, general guidelines on foundation modeling related to bridge structures have been provided in these *Guidelines* and can also be found in two existing MCEER publications [PoLam et al., 1998; PoLam and Law, 2000]. The latter publication addresses issues related to foundation modeling and soil-structure interaction analyses for large pile groups and caissons often associated with steel truss bridges. A third publication, *Bridge Foundations: Modeling Large Pile Groups and Caissons for Seismic Design* [Martin] is in preparation.

Theses *Guidelines* comprise seven chapters on the technical application of the seismic retrofitting of steel truss highway bridges:

1 Introduction

2 Retrofitting Philosophy and Process

3 Screening and Prioritization

4 Structural Analysis

5 Design Parameters

6 Evaluation of Members, Connections and Subsystems

7 Retrofit Measures, Approach and Strategy

Three additional chapers provide supporting information:

8 Case Studies

9 Glossary

10 References and Bibliography

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AISC	American Institute of Steel Construction
ARS	Acceleration Response Spectrum
ASD	Allowable Stress Design
ASL	Anticipated Service Life
ASTM	ASTM International; formerly American Society of Testing and Materials
ATC	Applied Technology Council
CQC	Complex Quadratic Combination
DS	Damage State
EBF	Eccentrically Braced Frame
ETHA	Elastic Time History Analysis
FCM	Fracture Critical Member
FHWA	Federal Highway Administration
FRP	Fiber Reinforced Polymer
ITHA	Inelastic Time History Analysis
Kgi	Kips per square inch
LFD	Load Factor Design
LL	Lower Level Earthquake
LRFD	Load and Resistance Factor Design
LRFR	Load and Resistance Factor Rating
MCEER	Multidisciplinary Center for Earthquake Engineering Research
MSTH	Multi-Support Time History
NSO	Non-seismic and other factors parameter
PL	Performance Level
PRP	Peer Review Panel
PS&E	Plans, Specifications and Estimate
RCR	Repair Cost Ratio
RMD	Response Modification Device
RSA	Response Spectrum Analysis
SC	Seismically Complex
SFI	Soil-Foundation Interaction
SFSI	Soil-Foundation-Structure Interaction
SHPO	State Historic Preservation Officer
SI	System International (i.e., metric)
SLC	Service Life Category
SRSS	Square Root of the Sum of the Squares
SS	Seismically Standard
TADAS	Triangular Added Damping and Stiffness
UL	Upper Level Earthquake
VSL	Vertical Shear Links

CHAPTER 1: INTRODUCTION

1.1 GENERAL

The *Seismic Retrofitting Guidelines for Complex Steel Truss Highway Bridges* (referred to herein as *Guidelines*) summarizes and extends the state-of-the-practice for retrofitting steel bridge trusses on the highway system within the United States. These *Guidelines* are based on recent advanced research in earthquake engineering and on experiences from steel truss highway bridge seismic retrofitting projects that formed the basis of the "State of Practice" to 2005.

Since 1971, every moderate or major earthquake in the United States has been followed by intense research that has led to a better understanding of the seismic behavior of bridges. The results of these seismic research projects are improvements and refinements to the standards of practice and the development of seismic retrofitting manuals and guidelines that can be used by bridge designers. An important product of past research is the new two-part *Seismic Retrofitting Manual for Highway Structures* [FHWA/MCEER 2006], developed by the Multidisciplinary Center for Earthquake Engineering Research (MCEER), and scheduled to be published by the Federal Highway Administration (FHWA) in 2006.

The *Seismic Retrofitting Manual for Highway Structures –Part 1: Bridges* (referred to in these *Guidelines* as the *Bridge Retrofitting Manual*) focuses on the seismic retrofitting of conventional or "ordinary" highway bridges. The seismic retrofitting processes for these ordinary highway bridges include seismic performance criteria, seismic and geotechnical hazards, performance-based seismic retrofit categories, screening and prioritization of bridges for seismic retrofitting, methods of evaluating bridges for seismic retrofitting, and seismic retrofitting strategies. *Part 2: Retaining Structures, Slopes, Tunnels, Culverts, and Pavements* focuses on non-bridge highway structures and therefore is not referred to in these *Guidelines*.

The *Bridge Retrofitting Manual* is intended to be applicable nationwide for all levels of seismic hazards, for conventional concrete substructures, and for steel and concrete girder-type highway bridges with spans less than 500 ft and a design life limited to 75 years. Suspension bridges, cable-stayed bridges, arches, long-span trusses, and movable bridges are not covered. However, many of the procedures and techniques presented in the *Bridge Retrofitting Manual* can be applied to these types of structures, if appropriate engineering judgment is used.

These *Guidelines* extend the retrofit provisions of the *Bridge Retrofitting Manual* to specifically cover the seismic retrofitting of both standard and complex steel truss highway bridges (see Section 2.2 for definitions). These *Guidelines* use U.S. customary units rather than SI units because U.S. customary units were used in the construction of most of the truss bridges that will require seismic retrofitting.

These *Guidelines* do not focus on nor prescribe specific requirements as to when steel truss highway bridges are to be retrofitted. Several engineering-based methods for screening truss bridges for seismic retrofitting are presented in Chapter 3; however, the decision to retrofit a bridge depends upon a number of factors, several of which are outside the realm of engineering. These include, but are not limited to, the availability of funding and a number of political, social, and economic issues. The focus of these *Guidelines* is exclusively on the engineering factors pertaining to the seismic retrofit of complex steel truss highway bridges, without restrictions on span length, design life, or seismic hazard, and is applicable to all regions of the United States.

These *Guidelines* are written primarily for practicing bridge design engineers who have some familiarity with the seismic retrofit design of ordinary steel and concrete girder bridges. Experience in the application of more advanced design techniques such as nonlinear analysis, soil-foundation-structure interaction, and in bridge construction methods, is helpful in applying these *Guidelines*. As it is not possible to cover all seismic retrofit situations, skill in applying engineering judgment is an asset that will also be helpful to bridge engineers using these *Guidelines*. A team approach to the retrofit design will be beneficial.

1.2 HISTORY OF SEISMIC PROVISIONS IN BRIDGE DESIGN PRACTICE

The American Association of State Highway and Transportation Officials (AASHTO) bridge design specifications have been the standard bridge design code since 1931 when their first publication codified the available data on bridge design. Since then, this document has been continually improved and expanded to become the current *AASHTO Bridge Design Specifications* used for the design of all types of ordinary bridges, for spans of 500 ft or less, and design life of 75 years.

Until 1971, the AASHTO seismic provisions required an equivalent static lateral force, usually applied at the deck level, of only two to six percent of the dead weight, depending on whether the footings were spread footings or supported on piles. The AASHTO bridge design specifications have never included any provisions for seismic retrofitting. For seismic retrofitting of existing bridges, engineers had to adapt the AASHTO seismic design requirements for new bridges as best they could to carry out the retrofit design of existing bridges.

The major turning point in modern seismic engineering of bridges in the United States was the 1971 earthquake in the San Fernando Valley of California. In the aftermath of that event, the California Department of Transportation (Caltrans) responded to public outcry to make highway bridges safe from earthquake collapse. In 1973, Caltrans issued new seismic design criteria for bridges in California. This was the first attempt in the United States to relate peak ground accelerations, as shown on seismic hazard maps to different soil types at bridge sites; to the dynamic-response characteristics of the structure; and to force-reduction factors that account for inelastic behavior. These California seismic design criteria formed the basis of the national seismic provisions published in the 1977 *AASHTO Standard Specifications for Highway Bridges.*

In 1978, FHWA awarded a contract to the Applied Technology Council (ATC) to develop improved seismic design guidelines for highway bridges that would be applicable to all regions in the U.S. The product of this effort was the *ATC-6 Seismic Design Guidelines for Highway Bridges* [ATC 1981]. FHWA followed this pioneering work by publishing Report No. FHWA/RD-83/007, *Seismic Retrofitting Guidelines for Highway Bridges* [FHWA 1983]. This was the first document to focus on the seismic evaluation and retrofitting of ordinary highway bridges and provided nationally applicable guidelines to the bridge design profession.

In 1987, FHWA published *Seismic Design and Retrofit Manual for Highway Bridges* [FHWA 1987], which updated and expanded the 1983 work into a manual for the design and retrofit of highway bridges.

The 1987 document was followed in 1995 by the FHWA publication *Seismic Retrofitting Manual for Highway Bridges* [FHWA 1995]. This 1995 manual incorporated the experience obtained from the use of the 1987 publication, new knowledge gained from analytical and experimental research, and reconnaissance trips to earthquake-devastated areas. The 1995 manual was developed as an interim document with the expectation that it would be revised and updated at a later date.

The 1995 manual has now been revised, updated, and expanded by MCEER as the two-part *Seismic Retrofitting Manual for Highway Structures*, scheduled to be published in 2006 by FHWA. Part 1 of the manual, referred to in these *Guidelines* as the *Bridge Retrofitting Manual*, incorporates experience gained from recent earthquakes with the intense seismic research effort that has recently been developed and conducted in several structural testing laboratories.

Another important product of recent research related to the seismic design of new bridges is the document entitled *Recommended LRFD Guidelines for the Seismic Design of Highway Bridges* [MCEER/ATC 2003]. This document was developed by a joint venture of MCEER and ATC for NCHRP Project 12-49 and focuses exclusively on seismic design requirements for the LRFD design of new bridges. It is applicable nationwide for all levels of seismic hazards. It does not, however, cover seismic retrofitting of existing highway bridges.

Seismic retrofitting of bridge substructures is discussed in the *Bridge Retrofitting Manual* and is not addressed in these *Guidelines*. These *Guidelines* focus on the seismic retrofitting of highway bridge steel truss superstructures and the on steel pier shafts and steel braced towers.

1.3 INTRODUCTION TO STEEL TRUSS HIGHWAY BRIDGES

Over the last 100 years or more, the steel truss has often been selected for highway bridges in the span range of about 150 ft to 1200 ft. Until nearly the end of the twentieth century, truss bridges served as the bridge of choice for long-span sites. Over 80 percent of bridges with spans longer than the AASHTO limit of 500 ft are truss bridges. Most of these structures were designed without the bridge engineering profession’s current understanding of earthquake ground motions, magnitude of seismic forces, and complex structural responses. This is the underlying reason for the production of these *Guidelines*.

The single most significant characteristic that nearly all highway truss bridges share is that the center of mass of the superstructure is located well above the supporting bearings; thus the inertia response of the superstructure to seismic excitation is similar to an inverted pendulum. In rare instances, a few deck trusses are hung below their support bearings; in these cases, the inertia response is similar to a pendulum under seismic excitation. Another general characteristic of highway truss bridges is that all trusses are composed of multiple members connected together. Each member carries either a tension or a compression force.

Trusses can be supported by tall steel braced towers, such as on the Astoria Bridge (Oregon 1966), or by unreinforced masonry piers, such as on the multi-span Queensboro Bridge (New York 1908). Multi-span trusses can be joined together to form a long structure of simple span trusses, such as the double-decked East spans of the San Francisco-Oakland Bay Bridge (California 1936). Trusses can be used for very long spans, such as the Longview Bridge cantilever truss with a span of 1200 ft (Washington 1930), or the 1400 ft cantilever truss of the East Bay span of the San Francisco-Oakland Bay Bridge (California 1936). Occasionally truss bridges, such as the Benicia Martinez Bridge (California 1962), utilize high piers and multiple spans to make a long bridge. This bridge has seven 525 ft truss spans, is over 3600 ft long, and is supported on concrete piers up to 130 ft high.

Trusses are also used in special applications, such as stiffening elements in suspension bridges, movable span bridges, or arch ribs in arch bridges. They are also commonly used for railroad bridges. The seismic retrofit design of these special applications of trusses is not specifically included in these *Guidelines*; however, an experienced bridge engineer can use the basic seismic retrofitting techniques presented here for these special applications of trusses.

1.3.1 STEEL TRUSS HIGHWAY BRIDGE CHARACTERISTICS

In these *Guidelines*, steel highway truss bridges are generally defined as a framework of straight steel members, forming triangular patterns, that are connected together to form a primary load-supporting system of axially loaded members called a truss. Trusses contain upper and lower chords that act in compression or tension. Vertical members, called verticals, posts, or plumb posts, connect the upper and lower chords and act in tension or compression. Sloping members, called diagonals, frame between the upper and lower chords and between the vertical members, and act in tension or compression.

The triangle is the basic unit of a truss bridge configuration, as it is the most inherently stable planar geometric shape. Each straight member of the triangular configuration acts either in tension or in compression, with the assumption that the end connections act as frictionless pin joints. In reality, small secondary bending stresses are induced into members because these connections have some stiffness and do not act as frictionless pinned joints.

In trusses with parallel chords, the vertical members form a rectangular shape called a panel. With variable-depth or haunched trusses, vertical members form trapezoidal-shaped panels with the horizontal and sloping chord members. The vertical members form the boundary of the panel and the centerline of the vertical members mark a series of locations along the truss called panel points. In trusses with diagonal members only, the apexes of the triangles form the panel points.

Generally, two primary trusses are spaced apart by floor beam members framed into the vertical posts or into the apex of the diagonals. The floor beams support longitudinal stringers which in turn support the deck on which vehicles travel. The deck is most frequently reinforced concrete but open steel grate decks or concrete-filled open steel grate decks have been used when a lighter deck was needed.

Secondary members fulfill important functions such as forming a lateral bracing system in a horizontal plane at the upper and lower chord levels. Originally designed for resisting lateral wind-induced loading, secondary bracing also plays an important role in resisting seismic loading in high seismic areas. In the vertical plane at the panel points, secondary cross bracing acts both to space the trusses apart to match the floor beam spacing, and to perform the important function of stiffening the rectangular cross-section of the truss against sway distortions. Tertiary bracing members fulfill the important functions of bracing long compression members within the plane of the trusses and also brace secondary compression members against buckling.

Through trusses have stiff sway-frames at each end of the truss, called *portals* or *portal frames*. The function of the portal frames is to carry wind loads and seismic loads, from the lateral bracing system in the plane of the upper chord, down to the bridge bearings that support the truss at each end.

Trusses are usually supported on structural steel bearings, sometimes called "shoes," that carry vertical gravity loads, horizontal wind loads, and seismic loads to the substructure. Rockers and nested rollers are usually used for bearings that are designed to move in order to accommodate thermal movements. Large-diameter steel pins are commonly used to attach the trusses to support bearings that are fixed to the substructure against longitudinal movement. The pins will allow the trusses to rotate under live load deflections and to carry the longitudinal forces from traffic, wind, or earthquakes to the fixed bearings. Large steel pins can also be used to connect the trusses to sliding bearings that are free to move longitudinally to accommodate thermal expansion and contraction.

Often the concrete decks of truss bridges have joints supported by the floor beams every three or four panel points. The function of these joints is both to reduce the structural participation of the concrete deck through composite action with the truss chords under live load flexure, and to accommodate small differential temperature expansion and contraction movements. They do not aid in resisting wind and seismic lateral loads.

The concrete decks of trusses are connected either to roadway slabs at each abutment, or to adjacent trusses in multi-span truss arrangements by thermal joints that accommodate temperature movements. These joints are often called expansion joints, although they also accommodate thermal contraction movements. These thermal joints allow vehicles to pass smoothly from the truss concrete deck to the concrete slab at each approach or to the concrete deck of the adjacent truss. Finger joints were used predominately for these connections because of their record of low maintenance, but other types such as modular joints have also been used since the 1960s.

1.3.2 DIFFERENCES BETWEEN STEEL TRUSS BRIDGES AND STEEL ARCH BRIDGES

The distinguishing feature between a steel arch bridge and a steel truss bridge is the diagonal thrust force that the arch bridge develops at each end of the arch rib, under gravity loads. A truss bridge develops only vertical forces at the supports under gravity loads. Some long-span arch bridges use a curved truss as the arch rib. These are properly called trussed arches although it is more common to call them arch trusses.

In a true arch, the diagonal thrust from the arch-action is taken into the supporting abutments at each end of the arch. In a tied-arch, the horizontal component of the diagonal thrust is taken into a tension tie that spans between the two ends of the arch; the tension tie usually carries the roadway. Similar to the truss, the vertical component of the arch-thrust in a tied-arch produces only vertical forces at the supports under gravity loads.

These *Guidelines* cover the seismic retrofitting of steel highway bridge trusses only. An experienced bridge engineer can use the principles presented herein for the general seismic retrofitting of arch bridges; however, the large variation in the arch-thrust force during seismic excitation must be specifically evaluated and accommodated.

1.3.3 STRUCTURAL CLASSIFICATIONS OF STEEL HIGHWAY TRUSS BRIDGES

Most steel truss highway bridges are structurally classified by the position of their deck within the truss cross section, the number of spans, the shape of the truss, their method of construction, the location of internal hinges in multiple-span trusses, or by a combination of these classifications. Some examples are:

Named for Position of Deck within Cross Section
- Through Truss
- Half-through Truss
- Deck Truss
- Pony Truss

Named for Truss Shape
- Camel-back Truss
- Bow-String Truss

Named for Number of Spans
- Simple Span Truss
- Continuous Truss
- Gerber Truss (internal hinges in alternate spans)

Named for Method of Construction
- Cantilever Truss (constructed by cantilevering arms from each side span)
- Suspended Span (simple span truss suspended between the two cantilever arms to complete the center span of a cantilever truss)

Combination of Names
- Cantilever Through Truss
- Gerber Deck Truss

1.3.4 TYPES OF SIMPLE-SPAN STEEL HIGHWAY TRUSS BRIDGES

The simple-span truss, viewed in elevation, forms identifiable patterns that have been developed over the 125 years that metal trusses have been used, as shown in Figure 1-1. Some are named for their designer, such as the Howe truss, with all sloped diagonals acting in compression, or the Pratt Truss, with all diagonals sloped in the opposite direction and acting in tension. Some trusses are named for the shape of the chords, such as the curved, upper chord Camel Back truss (sloping portal members at each end), or the Bowstring truss, which has a sloping upper chord without portals. Some are named for the configuration of the diagonals, such as the K-truss in which the diagonals form the letter "K."

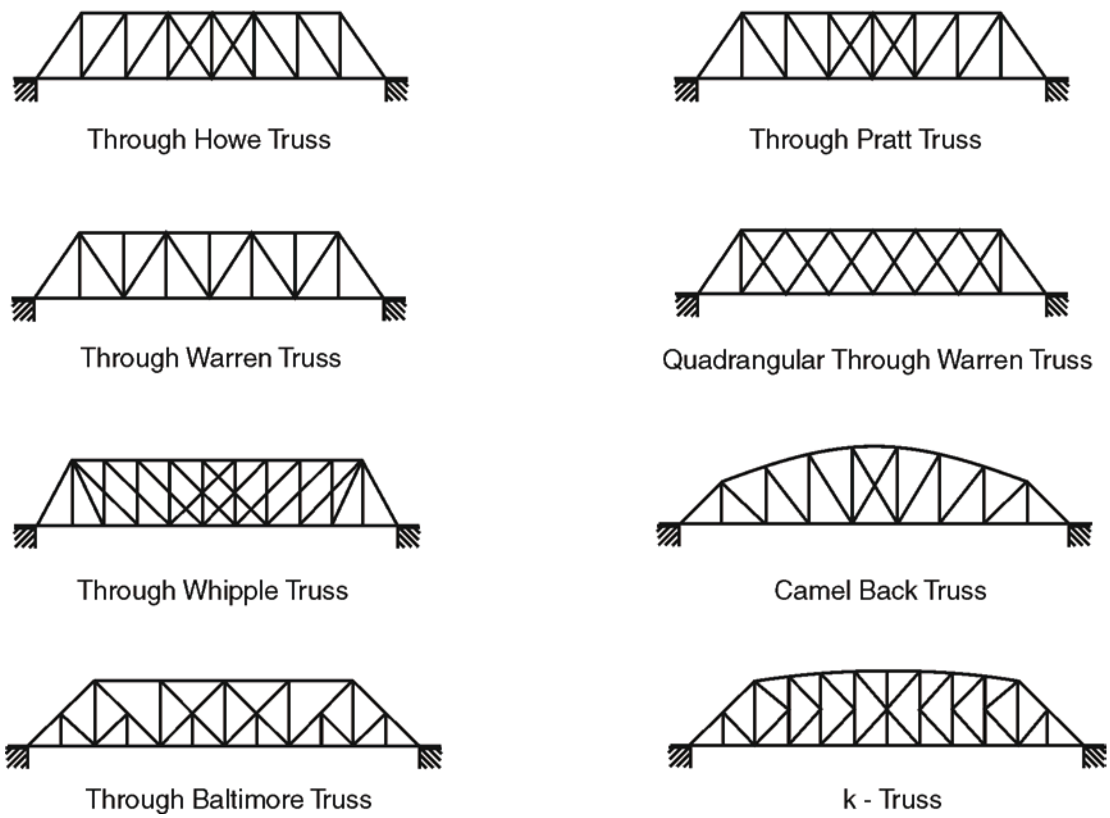

Figure 1-1. Examples of Simple Span Steel Truss Highway Bridges

The parallel-chord Warren truss has adjacent diagonals sloped in opposite directions such that one diagonal carries compression and the other diagonal carries tension. Warren trusses with verticals have been predominately used in the 20th century, but in the last two decades the Warren truss without verticals has emerged as a more aesthetically pleasing configuration, especially with parallel chords.

Trusses are classified by the position of the deck within the cross section of the bridge, as shown in Figure 1-2. If the deck is in the plane of the upper chords, the bridge is referred to as a deck truss; when the deck is in the plane of the lower chord, the bridge is referred to as a through truss. If the deck is between the top and bottom chords, the bridge is called a half-through truss. A truss with the deck in the plane of the lower chord, without a lateral bracing system in the plane of the upper chord, is called a Pony truss. Pony and Bowstring trusses were used in the early 20th century for comparatively short spans in the 130 ft or less range, but they have not been used in recent years because steel girders are typically more economical.

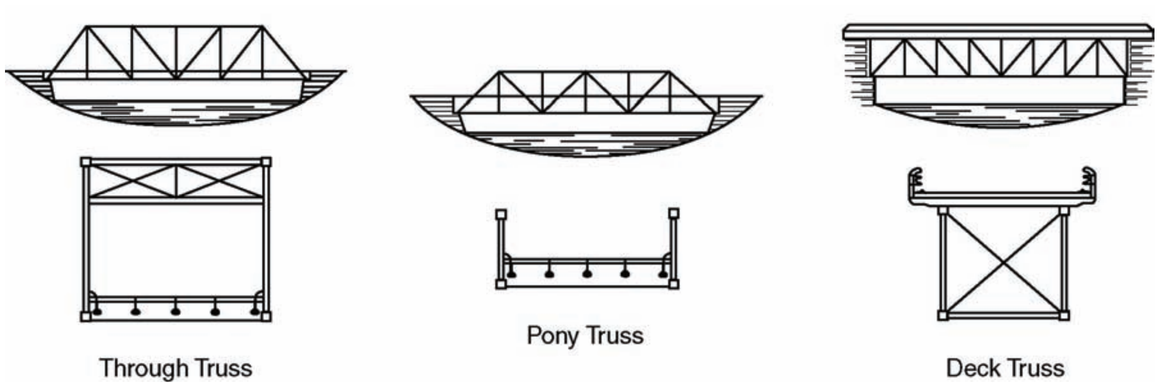

Figure 1-2. Classification of Truss Highway Bridges

1.3.5 TYPES OF CONTINUOUS AND MULTIPLE SPAN TRUSSES

A continuous truss is, as its name implies, continuous over each of the piers that support the truss as shown in Figure 1-3. Continuous trusses are generally limited to two, three, or four spans. Multiple span trusses are usually a series of simple spans linked together to form a long bridge. The position of the deck, as denoted by the terms "through," "deck," or "pony," is usually placed after the truss classification term, i.e., Simple Span Deck Truss or Cantilever Through Truss.

A cantilever truss refers to the construction method in which the truss is erected by balanced-cantilevering out from each side of the two center towers. Often the side spans are temporarily supported for stability by falsework near the center of the spans. When the side-span cantilevering arms reach the side towers (or anchor tower), the side spans then act as continuous trusses. When the two center cantilever arms join at the center, the two arms remain as cantilevers connected continuously over the center towers to the two side-spans. Often a suspended span is used to shorten the cantilever arms. The suspended span is usually connected to the tips of the cantilever arms by eye bars or by hanger plates and functions as a simple span.

A Gerber Truss arrangement has a continuous span with a cantilever arm reaching out into each of the two adjacent spans. A suspended span, often called a drop-in span, is connected to each cantilevered arm. This arrangement of alternating continuous spans and drop-in spans provides all the advantages of a continuous truss but accommodates thermal movements without flexing the support towers or piers. The Gerber system, in effect, allows many spans to be linked together with the suspended spans relieving the build-up of temperature stresses. The number of spans for fully- continuous spans is usually limited by the accumulation of temperature stresses or by the limitation of large roller or rocker bearings.

In the U.S., simple-span highway trusses are rarely used for spans exceeding 400 ft, but several multiple-span bridges have been constructed in the 600 ft range. For very long multiple-span bridges, truss spans are joined by using Gerber framing. Three-span continuous trusses have

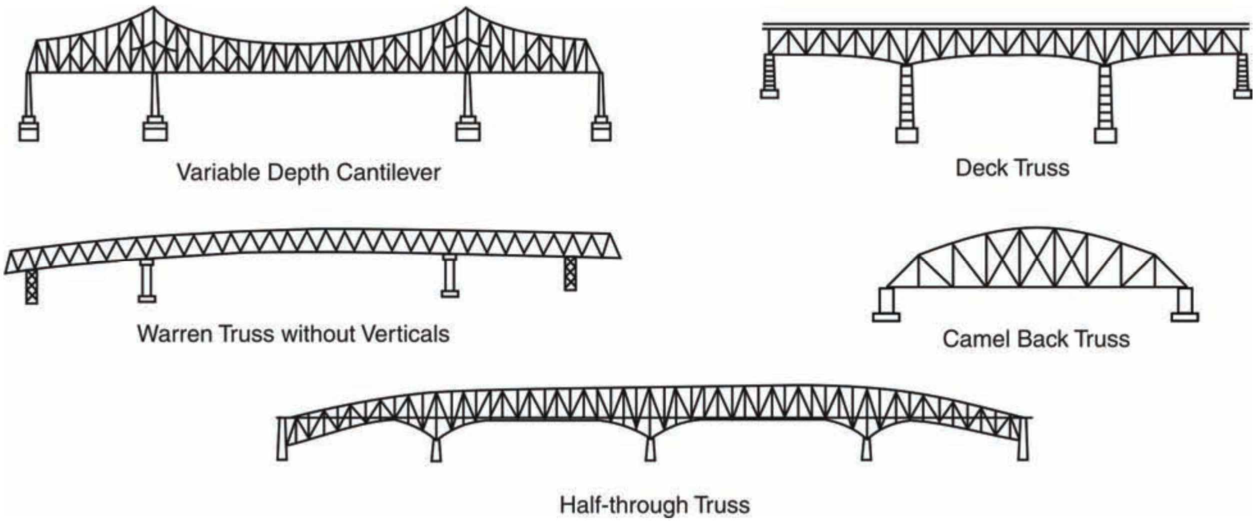

Figure 1-3. Examples of Continuous Truss Highway Bridges

been constructed in the intermediate range with spans up to about 750 ft. For spans greater than about 750 ft, the cantilever truss type is generally used with both deck and through truss configurations. For very long spans, up to about 1600 ft, the through-truss is used exclusively.

Through trusses are usually configured as cantilever spans with two back spans; some use Gerber framing and some are fully continuous over several spans. The three-span cantilever configuration usually has a suspended span in the center of the main span; occasionally the suspended span is not used and the center span is hinged at the center, or the cantilever truss can be continuous over the three spans. The term “cantilever” refers to the structural configuration in the erection stage of construction, when the center span is cantilevered out from the side spans. For spans longer than about 500 ft, the structural depth of the cantilever trusses is often increased at the two center supports, giving these bridges their characteristic up-swooping curves at the towers.

1.4 STRUCTURAL CHARACTERISTICS OF TRUSS BRIDGES

1.4.1 THROUGH TRUSSES

Long span through trusses generally use a Warren truss pattern with verticals, although other truss patterns have been used. As presented in Sections 1.3.3 and 1.3.4, the through truss type is characterized by the placing of the trusses and bracing system above the roadway, and the vehicles drive through the array of steel. The through truss type is often used for long truss spans over navigation channels because the structural depth of the deck system from the top of the deck to the lower chord clearance line is smaller than that of deck truss types. The height of the main span deck is thereby decreased, reducing the height and length of the approach spans; these reductions reduce the initial cost of the crossing.

The key elements of lateral force resisting elements in through trusses are the horizontal lateral bracing systems placed within the planes of the upper and lower chords. The upper lateral bracing system usually carries forces by truss action to the ends of the span, or toward the towers of the cantilevers or toward the piers of the continuous spans. At these points, the forces are transmitted down to the lower lateral bracing system through portal frames placed at the end of the suspended spans and at the end of the cantilevers, or by portal frames placed in the plane of the two diagonals that frame into the truss supports. The portal frames have cross bracing only at the top of the frames because the lower portion of the frames must be open for the passage of vehicles.

The lower lateral bracing system in a suspended span carries forces by horizontal truss action to each end of the suspended span and transmits these forces together with the forces from the end portals through shear locks (sometimes called wind tongues), to the cantilever arms. The lower lateral bracing systems in the cantilever arms carry forces by truss action to the truss support bearings. In continuous spans, there are no suspended spans or shear locks; hence the forces are carried directly to the support bearings by the lower lateral bracing systems.

Typically, these lateral bracing systems and shear locks were designed for equivalent static wind forces because wind loads usually controlled the design over seismic loads. Seismic codes were changed late in the 20th century to reflect the much larger demands imposed on bridges by the dynamic actions of earthquakes. The codes today specify much higher seismic loads that exceed wind loads in all but the lowest seismic area.

The mass of non-composite concrete decks and the deck-supporting systems contribute inertia demands on the lateral bracing system, but contribute little lateral force resisting capacity.

1.4.2 DECK AND DOUBLE-DECK TRUSSES

The deck type truss is characterized by placing trusses and bracing systems below the roadway so that vehicles drive on a deck placed in the plane of the top chords. As noted before, most long-span deck trusses use a Warren truss pattern with verticals, although other truss patterns have been used. The deck type truss is rarely used for long spans over navigation channels because the structural depth of the deck system from the top of the deck to the lower chord clearance line is much greater than that of the through truss type. The deck type truss increases the elevation of the roadway, which increases the height and length of the approach spans, and these raise the cost of the crossing.

Occasionally, double-deck trusses were used to increase the capacity of a crossing. For short and intermediate span lengths, double deck trusses usually have their decks in the plane of the top and bottom chords. For longer spans, the deck truss becomes so deep that the upper deck is placed at an intermediate level within the truss depth to lower the cost of the approach spans.

Deck or double deck trusses can also be configured as a cantilever span, as Gerber framing, and as a fully continuous truss over several spans. The cantilever configuration usually has a suspended span in the center of the main span; sometimes it may have a single hinge in the center of the main span or it may be continuous over the three spans. For spans longer than about 500 ft, the structural depth of the cantilever trusses is often increased below the deck at the two center supports, giving these bridges their characteristic down-swooping curves at the towers.

The Gerber framing, in effect, uses the cantilever with a suspended span configuration in every other span. This allows many spans to be linked together with the suspended spans, relieving the build-up of temperature stresses. The number of fully continuous spans is usually limited by the build-up of temperature stresses or by the rotational limitation of large roller or rocker bearings.

Similar to through trusses, the key elements of the lateral force resisting elements in deck and double deck trusses are the horizontal lateral bracing systems placed within the plane of the upper and lower chords. The lateral bracing systems in the suspended span carry forces to each end by horizontal truss action and transmit these forces and the forces from the end portals, through shear locks, to the cantilever arms. The lateral bracing systems in the cantilever arms carry forces by horizontal truss action to the portal frames placed at the towers, and then down to the truss support bearings. In continuous spans, there are no suspended spans or shear locks so hence the forces are carried directly to the portal frames and then to the support bearings.

In double deck trusses, the portal frames have cross bracing only at the top of the frames because the lower parts are open for passage of vehicles on the lower deck. For deck trusses the portal frames may have cross bracing for the full depth, because vehicles travel on the deck above.

Some single deck trusses are supported by bearings placed under the top chords. In this type of truss, the structural action of the portal framing is reversed and the lateral forces are carried to the support-bearings placed under the upper chords.

As noted in section 1.4.1, the lateral bracing systems and shear locks of through type trusses and deck and double-deck type trusses were designed for static wind forces because at the time of their original design, wind forces usually controlled over seismic forces. Similar to the through trusses, the mass of non-composite concrete decks and their deck supporting systems contributes inertia demands on the lateral bracing system, but contributes little lateral force resisting capacity.

For deck trusses supported by bearings under the lower chord, the mass of the non-composite concrete deck, acting through a moment arm equal to the depth of the truss, develops large inertia force demands on the lateral bracing systems. For double deck trusses, the lower deck adds additional lateral force demands.

For double deck through trusses, in addition to the effects noted above, the upper non-composite concrete deck delivers lateral forces to the vertical truss members that support the deck, inducing bending moments in these members.

CHAPTER 2: RETROFITTING PHILOSOPHY AND PROCESS

2.1 GENERAL

The seismic retrofitting philosophy developed for these *Guidelines* is based on a set of performance expectations listed in the AASHTO *Standard Bridge Specifications for Highway Bridges* [AASHTO 2002], and summarized in the *Bridge Retrofitting Manual:*

- Small to moderate earthquakes should be resisted within the elastic range without significant damage.
- Realistic seismic ground motion intensities and forces should be used in the design procedures.
- Exposure to shaking from large earthquakes should not cause collapse of all or part of the bridge. Where possible, damage that does occur should be readily detectable and accessible for inspection and repair.

A set of basic concepts for seismic design of new bridges, derived from this philosophy and the AASHTO set of performance expectations, is summarized in the *Bridge Retrofitting Manual* as*:*

- Hazard (risk) to life is to be minimized.
- Bridges may undergo damage but should have a low probability of collapse.
- The function of Essential Bridges should be maintained.
- Ground motions used in design should have a low probability of being exceeded in the normal life of the bridge.

All of this background development for the seismic design of new bridges is applicable to the seismic retrofit design of existing truss bridges. The concepts presented in the *Bridge Retrofitting Manual*, and in these *Guidelines* for retrofitting truss bridges, are based on the following retrofit philosophy:

- Provisions should minimize loss of life and serious injury to the traveling public from unacceptable bridge performance.
- Provisions should be applicable to all regions of the United States for high, moderate, and low seismicity.
- Provisions should not restrict retrofit designers from using new and innovative concepts and design approaches.

The essential concepts that govern development of seismic retrofitting of truss bridges are:

1. Enhancing the ductile deformation capacity of the substructure and the superstructure to ensure that both structural systems are capable of sustaining large lateral displacements without reaching a collapse limit state.
2. Reducing seismic inertial loading on the superstructure and substructure by using Response Modification Devices (RMDs), such as isolation bearings and energy-dissipation mechanisms, load-limiting methods such as capacity-protected designs, or load transfer to more robust systems by seismic lock-up devices.

3. Strengthening of the support bearings, members, and connections to ensure robust seismic inertial load transfer mechanisms within the substructure and the superstructure systems.
4. Minimizing the seismic inertial loads induced in the superstructure by reducing the dead load of the concrete deck by using a lighter material to reduce or avoid excessive strengthening of the substructure and the superstructure systems, as described in 3.
5. Improving superstructure redundancy.
6. Combinations of the concepts listed above.

These seismic retrofitting concepts essentially amount to introducing displacement ductility capacity in the structural systems by means of load-limiting mechanisms, establishing robust load paths, using energy-dissipation or load transfer mechanisms, or by reducing the dead load.

These concepts and related issues are discussed in these *Guidelines* which include: truss classifications, performance levels, earthquake levels, bridge importance and anticipated service life, seismic analysis, and retrofit design.

2.2 SEISMIC CLASSIFICATION OF TRUSS BRIDGES

The seismic retrofitting process for truss bridges starts with classifying truss bridges as either Seismically “Standard” Truss Bridges or Seismically “Complex” Truss Bridges, as defined in the following sections.

2.2.1 SEISMICALLY CLASSIFYING TRUSS BRIDGES

In the *AASHTO Standard Specifications for Highway Bridges* for both Allowable Stress Design (ASD) and for Load Factor Design (LFD), the introduction states, “[these specifications] apply to ordinary highway bridges and supplemental specifications may be required for unusual types and for bridges with spans longer than 500 ft.” This statement does not appear in the recently published AASHTO Load and Resistance Factor Design (LRFD) specifications (AASHTO 2004).

Thus the AASHTO ASD and LFD bridge design specifications recognize that for new bridge design projects there are both long-span and unusual types of bridges that are not covered within the AASHTO specifications. The design of “unusual” bridges requires special supplemental design specifications.

Similarly, for the seismic retrofit of “unusual” steel highway truss bridges, special supplemental retrofit design specifications are required. These *Guidelines* serve as the special supplemental retrofit design specifications in the form of guidelines. The category of “unusual” steel highway truss bridges are defined in these *Guidelines* as Seismically Complex Truss Bridges.

The *Bridge Retrofitting Manual* was developed to cover seismic retrofitting of “ordinary AASHTO bridges,” as defined by the AASHTO bridge design specifications, and it divides “ordinary AASHTO bridges” into two additional classifications by their seismic performance as “essential bridges” and “standard bridges:”

- *Essential* bridges are those that are expected to function by carrying traffic after an earthquake.
- All other bridges are classified as *standard* bridges. The *Bridge Retrofitting Manual* does not cover "Unusual Bridge" types.

These *Guidelines* extend the requirements of the *Bridge Retrofitting Manual* to include specific requirements for the seismic retrofitting of highway steel truss bridges, and recommend the following two classifications of bridge trusses by their seismic performance:

- Seismically standard trusses (ordinary trusses, according to AASHTO).
- Seismically complex trusses (unusual trusses, according to AASHTO).

Essential bridges that are truss bridges are classified as Seismically Complex (SC) trusses in these *Guidelines*, as defined in Section 2.2.2. Seismically Standard (SS) trusses are defined as all other trusses that are not classified as SC trusses.

Standard bridges, defined in the *Bridge Retrofitting Manual*, that are trusses can be retrofitted according to the requirements of these *Guidelines*, or at the option of the bridge owner or the retrofit bridge engineer, to the requirements of *Bridge Retrofitting Manual*. SC trusses should be retrofitted according to the requirements of these *Guidelines.*

2.2.2 CLASSIFICATION OF SEISMICALLY COMPLEX TRUSS BRIDGES

Damage to bridges from recent earthquakes and recent seismic investigations and analyses of bridge structures have demonstrated that, in addition to span length, there are also special configurations of bridge trusses that behave, under seismic excitation, in a complex manner. These special configurations of bridge trusses, are classified as SC trusses and should be seismically retrofitted in conformance with the specific requirements of these *Guidelines*.

An SC truss bridge is defined as meeting one or more of the truss bridge attributes and configurations listed below:

- Single truss span exceeding 500 ft.
- Deck trusses or double-deck trusses.
- A series of truss spans creating a long bridge with multiple supports exceeding seven spans.
- A series of truss spans creating a long bridge that have a total length exceeding 1600 ft.
- Truss bridges that have unusual geometry or alignment, skews exceeding 20 degrees, or unusual mass or stiffness distribution.
- Movable bridges that swing, lift, or tilt open with truss superstructures.
- Trusses supported on tall/slender concrete pier shafts.
- Trusses supported on braced-steel towers.
- Trusses supported on unreinforced masonry piers.
- Trusses supported on concrete piers or braced-steel towers with heights varying more than 25% between the highest and lowest pier or tower.
- Historic trusses listed on, or eligible to be listed on, a State or a Federal List of Historic Places.

The structural response aspects, under seismic loading, of these special truss bridge configurations that make them “seismically complex” are as follows:

- The large mass of a long-span truss can develop large seismic forces; displacements or instability in truss members and connections within the portal frames, at the support bearings, and within the lateral and sway-bracing systems.
- Deck trusses or trusses with double decks can generate, within the plane of the upper chords, large lateral forces that must be carried by the sway bracing and by the portal frames. High seismic forces can distort or buckle the sway and portal frames.
- Seismic waves passing within the earth, along the length of a multi-span truss bridge, can cause the supporting piers to move out-of-phase with one another, causing the piers to rotate separately and to move together and apart with the passage of the seismic wave. The pier movements can unseat spans, causing the trusses to drop off the piers.
- Unusual bridge alignment, mass, or stiffness distribution of the superstructure or the sub-structure, under seismic excitation, can subject the structure to large inertia forces; large displacements; drifts of tall columns; and high shear and moment demands on short columns. Truss bridges that have insufficient sway bracing, or flexible lateral bracing, will have greater displacement and ductility demands than standard truss bridges have.
- The common characteristic of movable bridges is their articulation of swing, lift, or tilt (bascule bridges), with the latter two types having heavy counterweights. The articulation and the counterweights can make movable bridges seismically vulnerable to bearing failure or to displacement at their supports. The counterweights can significantly increase inertial forces in the superstructure. Often these bridge types employ trusses as the primary load-carrying unit. The recommendations in these *Guidelines* may be used for the seismic retrofitting of the trusses of movable bridges, but these *Guidelines* do not cover the seismic retrofit of the movable portions of these types of bridges.
- Trusses supported on tall concrete pier shafts can develop large seismic displacements and “P-Delta” effects from seismic flexure of the shafts, foundation rotations, and seismic wave passage effects.
- Braced steel towers function as vertical trusses connected to the concrete piers, generally with anchor bolts, which restrict uplift. They are usually designed to carry lateral loads from wind forces. Under seismic excitation, bracing members can buckle, their connections can fail, the leg members of the tower can buckle, or the anchor bolts can fail in tension. If the bracing, connections and legs hold together and do not buckle in a seismic event, the tops of the towers may develop large seismic rotation and displacement of the bearings supporting the truss.
- Short piers or towers within a group of several taller piers or towers will attract disproportionately large seismic forces.
- Trusses, supported on unreinforced masonry piers, are vulnerable to severe damage or collapse from failure of the unreinforced masonry pier under seismic excitation.
- Historic trusses also require special consideration for seismic retrofitting to ensure that all applicable rules and regulations are followed to preserve, as much as is possible, the historic features of the original structure.

2.2.3 IDENTIFICATION OF SEISMICALLY COMPLEX TRUSS BRIDGES

In addition to the physical attributes presented in Section 2.2.2, truss bridges that meet the definition of Essential Bridges in the *Bridge Retrofitting Manual* should be classified as SC trusses.

Owners of SC steel truss highway bridges, with the help of the bridge engineer, may be able to identify these types because they are unusual and stand out as long span, multiple-span, or trusses supported on tall piers or towers. For a large inventory of bridges, a simple program can be written to search the inventory for the characteristics of complex bridges. The National Bridge Inventory listings provide a database of information for determining truss superstructure span lengths, pier heights, substructure types, seismic hazard zone, traffic volumes, and other important information about seismically complex bridges. Figure 2-1 shows a flow chart for identifying SC and SS truss bridges.

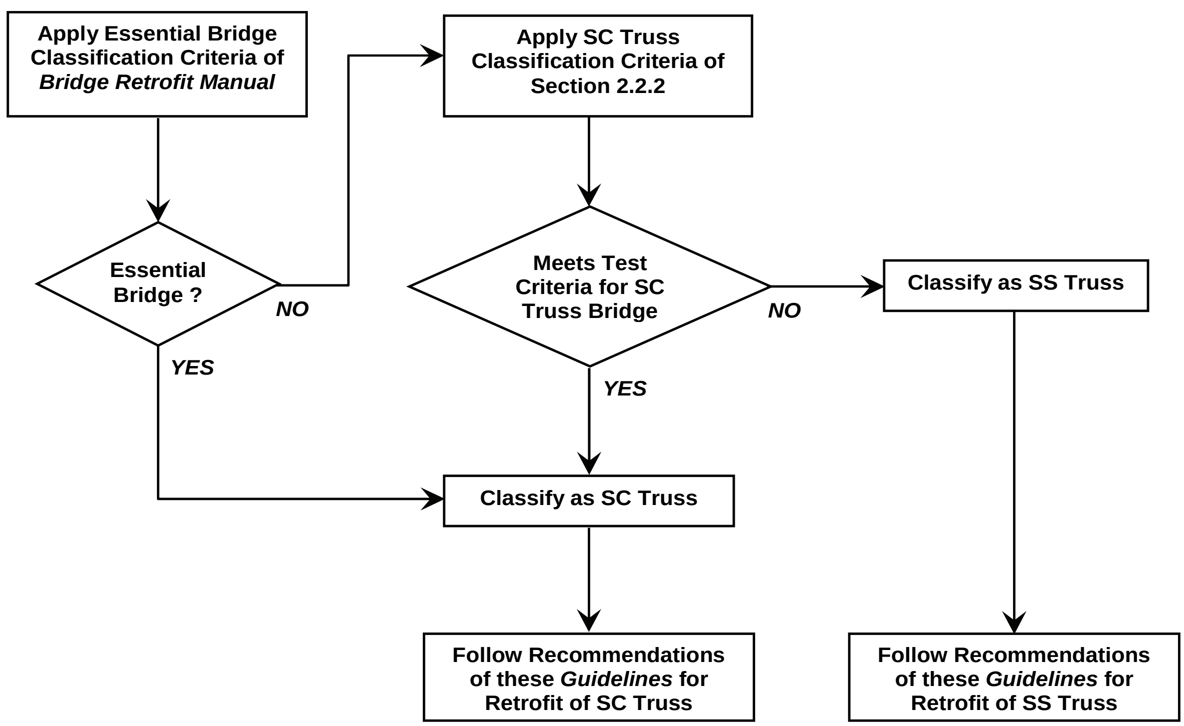

Figure 2-1. Seismic Classification Process for Steel Truss Highway Bridges

2.3 SEISMIC PERFORMANCE LEVELS

This section presents three seismic performance-based levels for the purposes of these *Guidelines*. The three performance objectives for the evaluation of truss bridges are specified as:

(1) *Life Safety*: Performance Level 1 (PL1).
(2) *Operational*: Performance Level 2 (PL2).
(3) *Fully Operational*: Performance Level 3 (PL3).

The descriptions of the performance levels in terms of permissible damage are given in Table 2-1. The applicability and selection of the appropriate performance level is a function of the seismic hazard level at the bridge site, the bridge importance to traffic flow or to the community, and the *Anticipated Service Life* (ASL) of the bridge. The seismic hazard levels are presented in Section 2.4. Bridge importance is presented in Section 2.5. Anticipated Service Life is presented in Section 2.6. Section 2.7 discusses the method of selecting the performance levels for the design of the seismic retrofit

Table 2-1. Seismic Performance Levels

Performance Level	Damage Level
1. PL1: Life Safety	Sustained damage is *significant* after a large earthquake and function is significantly disrupted, but life safety is assured. The bridge may need to be replaced after a large earthquake or a fault surface rupture.
2. PL2: Operational	Sustained damage is *minimal or repairable* and limited function for emergency vehicles is available after inspection and clearance of debris. The damage is repairable, with or without restrictions on traffic flow during repair.
3. PL3: Fully Operational	Sustained damage is negligible and full function for all traffic is available after inspection and clearance of debris. No repairs are required.

The terms *minimal, repairable,* and *significant* damage are used in the above performance criteria. These terms are explained below and are developed mathematically in Chapter 6:

- *Minimal* damage includes minor inelastic response in steel members and narrow cracking in concrete. Permanent deformations are not apparent and repairs can be made under non-emergency conditions, with the possible exception of superstructure expansion joints. These joints may need removal and either temporary replacement or bridging-over with temporary steel plates immediately after the seismic event.
- *Repairable* damage includes inelastic response. Members are damaged enough to require repairing but are still capable of carrying loads temporarily, with reduced traffic, until repairs can be made. Apparent damage constitutes small permanent offsets, small permanent deformations in members, closely-spaced or wide cracking of concrete.
- *Significant* damage includes large permanent offsets, buckled steel members, large concrete cracks, major spalling of concrete, and yielded reinforcement. Any of these damage states may require bridge closure to repair. Partial or complete replacement of columns may be required. Foundations are not damaged except in the event of large lateral flows due to liquefaction, in which case inelastic deformation in piles is acceptable.

The *Life Safety Level*, PL1, is the most basic level set to prevent bridge collapse and loss of life, although the bridge may need extensive repair or replacement after the earthquake. For bridges on lifeline routes, the performance level must be set to provide *Fully Operational*, PL3, soon after a seismic event, including the maximum expected event. Retrofit measures for these bridges must be designed to provide essentially elastic behavior during a seismic event. In between these two seismic levels is the *Operational*, PL2, level for bridges that are not required to perform at the *Fully Operational* level but are bridges the owner is willing to retrofit after an earthquake.

For any bridge that undergoes seismic retrofitting, obviously the minimum performance level expected is “No Collapse” after an “Upper Level” earthquake as defined in Section 2.4. The traveling public expects and demands that bridges not collapse during earthquakes. Thus, the trade-off in performance and cost must be made between the following two extremes: (1) no collapse as a minimum standard; and (2) no collapse as a high standard of retrofit, depending on what can be afforded to fit the traffic-service levels provided by the bridge. High traffic volume bridges serving as vital commerce links as well as important or historically valuable bridges will require a higher standard of retrofit. These categories of bridges will justify the high cost required for the higher standard of retrofit measures associated with the needed high performance level.

2.4 SEISMIC HAZARD LEVELS

Seismic *Hazard Levels* for the design of the seismic retrofit consist of two levels of earthquake ground motions with different return periods and a level associated with site geological hazards.

The *Lower Level* (LL) earthquake ground motion is the level of ground motion that is likely to occur within the lifetime of the bridge; i.e., it represents a relatively small but likely seismic event. In the *Bridge Retrofitting Manual* and in these *Guidelines*, the LL earthquake ground motion has a probability of exceedance of 50 percent during an assumed 75 year service life of the bridge, which corresponds to a mean return period for the LL ground motion of approximately 100 years.

The *Upper Level* (UL) earthquake ground motion represents a relatively large and rare event. In the *Bridge Retrofitting Manual* and in these *Guidelines*, the UL earthquake ground motion has a probability of exceedance of 7 percent in 75 years, which corresponds to a mean return period of approximately 1,000 years.

A third hazard characterization for specific sites, designated as *Site Hazard*, has been added to categorize sites with known active seismic faults, unstable geological slopes, or liquefiable soils. Occasionally, bridge alignments cross over known seismic faults for which seismologists can usually estimate surface offset movements both vertically and horizontally. Bridges can also cross sites with unstable slopes that may slide in an earthquake or are underlain with liquefiable soils. In most cases, geotechnical engineers can predict the potential extent, depth, and movement of soil slides and amount of softening and movement of liquefiable soil. The seismic retrofit designer must analyze the estimated extreme-event movement according to the AASHTO extreme event loading criteria. The seismic retrofit designer must provide retrofit measures to prevent collapse of the structure.

As examples, the San Pedro Suspension Bridge near Los Angeles Harbor crosses a known fault. Seismic analysis determined that the center span of 1500 ft was long enough to accommodate the estimated surface fault displacements without collapse. The 275 ft steel girder approach spans of the San Diego-Coronado Bay Bridge also cross a known fault in San Diego Bay. Analysis determined that isolation bearings, installed as part of the seismic retrofit, were capable of remaining stable under the extreme distortion that would be produced if the fault ruptures. If the fault ruptures, the bearings will be distorted, the girders will be displaced, and the bridge must be closed for repairs and for isolation bearing replacement. However, the bridge will not collapse and life-safety will be preserved.

Similar to these examples cited, long-span trusses crossing known seismic fault lines should be designed to the requirements of PL1 to preserve life safety.

2.5 BRIDGE IMPORTANCE

The *Bridge Retrofitting Manual* discusses bridge importance as it relates to the selection process for the seismic retrofitting of bridges, where *essential* bridges, which must function after an earthquake, are defined to be more important than *standard* bridges and therefore require a higher degree of engineering for seismic retrofitting. For this reason all truss bridge types that meet the definition of *essential* bridges in the *Bridge Retrofitting Manual* should also be classified in these *Guidelines* as SC trusses.

Similarly, all truss bridge types that meet the definition of *standard* bridges in the *Bridge Retrofitting Manual* should also be classified in these *Guidelines* as SS trusses.

2.6 SERVICE LIFE CATEGORIES (SLC)

An important factor in deciding the extent to which an SC truss should be seismically retrofitted is the Anticipated Service Life (ASL) of the structure. Seismic retrofitting of an SC truss with a short service life may be difficult to justify in view of the low likelihood that the design earthquake will occur during the remaining service life of the structure. On the other hand, a truss that is almost new, or one that is being considered for non-seismic rehabilitation to extend its service life, should be seismically retrofitted because of its extended anticipated service life.

SC trusses may have ASLs much longer than the 75 years assumed for new ordinary AASHTO truss bridges. Although these SC truss bridges may have been designed according to an AASHTO bridge design specification that implies a design life of 75 years, many have, in fact, a much longer service life. Trusses that meet the definitions of SC trusses are generally more costly to design and construct, which leads to a large initial investment in the facility. This investment in design and construction can provide for a longer life.

Additionally, several other issues exist: (1) a replacement bridge may be prohibitively costly to construct; (2) the prime location of an existing bridge may force a new and costly alignment for a new bridge; (3) an existing bridge may be a structural icon to a community that wishes to retain its bridge for symbolic reasons; (4) a well-maintained steel bridge can have a very long service life; and (5) even a neglected steel bridge can be rehabilitated to extend its service life.

For these reasons, the ASL of these SC truss bridges needs to be carefully evaluated. The basic question is: Which one of the following options is the best economic solution?

1. Replace the truss bridge with a new bridge designed to current seismic design requirements.
2. Rehabilitate the existing structure to extend its ASL and, at the same time, seismically retrofit it to the requirements of these *Guidelines.*
3. Seismically retrofit the bridge to the requirements of these *Guidelines* if the truss has been well-maintained and is in excellent condition.

Estimating the ASL remaining in a truss is not an exact science and depends on many factors such as age, structural condition, fatigue life, specifications used for design, and capacity to handle current and future traffic. Nevertheless, estimates can be made, at least within broad ranges, for assigning an anticipated service life to a truss. Three Service Life Categories (SLC) are defined in Table 2-2 as functions of three anticipated service lives.

SC trusses in category SLC1 are considered to be near the end of their service life and extensive seismic retrofitting may not be economically justified. Using the criteria presented herein, these SC trusses may be retrofitted to PL1; at the same time, they should be evaluated for non-seismic rehabilitation to increase their service life. Bridges in category SLC3 are almost new, or are in excellent condition because of good maintenance; seismically retrofitting these to the standard of a new design may be justified. Those in category SLC2 fall between these two extremes and would be seismically retrofitted to a lesser standard than SLC3 due to cost and other factors.

Table 2-2. Service Life Categories

Service Life Category (SLC)	Anticipated Service Life (ASL)
Not applicable	Replacement is scheduled within 5 years
SLC1	5-25 years
SLC2	26-50 years
SLC3	>50 years

Many bridges are rehabilitated when they are nearing the end of their service life by the repair of structural deficiencies that have accumulated over time, and also to improve their safety for the bridge user. Therefore a bridge classified in category SLC1 or SLC2, after structural rehabilitation, can gain additional service life of many years, and gain a higher level SLC category. The bridge should be re-evaluated for seismic retrofitting at the same time as the planning for non-seismic rehabilitation. In this way, retrofitting measures can be implemented at the same time as the non-seismic deficiencies are corrected. By taking advantage of the contractor being on-site, savings can be achieved if seismic retrofitting is undertaken at the same time as non-seismic rehabilitation work.

2.7 SELECTION OF PERFORMANCE LEVELS

Based on the seismic *Performance Levels* defined in Table 2-1, minimum performance levels for SC trusses are given in Table 2-3 according to earthquake *Hazard Levels*, as defined in Section 2.4, and *Service Life Category*, as defined in Section 2.6.

If retrofitting to these performance levels cannot be justified economically, the owner may choose a lower retrofit level. The owner may also choose a higher level than that recommended here for certain classes of bridges. For example, a bridge on the National Highway Network which is critically important to the operation of national or regional transportation routes, a life line bridge in a metropolitan area, or a truss bridge classified as an *essential* bridge according to the *Bridge Retrofitting Manual*.

Table 2-3. Minimum Performance Levels For Seismically Complex Trusses

	Service Life Category		
Seismic Hazard Levels	**SLC1**	**SLC2**	**SLC3**
Lower Level Earthquake (LL)	PL3	PL3	PL3
Upper Level Earthquake (UL)	PL1	PL1	PL2
Site Hazard	PL1	PL1	PL1

2.8 SEISMIC RETROFIT STRATEGIES, APPROACHES AND MEASURES

The objective of retrofitting a truss bridge is to ensure that it will meet the selected performance levels, presented in Section 2.7, if it is subjected to either one of the two levels of earthquakes or a site hazard as defined in Section 2.4. If a bridge is evaluated and found to have seismic deficiencies, it should be retrofitted to meet the seismic performance levels given in Section 2.3 and in Table 2-1. Table 2-3 shows the minimum performance levels as determined by the service life categories given in Section 2.6.

After a bridge is found to be seismically deficient, the next step in the seismic retrofit process is to decide what seismic retrofit, if any, should be applied to the bridge to correct these deficiencies. The seismic retrofitting process has three selection levels: (1) identifying *Retrofit Measures;* (2) developing *Retrofit Approaches*; and (3) selecting a *Retrofit Strategy.*

A *Retrofit Measure* is a modification of a component in the bridge for correcting seismic deficiencies of components or improving the seismic performance of the bridge. Replacing lacing bars with perforated cover plates on a truss member to strengthen the member, or replacing a truss bearing with a more robust bearing that can transmit seismic lateral loads to the substructure, are examples of retrofit measures.

A *Retrofit Approach* is a method of correcting seismic deficiencies of the bridge. Strengthening is an example of a retrofit approach. One or more retrofit approaches may be employed in the seismic retrofit of the bridge.

A *Retrofit Strategy* is the overall plan for the seismic retrofit of the bridge. The plan can employ more than one retrofit approach and several different retrofit measures.

2.9 SEISMIC RETROFITTING DESIGN PROCESS

Selecting the preferred seismic retrofit strategy is a multi-faceted problem. Not only is it often a challenge to find the right technical solution, it is also a challenge to satisfy a multitude of socio-economic constraints. Therefore, a systematic process, as outlined in this section, should aid as a guide to follow in selecting an appropriate seismic retrofit strategy. While these steps are outlined as precise subjects in an orderly flow, the actual retrofit design process is far from orderly. Some steps require backtracking, some may leap ahead, and some may not be needed. Nevertheless, with generous interpretation, these steps and the reference chapters in these *Guidelines* can help with the seismic retrofit design process.

Step 1. Screen the inventory of bridges for seismically complex trusses.

> Bridge owners should screen their inventory of truss bridges for those defined as an SC truss bridge, as listed in Section 2.2.2. General screening methods are presented in Chapter 3. From the resulting list of SC truss bridges produced by the screening process, prioritize deficient truss bridges for seismic retrofitting.

Step 2. Conduct a detailed structural inspection of the selected truss and a review of the maintenance records.

> The first phase of Step 2 is to perform a detailed structural inspection of the selected bridge. The inspection should determine structural damage from any cause such as vehicle impacts on truss members, vessel impacts on piers, structural deficiencies such as missing rivets, bolts, or nuts, sheared anchor bolts and, particularly, corrosion damage. Corroded members, connections, eyebar heads and pins have reduced strength. Often bearings are locked up due to corrosion and require a detailed inspection and evaluation.
>
> The next phase is to review the as-built drawings, in addition to all the original construction specifications, inspection reports, maintenance records, and whatever repair contract plans that are available. This material is helpful for augmenting the structural inspection, determining the existing condition of the bridge, and identifying any structural issues that may require non-seismic rehabilitation or will place constraints on the required retrofitting strategy.

Step 3. Develop a Condition Assessment Report.

The next step is to develop a *Condition Assessment Report*. This report should present the findings from the structural inspection and maintenance documents review so the retrofit designer has a clear understanding of structural damage or deficiencies in the bridge caused by accidents, poor maintenance, or neglect of the bridge. Structural specialists, geotechnical specialists, and the bridge owner should be involved in this phase of developing the *Condition Assessment Report.*

Step 4. Perform an analytical evaluation of the existing structure.

The next step is to perform a detailed analytical evaluation of the structure as it exists in the *Condition Assessment Report*. Analytical methods are presented in Chapter 4. The goal of this step is to assess the seismic performance of the bridge during either one of the two levels of earthquakes or ground fault conditions and to identify seismic deficiencies that can be corrected by seismic retrofitting. A corollary goal is to determine non-seismic structural deficiencies that can be corrected by structural rehabilitation.

Step 5. Develop conceptual seismic retrofit measures.

Often there is more than one way to improve the seismic performance of an existing truss, and it is important that the designer identify as many retrofit measures as possible. Retrofit design considerations are presented in Chapter 5. Many of the retrofit measures will be quickly eliminated because of excessive cost, constructability, traffic controls, or other issues. Solutions that appear viable should be further considered in Step 6.

Step 6. Evaluate alternative seismic retrofit approaches.

The truss was modeled in Step 4 in the un-retrofitted condition. The model is then modified in this step to evaluate the seismic retrofit measures developed in Step 5. Subsystem models may be required to evaluate important subsystems and detailed models may need to be developed for evaluating members and connections. The truss should also be modeled globally to determine the total system response. Evaluation of members, connections, and subsystems are presented in Chapter 6. This step should lead to one or more viable retrofit approaches.

Step 7. Evaluate constructability and cost of retrofit alternatives.

Each of the viable retrofit approaches should have a preliminary design prepared so that comparative cost estimates can be made to compare the cost of each alternative. Chapter 7 discusses Retrofit Measures, Approaches, and Strategies. At this stage, it is helpful to develop a constructability study to determine how contractors could actually install the retrofit. The constructability study can also help develop realistic cost estimates.

Step 8. Conduct meeting(s) for reviewing the seismic retrofit strategies and appoint a Peer Review Panel.

Since selecting a seismic retrofitting strategy involves many complex issues, consensus must be reached on the selection of the most appropriate method. This may be accomplished in strategy meetings. For small or simple projects, one or two meetings may be sufficient. On large or expensive projects, a *Peer Review Panel* (PRP) is often employed to provide experienced oversight review during the entire retrofit process.

These *Guidelines* highly recommend, for large or important bridge retrofit projects, that a PRP be assembled before the seismic retrofitting strategy meeting, beginning at Step 5. The PRP should follow the various steps in the process of selecting a seismic retrofitting strategy. The PRP participation has worked very well for many of the seismically retrofitted bridges that have been constructed. The constructive suggestions of the PRP assure the bridge owner that current seismic technologies are being applied in a professional, acceptable manner.

At the PRP review or strategy meetings, the retrofit designer presents to the attendees the condition assessment of the truss; the seismic and structural deficiencies found; and the alternative seismic retrofitting strategies, approaches, and measures investigated. The constructability study and cost estimates are reviewed and compared. The meetings should lead to a consensus in the retrofit selection and to the selection of non-seismic rehabilitation measures, if required.

Representatives of all the stakeholders should attend this meeting; i.e. the bridge owner, utility companies, federal, state and local government agencies, structural and geotechnical engineering specialists, environmental and citizen groups. The State Historical Preservation Office must approve retrofitting and rehabilitation work on historic bridges.

Step 9. Document the seismic retrofit strategy selection process.

The retrofit decision should be documented in a *Seismic Retrofitting Strategy Report* that becomes part of the permanent record for the project. This report should include all the calculations of the as-existing and the as-retrofitted structure, preliminary plans and sketches showing the proposed retrofit selection, a summary of conclusions and recommendations, preliminary cost estimates, and a summary of discussions from the seismic retrofitting selection meeting.

Step 10. Prepare construction plans, specifications, and estimates (PS&E).

The final step will be developing the construction document – the plans, specifications, and estimates (PS&E) for the seismic retrofit design.

Figure 2-2 outlines the 10-step process for seismic retrofit of truss bridges presented above.

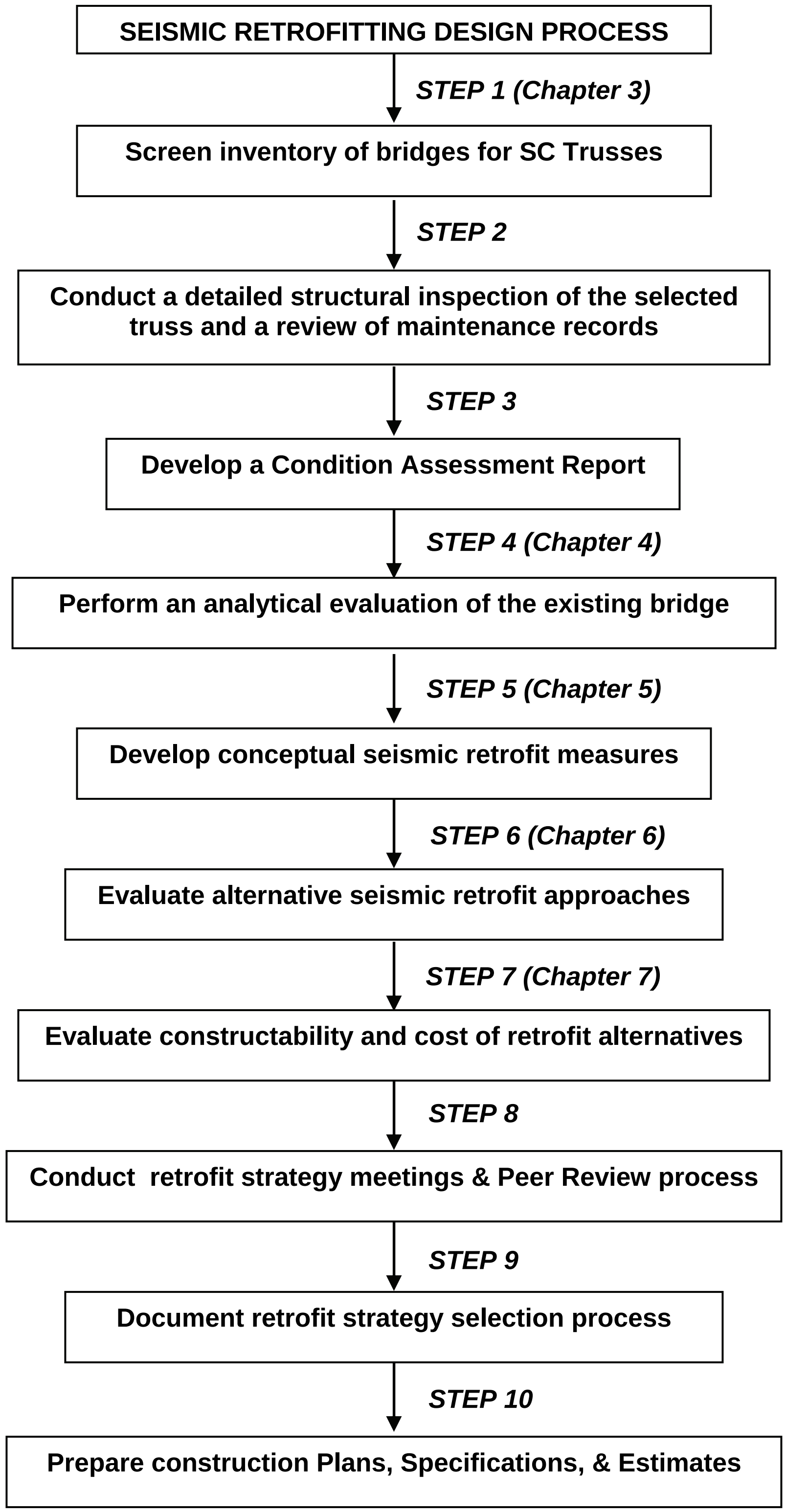

Figure 2-2. Seismic Retrofit Process

CHAPTER 3: SCREENING AND PRIORITIZATION

3.1 GENERAL

In the United States, there exists no standardized bridge procedure for prioritizing a population of bridges according to need for seismic retrofit. States such as California and New York have developed procedures based on their own special circumstances and needs, though it is not the function of these *Guidelines* to establish such procedures. These *Guidelines* do attempt to present information which may be useful for agencies which have not yet adopted standardized seismic methodologies and procedures.

This chapter summarizes the screening and prioritization methods covered in detail in the *Bridge Retrofitting Manual*, and highlights the aspects of relevance to steel truss highway bridges. Steel truss highway bridges, as a subset of all highway bridges, require special screening and prioritization methods because of their unique structural action of axial stressed members connected together into a triangular system. Truss bridges are often used at difficult bridge sites for which other bridge types are not suitable or too expensive. Truss bridges are also used for aesthetic reasons. In the past, trusses were often selected for sites which required long spans, high-level roadways, movable spans, or which were in remote areas. For the last decade in the U.S., cable-stayed structures have replaced trusses as the structure of choice for long spans. Nonetheless, there are a large number of truss bridges in the nation’s highway systems. Many of these structures may be seismically vulnerable, and because replacing all of them is not economically viable, they must be seismically screened and prioritized for seismic retrofitting to assure that the traveling public is protected.

3.2 SCREENING OF TRUSS BRIDGES

The first step in implementing a screening method for truss bridges is to compile a bridge inventory. This will provide the following basic information:

- The structural characteristics of each bridge. This is essential for determining the seismic vulnerability of the bridge which will be expressed as a rating.
- The soil conditions and seismicity at the bridge site. This is necessary to determine a seismic hazard rating.
- The truss bridge performance level for each event.
- The importance of the link to the transportation system and its value to the socio-economics of the community (*essential* or *standard* bridge classification).

This information may be obtained from the bridge owner's records, the National Bridge Inventory, “as-built” plans, maintenance records, the regional disaster plan, on-site bridge inspection records, and discussions with community leaders. Data that is not in the existing database should be made part of the official record and stored in the bridge history file.

Screening starts with an inventory of all bridges within the jurisdiction of the agency that owns or operates these facilities. The bridges are screened to identify those truss structures that are seismically deficient and to prioritize them in order of need for retrofitting. The process needs to be rapid, easy to apply, and conservative in classifying seismically vulnerable trusses. Truss bridges found to be deficient during the screening stage will be subjected to detailed evaluation in the next step. Any bridge that is found not to be seismically vulnerable is excluded from further study.

The *Bridge Retrofitting Manual* presents a detailed discussion of screening and prioritization methods. Any of the methods presented in the manual may be used for the first level of screening. Truss bridges classified as *essential* bridges are considered in these *Guidelines* to be SC truss bridges, as discussed in Section 2.2.2, although they may not meet the physical definition of SC truss bridges. The methods of analysis and seismic retrofitting of SC trusses, outlined in these *Guidelines*, should be applied to *essential* bridges to ensure that they will meet the seismic performance level required. The trusses classified as *standard* bridges from the first-screening level need to be further screened in a second screening level that determines if any of them meet the physical definition of SC truss bridges. The methods of analysis and seismic retrofitting of SC truss bridges, as outlined in these *Guidelines*, should also be applied to these structures.

3.3 PRIORITIZATION OF SEISMICALLY–COMPLEX TRUSS BRIDGES

The objective of a prioritization program is to determine the order of retrofitting truss bridges that were found to be SC truss bridges by the screening program. Truss bridges that were classified by the screening program can then be prioritized according to the methods presented in the *Bridge Retrofitting Manual*.

A number of factors must be considered in any prioritization program for SC truss bridges. Factors that will affect such a program are structural vulnerabilities, soil conditions, the level of the seismic hazard, as well as:

- The bridge’s function as a symbol and an economic link to the community.
- Bridge importance, as outlined in Section 2.5.
- Physical condition of the truss and of the substructure supporting the truss.
- Serviceability of the truss. Many truss bridges were built with two narrow lanes, light truck loading, or low clearance levels.
- Service Life Categories, as outlined in Section 2.6. A bridge in poor physical condition, or one that is already scheduled for structural or functional rehabilitation, may be given a higher priority for seismic retrofitting, since cost savings can be achieved by performing non-seismic and seismic work simultaneously.
- Cost of seismic retrofit. If the cost of the seismic retrofitting lies within 50% - 75% of the cost of a new bridge, bridge replacement should be seriously considered. This rule of thumb may not be applicable in the case of a historically significant bridge.

The above factors are not an exhaustive list, but they illustrate some of the principles involved in the assigning of priorities. In most cases, seismic ratings are used to guide decision-making, but are not the final word. Common sense, engineering judgment, and community feelings are inputs to the task of weighing the actual costs and benefits of seismic retrofitting (and structural rehabilitation) against the risks of doing nothing or some other alternative.

3.4 SEISMIC RATING PARAMETERS FOR SC TRUSS BRIDGES

The *Bridge Retrofitting Manual* (Chapter 4) contains two alternative seismic prioritization rating methods based on quantitative as well as qualitative considerations. Both methods are defined by a numerical prioritization index as a function of several quantitative and qualitative parameters. They are designated as: (a) Seismic Rating Method of Indices; and (b) Seismic Rating Method of Expected Damage. For the purpose of completeness, this section summarizes the definitions of these methods (presented in sections 4.2 and 4.3 of the *Bridge Retrofitting Manual*) as well as the parameters needed to define the prioritization algorithms .

The primary quantitative parameters required for the prioritization rating of SC truss bridges are derived based on the following behavioral and performance measures:

- Reliability or redundancy of lateral load resisting systems in structural subsystems such as support towers, piers, and foundations.
- Superstructure portal frame and sway frame racking drift capacities (fragilities).
- Eye-bar member capacities (fragilities).
- Effect of condition of structure in terms of degree of deterioration (section loss).
- Truss shoe pin and bearing vulnerabilities (fragility thresholds).
- Expansion joint seat lengths.

3.4.1 METHOD OF INDICES

The Seismic Rating Method of Indices is based on a two-stage process characterized by a function of quantitative and qualitative variables. The method is represented by a prioritization index function defined by:

$$P = P\,(R, I, NSO) \qquad (3\text{-}1)$$

where:
- P = Priority Index
- R = R (V,E) is the *bridge rank* based on structural vulnerability (V) and site seismicity (E)
- I = Importance of structure & route carried
- NSO = Non-Seismic & Other factors. (Other factors include: network redundancy; anticipated service life, etc.)

The *priority index*, P, obviously applies to a specific population of bridges that are to be ranked for seismic retrofit, given a limited source of resources allocated for a retrofit program. Recommendations for the evaluation of parameters R, I, NSO, and P for SC truss bridges are presented in the subsections that follow.

3.4.1.1 Bridge Rank, R

The enumeration of the quantitative stage involves the *rank* computation based on structural *vulnerability* (V) and site-specific *seismic hazard* (E), whereas the qualitative stage is based on the notion of societal and economic *importance* (I) of the bridge and the route it carries as well as other non-seismic factors such as network redundancy.

In this method, bridge rank may be computed as the product of the structural vulnerability and the site seismicity:

$$R = V\,E \tag{3-2}$$

where V and E take on values in the range of 0 – 10 each, thereby resulting in R to take on values in the 0 – 100 range. The enumeration of the vulnerability rating involves two distinct categories characterized by strength-based components (connections, bearings, etc.) and ductility-based components (lateral load resisting mechanisms) as follows:

$$V = \max\,\{V_1, V_2\} \tag{3-3}$$

where

V_1 = vulnerability rating for connections, bearings, and seat widths
V_2 = vulnerability rating of components and subsystems that make up the lateral load resisting system of the bridge that is susceptible to failure.

The V_1 rating is a strength-based indicator where the reliability of structural connection details, bearing type, load capacity, and seat lengths are evaluated. The enumeration procedure (in the 0 – 10 rating range) outlined in Section 4.2.1.1(a) of the *Bridge Retrofitting Manual* is directly applicable here.

The V_2 rating is a reflection of the susceptibility of the bridge to develop a collapse mechanism under earthquake-induced lateral loading and soil liquefaction. For SC truss bridges, this rating is best derived from a preliminary assessment of the structural reliability of primary subsystems that comprise the lateral load resisting system of the bridge, in terms of limit states. The enumeration of this rating may be determined as the weighted sum of the ratings for each primary lateral load resisting subsystem (foundations, piers, towers, and abutments) as follows:

$$V_2 = w_1\,V_{2,1} + w_2\,V_{2,2} + \ldots. + w_n\,V_{2,n} \tag{3-4}$$

where

w_i = relative weighting factors: $\sum_i w_i = 1$
$V_{2,i}$ = vulnerability rating of primary subsystem "i" ≤ 10

The $V_{2,i}$ rating values for each subsystem must be based on the degree of redundancy and a rough estimate of the available ductility in the lateral load resisting structural system. As an example, in the case of a foundation-pier-tower configuration, there are three primary subsystems: (1) foundation, (2) concrete pier, and (3) chevron-braced steel tower. Using lateral drift displacement as a performance measure, yield limit states may be defined (in view of the degree of structural redundancy) by means of preliminary analyses for each subsystem and used to obtain an approximate evaluation of available subsystem ductility. For a subsystem with estimated ductility between 1.0 – 2.0, a vulnerability rating of $V_{2,i}$ = 10 is appropriate, whereas for an estimated ductility capacity in the 5.0 – 6.0 range, a vulnerability rating of $V_{2,i} = 0$ may be assigned. Thus, depending upon the degree of structural redundancy, the vulnerability rating of each subsystem will ranges between 0 – 10, with the value of 10 representing the highest level of vulnerability. As to the weighting factors, they indicate the relative importance of each subsystem, with the weakest link having the highest weight. In the example considered, assuming that the weakest link is the steel tower (subsystem 3), followed by the foundation (subsystem 1) and the concrete pier (subsystem 2), the following respective weighting factors may be used $w_3 = 0.5$, $w_1 = 0.4$, and $w_2 = 0.1$.

In assessing system ductility for the evaluation of the $V_{2,i}$, care must be taken to ensure that a relatively high component ductility (say 4) does not automatically imply a relatively high system ductility. In fact, system ductility generally tends to be lower than component ductility, as it is largely dependent on the structural system.

The seismic hazard rating, E, may be used as defined in section 4.2.1.1 of the *Bridge Retrofit Manual*, given by:

$$E = 10\, F_v\, S_1 \leq 10 \tag{3-5}$$

where

F_v = is the site coefficient
S_1 = spectral acceleration (g) at 1-sec period

3.4.1.2 Importance Factor, I

The *importance* factor, I, like the *rank*, takes on a value in the 0 – 100 range. This may be determined based on owner agency criteria, if available. As an alternative, these *Guidelines* recommend the evaluation of this factor based on the formulation proposed by [Buckle 1990], as follows:

$$I = a\,[\, r_1(d_1/d_{ref}) + r_2(d_2/d_{ref}) + u + t\,] + b\,(\$retro / \$repl)$$

where: a,b = relative weighting factors with values in the 0 – 10 range, such that a+b=10
r_1 = route type *carried* by the ridge
r_2 = route type *crossed* by the bridge
r_1, r_2 = 0.8 for County Arterial
= 0.5 for County Highway
= 0.2 Local Access
d_1 = detour length of route *carried* by the bridge
d_2 = detour length of route *crossed* by the bridge
d_{ref} = reference detour length = max{d_1,d_2 - over group of trusses under study}
u = 1 if utilities are carried by bridge
= 0 if no utilities are carried
t = ADT / ADT_{ref} -- traffic factor
ADT = Average daily traffic
ADT_{ref} = Reference ADT = max {ADT of group of trusses under study}

$repl = Replacement cost estimate of bridge, $
$retro = Retrofit cost estimate of bridge, $

3.4.1.3 Non-Seismic Factors Parameter, NSO

The *Non-Seismic and Other Factors* parameter, NSO, takes on a value in the 0 – 100 range. By definition, this parameter does not lend itself to a direct and simple mathematical formulation. The evaluation of NSO is best determined based on owner agency criteria, if available. In the absence of such criteria, assumed values may be assigned on a relative basis with the other bridges in the group under study for prioritization.

3.4.1.4 Priority Index, P

In these *Guidelines*, it is recommended that the computation of the retrofit *priority index*, P, be performed by the algorithm proposed by [Buckle 1990], using a weighted sum function of R, I, and NSO as follows:

$$P = \alpha_1 R + \alpha_2 I + \alpha_3 NSO \quad ; \qquad \text{with} \quad \Sigma_i \alpha_i = 1 \tag{3-6}$$

This index takes on values in the 0 – 100 range, based on the allowable domain of values that the parameters R, I, and NSO assume. Thus, a group of SC truss bridges within a given inventory of truss bridges may be numerically ranked for retrofit evaluation and design, in accordance with the *priority index*, P, beginning with the highest number.

3.4.2 METHOD OF EXPECTED DAMAGE

The Seismic Rating Method of Expected Damage is also represented by a Prioritization Index, P, which is defined as a function of quantitative and qualitative variables. This method is based on a comparison of the severity of expected damage for each bridge in the inventory, based on the same earthquake. The prioritization index is given by:

$$P = P\,(R, H_{LOSS}, NSO) \tag{3-7}$$

where: P = Priority index
R = R *bridge rank* based on expected damage in terms of Fragility Curves
H_{LOSS} = Indirect losses (i.e., not directly related to physical damage)
NSO = Non-seismic & other factors. (Other factors include: network redundancy; anticipated service life, indirect losses, etc.)

As in the Method of Indices, the *priority index*, P, obviously applies to an inventory or group of bridges that are to be ranked for seismic retrofit, given a limited source of resources allocated for a retrofit program. Recommendations for the evaluation of parameters R, H_{LOSS}, NSO, and P for SC truss bridges are presented in the subsections that follow.

3.4.2.1 Bridge Rank, R

In this case, the quantitative stage involves the computation of the *rank*, R, based on the level of *expected damage* and the repair costs associated with each of them. Expected damage in this case is based on a probabilistic notion of damage characterized by *fragility curves*. In this method, the rank is computed as follows:

$$R = R\,(B_{LOSS}) \tag{3-8}$$

where:

B_{LOSS} = Direct losses computed from probabilistically-based *Fragility Curves* defined by seismic spectral acceleration and five levels of damage ($DS_1 \rightarrow DS_5$) and their respective repair costs

= U x RCR_T

Damage States:

DS_1 = No Damage
DS_2 = Slight Damage
DS_3 = Moderate Damage
DS_4 = Extensive Damage
DS_5 = Collapse

U = Replacement cost of bridge

RCR_T = Total Repair cost ratio

= $\Sigma_i\,(RCR_i \,.\, P[DS_i|S_a])$

RCR_i = Repair cost of Damage State "i" (DSi)

$P[DS_i|S_a(1)]$ = The probability of DSi occurrence, for a given spectral acceleration ordinate Sa @ 1-sec period (i.e., $S_a(1)$), obtained from fragility curves (see Fig. 3-1)

The parameter RCR_T is computed based on the notion of *fragility curves* representative of particular damage states (*DSi* given above), and are constructed on the basis of probabilistic reliability analyses.

Appendix C of the *Bridge Retrofit Manual* presents the notion of fragility curves in detail. As a matter of illustration, the simplified alternative version of the damage probability proposed in the *Bridge Retrofit Manual* is presented here for the derivation of sample fragility curves for damage state 1 through 5. Based on the simplified definition, the probability of being in a particular damage state D that is equal to or greater than damage state DS_i, for a given spectral acceleration at a structural period of 1 second, i.e., $S_a(1)$ = S1, is given by

$$P\,[\,D > DS_i \mid S_a(1)] = \frac{1}{1 + \{\,S_a(1)/\,A_i\,\}^{-1.7\,/\,\beta_c}} \tag{3-9}$$

where:

DS_i = Damage State "i"
S_a (1) = Spectral acceleration of seismic hazard response spectrum at 1-second period
A_i = Median spectral acceleration to cause damage state DS_i
β_c = Randomness and uncertainty parameter for demand and capacity
= 0.60 recommended value in Appendix C of *Bridge Retrofit Manual*

Equation (3-9) expresses the probability of exceeding a certain damage state, DS_i, as a function of the spectral acceleration S_a (1) – S_a evaluated at 1-sec structural period in the hazard response spectrum applicable to the site – parameterized by the median spectral acceleration, A_i, necessary to cause damage state DS_i to occur. Thus, the notion of the probabilistic distributions of both the seismic demand and the capacity of components and subsystems are reflected.

As an illustrative example, Table 3-1 depicts the distribution of the values of the median spectral acceleration, A_i, that causes damage state DS_i to occur.

Table 3-1. Sample Median Spectral Acceleration Distribution

Damage State	**Damage Type**	A_i (g)
1	None	0.10
2	Slight	0.15
3	Moderate	0.40
4	Extensive	0.60
5	Collapse	1.0

Based on the data in Table 3-1 and Equation (3-9), the fragility curves parameterized by the median spectral acceleration values that trigger the five damage states is shown in Figure 3-1.

Thus, based on such curves and relative repair costs, the *direct loss* parameter, B_{LOSS}, is evaluated consistently. In terms of the applicability of the method to SC truss bridges, the following guidelines may be followed in developing fragility curves:

- Determination of the load path in the structural system and potential damage mechanisms that best characterize the five damage states cited above, in terms of the behavior of its components and subsystems.
- Determination of limit states that characterize the five damage states of the bridge system. This involves the determination of the level of spectral acceleration or equivalently spectral displacement that triggers the limit states corresponding to the five damage states. This will require various levels of structural analysis.
- Computation of fragility curves using Equation (3-9).

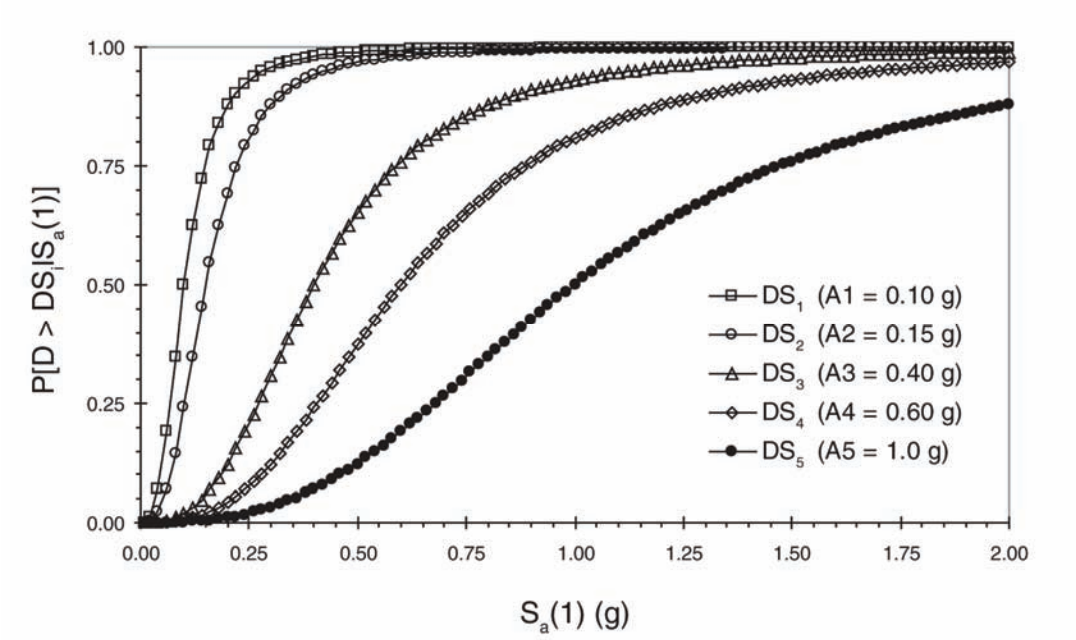

Figure 3-1. Fragility Curves for Damage States 1-5, parameterized by A_i = {0.1g, 0.15g, 0.4g, 0.6g, 1.0g}

In the absence of a direct algorithm for the evaluation of the *rank* parameter, R, as a function of B_{LOSS}, the following functional form (which restricts its value to the 0 – 100 range) is recommended for this computation:

$$R = 100\ B_{LOSS} / \max\{B_{LOSS} \text{ for group in inventory}\} \qquad (3\text{-}10)$$

3.4.2.2 Indirect Losses Parameter, H_{LOSS}

The *indirect losses* parameter, H_{LOSS}, takes on a value in the 0 – 100 range. By definition, this parameter does not lend itself to a direct and simple mathematical formulation. The evaluation of H_{LOSS} is best determined based on owner agency criteria, if available. In the absence of such criteria, assumed values may be assigned on a relative basis with the other bridges in the group under study for prioritization.

3.4.2.3 Non-Seismic Factors Parameter, NSO

The *Non-Seismic and Other Factors* parameter, NSO, takes on a value in the 0 – 100 range. By definition, this parameter does not lend itself to a direct and simple mathematical formulation. The evaluation of NSO is best determined based on owner agency criteria, if available. In the absence of such criteria, assumed values may be assigned on a relative basis with the other bridges in the group under study for prioritization.

3.4.2.4 Priority Index, P

While this alternative of retrofit prioritization is more complex and analytically intensive than the Method of Indices, it does provide a more comprehensive representation of expected damage of the bridge structural system, for a given earthquake. Again, as in the Method of Indices, in these *Guidelines* it is recommended that the computation of the retrofit *priority index*, P, be performed by the algorithm proposed by [Buckle 1990], using a weighted sum function of R, H_{LOSS}, and NSO as follows:

$$P = \alpha_1 R + \alpha_2 H_{LOSS} + \alpha_3 NSO \quad ; \quad \text{with} \quad \sum_i \alpha_i = 1 \tag{3-11}$$

Similarly, this index takes on values in the 0 – 100 range, based on the allowable domain of values that the parameters R, H_{LOSS}, and NSO assume. Thus, a group of SC truss bridges within a given inventory of truss bridges may be numerically ranked for retrofit evaluation and design, in accordance with the *priority index*, P, beginning with the highest number.

CHAPTER 4: STRUCTURAL ANALYSIS

4.1 GENERAL

The type and extent of the analytic effort required for the seismic performance evaluation of bridges depends upon the importance of the bridge, the complexity of the structural system, and the seismic environment in which it is located, as discussed in Chapter 5 of the *Bridge Retrofitting Manual*. This chapter presents the types of analyses appropriate for evaluating Seismically Complex (SC) and Seismically Standard (SS) truss bridges classified according to the criteria presented in Section 2.2.2. Due to the wide variety of existing truss bridges scattered throughout the United States, the level of structural analyses required – in terms of complexity – for seismic performance assessment and retrofit design projects could be quite varied, depending upon the classification of the bridge and the site-specific seismic hazard. Table 4-1 illustrates the range of possibilities for the level of demand analyses for truss bridges.

Table 4-1. Level of Structural Analysis Desired for Truss Bridges

Truss Bridge Type	**ANALYSIS COMPLEXITY (1)**		
	Seismic Hazard Level (2)		
	Low	**Moderate**	**High**
Seismically Standard (SS)	Simplified (Uniform Load)	Multi-Mode Spectral	Elastic & Inelastic Dynamic Analysis
Seismically Complex (SC)	Multi-Mode Spectral	Elastic & Inelastic Dynamic Analysis	Inelastic Dynamic Analysis

(1) See Chapter 5 of the Bridge Retrofitting Manual and Section 4.3 of these Guidelines.
(2) The Low and High seismic hazard levels correspond to the Lower Level (LL) and Upper Level (UL) earthquakes defined in Section 2.4. The Moderate hazard level here refers to a project-specific hazard criterion that lies between the LL and UL events, such as a 500-year mean return period scenario, for example.

4.1.1 OVERALL APPROACH TO ANALYSIS AND EVALUATION OF TRUSS BRIDGES

The major factors governing the seismic performance of SC truss bridges (or selected SS trusses that are essential bridges) include: moderate to large-scale multiple spans; location in high-seismic hazard zones; significantly long alignments spanning waterways of variable soil profiles; physical condition of existing structure; and deep foundations in soft soils. These types of bridges create challenges and complexities in terms of seismic performance assessments, and require a global strategy which includes: derivation of ground motions based on site-specific seismic hazard assessments; thorough geotechnical site investigation programs; and the use of mathematical models for demand and capacity analyses. The essential elements of the analytic approach required for such undertakings include:

- Development of local and subsystem mathematical models for capacity assessment of foundation, pier, tower, and superstructure systems.

- Development of global mathematical models – representative of the system inclusive of the soil-foundation, substructure, and superstructure system – for demand assessments that include strong nonlinearities of relevance in the structural system, in order to capture effects such as expansion hinge opening/impact and "engineered" plastic hinging of flexural members.
- Soil-foundation interaction (SFI) effects for modeling soil-foundation behavior.
- Multi-support-time-history (MSTH) analyses to capture alignment variable motions and soil-foundation-structure interaction (SFSI) effects.

4.1.2 ANALYSIS OBJECTIVES

Seismic retrofit projects of major bridge crossings and, by definition, SC truss bridges involve three phases:

- Phase 1: Seismic vulnerability assessment.
- Phase 2: Retrofit strategy development.
- Phase 3: Final retrofit design.

Within this framework, the objective of the analytic effort is twofold: (a) to assess the seismic demand on the overall system; and (b) to determine of the capacity of the structure in terms of strength and ductility. Based on these primary activities, a systematic assessment of the seismic performance of the structure under the design seismic event is achieved in terms of Capacity/Demand (C/D) ratios. The demand loads and capacities that enter the C/D computation must be factored in accordance to the provisions of the AASHTO LRFD or LFD Design Specifications [AASHTO 1986, 2002, 2004], and the C/D ratios computed for the appropriate load combinations.

In the initial vulnerability assessment phase (Phase 1), which is a diagnostic phase, the objective of the analyses is to facilitate seismic vulnerability assessment of the structure and identification of preliminary retrofit measures. These analyses must be conducted using linear-elastic models; in some cases, however, the incorporation of "geometric" nonlinearities characterizing the behavior of expansion joint opening-impact is essential. The inclusion of nonlinearities characteristic of plastic hinges in existing flexural members at this stage is not necessary; in fact it may be counterproductive because potentially deficient flexural members will be incapable of developing them and a false sense of performance will be obtained by their inclusion.

In the strategy phase (Phase 2), the analyses are directed towards examining viable retrofit measures, approaches, and strategies. They include more complex mathematical models with a number of nonlinearities characterizing: expansion joint opening-impact; load-limiting ductile structural systems such as frames or individual members with flexural plastic hinges; and response modification devices (RMDs) such as isolation bearings and dampers.

In the final design phase (Phase 3), the analyses serve to validate the structural design philosophy implemented and to verify the seismic performance of the retrofitted system. These analyses also are typically based on relatively complex mathematical models that include all essential geometric nonlinearities as well as "engineered" nonlinearities, such as ductile frames and

RMDs. The degree of complexity of the modeling and analytic effort must directly serve the purposes of the retrofit strategy and the design philosophy being investigated.

4.1.3 SEISMIC PERFORMANCE MEASURES AND EVALUATION

The seismic performance evaluation of the entire bridge structural system requires the establishment of performance measures for individual components and subsystems, in terms of strength and deformation parameters. The *strength-based* measures are expressed in terms of resultant loads, and the *deformation-based* measures in terms of ductility and displacements. In either case, the performance of the structure under the design seismic event is evaluated by C/D ratios, with the demands and the corresponding capacities defined by their respective measures discussed below. As stated in section 4.1.2, the computation of C/D ratios is based on factored loads and resistances.

In the case of SC truss bridges, the general category of structural components and subsystems comprising superstructure members, steel towers, and support components, and the type of performance measure applicable to them are identified in Table 4-2. The performance assessment of the bridge and its components are ultimately based on these measures quantified in terms of appropriate *limit states*; the notion of limit state here is taken as a condition beyond which the structural component or the system experiences any of the following:

1. Unserviceability (excessive deformations – i.e., yield and beyond; cracking; etc.).
2. Loss of stability (high potential for collapse).
3. Violation of design provisions (i.e., strength degradation; exceedance of specified ultimate load capacities).

With this definition, the notions of *yield* and *ultimate* limit states go into the definitions of deformation-based as well as strength-based measures alluded to in Table 4-2.

4.1.3.1 Strength-Based Measures

The strength-based measure is applicable to all components whose primary intended function is to transfer loads between structural members and structural subsystems while maintaining deformations well within the elastic range of the material. This is expressed in terms of load resultants on the components under consideration.

For example, the performance of a mechanical support bearing unit is measured by its resistance to applied shear forces, axial forces, and bending moments. Component performance is thus assessed by computing C/D ratios for each applicable load resultant combination, where the capacity is specified as an ultimate limit state load.

Table 4-2. SC Truss System Performance Measure Categories

Components & Subsystems	Performance Measure Category		
	Strength-Based	**Displacement-Based**	**Ductility-Based**
Superstructure Members:			
Verticals	X	-	X(1)
Diagonals	X	-	-
Chords	X	-	X
Eye-bars	X	-	-
Stringers	X	-	-
Floor Beams	X	-	-
Horizontal Bracing (wind)	-	-	X(1)
Superstructure Subsystems:			
Portal Frames	-	-	X
Sway Frames	-	-	X
Superstructure Components:			
Gusset Plates	X	-	-
Eye-bar Pins	X	-	-
Truss Shoes and Pins	X	-	-
Support Components:			
Fixed Bearings	X	-	-
Roller Bearings	X	X	-
Rocker Bearings	X(2)	X(2)	-
Eye-bar Hangers	X	X	-
Expansion Joints (Seats)	-	X	-
Towers:	-		
Braced Steel Towers	-	-	X
Steel Split Bents (expansion)	-	X	-
Concrete shafts	-	-	X
Substructure:			
Concrete Columns	-	-	X
Multi-column Bents	-	-	X
Rigid Piers & Abutments	X	-	-
Steel-Concrete Connections:			
Steel Tower Base	X	-	-
Bearing Anchorages	X	-	-
Foundations:			
Rock-bearing caissons	X	-	-
Pile groups	X	-	X(3)

*(**1**) For transverse frame action, in sway frames and portal frames.*
*(**2**) Rocker bearings are easily destabilized; replacement is preferable.*
*(**3**) Pile group foundations must have adequate ductility capacity in order to avoid brittle failure and loss of vertical load support capacity.*

4.1.3.2 Ductility-Based Measures

The ductility-based measure is applicable to all subsystems in the structure that comprise the primary mechanism for earthquake-induced lateral load resistance. These subsystems are expected to sustain deformations beyond the elastic range of the material. This measure is intended to assess the extent of inelastic deformation sustained by the subsystem, and it is essential for the performance assessment of structural subsystems that includes towers; transverse bents; longitudinal bridge frames; portal and sway frames in the superstructure truss system; and foundation pile groups. The ductility-based measure is traditionally expressed as:

$$\mu = \delta / \delta_Y \qquad (4\text{–}1a)$$

where:

- δ = Deformation sustained by the subsystem under lateral loading, in terms of the displacement of a characteristic degree of freedom of the subsystem. This response parameter is analytically evaluated for both demand and capacity assessments.
- δ_Y = Deformation of the subsystem at the *yield limit state*. This parameter is estimated by capacity analyses or available empirical and experimental data.

Figure 4-1 depicts the notion of lateral load-induced deformations such as racking drift displacements for portal frames and sway frames in the superstructure, drift displacements of piers and towers, and drift displacement of soil-foundation subsystems.

The δ_Y parameter, which designates the yield limit state of the subsystem, is identified from load-deflection curves obtained by static push-over analyses (see Section 4.2.2). For a portal frame, the load-deflection curve consists of the base shear versus the racking drift degree-of-freedom at the top of the frame, as depicted in Figure 4-2. The load-deflection curve also defines the ultimate limit state of the subsystem from which the ductility capacity or available ductility of the subsystem is evaluated:

$$\mu_{available} = \delta_{ULT} / \delta_Y \qquad (4\text{–}1b)$$

where, δ_{ULT} = deformation of system at *ultimate limit state*. The performance of the subsystem is thus evaluated by computing the C/D ratio of the available ductility to the ductility demanded.

4.1.3.3 Displacement-Based Measure

The purely displacement-based measure is applicable to components that must maintain functionality while undergoing large horizontal displacements without undergoing material deformations. These components include supports such as roller bearings and rocker bearings, eye-bar hangers at expansion joints of drop-in or suspended spans (pendular motion), and split bents acting as expansion joint mechanisms.

The performance of theses components are assessed in terms of the C/D ratio of the available displacement capacity to the demand displacement estimates.

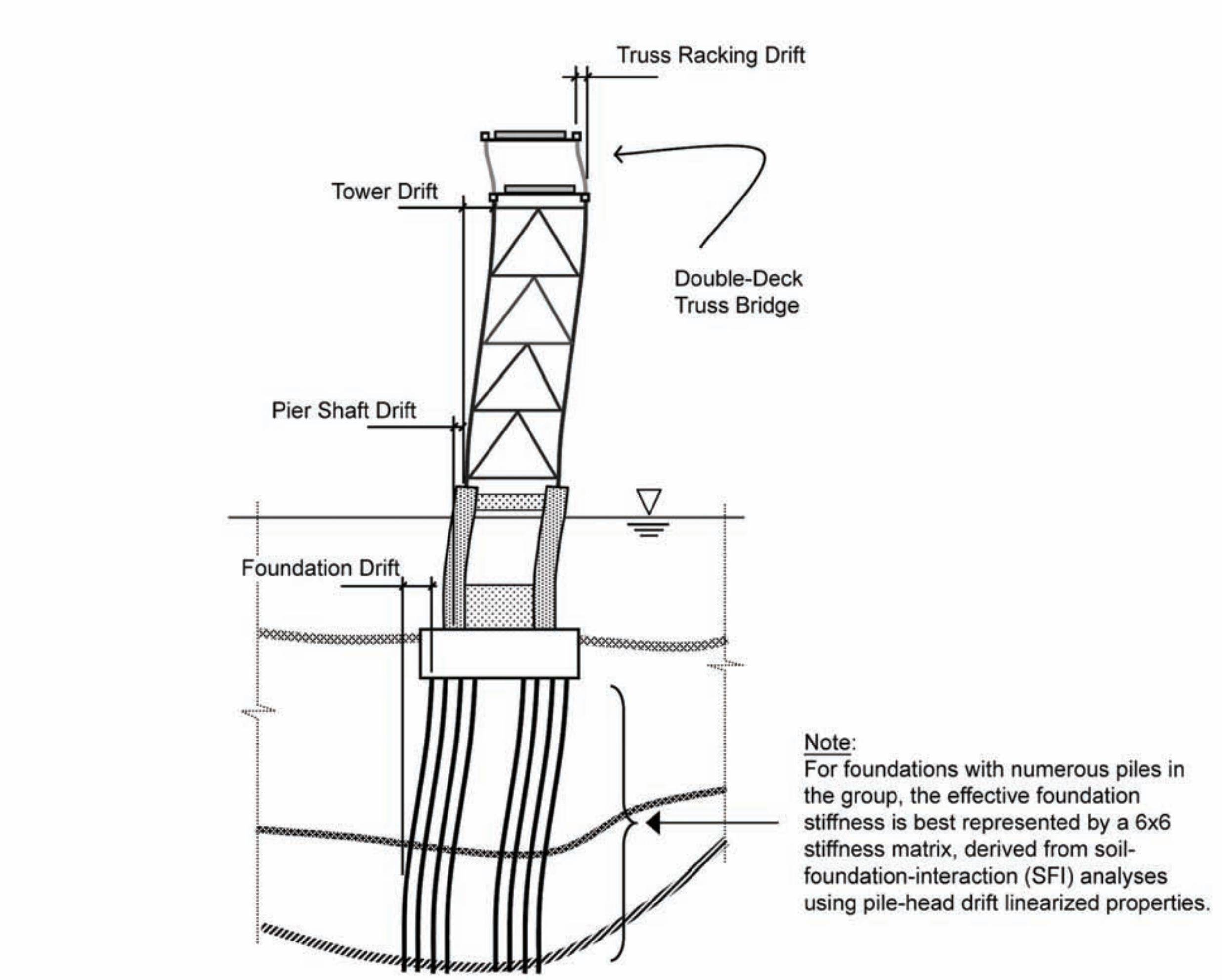

Figure 4-1. Drift Displacement Measures for Various Structural Subsystems

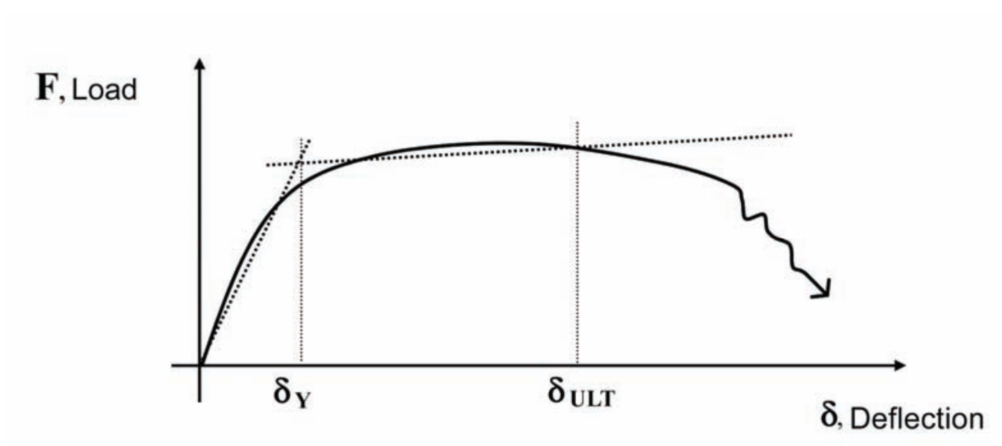

Figure 4-2. Typical Idealized Load-Deflection Curve

4.1.3.4 Retrofit Strategies

Based upon the desired performance criteria specified in terms of the performance measures outlined above, among the viable retrofit strategies that are likely to be examined for SC truss bridges include the following considerations:

- Strengthening individual superstructure members, such as truss chords, verticals, eye-bars, bracing members, bearings, and riveted, bolted, and welded connections, with the intent of enhancing their load carrying and deformation capacities, in terms of yield and ultimate load limit states.
- Installation of seat extenders at expansion joints to increase their displacement capacities.
- Strengthening tower structural members, such as legs, bracing members, and cross beams, to enhance their ductility capacities, or the introduction of additional ductile lateral frames to control tower drift-displacements.
- Introduction of pile-cap collar-frames supported by spud piles to control foundation drift displacements.
- Limiting the load transmitted to the superstructure by introducing isolation devices between the bridge truss and its supports, tower rocking mechanisms, and other RMDs such as damping devices across expansion joints. The intent is to keep the truss superstructure members within the essentially elastic range, even after the implementation of retrofit modifications.
- "Period-tuning" of bridge frames and bents with the intent of load re-distribution away from "stiff" piers – particularly with the objective of minimizing longitudinal loading on "anchor piers."

4.1.4 ANALYSIS TYPES

The computation of seismic demands on SC truss bridges will require analyses using either elastic or inelastic dynamic analysis methods, by means of finite-element computer analysis programs. An elastic analysis might be based on either a *multi-mode response spectrum analysis* or a *time history analysis*. For many structures, a response spectrum analysis will be sufficient. A time history analysis will be necessary to obtain simultaneous force demands on a member, for evaluation of a very critical element, or if the variation of ground motions along the bridge alignment turns out to be significant. An inelastic dynamic analysis will be required to capture the nonlinear response of a support tower due to flexural hinging or rocking action, for instance. These various approaches are discussed in more detail in the following section.

The establishment of capacities (limit states) is achieved by a series of static analyses, which may be performed manually in some cases or with the use of finite-element computer analysis software. As discussed in section 4.1.3, the capacity of members can be expressed either in terms of *force*—the strength of the member—or *deformations*—the ductility of the member. A force approach will be suitable for most structures**.** It will be necessary to consider the ductility of the members for only those members subjected to inelastic demands. Members that are particularly likely to be subjected to inelastic demands are those undergoing large displacements as occurs in members supporting braced steel towers. In this particular case, it may be effective to consider the support tower as a subsystem and to compute its displacement capacity from a pushover analysis. The capacity of members and subsystems is discussed in Chapter 6**.**

4.2 METHODS OF ANALYSIS

A number of mathematical models and structural analyses with varying degrees of complexity are required for capacity and demand assessments. These range from manual or semi-manual methods (e.g., moment curvature analyses for reinforced concrete sections to establish displacement ductility of single flexural members and portal frames) to large-scale nonlinear dynamic analyses using finite element structural analysis computer programs. This section summarizes the essential elements of the types of analyses relevant for the evaluation of SC truss bridges.

4.2.1 DEMAND ANALYSES

Seismic demand analyses of SC truss bridges (and selected SS trusses) require the use of mathematical models that reflect the dynamic properties of the structure, i.e., natural vibration periods and mode shapes, as well as the distribution and frequency content of the earthquake loading – characterized by design response spectra or, equivalently, by spectrum-compatible ground motion time histories. Generally, three types of analyses may be performed using commercially available finite element computer programs. In order of complexity, these methods are:

- Conventional Response Spectrum Analysis (RSA) or Multi-Mode Spectral Analysis.
- Elastic Time History Analysis (ETHA).
- Inelastic Time History Analysis (ITHA).

The choice of method depends on the objectives of the analyses cited in Section 4.1.2, the level of seismic ground motion intensity, and on the complexity and importance of the structure. Typically, all three methods are utilized in the various phases of seismic retrofit projects. An inelastic analysis would only be warranted if the demands were large enough to produce inelastic response, which may be identified by the use of the elastic dynamic analyses as the initial "diagnostic" step in the evaluation process. As to the merits of using time history analysis versus RSA, a complex structural detail or an important structure might warrant a time history analysis to obtain simultaneous force demands in order to realize an efficient retrofit or a more accurate evaluation that would make it possible to avoid a retrofit altogether. Each of these approaches is discussed in following sections. The subject of modeling is discussed in Section 4.3.

4.2.1.1 Conventional Response Spectrum Analysis (RSA)

The conventional response spectrum analysis technique (also known as Multi-Mode Spectral Analysis Method) is based on the modal superposition principle of dynamic response evaluation, where every ground support point of the structure is excited uniformly by the same input design acceleration response spectrum (ARS). Because of this, the applicability of the RSA method for the seismic analysis of truss bridges is limited to configurations in which the variability of the ground motions along the bridge alignment is not significant. Variability of ground motions along the alignment result in variable support-specific excitation, which may arise in cases for long multi-span truss bridges or in deep gorge crossings with highly variable subsurface conditions.

These types of analyses are performed using three-dimensional finite-element models. Specific advice regarding the modeling of truss bridges is given in Section 4.3. General guidelines for the response spectrum analysis of bridges are given in Section 5.4.2.2 of the *Bridge Retrofitting Manual*. Those guidelines are fully applicable to truss bridges. Some of the key principles from the manual are:

- A modal analysis is required in order to extract the natural vibration periods and corresponding mode shapes of the structure. Enough modes must be extracted in order to capture 90-95% of the generalized (modal) mass. For bridges with massive substructures, it is necessary to pay particular attention to the distribution of modes to ensure that enough of them are extracted to capture the dynamic response of the (lighter) superstructure truss members. (Note: the use of Load-Dependent or Derived Ritz Vectors available in some computer programs significantly reduces the number of modes required to capture virtually all of the generalized mass of the system).
- The CQC (Complete Quadratic Combination) method should be used to combine modal responses. This method ensures that vibration modes with closely spaced frequencies are properly accounted for in the modal superposition calculation of the response.
- The SRSS (Square Root of the Sum of the Square) method may be employed to obtain the directionally combined response quantities of interest due to seismic loading components associated with the three mutually orthogonal directions (2 horizontals + vertical). Alternatively, the familiar "30% rule" (i.e., $Q = Q_{1.0\,X} + Q_{0.30\,Y}$, where Q is a response parameter and X & Y indicate mutually orthogonal loading directions) may also be used for this.

The results of a response spectrum analysis represents the peak dynamic loads in members and the peak dynamic displacements of the structure. The dynamic loads may be combined—both positive and negative signs should be considered in the combinations—with dead load and service loads to obtain the total resultant loads for comparison with member capacity, as discussed in Chapter 6. This is Method C presented in Section 5.4.3 of the *Bridge Retrofitting Manual*.

A significant drawback of response spectrum analysis is that the calculated load components in a structural member (i.e., the peak axial force, shear forces, and flexural moments) are assumed to occur simultaneously in time, which, in reality, is unlikely to occur for every member in the system. For instance, since the "yield surface" of a flexural member depends on the interaction of axial force and moment, the capacity/demand evaluation of the member will typically be under-predicted, which may result in a high degree of conservatism. This could be the case for a steel compression member, for instance, where the combined axial-flexural capacity is determined by the AASHTO interaction equations of the form:

For $\dfrac{P_u}{P_r} < 0.2$: $\quad \dfrac{P_u}{2P_r} + \left(\dfrac{M_{ux}}{M_{rx}} + \dfrac{M_{uy}}{M_{ry}} \right) \le 1$ (4-2a)

For $\dfrac{P_u}{P_r} \ge 0.2$: $\quad \dfrac{P_u}{P_r} + \dfrac{8}{9}\left(\dfrac{M_{ux}}{M_{rx}} + \dfrac{M_{uy}}{M_{ry}} \right) \le 1$ (4-2b)

where: P_u = axial compressive load; M_{ux}, M_{uy} = factored concurrent flexural moments about the member local axes x and y, respectively; P_r = factored compressive resistance; and M_{rx}, M_{ry} = factored flexural resistance about the member local axes x and y, respectively.

Structural displacements and deformations may be similarly computed and compared with the displacement drift capacities (limit states) of subsystems such as support towers, as determined by pushover analyses. This is Method D2 of the *Bridge Retrofitting Manual*. Pushover analysis is discussed generally in Section 4.3 Evaluation of the displacement capacity of support towers is further discussed in Section 6.5.

4.2.1.2 Time History Analyses – Elastic & Inelastic

The time history analysis method for the dynamic structural response evaluation of SC truss bridges lends itself to a wide range of applicability in terms of: (a) consistent combination of resultant loads on individual structural members; (b) capturing the effects of alignment-variable input ground motions; (c) soil-foundation-structure interaction effects; and (d) modeling nonlinear characteristics. As stated earlier, the dynamic demand analyses may be categorized as ETHA and ITHA, depending upon the presence of nonlinearities in the models.

4.2.1.2-1 Elastic Systems (ETHA)

The general governing equations of motion for the seismic analysis of multi-span bridges with linear-elastic behavior (generally modeled by means of the finite-element method - see Section 4.3), are conveniently represented as follows:

$$M\ \ddot{x}(t) + C\ \dot{x}(t) + K\ x(t) = F(t) \tag{4-3a}$$

where *M*, *C* and *K*, are the system mass, damping, and stiffness matrices, respectively; $x, \dot{x}, \ddot{x}$ are the displacement, velocity, and acceleration vectors, respectively; and *F(t)* is the externally applied loading as a function of time, *t*. The matrix representation of Eq.(4-3a) incorporating effects due to soil-foundation-structure interaction (SFSI) is as follows:

$$\begin{bmatrix} M_{aa} M_{ab} \\ M_{ba} M_{bb} \end{bmatrix} \begin{Bmatrix} \ddot{x}_a(t) \\ \ddot{x}_b(t) \end{Bmatrix} + \begin{bmatrix} C_{aa} C_{ab} \\ C_{ba} C_{bb} \end{bmatrix} \begin{Bmatrix} \dot{x}_a(t) \\ \dot{x}_b(t) \end{Bmatrix} + \begin{bmatrix} K_{aa} & K_{ab} \\ K_{ba} & (K_{bb} + G_{bb}) \end{bmatrix} \begin{Bmatrix} x_a(t) \\ x_b(t) \end{Bmatrix} = \begin{Bmatrix} o \\ f_b(t) \end{Bmatrix} \tag{4-3b}$$

where:

$x_a(t)$ = structure displacement vector of "*a* "degrees-of-freedom (dof) as a function of time, *t*

$x_b(t)$ = soil-foundation system displacement vector of "*b* " DOF as a function of time, *t*

$\dot{x}_i(t)$, $\ddot{x}_i(t)$ = velocity & acceleration vectors corresponding to degrees-of-freedom "$i = a,b$" as a function of time, t

M_{ij} = subsystem *mass* matrices

C_{ij} = subsystem *damping* matrices

K_{ij} = subsystem *stiffness* matrices

$f_b(t)$ = $G_{bb}\, v_b(t)$ ~ b-dof effective driving force vector

G_{bb} = soil-foundation system stiffness matrix

$v_b(t)$ = ground displacements due to kinematic interaction between the free-field motions and the soil-foundation system (also known as "scattered" motions)

The formulation expressed by Eq.(4-3b), whose derivation may be found in [Clough & Penzien, 1993] and [Wolf 1985], is based on the mathematical sub-structuring concept – where the system matrices are partitioned based on the degrees of freedom of the structural and soil subsystems designated by the displacement vectors x_a and x_b, respectively. This procedure facilitates the application of multi-support excitation to the structure – through the soil-foundation system – and also captures the soil-foundation-structure interaction (SFSI) effect.

The soil-foundation system stiffness matrix, G_{bb}, and the motions due to kinematic interaction, $v_b(t)$, must be derived from soil-foundation interaction (SFI) analyses, which require special expertise. The essential elements of this methodology are further discussed in subsection 4.2.1.3 presented below. In problems where the foundations are not deep and are based in stiff geo-materials, ranging from hard soil/soft rock to solid rock, soil springs and rock motions can be substituted directly for G_{bb} and v_b directly, using conventional calculation methods.

4.2.1.2-2 Inelastic Systems (ITHA)

Inelastic analyses may be required for the retrofit strategy development and design phases of major truss retrofit projects. The nonlinear behaviors of significance in truss bridges may include: yielding of support tower flexural members; introduction of mechanical RMDs; ductile portal and sway frames in the superstructure; tension-only truss members; buckling or yielding of steel bracing members; geometric nonlinearities as a result of pronounced P-Δ ("P-Delta") effects in slender towers; and "strong" nonlinearities such as gap-impact behavior at expansion joints. As a result of these inelastic actions, the system stiffness and damping coefficients become functions of the displacement and velocity as well (dependence on the time variable is implicit). The general equation of motions for the nonlinear case is expressed by:

$$F_i(\ddot{x};t) + F_d(x,\dot{x};t) + F_s(x;t) = F(t) \tag{4-4a}$$

where, F_i, F_d, and F_s are the system inertia, damping, and stiffness force vectors, respectively, and $F(t)$ is the applied external load vector. The numerical solution of this nonlinear system requires discretizing the problem into incremental equations, comprising a step-wise progression of system equations assumed to be linear over small time intervals, Δt. The incremental equations of motion are formulated by: (a) discretizing the time domain into a discrete set: t_o,

$t_1 = t_o + \Delta t$, $t_2 = t_1 + \Delta t, \ldots$, $t_n = t_{n-1} + \Delta t, \ldots$; and (b) subtracting evaluations of Eq.(4-4a) at two adjacent points in time, say $t = t_n$ and $t = t_n + \Delta t$, as follows:

$$M\ \Delta\ddot{x} + C(t_n)\ \Delta\dot{x} + K(t_n)\ \Delta x = \Delta F \qquad (4\text{-}4b)$$

where:

M = Mass matrix
$C(t_n)$ = Damping matrix (approximate) valid over time interval $[t_n, t_n + \Delta t]$
$K(t_n)$ = Stiffness matrix (approximate) valid over time interval $[t_n, t_n + \Delta t]$
$\Delta F = F(t_n + \Delta t) - F(t_n)$
$\Delta x = x(t_n + \Delta t) - x(t_n)$
$\Delta\dot{x} = \dot{x}(t_n + \Delta t) - \dot{x}(t_n)$
$\Delta\ddot{x} = \ddot{x}(t_n + \Delta t) - \ddot{x}(t_n)$

This formulation may also be found in [Clough & Penzien, 1993]. The type of numerical integration scheme required to march Eq.(4-4b) forward in time is determined by the form of the variation of the acceleration that is assumed over the integration time step interval Δt (i.e., acceleration is assumed to be constant, linear, etc. over Δt), and iterations required to enforce equilibrium. The discussion of particular numerical schemes is beyond the scope of this section. Note that, in expanded form, the coefficient matrices and load vector are identical in form as for the linear case given by Eq. (4-3b), including the soil-foundation system stiffness sub-matrix, G_{bb}.

4.2.1.2-3 Solution Algorithms

The solution of the system of Equations (4-3) and (4-4) requires a numerical algorithm whose choice depends upon the following three main factors: (a) the presence of nonlinear characteristics and their nature; (b) mathematical model size; and (c) the significance of special effects such as SFSI and support-specific excitation. For the analysis of SC truss bridges by means of the time history analysis method, the following aspects require consideration:

- For diagnostic purposes, a fully linear-elastic model is sufficient in the preliminary phase of a seismic vulnerability analysis. Thus, the diagnostic ETHA may be performed by either the modal superposition technique or the direct integration technique; the choice is dependent upon model size and the capabilities of the computer analysis software being utilized. The Capacity/Demand (C/D) ratios (see Chapter 6) computed for each structural member will provide a good indication of the level of inelasticity – and therefore damage – that may be expected in the overall structural system. Inclusion of limited nonlinearities such as gap-impact at expansion joints are generally required at this stage.
- Following the preliminary diagnostic analyses, inelastic behaviors may need to be considered and modeled for a more refined evaluation and for the retrofit strategy development phase. While the behavior of the truss superstructure (existing or retrofitted) – especially its primary members as defined in Chapter 5 – must be maintained within the linear-elastic material range, support truss towers (existing or retrofit) may be permitted to undergo inelastic deformations provided their vertical load carrying capability is not compromised.

- In using the direct integration time history analysis method (for either the ETHA or ITHA methods), it is important to select an unconditionally stable integration algorithm (implicit) to ensure incremental solution stability, along with a small enough time step size to ensure accuracy.

A significant advantage of the time history analysis method for the evaluation of the dynamic response of bridge structures is that resultant load components on structural members can be combined in a consistent manner for every discrete time point in the solution, as opposed to the RSA method where peak responses of each load component are considered to occur simultaneously. Directional loading combinations are also non-problematic because the loading is also applied consistently in terms of a tri-axial set of motions at each support location.

4.2.1.3 Soil-Foundation Interaction Analysis and Effective Input Motions

Conceptually, the system of equations given by Eq. (4-3b) and Eq. (4-4b), which represents the soil-foundation-structure system, demonstrates the interrelationship between the structure and the soil-foundation system, by means of the cross-coupling terms in the system matrices, as well as the source of the seismic loading. Figure 4-3 depicts a typical representation of this concept.

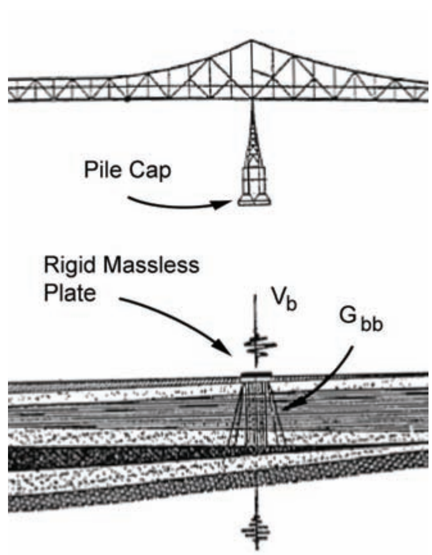

Figure 4-3. Superstructure and Soil-Foundation Models

The determination of model parameters for the foundations of an SC truss bridge obviously depends upon the type of foundation. Some of the most likely types encountered in practice are shown in Figure 4-4, which depicts three types including: multi-pile groups; large-diameter shafts; and caissons. In all of these cases the challenge is in the determination of the stiffness of the soil-foundation system and the effective seismic input motions. The soil model in each case comprises depth-variable soil resistance that can be represented by equivalent “springs” that may be linear or nonlinear. For cases (a) and (b), the depth-variable soil characterization is generally accomplished by the use of nonlinear ‘p-y’, ‘t-z’, and ‘Q-u’ curves, which represent lateral, side-shear, and tip resistance functions, respectively. For the case of the caisson foundation, case (c), similar resistance functions may also be used.

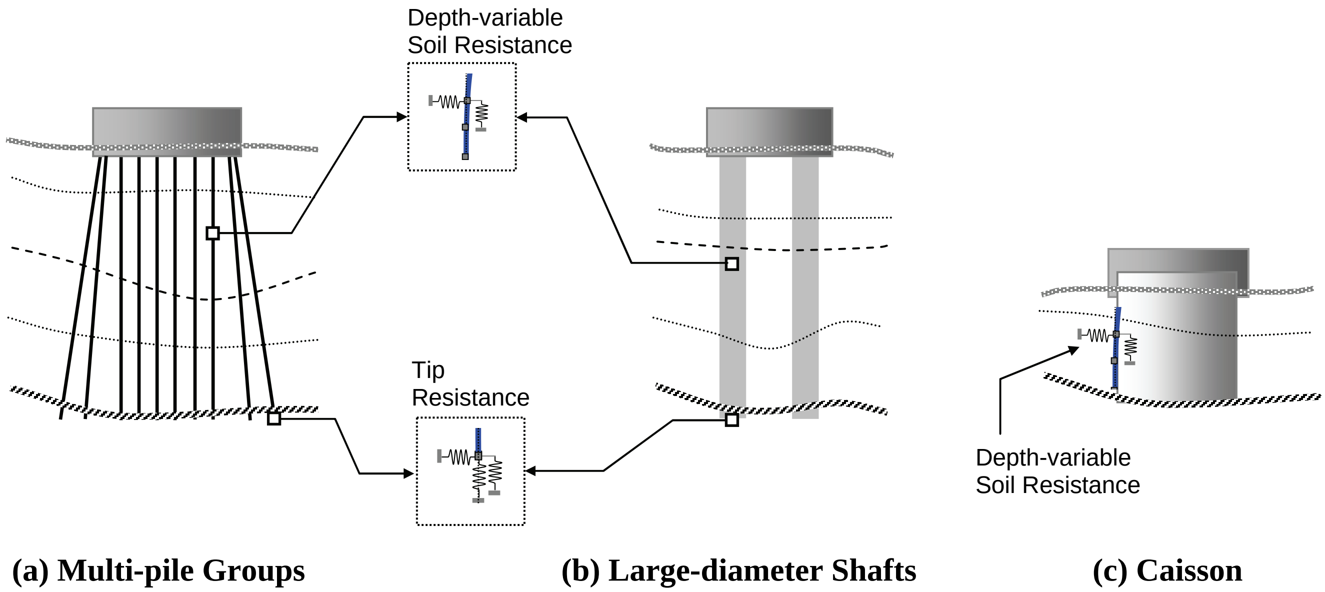

Figure 4-4. Typical Foundation Types for SC Truss Bridges

The approach and degree of detail in modeling the soil-foundation system depends upon the type of foundation. As shown in Figure 4-4, the treatment of pile-supported or large-diameter shaft foundations (governed by 'p-y, t-z, & Q-u' characterization of the soil medium) will be quite different than that for caisson-type foundations (based on the 'elasto-dynamic' approach). A review of these approaches is available in [Tseng & Penzien, 2000].

The following stepwise process is recommended for modeling foundations and the corresponding effective input motions for implementation in a global model for the seismic evaluation and retrofit of SC truss bridges.

1. <u>Reference Ground Motions</u>
   - Establish reference ground motions at firm ground (rock or hard soil).
   - If applicable, develop multiple-support motions to simulate incoherency and wave-passage effects of seismic wave propagation along the bridge alignment.

2. <u>Site Geology, Soil Profile, & Site Response</u>
   - Review foundation type of the existing structure (or retrofitted configuration) and determine extent of embedment.
   - Review soil profile, and categorize soil type as "soft / medium / hard."
   - Obtain appropriate geotechnical characterization of the soil.
   - For deep foundations obtain depth-variable free-field motions at selected pier locations, based on site response-analyses. (Site-response analyses are not needed for shallow foundations in stiff soils).

3. <u>Pile/Shaft Foundations (Configurations (a) and (b))</u>
   - Assemble soil-foundation models using individual pile/shaft units supported by depth-vertical lateral and vertical springs (nonlinear or linearized; linearization must

be based on the lateral deflection profile of the embedded pile/shaft element parameterized by a set of pile-head drift displacements).

- For pile groups with a large number of piles (Case (a)), assemble a foundation subsystem using linearized individual soil-pile units. Derive 6x6 stiffness matrix representation of the entire foundation system and the corresponding effective input motions from SFI analyses, for implementation in the global model.
- For foundations with few shaft elements (Case (b)), either the above procedure may be used to condense the system, or the individual soil-shaft units may be directly implemented in the global model, using the depth-variable free-field motions as the seismic input.

4. Caisson Foundations
   - Use either one of the following two alternatives: (i) discrete model with the caisson represented by a block of solid (3D) finite elements supported by depth-variable springs (linear or nonlinear); or (ii) caisson block embedded in a continuum model of the soil (this will require special computer programs).

Based on the procedure outlined above, the dynamic properties of the soil-foundation system along with its response, computed from the time history analyses of the subsystem (subjected to the free-field depth-variable ground motions), are used to obtain the soil-foundation system stiffness matrix (G_{bb}) and the motions due to kinematic interaction ($v_b(t)$). In view of inherent uncertainties associated with the soil medium, it is generally preferable to develop linearized foundation model parameters in developing models for the majority of retrofit projects.

4.2.2 CAPACITY ANALYSES

There are two general categories of capacity analyses that are required for the evaluation of component and subsystem capacities for use in the C/D ratios: (a) static push-over analyses for portal frames, such as braced steel towers or multi-column bents; and (b) static structural/stress analyses of components such as steel tower base anchorages, bearing assemblies, steel tower legs, and laced bracing members, using detailed finite-element models.

4.2.2.1 Static Push-Over Analyses

Static push-over analyses are needed to establish the deformation-based limit states cited in Section 4.1.3, for the evaluation of all the subsystems in the structure that comprise the lateral load resisting mechanism. These analyses are performed by means of finite-element computer codes with the capability to model plastic hinging behavior in flexural members and P-Δ effects.

The objective of these analyses is to determine the inelastic behavior of the subsystem under study (i.e., portal frames, sway frames, support towers, etc.) and to characterize it by means of a load-deflection curve, an example of which is shown in Figure 4-2. In the case of a portal frame or braced tower, for instance, this consists of the base-shear as a function of the tower-top racking drift displacement. Figure 4-5 depicts a push-over analysis model of a typical braced steel tower under static loading.

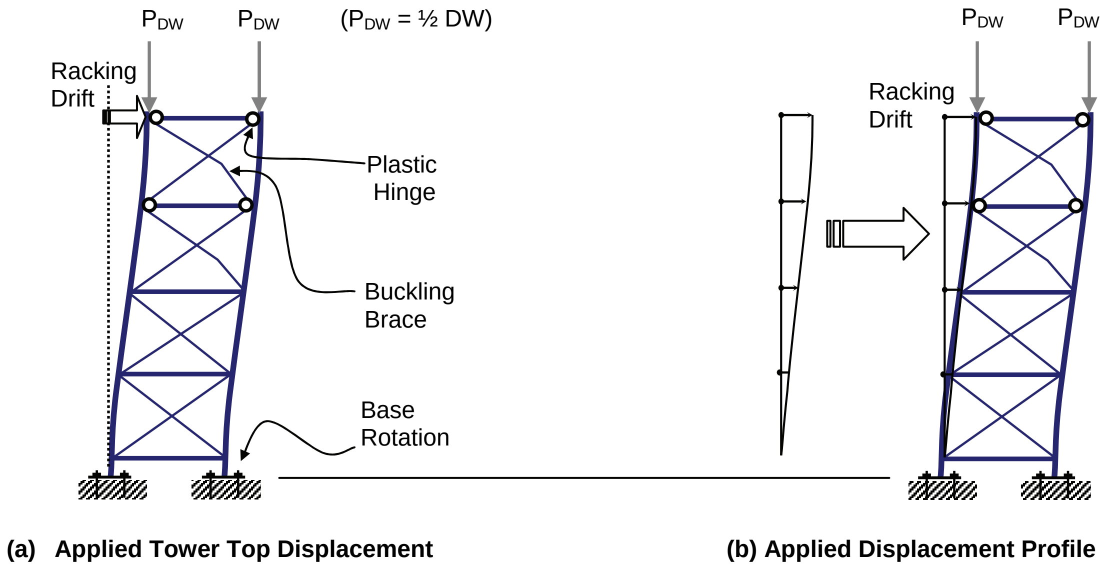

Figure 4-5. Push-over Analysis of Braced Steel Tower

A primary aspect of nonlinear push-over analysis concerns the selection of the appropriate degrees of freedom and the corresponding loading mechanism. For example, the racking deformation of a braced steel tower may be characterized by the horizontal translational degree of freedom at the top of the tower with a fixed-base condition, as depicted in Figure 4-5(a). The appropriate loading mechanism for this configuration consists of a displacement-controlled loading applied along the translational degree of freedom. Alternatively, a more complicated displacement profile in the form of the first lateral mode shape of the frame may be applied instead (Figure 4-5(b)); the choice will depend upon the expected dynamic behavior of the system. For most braced towers of traditional truss bridges the simple lateral racking drift displacement applied to the top of the frame will suffice (Figure 4-5(a)).

With respect to static push-over analyses of structural models with plasticity, it is important to note that the *displacement-controlled* loading approach is preferable over the *load-controlled* method because of its inherent stability. This is by virtue of the fact that for every prescribed displacement in the plastic range, the resultant load that sustains the prescribed displacement can be readily computed (uniquely) throughout the plastic range (see load-deflection curve in Figure 4-2). Conversely, for a load-controlled procedure the displacement solution is difficult to obtain in the post-yield region of the system because of the “wide” plastic displacement range that corresponds to a very “narrow” load range on the flat portion of the curve.

Effects due to high compressive loads as well as effects due to large-displacement geometry are important for certain structural members, such as tall and slender support towers, where the P-Δ effect tends to reduce system stiffness (“geometric stiffness” effect) as well as system ductility. Figure 4-6 depicts the influence of the P-Δ effect on the load-deflection curve (idealized) of a tower structure, which shows: (a) loss of plastic shear capacity of the system; and (b) reduction of the post-yield stiffness (i.e., loss of strain hardening) – thereby reducing the threshold drift displacement for instability.

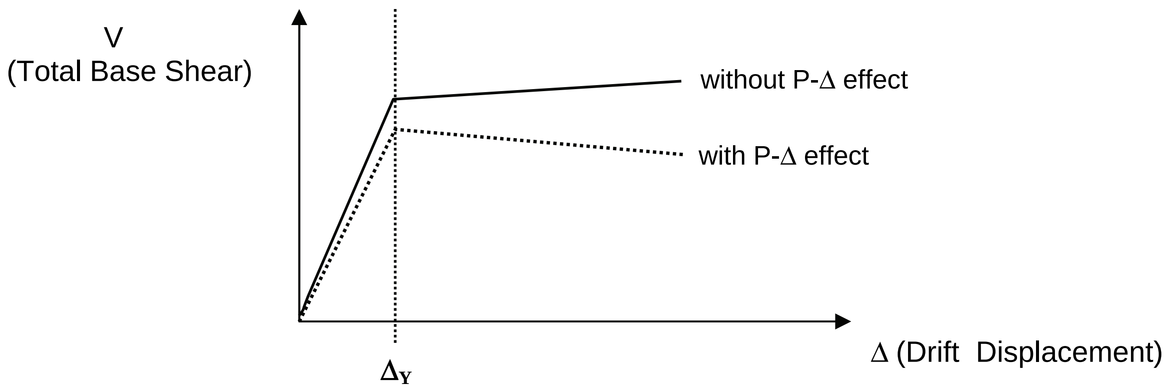

Figure 4-6. Idealized Load-Deflection Curves for Steel Towers

4.2.2.2 Detailed Component Analyses

For the evaluation of the strength-based capacities of complex components, static structural/stress analyses need to be performed in order to identify the governing mode of failure or loss of functionality of the component under study. Such components include: large mechanical bearing units with steel pins; steel tower base anchorage assemblages; laced bracing members; multi-member truss connections; welded, bolted, and riveted connections, etc.

The analyses required here involve detailed finite-element models with a large number of various element types including shell and 3D solid finite elements. Primarily, the analyses need to be performed to identify failure modes of the assemblage, which may involve a series of limit states such as onset of local buckling of plates, and onset of brittle failures of sub-components that make up the unit. These types of analyses may involve modeling some inelastic material behaviors in order to obtain realistic failure modes of the assemblage.

The failure load set (i.e, ultimate limit state) identified by these types of analyses define the strength-based capacity needed for the C/D evaluation of the component under study.

4.3 MODELING TECHNIQUES

4.3.1 GLOBAL MODEL DEVELOPMENT

Seismic analysis of SC truss bridges are best accomplished using mathematical models developed with the aid of three-dimensional (3D) structural analysis computer codes. For a consistent seismic performance assessment of the entire bridge system, a 3D global finite-element model is required for the demand analyses. Figure 4-7 depicts an example of a global model developed for the seismic retrofit design of a major SC truss bridge.

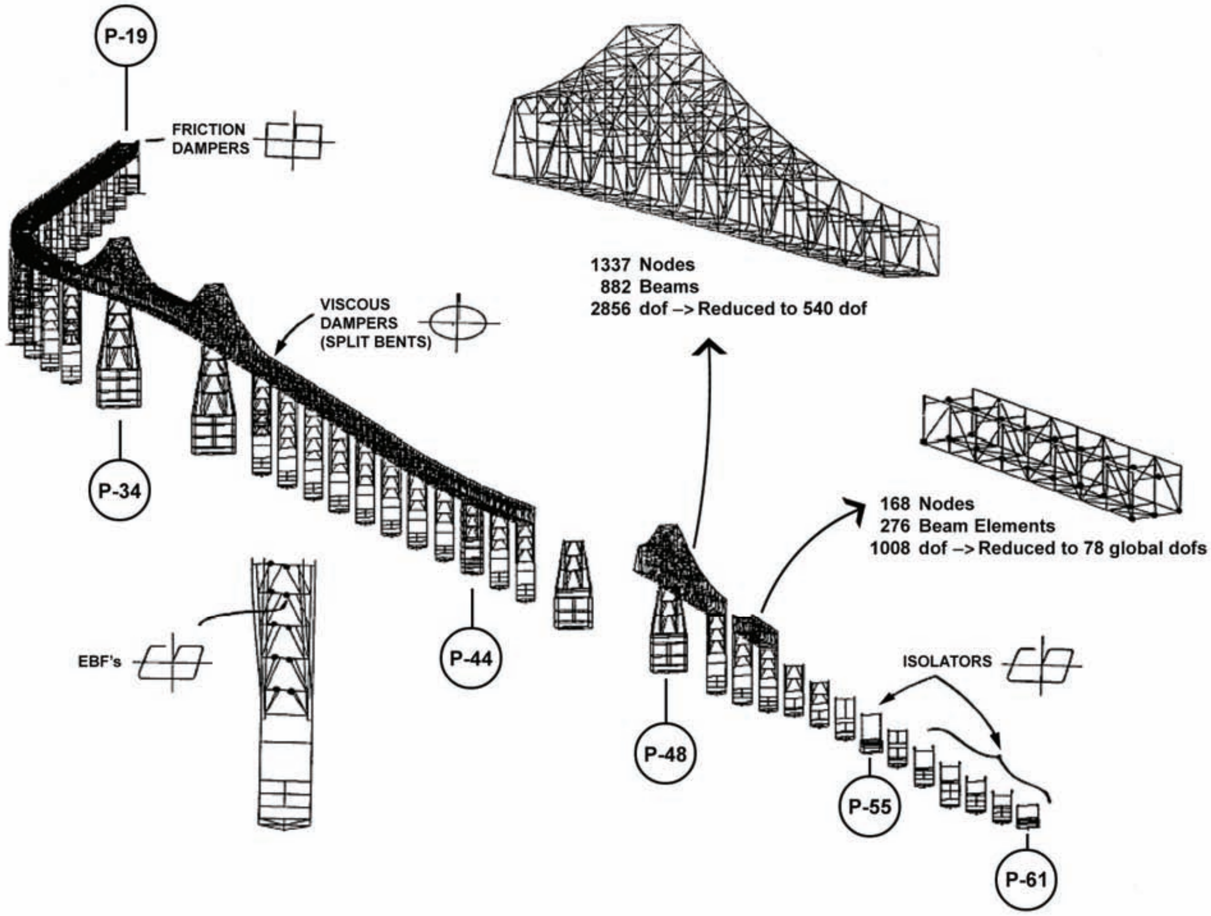

Figure 4-7. Large-Scale Global Model

Global finite-element models comprise an assemblage of local and subsystem units, in turn composed of "truss," "beam-column," and "shell" finite-elements. Among behaviors that are important to capture by a global model (generally for the purposes of a time history analysis) are:

- Essential nonlinearities such as expansion joint opening and impact.
- Nonlinearities associated with ductile mechanisms in either the existing piers or in potential retrofit schemes.
- P-Δ effects (geometric stiffness) in slender members and towers.
- Nonlinearities characteristic of RMDs (isolators and dampers).
- Soil-foundation system stiffness and damping.

The types of elements required to achieve these behaviors, either individually or grouped in subsystems, are available in some modern commercially available finite-element software and include:

1. Conventional beam-column elements (linear) with geometric stiffness.
2. Plastic hinge elements defined by load-deflection curves or moment-curvature elements.

3. Gap elements to simulate expansion joint behavior: opening (no resistance) and closure (impact).
4. Tension-only elements to model eye-bars and cables.
5. One-dimensional (1D) or composite elements with plasticity to simulate load-deflection curves of isolation bearings.
6. Viscous damper elements with optional exponents.
7. Mathematical sub-structures (or super-elements) that condense large structural units (truss spans or foundation pile groups) by the *static condensation* technique.

The subsections that follow present examples and key modeling aspects of "truss bridges" and "bridge trusses" discussed throughout the present chapter, and they include: truss superstructure; piers and towers; RMDs; expansion joint models; built-up laced members; eye-bars; and buckling compression members.

4.3.2 TRUSS SUPERSTRUCTURE MODELING

Figure 4-8 depicts a typical truss unit modeled with beam finite-elements. Note that although truss structural members primarily resist tension and compression, it is usually preferable to model them using 3D beam-column elements with flexural properties; this facilitates the evaluation of connections such as gusset plates. Shell finite-elements are also very effective in modeling bridge deck slabs, specifically to achieve the proper mass distribution and in-plane stiffness of the deck system.

4.3.3 PIER AND TOWER MODELS

Figure 4-9 depicts the elements of a truss bridge pier that includes the concrete substructure, steel tower, and partial superstructure. The bridge truss and steel tower are modeled with beam-column finite-elements, the concrete pier is modeled with beam-column elements to simulate the shafts, and shell elements to represent the diaphragm (shear) walls. The foundation is represented by a 6x6 stiffness matrix in this case (see Figure 4-1).

4.3.4 RESPONSE MODIFICATION DEVICES

Figure 4-10 depicts the elements of a retrofit measure utilizing RMDs. As shown, the isolation device is modeled as a composite 6-element system (4 elastic members, and 2 inelastic members) that represents the bi-axial hysteretic behavior represented by the load-deflection curve (F-Δ) in each direction (longitudinal and transverse).

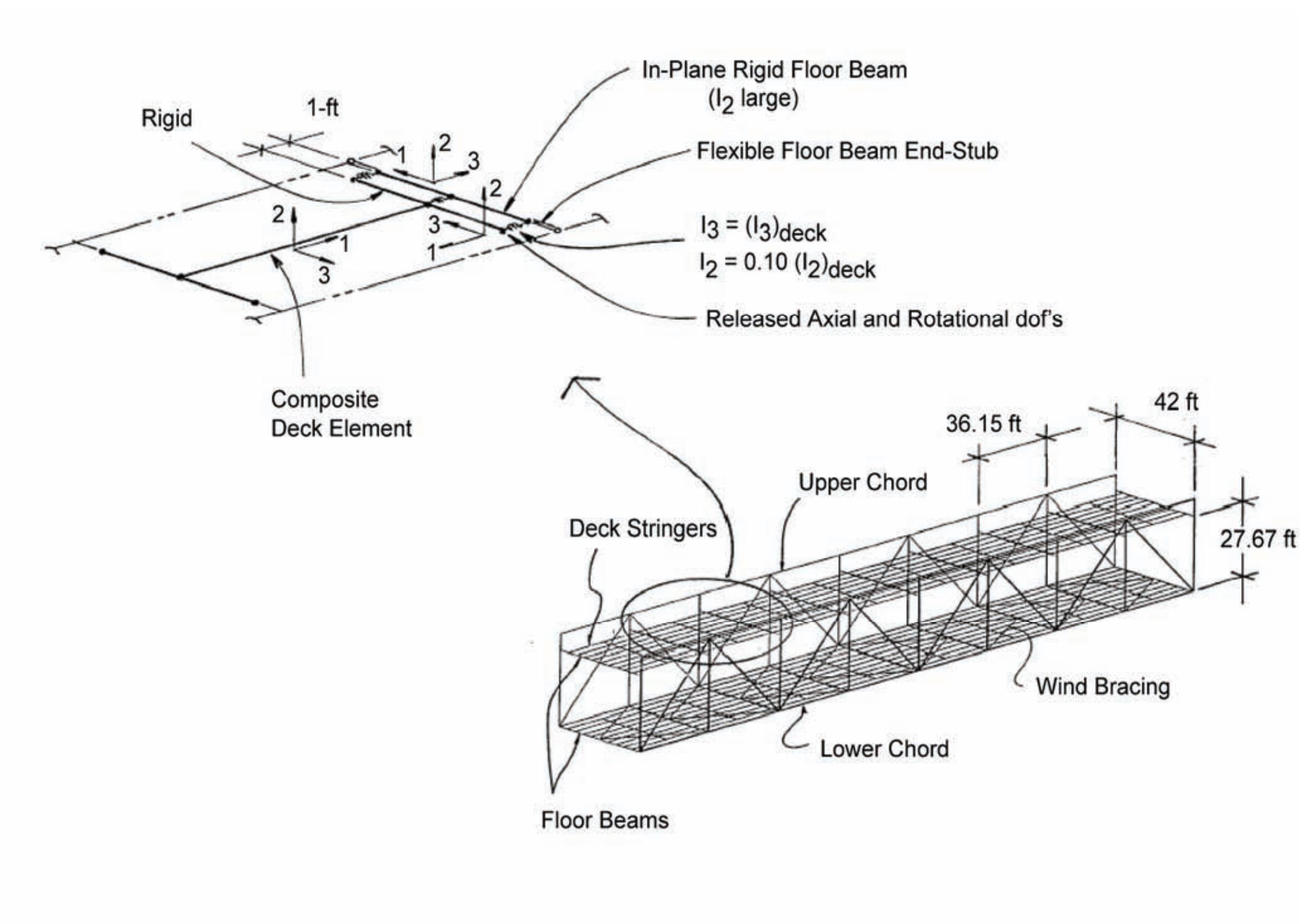

Figure 4-8. Typical Truss Unit Structural Model Features

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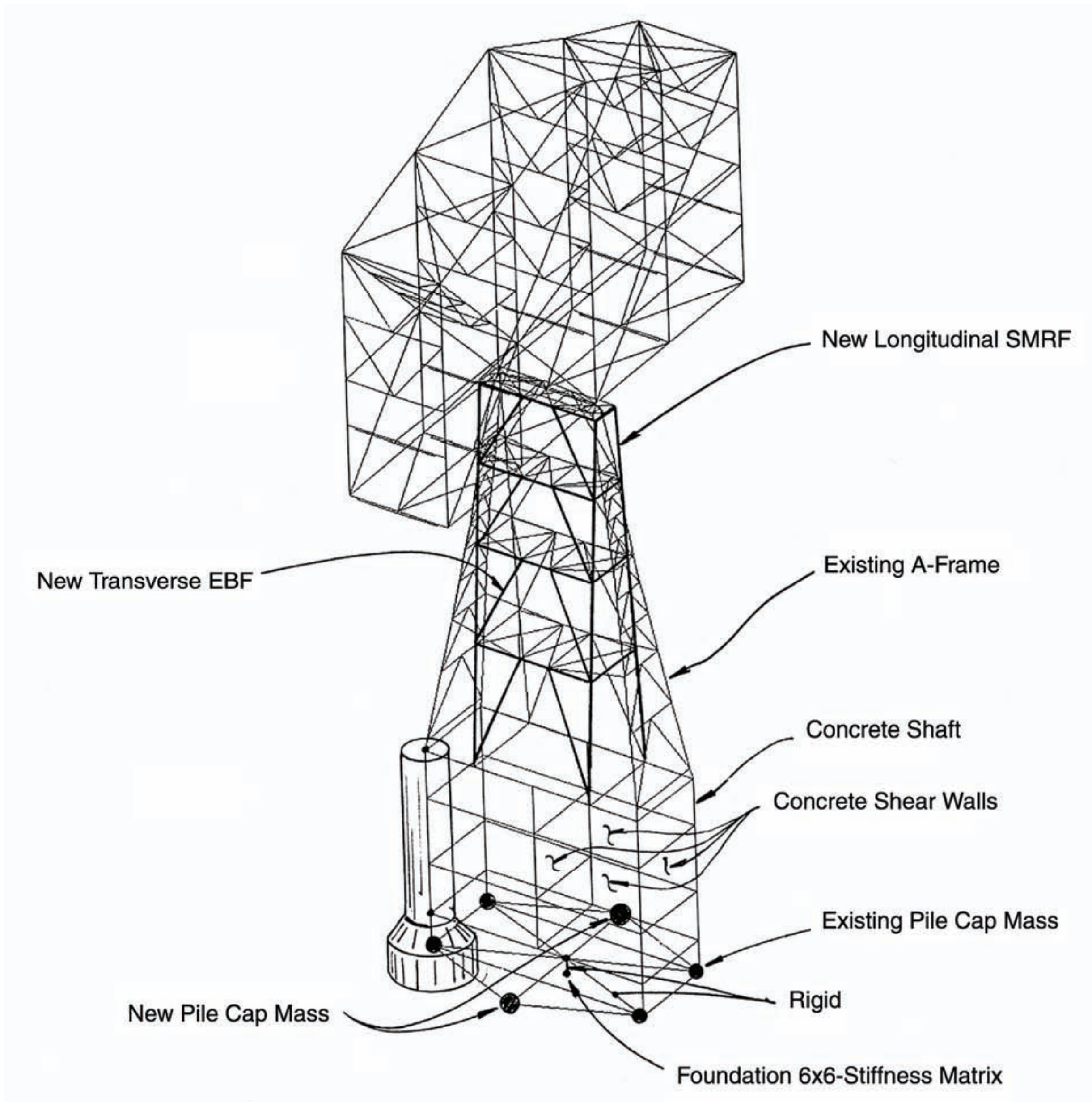

Figure 4-9. Typical Truss Bridge Pier Model

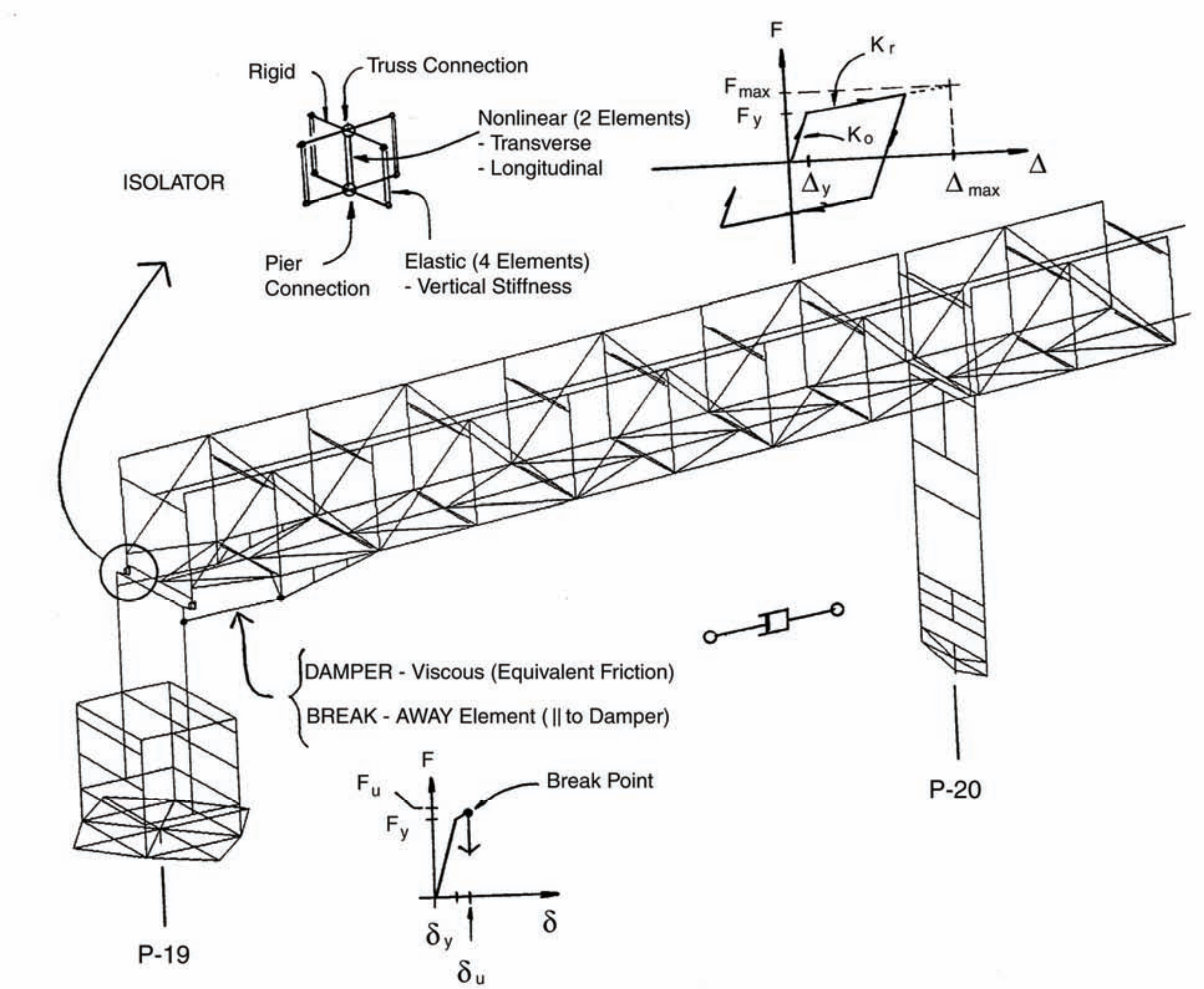

Figure 4-10. Typical Simple Truss Span Model With RMD Retrofit

4.3.5 EXPANSION JOINT IMPACT AND DAMPING DEVICES

Figure 4-11 depicts a typical split-bent tower with the behavior of the expansion joint represented by a "gap-crush" element which simulates opening and closure/impact of the expansion joint. This behavior is represented by the shifted bi-linear load-deflection relationship curve shown in the figure.

Also shown in the figure is the implementation of a fluid viscous damping devices placed across the joint as a retrofit measure intended to control the joint impact. The constitutive relation for the fluid viscous damping device is given by:

$$F = C\,|V|^n\,Sgn\,\{V\} \tag{4-5}$$

where: F = damper force; V = damper piston velocity; C = amplitude scaling coefficient; n = damper exponent; and $Sgn\{V\} = |V|/V$, i.e., sign function of V.

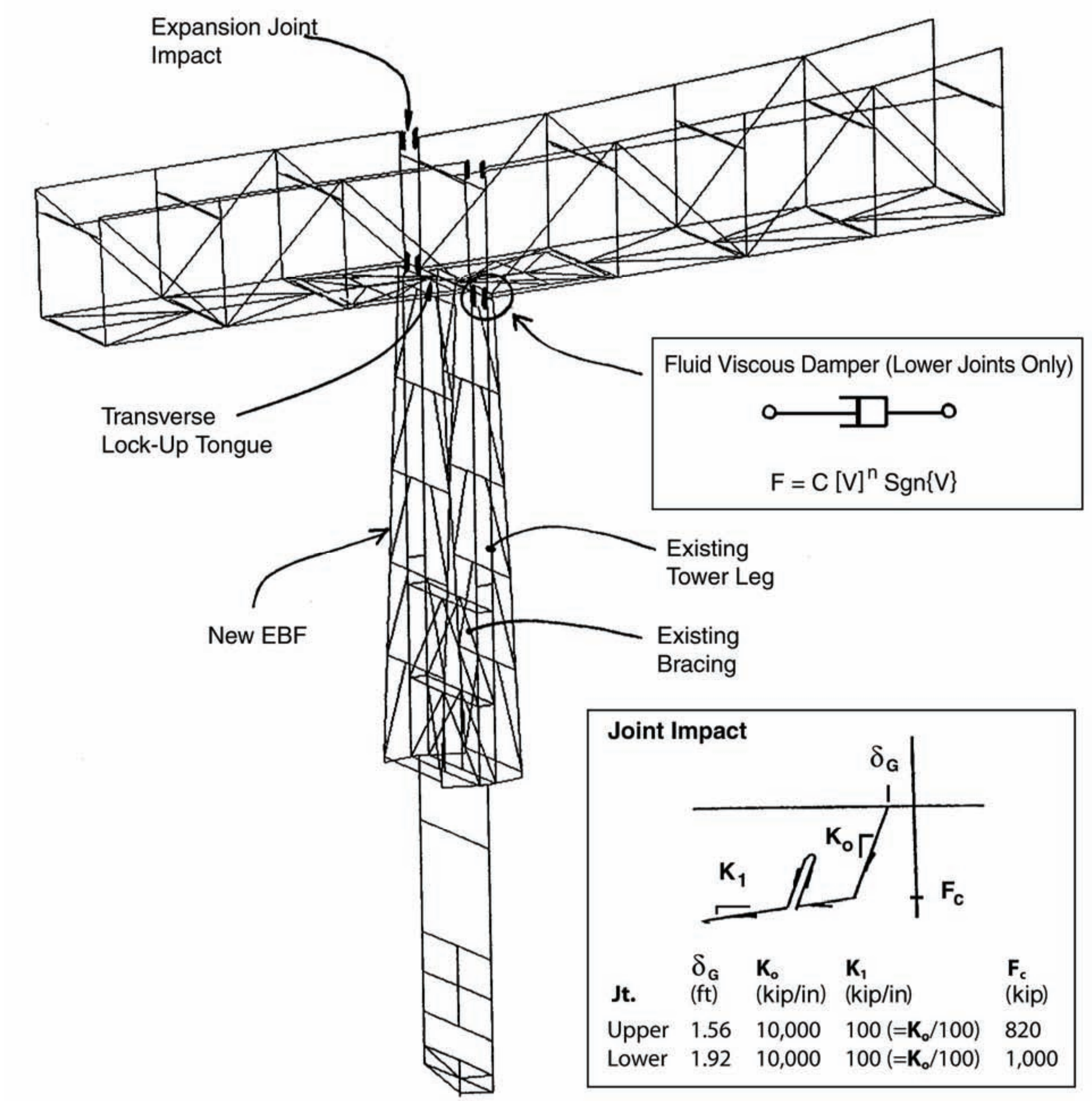

Figure 4-11. Expansion Joint Model With Viscous Damper

4.3.6 PROPERTIES OF BUILT-UP MEMBERS

The section properties of built-up members may be computed by integration over the cross-section, using the same mechanics of materials approach as that of solid sections. Laces used to connect different elements of a cross-section together usually do not contribute to the axial or the flexural stiffness of the section. The laces do contribute to the shear and torsional stiffness of a built-up member, however. Indeed, the laces are the major component of the shear and torsional stiffness of many members. Also, shear-induced deformations in laced members tend to considerably reduce the buckling strength of the members [Structural Stability Research Council 1988].

The diagonal member, shown in Figure 4-12, is a member of this type; the laces provide most of the shear stiffness in the horizontal direction, and the torsional stiffness of the member would be very small if the laces were absent.

Figure 4-12. Typical Laced Member

The *shear* and *torsional* properties of a laced member may be computed by the usual mechanics of materials formulas, if the laces are considered a plate with equivalent thickness:

$$t = 2(1+\nu)na_{lace}\frac{LT}{\sqrt[3/2]{L^2+T^2}} \tag{4-6}$$

where ν is Poisson's ratio, n is an integer based on the type of lacing, a_{lace} is the area of a single lacing bar, and L and T are the longitudinal and transverse spans of the bar. These latter parameters are illustrated in Figure 4-13.

The parameter n is 1 for single lacing, as illustrated in Figure 4-13; and it is 2 for double lacing, as in Figure 4-12. In general n is equal to the number of lacing bars crossed by a line perpendicular to the member axis; it may equal three or four in some unusual members.

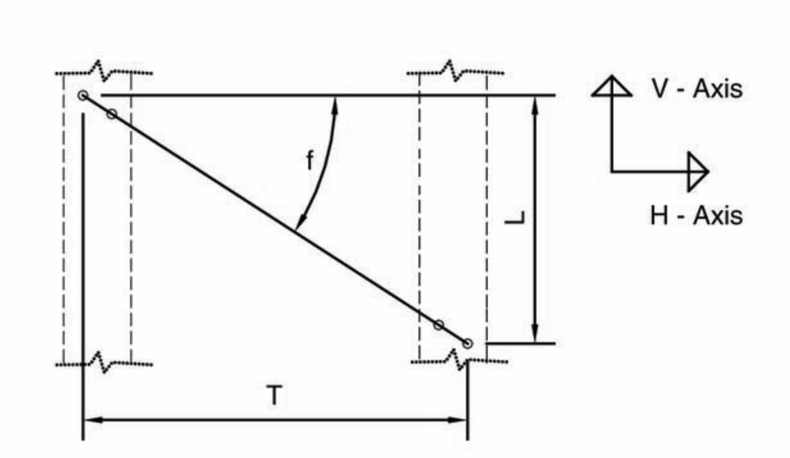

Figure 4-13. Lacing Bar Dimensions

Using this formulation the member shown in Figure 4-12 is equivalent to a closed box with two webs of normal thickness, and two thin webs corresponding to the laced faces. The torsional stiffness of the member may be computed from:

$$J = \frac{4A_{box}{}^2}{\oint \frac{ds}{t}} = \frac{4(ST)^2}{2\frac{S}{t_{web}} + 2\frac{T}{t_e}} \tag{4-7}$$

where S is the dimension of the web out-of-the-paper in Figure 4-13, and t_{web} is its thickness. The shear areas of the member are:

$$A_s = 2St_{web} \tag{4-8a}$$

$$A_t = 2Tt_e \tag{4-8b}$$

in the two principal directions. The shear areas of members should be utilized in computer programs that include the shear deformations of beam elements.

Where the shear areas of members cannot be included in the analysis, an alternative approach is given by [Duan et al., 2000], wherein the moments of inertia of members are reduced to account for the effects of the lacing.

The lace bars should also be considered as an added mass; the additional volume of material per unit length of member is:

$$A' = na_{lace}\sqrt{1+\left(\frac{T}{L}\right)^2} \tag{4-9}$$

which has the unit of area. Again, this area does not contribute to the axial stiffness of the member; it may be reflected in the mass by increasing the mass density of the material used to describe the member.

4.3.7 MODELING OF TENSION ONLY MEMBERS (EYEBARS)

Eye-bars and similar members constructed from flat plates may be unable to resist compression forces. Typically, these members are under dead load tension, and the issue of their compression capacity—or lack thereof—only arises if the dynamic force in a member exceeds its initial tension. In this case, an elastic analysis will predict a net compression in the member, which is not physically realizable.

It is a matter of judgment whether or not this modeling inaccuracy compromises the results to such an extent that they are invalid. Often the predicted compression will be small. The implications are minor buckling of the member—which should be elastic and reversible—and shedding of load to adjacent members.

If the predicted compression is large, steps need to be taken to improve the analysis. Within the context of a geometrically nonlinear time history analysis, this may be done by modeling the member with several elements—from two to five. Each element should have the cross-sectional properties of the whole member, including its small moment of inertia, about at least one principal axis. If the finite element program used for the analysis is able to perform geometrically nonlinear analysis, this series of elements will then buckle whenever the axial load in the member approaches the Euler load of the member. Of course, if the computer program has a tensile-only truss element, this may be preferable, subject to the limitations on global model stability discussed above.

4.3.8 MODELING OF BUCKLING OF COMPRESSION MEMBERS

It may occur that the predicted compression force (dead load plus dynamic force) in a compression member exceeds its compression capacity, which may be limited by buckling. In any case, the behavior is likely to be inelastic—either straightforward yielding or inelastic buckling—with the formation of a plastic hinge near the middle of the member. Again, a small overstress may be considered acceptable, if the member in question is reasonably compact and is able to withstand inelastic straining without too much degradation of response.

In cases of significant overstress, or if a member isn't compact**,** it is probably wise to model its inelastic response. This can be done by modeling the member with at least two elements to introduce a node near the middle of the member (similar to the modeling of eye-bars described above) and at this middle node a plastic hinging element is introduced. This is an element able to reproduce the force-moment yield interaction surface of the cross-section. Depending on the finite element program used, this plastic hinging element may be a conventional beam element with an inelastic material assigned to it, or it may be a special-purpose moment-curvature element in which the axial force is a parameter determining the moment-curvature relationship. Within the context of a geometrically nonlinear time history analysis, this type of model will properly limit the compression force in the member to its compression capacity, and will permit some estimate of the inelastic ductility demands on the member.

4.3.9 INELASTIC RESPONSE OF BUILT-UP MEMBERS

The ability of built-up members to sustain inelastic demands—either in compression or in flexure—should be viewed with skepticism. The ability of a cross-section to sustain inelastic strains depends on its compactness—on the compactness of its individual components in the case of a built-up member. The compactness of a built-up member also depends on how well the individual components are connected together—i.e., on the spacing of the stitch and sealing rivets—and on the span of the components between the points where they are connected together with laces. As pointed out in section 4.3.6, shear-induced deformations in laced members tend to considerably reduce the buckling strength of the members.

It is difficult to give general rules. Important cases, with significant inelastic demand(s), deserve careful investigation. Of course, the testing of large-scale models is a reliable method of proving the capability of a member. However, useful results can also be obtained by inelastic finite element analysis, if this is performed by experienced individuals. Such an analysis, of a solid and shell element model of a typical length of member, may need to include such refinements as: geometric nonlinearity, inelastic material (perhaps with Bauschinger effect), initial imperfections, and residual stresses, in order to obtain meaningful results.

CHAPTER 5: DESIGN PARAMETERS

5.1 GENERAL

This chapter covers the main design parameters pertaining to steel structural elements of truss bridges. In addition to superstructure truss members, these design parameters are also applicable to braced steel support tower members. The retrofit design considerations for concrete piers and substructure elements covered in the *Bridge Retrofit Manual* are fully applicable to truss bridge substructures, and therefore are not addressed here.

5.2 STEEL USED FOR TRUSS BRIDGES

Around the 1920s, steel plates and rolled sections conforming to ASTM specifications began to be supplied by steel mills. The ASTM specifications produced standards that steel manufacturers were required to meet and that bridge designers could specify with reliability.

Older bridges used cast iron, wrought iron, and steels of unknown type and strength. Before the development of ASTM specifications, most steel used in bridges was called “medium steel” for nominal strength levels**,** and “high-carbon steel” for higher strength requirements. During this same period, high-strength silicon and nickel steel were also developed. Steel eye-bars, essentially flat bars with large rounded ends that had holes, or “eyes,” were forged from higher-strength steel. They were used for tension-carrying truss members and were connected with forged steel pins through the hole. Most of these early steels are considered unweldable.

After the Second World War, low-alloy steels became available with higher strengths, and some were formulated for welding. In the 1950s, welded, high-yield-strength quenched and tempered steels were developed and were used as tension members in place of eye-bars, which were not produced after the war. Steels were produced without a toughness (Charpy Impact[1]) specification requirement. In the 1970s and 1980s, several bridges developed brittle fractures, resulting in collapses or closures for bridge repairs. Steel with toughness requirements are now part of ASTM Specifications A709. AASHTO bridge design specifications require different toughness values according to the temperature zone in which the bridge is located.

These *Guidelines* do not cover truss bridges fabricated from cast iron or from wrought iron. Bridges constructed using these older materials are historic structures and require special care to preserve their aesthetics and their historic significance, as well as their structural integrity, during seismic retrofit design and seismic retrofit construction.

The bridge designers who are undertaking a seismic retrofitting project should attempt to find out what steels were used in the various members of the truss. Often this information is shown on the construction plans. If the steel is not covered by the ASTM**,** or if the construction plans or specifications are not available and the steel is unknown, steel samples can be cut from the truss

[1] As in ASTM E23-00

in low-stressed areas for laboratory testing for physical properties. After the type of steel or the properties of the steel are known, the principles in these *Guidelines* can be applied. As with most bridge design projects, the designer will have to apply fundamental engineering principles and exercise good engineering judgment to the steel members of the truss during the seismic retrofit process. In the absence of test data, the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*, [AASHTO 2003] contains recommendations for material properties based on the year of construction of a structure.

As guidance to the bridge designer, Table 5-1 presents the most common steels listed under an ASTM Designation and used in bridge construction since about 1925.

After 1933, rivet steel was usually specified as ASTM A141 – Specification for Rivet Steel. This ASTM Specification was withdrawn in 1967 and replaced with ASTM A502, which was withdrawn in 1999 with no replacement. The ASTM A141 requirement for ultimate tensile strength was 52-ksi minimum and 62-ksi maximum, with a minimum yield point of 50 percent of the tensile strength**,** but not less than 28 ksi.

For materials not covered herein, for unknown materials, and for very old bridges, additional advice and presumptive material properties are given in Section 6.6.2 of the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges* [AASHTO 2003].

5.2.1 NOMINAL, EXPECTED, AND OVER-STRENGTH MATERIAL PROPERTIES

Material property parameters relevant to structural member strength evaluations are conventionally characterized in terms of statistically distributed strength designations, such as: Nominal Strength, Expected Strength, and Over Strength. Nominal Strength refers to conservatively estimated material properties reflecting values in the low-percentile range of the statistical distribution (less than 5%) or minimum published values. Expected Strength is a representation of the mean value of the statistical distribution, i.e., 50-th percentile. Over Strength defines the upper percentile range of the distribution, where the probability of being exceeded is low. Over Strength is commonly expressed as a factored value of Nominal Strength.

For seismic retrofit projects, evaluation and rehabilitation design strategies are based on the objective of a “best-estimate” assessment of the overall structural response. This is especially the case where the evaluation is based on the assessment of the displacement capacity or ductility capacity of the structure. In this case, expected strength parameters are appropriate for use in material property calculations. In the absence of laboratory test data of core samples obtained from the actual bridge**,** or mill certificates for steel structural members and reinforcement, expected strength parameters are estimated as factored values of the nominal or specified strength. Commonly, the following factored values are used for the yield strength of steel, f_y, and the compressive strength of concrete, f'_c [ATC 1996]

$$f_y = 1.1 f_{yn} \text{ for steel} \tag{5-1}$$

$$f'_c = 1.3 f'_{cn} \text{ for concrete} \tag{5-2}$$

Table 5-1a. Steel Grades

ASTM Designation	ASTM Description	Comments	Plate Thickness	Yield Point or Yield Strength, ksi	Tensile Strength, ksi
A6	General Requirements for Plates, Shapes, Sheet Piling, and Bars for Structural Use.	Covers a group of common requirements that apply to steel plates, shapes, sheet piling, and bars to many ASTM- listed steels and to all of the steels listed in this Table			
A7	Steel for Bridges and Buildings. Discontinued in 1967.	Specified for most riveted and bolted bridges until A36 was developed as a weldable steel.		½ Fu (1914-1923) ½ Fu $\geq$ 30 (1924-1933) ½ Fu $\geq$ 33 (1934-1967)	55 – 65 (1914-1923) 55 – 65 (1924-1933) 60 – 72 (1934-1967)
A36	Structural Steel. Now listed under A709 as Grade 36	Steel for riveted, bolted, or welded construction of bridges and buildings	To 8”	36	58-80
			Over 8”	32	58-80
A94	Structural Silicon Steel. Discontinued in 1966.	Generally considered unsuitable for welding.			
A242	High-Strength Low-Alloy Structural Steel.	Steel for riveted, bolted, or welded construction where savings in weight and enhanced corrosion resistance are important	1½” to 4”	42	63
			¾” to 1½”	46	67
			To ¾”	50	70
A440	High-Strength Structural Steel. Discontinued in 1979.	Generally considered unsuitable for welding. Specified for bolted trusses only.			
A441	High-Strength Low-Alloy Structural Manganese Vanadium Steel.	Steel for welded bridges and buildings where savings in weight and enhanced corrosion resistance are important	4” to 8”	40	60
			1½” to 4”	42	63
			¾” to 1¼”	46	67
			To 1¼”	50	70
A514	High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding. Now listed under A709 as Grade 50	Developed after the 1945 as a super-high strength (100 ksi, 690 MPa), weldable steel for tension and compression members in long-span trusses	2½” to 6”	90	100-130
			To 2½”	100	110-130
A572	High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality.	Steel Grades 42 and 50 specified for welded bridges and buildings. Grades 60 and 65 specified for riveted or bolted construction of bridges	To 6”	42	60
			To 4”	50	65
			To 1¼”	60	75
			To 1¼”	65	80
A588	High-Strength Low-Alloy	Steel for welded bridges and buildings where	4” to 8”	42	63

Table 5-1a. Steel Grades

ASTM Designation	ASTM Description	Comments	Plate Thickness	Yield Point or Yield Strength, ksi	Tensile Strength, ksi
	Structural Steel with 50 ksi (345 MPa) Minimum Yield Point to 4 in (100 mm) Thick. Now listed under A709 as Grade 50	savings in weight and enhanced corrosion resistance are important	4” to 5”	46	67
			To 4”	50	70
A709	Carbon and High-Strength Low Alloy Structural Steel Shapes, Plates, and Bars and Quenched and Tempered Alloy Structural Steel Plates for Bridges.	Currently the steel of choiace for the design of most new bridge projects. Several choices of strengths, toughness, and corrosion resistance. All Grades are weldable and offer good ductility.	To 4”	36	58
			To 4”	50	65
			To 4”	50W	70
			To 4”	70W	90
			To 2½”	100	110
			2½” to 4”	90	100

Table 5-1b. Steel Grades

ASTM Designation	ASTM Description	Comments	Grade or Diameter	Yield Point or Yield Strength, ksi	Tensile Strength, ksi
A27	Steel Castings, Carbon, for General Application		U60-30	60	30
			60-30	60	30
			65-35	65	35
			70-36	70	36
			70-40	70	40
A237	Alloy Steel Forgings for General Industrial Use	Tensile and yield strengths are the minimum in each grade, for thick material. Thin material may be slightly stronger.	A	50	80
			B	55	80
			C	58 min	90
			C1	85	110
			D	65 min	90 min
			E	70 min	95 min
			F	85 min	110 min
			G	110 min	135 min
			H	130 min	160 min
A141	Structural Rivet Steel	Superseded by A502, Gr. 1		28	52-62
A195	High-Strength Structural Rivet Steel	Superseded by A502, Gr. 2		38	68-82
A502	Steel Structural Rivets		1	28	52-62
			2	38	68-82
A307	Low-Carbon Steel Externally and Internally Threaded Standard Fasteners			NA	60
A325	High-Strength Bolts for Structural Steel Joints, Including Suitable Nuts and Plain Hardened Washers		½ to 1”	NA	120
			To 1½”	NA	105
A490	Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints		½ to 1½”	NA	150

where f_{yn} and f'_{cn} are nominal or specified values.

Evaluation and design of new structural steel members used for seismic retrofit should utilize specified material properties.

5.3 DESIGN METHODS

Materials in a bridge respond to the laws and forces of nature and not to the method of design. However, the method of design is important when retrofitting a bridge, as different design methods produce slightly different placements, amounts of materials, and different force levels within the structure. Over the years, the AASHTO specifications for bridge design have undergone three significant changes in its approach to bridge design practice.

5.3.1 ALLOWABLE STRESS DESIGN (ASD)

Starting with the first edition in 1927, the AASHTO bridge design specifications have been historically based on the Allowable Stress Design (ASD) philosophy. Many truss bridges have been designed using the ASD method, since this was the only design method available during the time when truss bridges were the popular structure of choice for highway bridges. The ASD method usually assumed frictionless pin connections and the structural system was usually statically determinant for ease of slide rule and manual analyses.

If available, the design plans may contain a “stress sheet” that lists the elastic stress levels for dead and live loads, wind and earthquake, if any. We know today that earthquake loads were greatly underestimated during the early periods of bridge design. A truss bridge seismic retrofit designer should treat the stresses on the stress sheet with caution, as they may not be reliable for a seismic evaluation of the bridge. The seismic retrofit designer of complex truss bridges should use a modern structural program specifically designed to analyze the seismic demands placed on the bridge structures. The “stress sheet” may be useful for validating a computer model for dead load, however.

5.3.2 LOAD FACTOR DESIGN (LFD)

In 1967, AASHTO introduced the Load Factor Design (LFD), an alternative design method using load factors and a limit-state design approach. The change-over from ASD to LFD was very slowly adopted by designers and, although the LFD method was available to designers, many truss bridges continued to be designed by ASD.

5.3.3 LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

In 1994, AASHTO introduced another alternative design procedure called Load and Resistance Factor Design (LRFD), which is an extension of the LFD limit-state design philosophy with load and resistance factors developed through a statistical approach. The first edition of the AASHTO *LRFD Bridge Design Specification* was updated by the publication of a second edition in 1998 and a third edition in 2004. Load factors and resistance factors are calibrated to achieve a uniform reliability index for span lengths, bridge types, and the statistical variability of the loads and materials. Ductility, redundancy, fatigue, fracture, and operational importance are the bases of the LRFD method of design. It is the design method of choice today. AASHTO has voted to discontinue maintenance of the Standard Specifications for Highway Bridges (ASD and LFD) and fully implement LRFD by the year 2007.

New seismic provisions using LRFD approach were published in 2003 with the "*Recommended LRFD Guidelines for the Seismic Design of Highway Bridges*" [MCEER/ATC 2003]. Though not adopted by AASHTO, it is a nationally applicable standard with provisions for all seismic zones and all bridge types and materials.

5.3.4 CHOICE OF DESIGN METHOD FOR SEISMIC RETROFITTING

For highway truss bridges undergoing seismic retrofit, the design method and the year of, or the edition of, the AASHTO bridge design specifications used for the design usually can be found on the first few sheets, or on the stress sheet, of the original contract drawings. Sometimes the original contract drawings are not available and the method of design must be assumed by comparing the year of construction with the development of the AASHTO specifications.

Both the *Bridge Retrofitting Manual* and these *Guidelines* are based on the LRFD method of design.

5.4 CATEGORIZATION OF MEMBERS

Older truss bridge members are often small rolled sections, round rods, flat bars, and eyebars. Eyebars were used for tension members exclusively because the pins allowed easily-made connections to the gusset plates. When larger members were required, flat plates, rolled sections, and angles were riveted together, usually in the form of I beams or H-shapes**,** or in an open box-shape with plates on three sides and the fourth side covered with a lattice of bars or angles. Many member have lacing on two sides.

After the Second World War, truss plate units were usually welded together to form members with an H-shape**,** or a box-shape with plates on all four sides. Generally one, and sometimes two, plates on opposite sides to each other were perforated with large rectangular cutouts to allow the painting of inside surfaces. In the last 25 years the box sections were often welded with solid plates and with welded diaphragms at each end, which sealed the box to prevent inside corrosion.

Members are categorized as follows:

1. Main members with no redundant load path. The criteria for these members are strict, since any loss of capacity of these members would seriously reduce the structural integrity of the bridge.
2. Main members with redundancy. The criteria for these members are less strict than for non-redundant members, since any loss of capacity of these members would have a less debilitating effect on the structural integrity of the bridge.
3. Bracing and Secondary Members. This classification includes members that primarily brace other members. The criteria for these members are more liberal than for the main members.

Members are also categorized depending on whether or not they carry dead load:

A. Members that carry dead loads. Inelastic response of members that carry dead load should be *limited* and in *flexure* only.
B. Members that don't carry dead loads.

This latter categorization is necessarily subjective, since most members are subjected to at least minor stresses under dead load. For the purposes of these guidelines, however, members with dead load forces (or moments) exceeding 15% of the corresponding capacity are considered to carry dead loads.

The permitted ductility demands, limiting member-slenderness ratios**,** and component width-to-thickness ratios depend on the category of the member. Existing members, if deficient, should be retrofitted to meet the stated criteria and new members, required for retrofitting the structure, shall be designed to meet the stated criteria.

5.4.1 BRACING AND SECONDARY MEMBERS

Inelastic response of bracing members, where axial ductility is achieved by formation of a plastic hinge near the middle of the member, is permitted. The distribution of load between tension and compression braces, and other participating members, shall be determined by inelastic analysis.

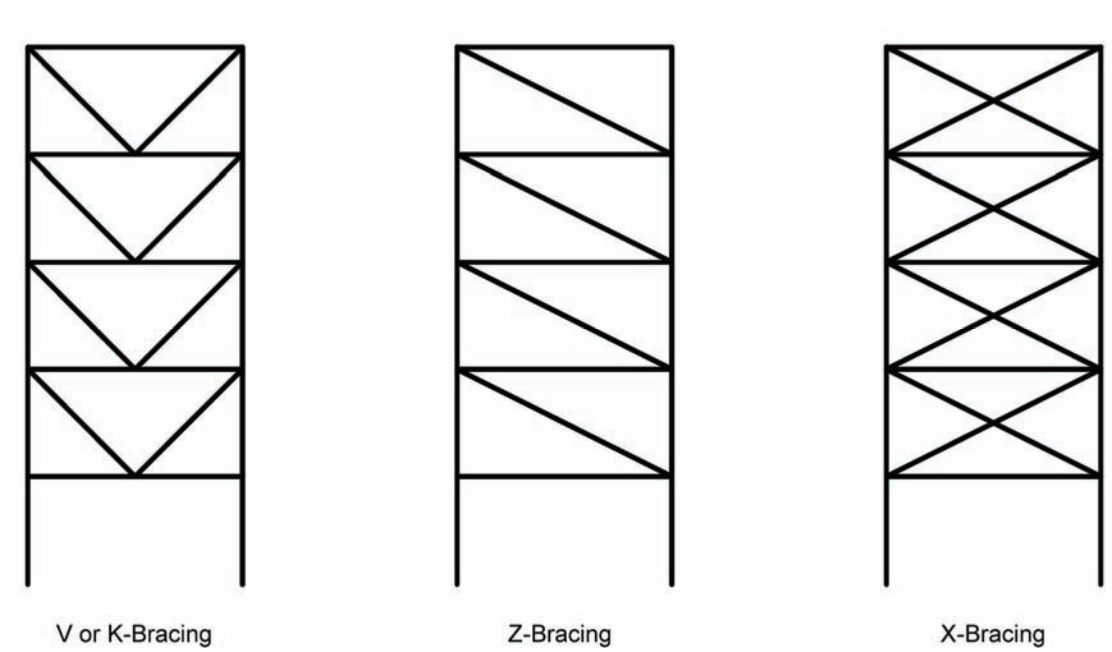

Figure 5-1. Bracing Types

Inelastic response of V-braced or K-braced systems, or other systems where tension forces in the braces are resisted by compression braces only, should be viewed with caution. New lateral bracing systems shall be of the X-braced or Z-braced type only, so that tension forces in the braces are resisted by frame elements other than the compression braces. The arrangement of braces shall be symmetrical about a vertical axis.

5.5 DESIGN REQUIREMENTS

5.5.1 EFFECTIVE LENGTH FACTORS

Truss members shall be evaluated using effective length factors provided in Table 5-2. The different web systems referred to in the table are illustrated in Figure 5-2.

Table 5-2. Effective Length Factors, K

Members		**Buckling in plane of Truss**	**Buckling out of plane of Truss**
Chord		0.85	0.85
Web	Single Triangulated System	0.70	0.85
	Multiple Inter-section System	0.85	0.70

The factors assume that truss chords are effectively braced by a system of lateral braces. For chords and web members in a single triangulated system, the lengths of members are to be taken between the points of intersection of the members. For webs in a multiple intersection system, the lengths of members are to be taken between successive intersections of the members (if they are adequately connected) when considering in-plane buckling; and between intersections with the chords when considering out-of-plane buckling.

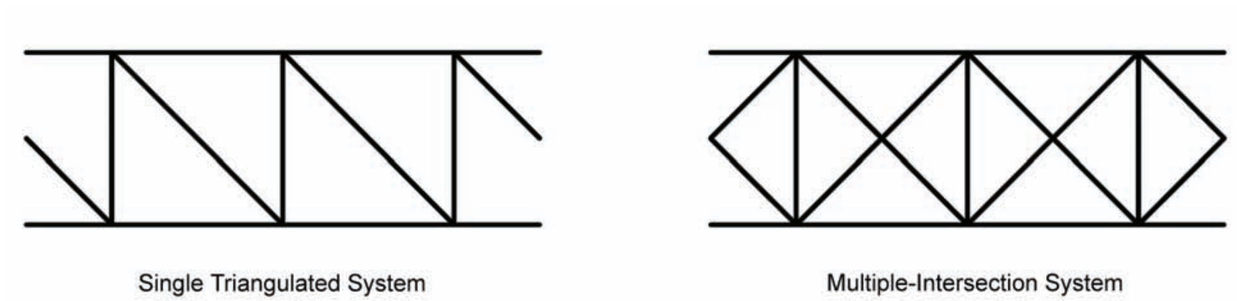

Figure 5-2. Web Systems

The effective length factors of laced members shall be increased to account for the effect of the laces on the stability of the members as described in Section 6.2.1.

5.5.2 LIMITING WIDTH-TO-THICKNESS RATIOS

The width-to-thickness ratios of components of cross-sections must be limited to prevent local buckling during both elastic and inelastic deformation, and during both flexure and compression. The width-to-thickness ratio of a component is defined by the applicable of Eqs. (5-3). For a plate, or for the outstanding leg of an angle or flange:

$$\lambda = \frac{b}{t} \tag{5-3a}$$

where b is the width of the element and t is its thickness. For a flange, b is one-half of the total width, i.e., the outstanding width of the flange. For a web:

$$\lambda = \frac{h}{t} \tag{5-3b}$$

where h is the height of a web. For a circular section:

$$\lambda = \frac{D}{t} \tag{5.3c}$$

where D is the diameter.

The width-to-thickness ratios of cross section components shall be limited to the values given in Table 5-3. The various limiting ratios appearing in the table are defined in Table 5-4.

Table 5-3. Limiting Width-to-Thickness Ratios

Members	**Deformation Ductility, μ**	**Group**	**Limiting Ratio**
Existing	≤ 1.0	main, non-redundant	λ_r (1)
		main, redundant	λ_r (1)
		secondary, bracing	λ_r (1)
	μ ≥ 1.0	main, non-redundant	λ_p
		main, redundant	λ_p
		secondary, bracing	λ_p
New	≤ 1.0	main, non-redundant	λ_p
		main, redundant	λ_p
		secondary, bracing	λ_p
	μ ≥ 1.0	main, non-redundant	λ_s
		main, redundant	λ_p
		secondary, bracing	λ_p

(1) Lower slenderness ratios may be used if members are evaluated according to the requirements of the Appendix B of the Manual of Steel Construction, or Section 6.9.4.2 of LRFD Bridge Design Specifications.

The limiting width-to-thickness ratios λ_r, λ_p, and λ_s are given in Table 5-4.

Table 5-4. Width-to-Thickness Ratios (ksi units)

Member	**Ratio**	λ_r **(1)**	λ_p **(1)**	λ_s **(2)**
Flanges of I-shaped rolled beams and channels in flexure.	b/t	$\frac{141}{\sqrt{F_y - 10}}$	$\frac{65}{\sqrt{F_y}}$	$\frac{52}{\sqrt{F_y}}$
Flanges of I-shaped hybrid or welded beams in flexure.	b/t	$\frac{106}{\sqrt{F_{yw} - 16.5}}$	$\frac{65}{\sqrt{F_{yf}}}$	$\frac{52}{\sqrt{F_{yf}}}$
Flanges of I-shaped sections in pure compression; plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact; flanges of channels in pure compression.	b/t	$\frac{95}{\sqrt{F_y}}$		
Flanges of square and rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds.	b/t	$\frac{238}{\sqrt{F_y - F_r}}$	$\frac{190}{\sqrt{F_y}}$	$\frac{110}{\sqrt{F_y}}$ (3)
Unsupported width of cover plates perforated with a succession of access holes.	b/t	$\frac{317}{\sqrt{F_y - F_r}}$		
Legs of single angle struts; legs of double angle struts with separators; unstiffened elements, i.e., supported along one edge. (4)	b/t	$\frac{76}{\sqrt{F_y}}$		
Stems of tees (4)	d/t	$\frac{127}{\sqrt{F_y}}$		
All other uniformly compressed stiffened elements, i.e., supported along two edges.	h_c/t_w	$\frac{253}{\sqrt{F_y}}$		

Table 5-4. Width-to-Thickness Ratios (ksi units)

Member	**Ratio**	λ_r **(1)**	λ_p **(1)**	λ_s **(2)**
Webs in flexural compression	h_c/t_w	$\frac{970}{\sqrt{F_y}}$	$\frac{640}{\sqrt{F_y}}$	$\frac{520}{\sqrt{F_y}}$
Webs in combined flexural and axial compression	h_c/t_w	$\frac{970}{\sqrt{F_y}}$	For $P_u \le 0.125\phi_b P_y$ $\frac{640}{\sqrt{F_y}}(1-\frac{2.75P_u}{\phi_b P_y})$ For $P_u > 0.125\phi_b P_y$ $\frac{191}{\sqrt{F_y}}(2.33-\frac{P_u}{\phi_b P_y}) \ge \frac{253}{\sqrt{F_y}}$	For $P_u \le 0.125\phi_b P_y$ $\frac{520}{\sqrt{F_y}}(1-\frac{1.54P_u}{\phi_b P_y})$ For $P_u > 0.125\phi_b P_y$ $\frac{191}{\sqrt{F_y}}(2.33-\frac{P_u}{\phi_b P_y}) \ge \frac{253}{\sqrt{F_y}}$
Circular hollow sections in compression	D/t	$\frac{3300}{F_y}$	$\frac{2070}{F_y}$	$\frac{1300}{F_y}$ (3)
Circular hollow sections in flexure	D/t	$\frac{8970}{F_y}$	$\frac{2070}{F_y}$	

(1) These values are from the AISC LRFD [AISC 2001]
(2) These values are from the AISC Seismic Provisions for Structural Steel Buildings [AISC 1997]
(3) These values are from the Improved Seismic Design Criteria for California Bridges: Provisional Recommendations [ATC 1996]
(4) New flexural members shall not be constructed from these elements.

5.5.3 RESISTANCE FACTORS

Resistance factors shall be in accordance with article 6.5.4.2 of the AASHTO LRFD [AASHTO 2004] except as modified in Table 5-5. See Section 5.4 regarding the categorization of members.

Table 5-5. Resistance Factors

Action/Element	**Resistance Factor, ϕ**
Compression, member carries dead load	0.90
Compression, member doesn’t carry dead load	1.00
Tension, yielding of gross section	1.00

CHAPTER 6: EVALUATION OF MEMBERS, CONNECTIONS AND SUBSYSTEMS

6.1 GENERAL

When preliminary evaluation of a highway truss bridge shows potential seismic deficiency – as determined by a vulnerability assessment described in Section 4.1.1 – the next step is the performance of an analysis and then a detailed evaluation, using one or more of the methods described in this section in conjunction with the analytical procedures discussed in Chapter 4. The evaluation methods and steps are also discussed in the *Bridge Retrofitting Manual*, which lists six steps for ordinary bridges. For SC trusses the methods and steps are reduced to the following three:

- Individual truss member performance evaluation in terms of strength and ductility.
- Connection performance in terms of strength.
- Subsystem performance in terms of displacement ductility.

Methodologies for evaluating the strength and/or ductility of members, connections, and subsystems are discussed in this chapter.

The detailed seismic evaluation of a steel truss highway bridge is a two-part process consisting of a demand analysis and a capacity assessment. A demand analysis is required to determine the forces and displacements imposed on the structure by the design-earthquake. Analytic methods for the modeling of SC truss bridges are covered in Chapter 4 of these *Guidelines*. This demand analysis is then followed by an assessment of the capacity of the truss and its components. Most evaluation methods express their results as Capacity/Demand (C/D) ratios, calculated on a component-by component basis, or for structural subsystems. Where demands and capacities are expressed in terms of forces, a C/D ratio above 1.0 implies elastic response, whereas a C/D ratio below 1.0 implies some damage as a result of inelastic deformation, and a possible need to retrofit the component and/or the structure. Section 6.9 has a detailed discussion on the subject of C/D ratio.

In assessing the capacities of various structural members of existing steel truss bridges that require retrofit, the condition of the existing components must be factored in. A thorough inspection of the entire structure is required in order to assess the extent of deterioration of individual members and their connections, as well as cross-section material losses through corrosion. The degree of deterioration must be accounted for in the member and connection capacity (as well as stiffness) assessments that are presented in the sections that follow. The reader is referred to the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges*, [AASHTO 2003], for a quantitative approach to this issue.

6.2 MEMBER CAPACITY

6.2.1 AXIAL STRENGTH

6.2.1.1 Prismatic Members

The tensile capacity of prismatic members made from rolled shapes or plates welded together—e.g., to form a box section—may be computed using the *AASHTO LRFD Specifications* [AASHTO 2004]. The nominal tensile capacity is the lesser of the gross section capacity and the net section capacity, which are, respectively:

$$P_n = f_y A_g \tag{6-1a}$$

$$P_n = f_u A_n U \tag{6-1b}$$

where:

f_y = yield strength of the steel
f_u = tensile strength of the steel
A_g = gross area of the section
A_n = area of the net section
U = reduction factor for shear lag, see section 6.8.2.2 of the AASHTO LRFD.

The compressive capacity of a member with a *non-slender* cross-section may be computed using the *AASHTO LRFD Specifications* and/or the following:

$$\text{If } \lambda \le 2.25\text{, then } P_n = 0.66^{\lambda} f_y A_g \tag{6-2a}$$

$$\text{If } \lambda > 2.25\text{, then } P_n = \frac{0.88 f_y A_g}{\lambda} \tag{6-2b}$$

where:

$$\lambda = \left(\frac{K\,l}{\pi\,r}\right)^2 \frac{f_y}{E} \tag{6-3}$$

with:

K = the effective length factor, in the direction considered
l = the unbraced length of the member
r = the radius of gyration of the section, about the axis perpendicular to the direction considered
E = the modulus of elasticity of steel, 29,000 ksi

The compressive capacity should be computed for both of the principal axes of the member, with the smaller value controlling.

A non-slender cross-section is one in which each of the component parts of the section, i.e., the flanges, webs, etc., satisfy the λ_r requirements of Table 5-5. Thus the yield strain of the material may be realized without local buckling of the parts of the section.

The compressive capacity of a member with a slender cross-section may be computed following the procedure described in Appendix B of the Load and Resistance Factor Design Specification for Structural Steel Buildings [AISC 2001]. For unstiffened elements of a cross-section, e.g., for the leg of an angle, a reduction factor Q_s is a function of width-to-thickness ratio. For the leg of an angle, for instance,

$$Q_s = 1.415 - 0.00437\frac{b}{t}\sqrt{f_y} \text{ if } \frac{95.0}{\sqrt{f_y}} < \frac{b}{t} < \frac{176}{\sqrt{f_y}} \tag{6-4a}$$

$$Q_s = \frac{20000}{f_y\left(\frac{\text{b}}{\text{t}}\right)^2} \text{ if } \frac{176}{\sqrt{f_y}} \le \frac{b}{t} \tag{6-4b}$$

This factor reflects the reduced critical stress of elements with width-to-thickness ratio greater than λ_r.

For stiffened elements, a reduction factor is defined as a function of stress $Q_a(f)$. For the flange of a box section, for instance,

$$Q_a(f) = \frac{b_e(f)}{b} = \frac{326}{\sqrt{f}}\frac{t}{b}\left(1 - \frac{64.9}{\frac{b}{t}\sqrt{f}}\right) \tag{6-5a}$$

with:

b_e = the effective width of the flange (or web),

b = the actual width of the flange (or web)

This factor reflects the effective width of slender elements subject to stress *f*. For a cross-section with more than one slender flange or web,

$$Q_a(f) = \frac{\sum b_e(f)t}{\sum bt} \tag{6-5b}$$

where the summation is over the slender plates. When a cross-section has both unstiffened and stiffened elements the reduction factor is $Q = Q_s Q_a$.

The compressive capacity of a member with a slender cross-section is

$$\text{If } Q\lambda \le 2.25\text{, then } P_n = Q\left(0.66^{Q\lambda}\right) f_y A_g \tag{6-6a}$$

$$\text{If } Q\lambda > 2.25\text{, then } P_n = \frac{0.88 f_y A_g}{\lambda} \tag{6-6b}$$

If the cross-section has stiffened elements and Eq. (6-6a) controls, an iterative solution is required to find the critical stress $f_{cr} = Q(f_{cr})\left(0.66^{Q(f_{cr})\lambda}\right) f_y$. The capacity of the member is unaffected if it is itself slender, so that it buckles globally before the onset of local buckling. Thus, Eq. (6-6b) is identical to Eq. (6-2b). Note that the area A_g in Equations (6-6) is the gross area of the cross-section; all reductions are built into the factors Q_s and Q_a.

6.2.1.2 Built-Up Members

The members of old truss bridges were often built-up out of plates, channels, and angles, as illustrated in Figure 6-1. The behavior of these members is *generally* similar to that of rolled shapes and their tensile and compressive capacity may be computed in accordance with the previous section.

Figure 6-1. Typical Built-Up Member

One difference between rolled shapes and built-up members is that built-up members have a net section within the body of the member. Wherever its elements are connected together, a hole is created, and hence a net section is created also, see Figure 6-2. Also, the flanges and webs of heavy members are often constructed from multiple plies, and the connections of these plies together creates a net section. This is the case in Figure 6-2, for instance, where the rivets in the middle of the web are evidence of multiple plies. These net sections should be checked for fracture using the same methodology employed for connections.

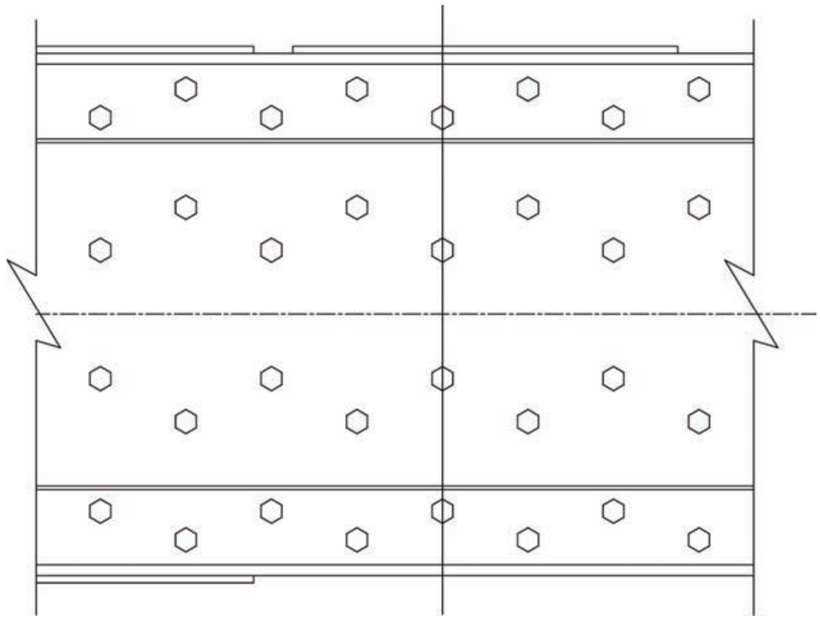

Figure 6-2. Net Section of Built-Up Member

According to the *AASHTO LRDF Specifications* [AASHTO 2004], the slenderness ratios of elements between connection points must not exceed 75% of the controlling slenderness ratio of the whole member, for the code provisions to be applicable. This requirement applies to a member like that shown in Figure 6-3, where the connection points must be spaced so as to meet the requirement. If the requirement is not met, the member falls outside of the scope of the code, and more sophisticated techniques must be used to compute the capacity of the member.

As a practical matter, the compressive capacity of a member such as that shown in Fig. 6-3 may be small and of little importance in the overall behavior of a truss. It may suffice to treat the member as a tension-only member (when the code requirements aren't satisfied); its tensile capacity, at least, can be computed from Eq. (6-2).

6.2.1.3 Laced Members

The axial capacity of laced members is not treated in modern bridge design codes, including the *AASHTO LRDF Specifications*. Nevertheless, the tensile capacity may be computed from Equations (6-1). The gross area to be used in Equations (6-1) and (6-2) is only the area of the longitudinal elements of the cross-section. In the diagonal member shown in Figure 6-1, for instance, the web plates and angles contribute to the gross area, but the laces do not.

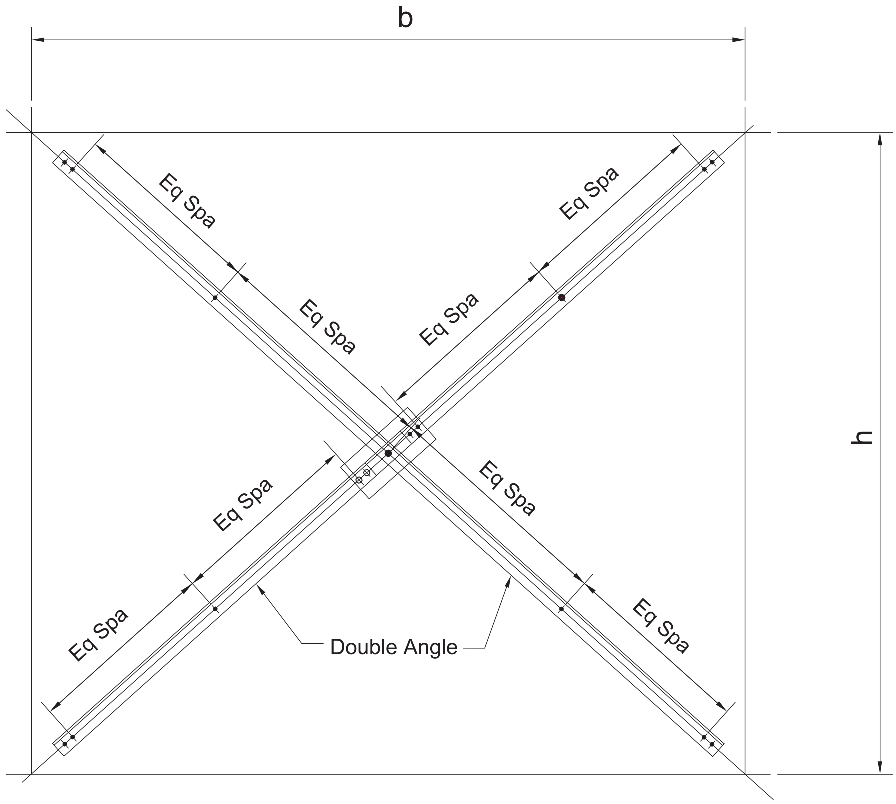

Figure 6-3. Member Built-Up from Angles

Similarly to built-up members, particular attention should be paid to the net section capacity of laced members. The gross section is reduced, not only at the connections of the member, but at each of the rivets connecting the lacing bars to the longitudinal elements of the cross-section.

The compressive strength of laced members shall be the minimum of the:

- Member strength and stability, based on the member's KL/r .
- Component strength and stability, based on the component's $a/r_{\min}$ between the points of attachment of the laces.
- Section strength and stability, based on the controlling (largest) width-to-thickness ratio, b/t or h_c/t_w .

The compressive capacity of laced members may be computed from Equations (6-2), with some modifications and limitations. Because lacing bars are usually small in section, laced members are more flexible in shear than are members made from rolled shapes, for instance. This additional shear flexibility reduces the compressive capacity of the member. According to [Structural Stability Research Council 1988] the reduced capacity may be obtained by using a modified effective-length factor in Equations (6-7):

$$\text{For } \frac{Kl}{r} > 40\,,\ K' = K\sqrt{1+\frac{300}{\left(KL/r\right)^2}} \tag{6-7a}$$

$$\text{For } \frac{Kl}{r} \leq 40\,,\ K' = 1.1K \tag{6-7b}$$

In a well-proportioned laced member, the slenderness of the longitudinal components of the member, between connections of the lacing bars, will be less than the overall slenderness of the member. This limitation is illustrated in Figure 6-4, where the slenderness ratio of the longitudinal element is $a/r_{\min}$ where a is the distance between lacing bar connections and where $r_{\min}$ is the minimum radius of gyration of the longitudinal element of the member.

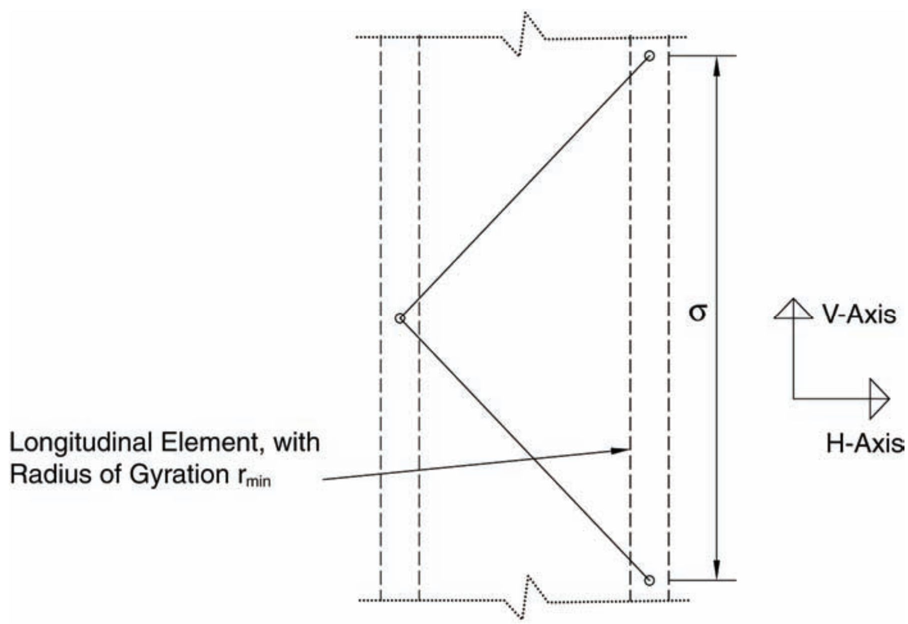

Figure 6-4. Slenderness of Longitudinal Elements of a Laced Member

The limitation on the slenderness of the longitudinal elements is to prevent local buckling of the member before the onset of global buckling, and also to minimize interaction between the local and the global modes of buckling—which interaction can potentially reduce the compressive capacity of the member. According to the *AASHTO Standard Specifications* [AASHTO 2002] the slenderness ratio of longitudinal elements must satisfy

$$\frac{a}{r_{\min}} \leq \min\{40, \frac{2}{3}\frac{Kl}{r}\} \tag{6-8}$$

If this limitation is satisfied, the compressive capacity of a laced member may be computed from Equations (6-2).

If the limitation on the slenderness of the longitudinal elements isn't satisfied, the interaction of the local and global buckling modes must be considered when computing the compressive capacity of a laced member. A methodology for this is described in [Bazant and Cedolin, 1991], where the Euler load of a laced member is reduced by a factor that depends on the critical load of the longitudinal elements—between lacing bar connections—and an imperfection parameter.

The compressive capacity of a laced member with a slender cross-section may be computed as described for prismatic members in section 6.2.1.1 based on the controlling (largest) width-to-thickness ratio, b/t or h_c/t_w.

6.2.1.4 Eyebars

Eyebars are tension-only elements. Their tensile capacity may be computed using Equation (6-1a) based on the gross area of the body of the eyebar, assuming that the head of the eyebar is properly proportioned to have a net section capacity exceeding that of the gross section. The AASHTO LRFD [AASHTO 2004] specification rules for eyebars may be used to check the head dimensions in order to verify this. Where the head dimensions do not satisfy the AASHTO requirements, the tensile capacity may be computed from a reduced gross area, such that the dimensioning requirements are met.

6.2.2 SPLICE CAPACITY

The tensile capacity of splices between, or within, members should be evaluated using Equations (6-1). The gross area is that of the splice plates; the net section is the gross area of the splice plates minus the area of rivet or bolt holes. Large members are often connected by a series of overlapping splice plates. In this case it is possible that a critical section exists through the intermediate plates. The transfer of load from the member into the splice plates—typically through rivets or bolts—should also be considered, as transfer of load may limit the splice capacity.

Existing splices between, or within, members subject to ductility demands greater than unity, must be strengthened to increase their nominal capacity to a level 25% higher than the nominal capacity of the members they connect. Splices between or within new members must have nominal capacities 25% higher than the nominal capacity of the members they connect. These provisions ensure that the splices are not a weak link in a member, considering overstrength and strain hardening of the member material. Splices—and connections generally—are not ductile; they must be reliably stronger than the members they connect in order to develop inelastic action in the members.

6.2.3 SHEAR STRENGTH

The shear strength of a laced member may depend entirely on its laces. This is case, for instance, for the member shown in Fig. 6-1, where the shear strength in the horizontal direction is provided by the laces alone. In an existing structure, its likely that members have been designed and oriented so that dead load shears, live load shears, and other large shears are resisted by solid webs, or by heavily laced webs. It's possible that during an earthquake, however, that previously unanticipated shears will occur in a direction where members are lightly laced.

The shear strength of a laced member may be computed from a truss analogy, considering each web of lacing independently. For the case of single lacing shown in Fig. 6-4, the shear strength is limited by the compressive capacity of a single lacing bar. This is a function of the bar area and length, $P_n\left(\sqrt{L^2+T^2}\right)$, according to Eq. (6-2). The shear strength of a single web (plane of lacing bars) is then

$$V_1 = \frac{T}{\sqrt{L^2+T^2}} P_n\left(\sqrt{L^2+T^2}\right) \tag{6-9a}$$

For the case of double (x) lacing (as shown in Fig. 6-1), the shear strength is also limited by the compressive capacity of a single lacing bar, but the strength is doubled because of the corresponding tension bar

$$V_2 = \frac{2T}{\sqrt{L^2+T^2}} P_n\left(0.7\sqrt{L^2+T^2}\right) \tag{6-9b}$$

The effective length of the lacing bar in compression is reduced by 30% because of the restraint offered by the corresponding tension bar.

The laces should be checked for the shear due to external loads, plus an additional shear equal to 2% of the compressive capacity of the member [AISC 2001].

6.2.4 FLEXURAL STRENGTH

The flexural capacity of prismatic, built-up, and laced members may be computed following the *AASHTO LRFD Specifications* [AASHTO 2004]. The flexural capacity of a member with a compact cross-section—also satisfying net section requirements—is equal to the plastic moment $M_p = ZF_y$, where Z is the plastic modulus of the cross-section. For a non-compact cross-section, with $\lambda_p \le \lambda \le \lambda_r$, the flexural capacity may be computed from

$$M_n = M_p - \left(M_p - M_r\right)\left(\frac{\lambda-\lambda_p}{\lambda_r-\lambda_p}\right) \tag{6-10}$$

based on the controlling (largest) width-to-thickness ratio, b/t or h_c/t_w, and where $M_r = SF_y$ where S is the section modulus.

Built-up and laced members should also satisfy net section requirements if they are to reliably develop the plastic moment. Specifically, the components of the cross-section subjected to tensile stress should satisfy Eq. (6-14) in order to prevent net-section fracture. If this requirement is not satisfied it is recommended that the flexural capacity of the member be limited to $M_r = SF_y$.

The flexural capacity of members may also be limited by lateral-torsional buckling. For box-shaped members this may be treated in accordance with article 6.12.2.2.2 of the *AASHTO LRDF Specifications* [AASHTO 2004]. For I and H-shaped members, the reader is referred to the *AISC LRFD Specifications* [AISC 2001].

6.2.5 DUCTILITY CAPACITY

The ductility of a member depends on the compactness of its cross-section. The *AISC LRFD Specification for Structural Steel Buildings* [AISC 2001] and the *AISC Seismic Provisions for Structural Steel Buildings* [AISC 1997] together, define three sets of limiting width-to-thickness ratios for the longitudinal elements of cross-sections. These, in turn, define four levels of compactness that can be applied to truss members. These levels are given in Table 6-1.

Table 6-1. Compactness Levels

Term	Width-to-Thickness Ratios	Achievable Ductility
Slender	$\lambda_r < x$	None. The longitudinal elements of a cross-section will buckle locally before strains reach the yield strain.
Non-Compact	$\lambda_p < x \le \lambda_r$	Little. The longitudinal elements of a cross-section will yield, but they won't sustain inelastic strains large enough to achieve the plastic capacity of the whole section.
Compact	$\lambda_s < x \le \lambda_p$	Modest. The λ_p requirements are intended for the plastic design of structures. The longitudinal elements of a cross-section will achieve strains of about 7 to 9 times the yield strain before buckling inelastically. A wide flange section should achieve a rotational ductility of at least 4.
Ductile	$x \le \lambda_s$	Significant. The λ_s requirements are intended for the seismic design of structures. A wide flange section should achieve a rotational ductility of about 8 to 10.

6.2.5.1 Damage Levels

It may also be desirable to limit the flexural ductility of a member in order to ensure the post-earthquake reparability and/or serviceability of the member and the structure. There is little research on this subject on which to base recommendations, but following reference [ATC 1996].

6.2.5.1-1 No Damage

No damage is presumed to occur if loads don't exceed the nominal capacity of a member computed in accordance with the AASHTO LFRD, or similar, code.

6.2.5.1-2 Minimal Damage

Minimal damage implies essentially elastic performance, and is characterized by:

- Minor inelastic response.
- No apparent permanent deformations.
- Inconsequential yielding of secondary steel members.

6.2.5.1-3 Repairable Damage

Repairable damage is damage that can be repaired with a minimum risk of losing functionality—i.e., without closing the bridge—and is characterized by:

- Yield of steel members, although replacement should not be necessary.
- Small permanent offsets, not interfering with functionality[1].

6.2.5.1-4 Significant Damage

Although the risk of collapse should be minimal, significant damage may require closure of the structure for repair, and is characterized by:

- Yield of steel members, possibly requiring replacement.
- Permanent offset of the structure.

6.2.5.2 Flexural

The flexural ductility of members forming the vertical elements of superstructure portal frames, sway frames, or support towers, expressed in terms of rotational ductility, is an important parameter in assessing the displacement ductility of those subsystems.

6.2.5.2-1 Prismatic Members

The ability of prismatic members made from rolled shapes or plates welded together—e.g., to form a box section—to sustain inelastic flexural deformations depends upon the compactness of their cross-section. Suggested values of allowable flexural ductility of members are given in Table 6-2.

The controlling width-to-thickness ratio, λ, in Table 6-2, is the largest value of b/t or h_c/t_w over the flanges, webs, etc. that make up the cross-section. The limiting width-to-thickness ratios λ_r, λ_p, and λ_s are given in Table 6-1.

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[1] The commentary to ATC-32 says that "permanent offsets should be avoided" for repairable damage.

Table 6-2. Suggested Flexural Ductility for Structural Steel, *R*

Damage Level	Controlling Width-To-Thickness Ratio, λ		
	$\lambda \le \lambda_s$	$\lambda \le \lambda_p$	$\lambda \le \lambda_r$
Significant	8	3	1
Repairable	4	2.1	1
Minimal	2	1.4	1
No	1	1	1

The values for significant damage are from "Ductile Design of Steel Structures" [Bruneau et al., 1998]. The values for repairable and minimal damage are obtained by geometric interpolation from those values to a value of unity for no damage.

The flexural ductility used in Table 6-2 is the *rotational* ductility

$$R = \frac{\theta_h}{\theta_p} \tag{6-11}$$

where θ_h and θ_p are illustrated in Figure 6-5.

θ_h is the ultimate rotation (capacity) calculated from

$$\theta_h = R\theta_p \,. \tag{6-12a}$$

The plastic rotation, θ_p, is related to the yield rotation, θ_y, through the shape factor of the cross-section

$$\theta_p = \frac{M_p}{M_y}\theta_y \tag{6-12b}$$

The yield rotation, θ_y, may be calculated from

$$\theta_y = \frac{M_y}{EI} \cdot L_p \tag{6-12c}$$

where L_p is the plastic hinge length. Following [Bruneau et al., 1998] this may be taken to be equal to 10% of the distance from the point of maximum moment to the point of inflection.

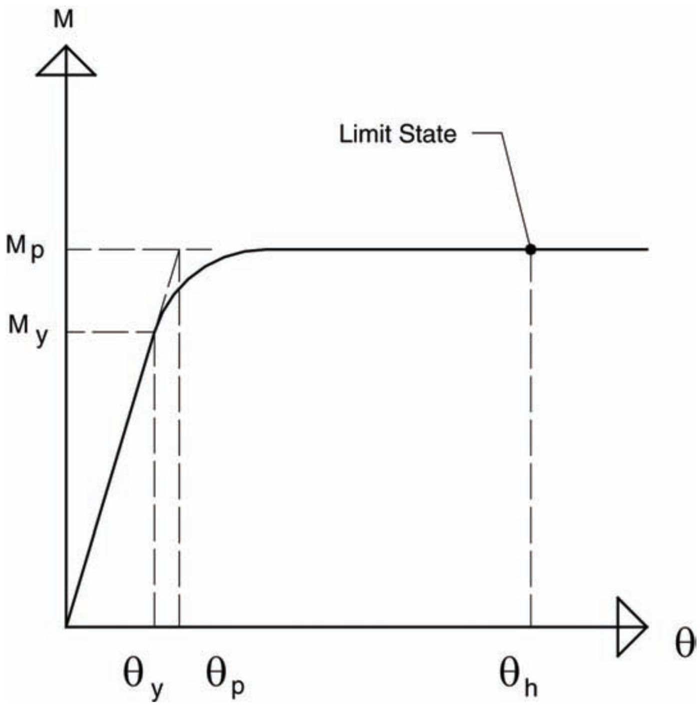

Figure 6-5. Definition of Rotational Ductility

The values of flexural ductility given in Table 6-2 apply if the axial load on a member—coinciding with the peak flexural demand—is less than one-half of its yield strength, $A_g F_y$. For values of axial load falling between $0.5A_g F_y$ and $1.0A_g F_y$ the allowable ductility may be obtained by linear interpolation between the tabulated value and unity.

6.2.5.2-2 Built-Up and Laced Members

There have been only a few tests of the cyclic behavior of laced members. Based on these few tests, it is recommended that the flexural ductility of laced members be limited to 2.0 for significant damage, 1.6 for repairable damage, and 1.3 for minimal damage [Lee and Bruneau, 2004; Uang and Kleiser, 1997; Dietrich and Itani 1999]; not to exceed the corresponding value in Table 6-2 based on controlling width-to-thickness ratio. Higher values may be established by inelastic finite element analysis, test, or other means.

There is no data relating the flexural ductility of laced members to axial load level. It may be reasonable to apply the same rule as for prismatic members, i.e., allow full ductility for axial loads less than $0.5A_g F_y$, and linearly reduce the allowable ductility to zero as the axial load approaches $1.0A_g F_y$.

6.2.5.2-3 Perforated Members

Based on [Dietrich and Itani, 1999], it is recommended that the flexural ductility of perforated members be limited to 3.0 for significant damage, 2.1 for repairable damage, and 1.4 for minimal damage; not to exceed the corresponding value in Table 6-2 based on controlling width-to-thickness ratio. Higher values may be established by inelastic finite element analysis, test, or other means.

There is no data relating the flexural ductility of perforated members to axial load level. It may be reasonable to apply the same rule as for prismatic members, i.e., allow full ductility for axial loads less than $0.5A_gF_y$, and linearly reduce the allowable ductility to zero as the axial load approaches $1.0A_gF_y$.

6.2.5.3 Axial

Inelastic *axial* response is acceptable for bracing and secondary members, but isn't recommended for members that carry dead load. The axial ductility of a member depends on its behavior in both tension and compression. The tensile ductility of a member depends on the area of its net section relative to its gross section and on the edge distance of the rivet and/or bolt holes where the longitudinal elements of the cross-section are connected together, or where laces are attached.

The compressive ductility of a member may be related to its flexural ductility. This is because, except for very squat members, the failure of a compression member occurs at a flexural hinge that forms near the middle of the member (and/or at hinges that form at the ends of the member if it's fixed). This flexural hinge is caused by the P-Δ effect; the moment carried by the flexural hinge is literally P×Δ, where Δ is the lateral deformation of the member. Since the lateral deformation increases with increasing axial deformation, the compressive load carried by the member decreases with increasing axial deformation.

The compressive ductility of a member also depends on its slenderness. The relationship between member ductility and subsystem ductility is an important consideration also. High values of member ductility do not necessarily imply a high level of subsystem ductility, as illustrated in Figure 6-12.

For members to be fully ductile, it is essential that the connections of the members be robust. The capacity of connections is addressed in the next section, where it is required that the connections of members subjected to ductility demands greater than unity be at least 25% stronger than the nominal capacity of the members they connect.

6.2.5.3-1 Prismatic Members

The ability of prismatic members made from rolled shapes or plates welded together to sustain inelastic axial deformations depends on their slenderness. Suggested values of axial ductility for *compact* members are given in Table 6-3.

Table 6-3. Suggested Axial Ductility for Structural Steel, R

Values are for X or Z-braces / V or K-braces			
Damage Level	**Member Slenderness Ratio, $\frac{Kl}{r}\sqrt{\frac{F_y}{36\,\text{ksi}}}$**		
	< 40	**< 80**	**< 120**
Significant	4.5 / 3.0	3.5 / 1.8	2.4 / 1.2
Repairable	2.7 / 2.1	2.3 / 1.5	1.8 / 1.1
Minimal	1.7 / 1.4	1.5 / 1.2	1.3 / 1.1
No	1.0 / 1.0	1.0 / 1.0	1.0 / 1.0

The values for significant damage are from [Dowrick 1991]. The values for repairable and minimal damage are obtained by geometric interpolation from those values to a value of unity for no damage.

The values given in Table 6-3 are applicable to members with compact cross-section only, i.e., members with $\lambda \le \lambda_p$, where λ is the largest value of b/t or h_c/t_w over the flanges, webs, etc. that make up the cross-section. The width-to-thickness ratio, λ_p, for a compact section is given in Table 5-4. The axial ductility of *non-compact* members should be established by inelastic finite element analysis, test, or other means.

6.2.5.3-2 Built-Up and Laced Members

Based on [Lee and Bruneau, 2004; Uang and Kleiser, 1997; Dietrich and Itani, 1999], it is recommended that the axial ductility of laced members be limited to 2.0 for significant damage, 1.6 for repairable damage, and 1.3 for minimal damage; not to exceed the corresponding value in Table 6-3 based on controlling slenderness ratio. Higher values may be established by inelastic finite element analysis, test, or other means.

6.2.5.3-3 Perforated Members

Based on [Dietrich and Itani, 1999], it is recommended that the axial ductility of perforated members be limited to 3.0 for significant damage, 2.1 for repairable damage, and 1.4 for minimal damage; not to exceed the corresponding value in Table 6-3 based on controlling slenderness ratio. Higher values may be established by inelastic finite element analysis, test, or other means.

6.3 CONNECTION CAPACITY

6.3.1 RIVETED CONNECTIONS

Riveted connections should be evaluated as bearing type connections, as described in the *AASHTO LRFD Specifications* [AASHTO 2004]. The shear strength of rivets shall be taken as:

$$\phi \cdot R_n = \phi \cdot F_r m A_r \qquad (6\text{-}13)$$

where:

ϕ = 0.80
F_r = the ultimate strength of the rivets in single shear
m = the number of shear planes
A_r = the nominal area of each rivet (before driving)

The ultimate strength F_r may be taken from Table 6-4 (from the AASHTO *Manual for Condition Evaluation and Load Resistance Factor Rating (LRFR) of Highway Bridges* [AASHTO 2003])

Table 6-4. Rivet Ultimate Shear Strength

Rivet Type or Year of Construction	F_r**, ksi**
Constructed prior to 1936 or of unknown origin	22
Constructed after 1936 but of unknown origin	26
ASTM A502 Grade I	31
ASTM A502 Grade II	37

For large bridges it may be warranted to test the rivet material in order to justify a higher shear strength than indicated in Table 6-4. This may be done by driving out typical rivets, machining their bodies into test specimens, and determining the tensile strength. Following the AASHTO LRFR, the tensile strength used for evaluation should be the mean strength minus 1.65 standard deviations, to obtain a 95% confidence limit. Following [Kulak et al., 1987], the ultimate shear strength of the rivet is 0.75 times the ultimate tensile strength of the material.

Kulak et al. [1987] report tensile and shear strengths for driven ASTM A502 rivets substantially higher than those given by AASHTO. They report that 60 ksi is a reasonable lower bound estimate of the tensile strength of Grade I rivets, implying that $F_r = 0.75 \times 60 = 45\,\text{ksi}$. For Grade II rivets they suggest $F_r = 0.75 \times 80 = 60\,\text{ksi}$.

The bearing-strength of rivets should be evaluated in accordance with the *AASHTO LRFD Specifications*. Connections containing both rivets and high-strength bolts should be designed in accordance with section J2.8 of the AISC LRFD Specifications and Chapter 14 of [Kulak et al., 1987].

6.3.2 BOLTED CONNECTIONS

Existing and new bolted connections may be evaluated in accordance with the *AASHTO LRFD Specifications* [AASHTO 2004]. Existing connections, subject to ductility demand greater than unity, should be strengthened to increase their nominal capacity to a level 25% higher than the nominal capacity of the members they connect.

6.3.3 WELDED CONNECTIONS

Existing and new welded connections may be evaluated in accordance with the *AASHTO LRDF Specifications*. Existing connections, subject to ductility demand greater than unity, should be strengthened to increase their nominal capacity to a level 25% higher than the nominal capacity of the members they connect.

For new welded connections, partial penetration welds should not be used in regions of members subject to inelastic deformation. Outside of those regions, partial penetration welds should provide at least 150% of the strength required by calculation, and not less than 75 % of the strength of the connected parts, regardless of the action of the weld.

6.3.4 PINNED CONNECTIONS

Pinned connections with mechanical steel pins within components such as truss shoes, eyebar connections, suspended truss span connections, and those that support tower-to-main cantilever truss member connections, shall be evaluated for strength, in accordance with the *AASHTO LRDF Specifications*.

6.3.5 GUSSET PLATES

Gusset plates shall be evaluated for strength and for stability, in accordance with the following requirements:

- Gusset plates shall be designed to F_y for combined axial forces and moments.
- The methodology described by [Thornton 1984] shall be used to calculate gusset plate forces and moments. Forces on plate edges shall be the corresponding component of the member force; moments on plate edges shall be the component force times the lever arm from the plate edge to the work point.
- Gusset plates shall be designed to $F_u / \sqrt{3}$ for uniform shear force and to $0.74 F_u / \sqrt{3}$ for flexural shear.
- Buckling of gusset plates shall be investigated by idealizing them as two truss elements. The truss elements shall be investigated as columns with $K = 0.65$.
- Unsupported edges of gusset plates with slenderness l/t (where l is the length of the unsupported edge and t is the thickness of the plate) exceeding $1.6\sqrt{E/F_y}$ shall be stiffened[2]. The stiffener moment of inertia (about its own centroid) shall satisfy [Brockenbrough 1981]
  $I_s \geq \max\left(9.2\,\text{in}^4, 1.83\sqrt{(b/t)^2 - 144}\right)t^4$.

---

[2] "Design Criteria for Seismic Retrofit of the San Francisco-Oakland Bay Bridge," California Department of Transportation, 1997.

Gusset plates should also have a capacity 25% higher than the nominal capacity of the members they connect, when those members are subjected to ductility demands greater than unity.

6.3.6 FRACTURE OF THE NET SECTION

Fracture of the net section shall be avoided by ensuring that the following requirement is met:

$$\frac{A_e}{A_g} \geq \frac{1.2\,\alpha D}{A_g f_u} \tag{6-14}$$

where:

A_e = the effective net area
A_g = the gross area of the member
f_u = minimum tensile strength of member
α = the fraction of the member force transferred across the net section
D = min{ $A_g f_y$, F_{eq} , F_{max}}
f_y = yield strength of member
F_{eq} = elastic seismic demand force
$F_{\max}$ = maximum force that can be transferred to the member

6.4 DISPLACEMENT CAPACITY OF PORTAL AND SWAY BRACING SYSTEMS

The displacement capacity of portal frames and sway frames in truss bridge superstructures may be evaluated in terms of limit-states characterized by racking and drift-displacement mechanisms. This will generally require an inelastic push-over analysis, similar to the case of support towers covered in Section 6.5 below. For portals and sway frames in double-decker truss bridges, the push-over analyses will require the imposition of higher-order lateral-deformation modes in order to capture the effect of the lateral-inertial loading from the upper deck.

6.5 DISPLACEMENT CAPACITY OF SUPPORT TOWERS

Steel truss bridges are often supported on steel towers. This is the case with the Richmond-San Rafael Bridge, across San Francisco Bay in California, shown in Figure 6-6.

A support tower of this type may be evaluated on a member-by-member basis, as described in Chapter 2. In this case, all of the elements of the support tower are included in a global model of the bridge, and the demands on each member are compared with member capacity, to derive capacity/demand ratios. These may be either force or ductility C/D ratios, depending on the modeling and the analysis approach used.

Figure 6-6. Support Towers of the Richmond-San Rafael Bridge

Inelastic response of a support tower is much more likely than for members of the superstructure, however, and it may be more convenient to treat a tower as a separate substructure. Detailed analysis of the inelastic response of the structure may then be confined to a model of the substructure. Conceptually, this approach is similar to that employed for the evaluation of typical highway bridges, where a pushover analysis of the inelastic response of the substructure—usually reinforced concrete piers or bents—is made using separate models. One advantage of this approach is that the detailed analysis of the inelastic response of the substructure is a static analysis, whereas a dynamic, inelastic analysis is required if the evaluation is to be on a member-by-member basis.

The pushover analysis of a hypothetical support tower is illustrated in Figure 6-7. The essential elements of the inelastic response of this K-braced tower are the formation of plastic hinges in the tower legs, the yielding in tension of one-half of the braces, and the buckling in compression of the remaining braces. Of course, in an earthquake, the deformation of the tower will reverse direction and the braces will be in alternating tension and compression (thus a cyclic pushover analysis may be warranted to obtain a backbone curve for global analysis).

It is important that the axial forces acting on the tower be included in the analysis, so that its effect on the flexural behavior and capacity of the tower legs is accounted for. Ideally, the analysis would be a geometrically nonlinear analysis, so that P-Δ effects are properly considered.

The modeling required to accomplish the pushover analysis illustrated in Figure 6-6 includes a plastic hinging element for the tower legs. Ideally, this element would take account of the interaction of axial force and flexure, since the axial force in one tower leg will decrease while the axial force in the other leg will increase during the pushover; this will affect both the flexural capacity and the flexural ductility of the member. The effects of axial load on the flexural ductility of members is discussed in Section 6.2.5.2.

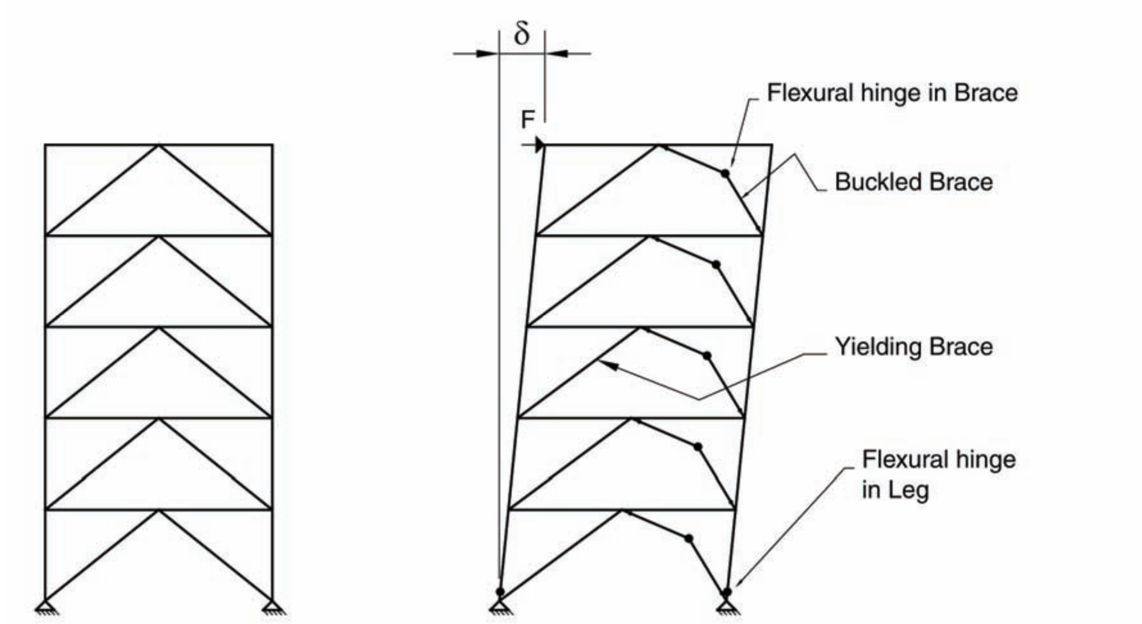

Figure 6-7. Pushover Analysis of a Support Tower

Three approaches are available for modeling the braces. The first approach utilizes a phenomenological model of brace behavior, such as the Ikeda, et al. [1984] model illustrated in Figure 6-8. A phenomenological model is a mathematical model that mimics the tensile and compressive behavior of both compact and slender braces, depending upon the chosen parameters. The second approach utilizes a physical model of brace behavior, such as the Ikeda and Mahin [1984] model shown in Figure 6-9. Physical models are based on a structural mechanics analysis of brace behavior. Both the phenomenological model and the physical model must be programmed into the analysis program used for pushover analysis.

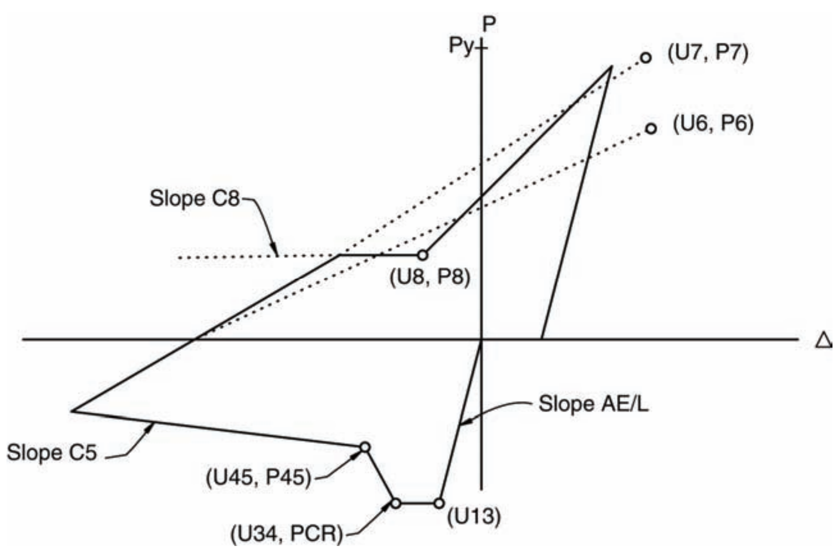

Figure 6-8. Phenomenological Model of Brace Behavior

The third approach utilizes a finite element model of the brace. In this approach, each brace member is modeled with several elements, including a plastic hinging element—with axial force and flexure interaction—at the mid-point of the member, and possibly at the ends of the member also. This modeling is illustrated in Figure 6-10. The pushover analysis must then be a geometrically nonlinear analysis. This modeling captures the important characteristics of brace behavior: yielding in tension, buckling in compression, and the subsequent formation of a plastic hinge at the middle of the member.

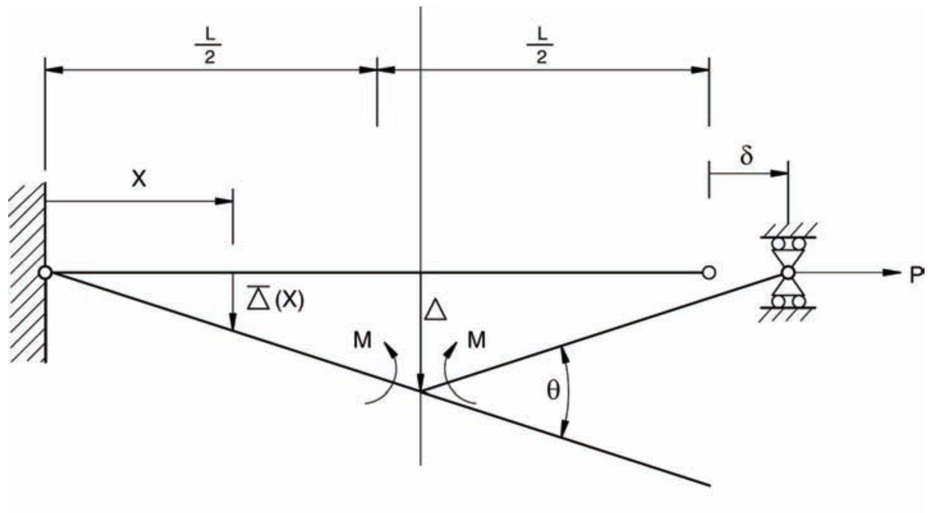

Figure 6-9. Physical Model of Brace Behavior

Yielding in tension will occur within the plastic hinging element(s) when the tensile force in the member exceeds the tensile capacity of the plastic hinge(s). Because the analysis is a geometrically nonlinear, the member will buckle in compression when the critical load of the member is reached—so the moments of inertia of the elements should be adjusted so as to achieve the critical load calculated in accordance with the procedures discussed in this manual. The element in the middle of the member will eventually yield in flexure due to P-Δ effects within the member itself—since buckling is accompanied by lateral deformations of the member.

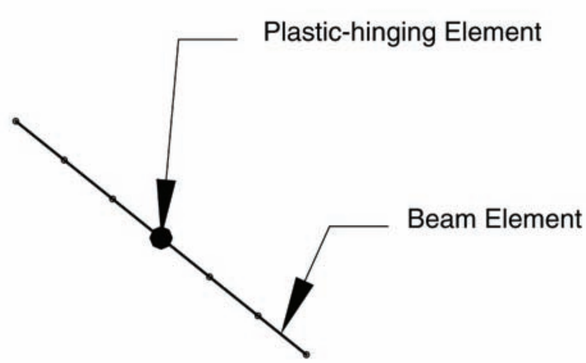

Figure 6-10. Finite Element Model of Brace Behavior

In a pushover analysis, a lateral force or displacement is applied to the top of the tower. The resulting force-displacement response will generally be of the form shown in Figure 6-11. Application of displacements is generally preferable for pushover analysis, since if forces are applied, convergence becomes very difficult when the strength of the frame is reached.

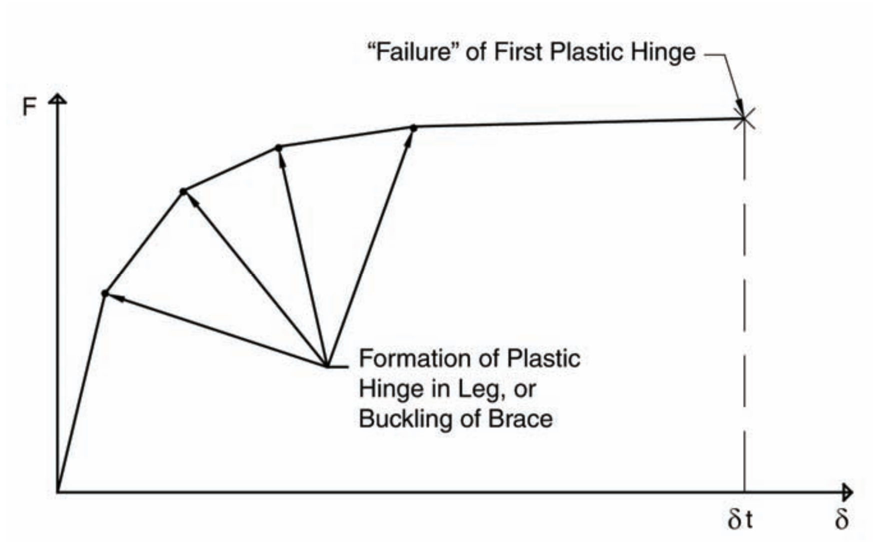

Figure 6-11. Force-Displacement Response from Pushover Analysis

The stiffness of the tower decreases progressively as plastic hinges form in the tower legs and as the braces progressively buckle. The displacement capacity of the tower may be defined by the "failure" of one of the plastic hinges. This will occur when the rotational ductility capacity of the hinge is reached. This depends on the member type, its compactness, and the degree of damage allowed, as discussed in Section 6.2.5.2. If a phenomenological model or physical brace model is used – or even a finite element model of a brace – the displacement capacity of the tower may be reached when the axial ductility capacity the braces is reached. The axial ductility capacity of member is discussed in Section 6.2.5.3. Whether the flexural ductility or the axial ductility capacity of members is used to define the failure limit state of a member depends both on the behavior of member in question, and on the modeling technique used.

It is also possible that the force-displacement response of a tower appears as shown in Figure 6-12. The resistance of the tower may decrease with increasing deformation because of decreasing resistance of the lateral braces, or because of second order – P-Δ – effects. In this case, the displacement capacity of the tower may be defined corresponding to a 20% reduction in resistance. This point may be reached before any of the structural elements have reached their allowable ductility capacity.

The force-displacement response illustrated in Figure 6-10 or 6-11 should be implemented in the global model of the bridge. In a time history analysis, it may be possible to implement the response utilizing an inelastic element with hysteresis. Thus any energy absorbed by the inelastic response of the tower may be directly accounted for in the global analysis. A cyclic pushover analysis is recommended to obtain a backbone curve that reflects the degradation of response with multiple cycles, and that is appropriate for the level of deformation observed in the global analysis. Care must be taken not to overdamp the global analysis. In a response spectrum analysis, it will be necessary to linearize the response. In either case, the displacement of the tower should be monitored during the analysis and the peak displacement of the tower should be noted for comparison with its displacement capacity.

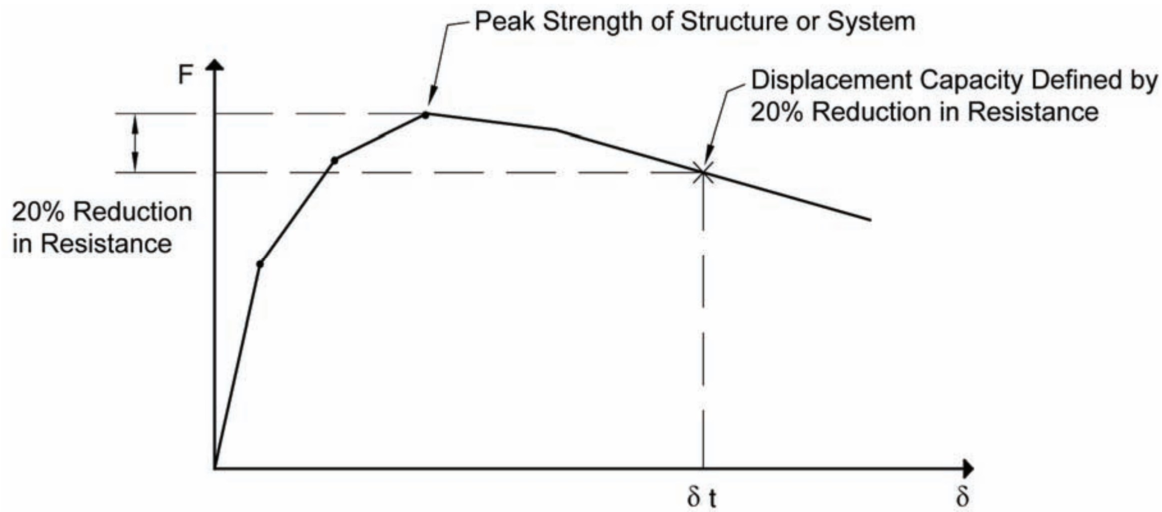

Figure 6-12. Force-Displacement Response from Pushover Analysis with P-Delta

In terms of displacement ductility, Figure 6-13 depicts the member ductility demand of a typical chevron braced truss tower (Richmond-San Rafael Bridge shown in Figure 6-6) as a function of the system displacement ductility, for loading in one direction (i.e., non-cyclic). As shown in the figure, the ductility of the tower subsystem is governed by the compression-brace limit state, in which a member ductility of 4 results in a lower system ductility of only 2. The retrofit of this type of tower will typically require the addition of ductile frames in order to significantly increase the lateral seismic resistance of the towers.

Pushover analysis of complex structures where many modes of vibration contribute to the response is problematic, since a pushover analysis involves only a single load or deformation pattern. For a complex structure, it might be prudent to perform several pushover analyses using different load and/or deformation patterns. These might include:

- A concentrated load at the top of the structure.
- A uniform load.
- A uniform acceleration, or mass proportional load.
- A concentrated displacement at the top of the structure.
- A displacement pattern derived from the displaced shape corresponding to the peak drift of the structure relative to its base.
- A displacement pattern equal to one or more relevant mode shapes.

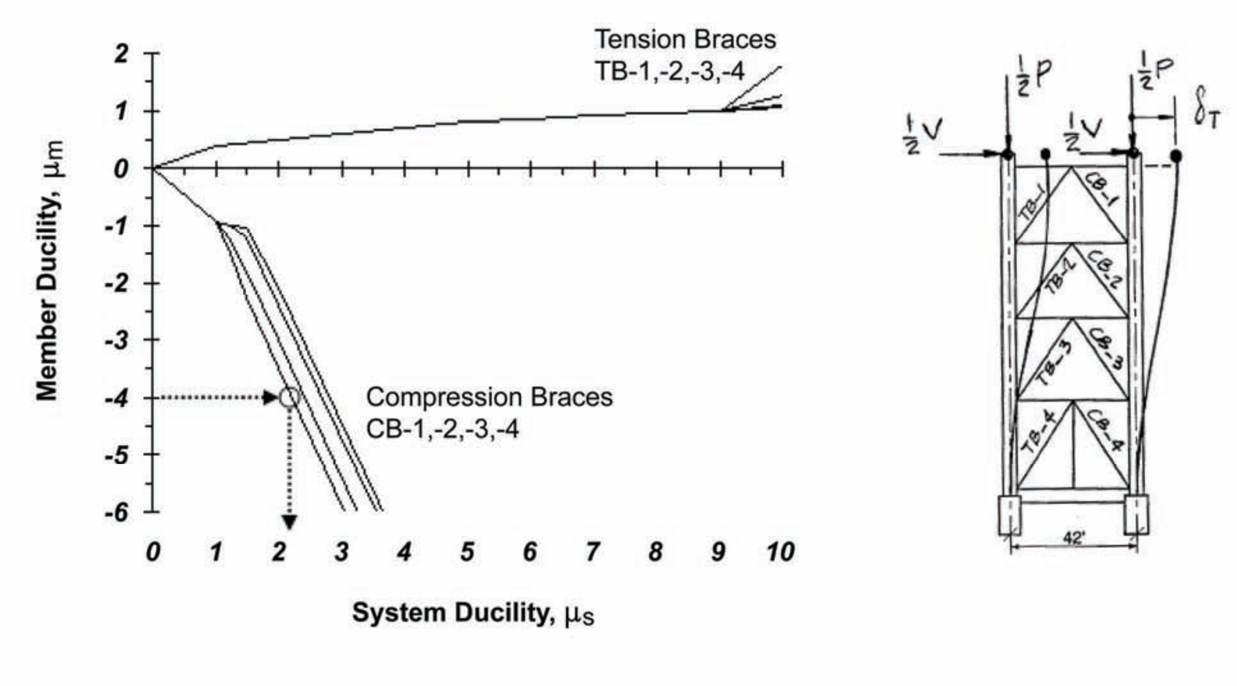

Figure 6-13. Displacement Ductility from Pushover Analysis (Richmond-San Rafael Bridge Tower-No. 29)

6.6 BEARINGS

Conventional bridge bearings should be evaluated for strength under transverse shear, longitudinal shear, vertical loading due to earthquake-induced overturning effects, and in some cases,, longitudinal displacements for "slider" and rocker bearings. Rotational displacement demands on some types of bearings, such as pot bearings, may also be critical. Bearing-component evaluation and design criteria are the same as those provided in the *Bridge Retrofit Manual.*

6.7 SHEAR LOCKS (WIND TONGUES)

Earthquake-induced loads on wind locks, also called wind tongues (which transmit transverse wind-induced loading on the truss superstructure to the substructure) in existing truss bridges are very likely to exceed their original wind-design loads. The tongue element, which essentially consist of cantilevered structural members, should be evaluated for transverse shear, flexural bending, as well as for longitudinal displacements that tend to disengage the tongues from their groves. The retrofit design intent here is to eliminate the inherent shear failure mechanism by strengthening.

6.8 EXPANSION JOINTS

Relatively long-span SC truss bridges, supported on tall steel towers, may have split bent expansion joints, which are beneficial in that span unseating is not possible. An example of a split bent is shown in Figure 6-14. The retrofit design requirements of these types of joints may require strengthening of the "tuning-fork" configuration of the split bents in order to increase the moment capacities of the legs for bending about the transverse axis of the bent**.** The use of damping devices such as fluid dampers across the joints, attached to the chord members of the adjoining truss spans, may be effectively used to control opening displacements and impact loading across the expansion joints.

For truss bridges, directly supported on concrete piers, the expansion joints are similar to those of ordinary bridges; retrofit design alternatives include the addition of seat extenders, restrainers, and dampers. The evaluation criteria for these types of joints, and the design criteria, which are presented in the *Bridge Retrofitting Manual*, are applicable.

Figure 6-14. Typical Split Bent (Richmond-San Rafael Bridge)

6.9 CAPACITY/DEMAND ANALYSIS

Capacity/demand analysis is a commonly-used way to summarize the vulnerability of a structure to a given event. The general approach to capacity/demand analysis is fully described in the *Bridge Retrofitting Manual*. For truss bridges, the capacity of members and structural systems such as support towers or longitudinal bracing may be determined by using the methods described in this chapter.

$$\mathrm{C/D} = \frac{\text{Member or System Capacity}}{\text{Member or System Demand}} \tag{6-15}$$

Simplistically, a capacity/demand ratio greater than unity indicates satisfactory behavior, and a ratio less than unity indicates unsatisfactory behavior that may require retrofitting.

6.9.1 MEMBER AND CONNECTION FORCE CAPACITY/DEMAND

If elastic behavior of a member is presumed—e.g., if force demands are computed from an elastic analysis—or is the goal of a seismic retrofit, then the force demands on that member with respect to its tensile and compressive strengths are of greatest interest. The strength of the connections of the member and of any splices within the member should also be considered.

For a given member there are three C/D ratios:

$$C/D_T = \frac{T}{D} = \frac{\text{Tensile Strength}}{\text{Calculated Demand}} \tag{6-16a}$$

$$C/D_C = \frac{C}{D} = \frac{\text{Compressive Strength}}{\text{Calculated Demand}} \tag{6-16b}$$

$$C/D_S = \frac{S}{D} = \frac{\text{Connection and/or Splice Strength}}{\text{Calculated Demand}} \tag{6-16b}$$

If the tensile strength is governed by the net section strength, then the tensile C/D ratio is denoted

$$C/D_N = \frac{N}{D} = \frac{\text{Net Section Strength}}{\text{Calculated Demand}} \tag{6-17}$$

Values of C/D ratios are treated differently if the net section strength controls**,** rather than the gross section strength, since the member behavior is brittle in the former case, and ductile in the latter case. Note that all remarks with respect to the connections of members also apply to the splices.

In principle, retrofit should be pursued whenever C/D ratios are less than unity. In practice, however, it may be reasonable to compare C/D ratios to a threshold value, or values less than unity above which retrofit need not be pursued. This concept recognizes that retrofit of a large number of slightly overloaded members might be very costly relative to the benefit obtained, considering the uncertainties of both seismic analysis and capacity evaluation, which may be conservative.

Such threshold values could depend on several factors. These include member type; since the consequences of failure of redundant main members, non-redundant main members, and secondary and bracing members are different. The capacity being considered is another factor, since failure of the gross section is likely to be ductile; overload in compression may or may not be ductile; and the failure of the net section and of connections or splices are likely to be brittle. Bridge importance may add another factor. Possible threshold values are given in Table 6-2.

Force capacity/demand ratios less than unity are fictitious, since the demands on a member can't actually exceed its capacity. A computed ratio less than unity implies one of two things:

1. If the computed ratio is only slightly less than unity, the inelastic response of the member is limited and it may not be worthwhile to conduct inelastic or pushover analysis in order to evaluate the actual response of the member. This notion is reflected in the threshold values of capacity/demand ratio given above. These also consider that the computation of capacity may be conservative, and that the consequences of a minor amount of yielding are probably not great.

2. If the computed ratio is large, then the member may fail in reality, or be subject to significant yielding. In this case some sort of inelastic analysis is probably warranted in order to determine the actual ductility demands on the member and the redistribution of force to other members.

Many cases may arise when calculated capacity/demand ratios will be compared to the threshold values.

Table 6-2. Suggested Threshold Values of Capacity/Demand Ratio, γ

	Member Type		
Failure Mode	**Main, Non-Redundant**	**Main, Redundant**	**Secondary, Bracing**
Yielding of Gross Section	1.0	0.8	0.67
Fracture of Net Section	1.0	1.0	1.0
Buckling in Compression	1.0	0.8	0.67
Connection or Splice Failure	1.0	1.0	1.0

6.9.1.1 Case 1 - $C/D_T < 1$

The tensile capacity is less than the demand.

Retrofit may not be required if the C/D ratio is above the specified threshold value.

A capacity/demand ratio less than unity implies that the response of the member is inelastic. Fracture of the net section should then be prevented by adherence to the requirements of Section 6.2. If necessary, connections and splices should be strengthened so that they are at least 25 % stronger than the member.

6.9.1.2 Case 2 - $C/D_T < \gamma_T$

The tensile capacity is less than the (threshold) demand.

Retrofit should be pursued to increase the tensile strength of the member, through the addition of cover plates, for instance. Capacity/demand ratios should be reevaluated for the retrofitted member, since connection or splice retrofit may then be required.

6.9.1.3 Case 3 - $C/D_C < \gamma_C$ and $\gamma_T < C/D_T$

The compression capacity is less than the (threshold) demand. The tensile capacity exceeds the (threshold) demand.

Retrofit should be pursued to increase the compressive strength of the member. Depending on the member, this may be pursued through the addition of cover plates. If the compressive strength is limited by the slenderness of the longitudinal elements of the cross-section, the compressive strength of the member may be improved by stiffening the cross-section. Capacity/demand ratios should be reevaluated for the retrofitted member, since connection or splice retrofit may then be required.

6.9.1.4 Case 4 - $\gamma_C < C/D_C$ and $\gamma_T < C/D_T$

Both the compression and tensile capacity exceed the (threshold) demand.
The connection and splice strength must still be evaluated, as follows.

6.9.1.4-1 Case 4a - $C/D_S < \gamma_S$

The connection or splice capacity is less than the (threshold) demand.
The connection or splice should be strengthened.

6.9.1.4-2 Case 4b - $\gamma_S < C/D_S$

The connection or splice capacity exceeds the (threshold) demand.
No retrofit is required, *if the tensile capacity exceeds the demand, i.e., if* $1 < C/D_T$. Otherwise, the member response is inelastic, Case 1 applies, and connections and splices should be strengthened so that they are at least 20% stronger than the member; see Section 6.9.1.1.

6.9.1.5 Interaction of Flexure

The AASHTO LRFD interaction equations for axial force and flexure are

$$\frac{P_u}{2.0P_r} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \text{ if } \frac{P_u}{P_r} < 0.2 \tag{6-18a}$$

$$\frac{P_u}{P_r} + \frac{8}{9}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \le 1.0 \text{ if } \frac{P_u}{P_r} \ge 0.2 \tag{6-18b}$$

where: P_u = axial compressive demand load; M_{ux}, M_{uy} = factored concurrent demand flexural moments about the member local axes x and y, respectively; P_r = factored compressive resistance; and M_{rx}, M_{ry} = factored flexural resistance about the member local axes x and y, respectively. These equations have the general form of "demand" divided by "capacity" since, if demands—in aggregate—exceed capacity, their left-hand sides will be less than unity. In order to develop equations in the form of "capacity" divided by "demand," the expressions on the left-hand side of each equation may be inverted, giving:

$$\frac{2\left(\frac{P_r}{P_u}\right)\left(\frac{M_{rx}M_{ry}}{M_{ux}M_{ry} + M_{uy}M_{rx}}\right)}{2\left(\frac{P_r}{P_u}\right) + \left(\frac{M_{rx}M_{ry}}{M_{ux}M_{ry} + M_{uy}M_{rx}}\right)} \ge 1.0 \text{ if } \frac{P_r}{P_u} > 5 \tag{6-19a}$$

$$\frac{\frac{9}{8}\left(\frac{P_r}{P_u}\right)\left(\frac{M_{rx}M_{ry}}{M_{ux}M_{ry} + M_{uy}M_{rx}}\right)}{\left(\frac{P_r}{P_u}\right) + \frac{9}{8}\left(\frac{M_{rx}M_{ry}}{M_{ux}M_{ry} + M_{uy}M_{rx}}\right)} \ge 1.0 \text{ if } \frac{P_r}{P_r} \le 5 \tag{6-19b}$$

Thus we may define a capacity/demand ratio for combined axial force and flexure

$$C/D = \frac{2\left(\frac{P_r}{P_u}\right)\left(\frac{M_{rx}M_{ry}}{M_{ux}M_{ry} + M_{uy}M_{rx}}\right)}{2\left(\frac{P_r}{P_u}\right) + \left(\frac{M_{rx}M_{ry}}{M_{ux}M_{ry} + M_{uy}M_{rx}}\right)} \text{ if } \frac{P_r}{P_u} > 5 \tag{6-20a}$$

$$C/D = \frac{\frac{9}{8}\left(\frac{P_r}{P_u}\right)\left(\frac{M_{rx}M_{ry}}{M_{ux}M_{ry}+M_{uy}M_{rx}}\right)}{\left(\frac{P_r}{P_u}\right)+\frac{9}{8}\left(\frac{M_{rx}M_{ry}}{M_{ux}M_{ry}+M_{uy}M_{rx}}\right)} \text{ if } \frac{P_r}{P_r} \le 5 \tag{6-20b}$$

6.9.2 MEMBER DUCTILITY CAPACITY/DEMAND

The member ductility C/D ratios are computed by division of the ductility capacities, addressed in Section 6.2, by the corresponding demand deformation ductilities and computed by dividing the demand member deformations (extensional, compressive, or hinge rotational) with the respective limit states defining the onset of inelastic action, as follows:

$$\mathrm{C/D} = \frac{\text{Member Deformation Ductility Capacity}}{\text{Member Deformation Ductility Demand}} \tag{6-14}$$

6.9.3 SUBSYSTEM DISPLACEMENT CAPACITY/DEMAND

The demand on a subsystem of a truss, e.g., on a portal frame or a support tower, may be related to the capacity of that subsystem through the displacement of that system. For instance, the displacement capacity of a support tower may be obtained from a pushover analysis, as described in Section 6.5. If that same displacement is monitored in an analysis, then a capacity/demand ratio may be computed:

$$\mathrm{C/D} = \frac{\text{Subsystem Displacement Capacity}}{\text{Subsystem Displacement Demand}} \tag{6-15}$$

Generally, retrofit is required if the computed capacity/demand ratio is less than unity.

CHAPTER 7: RETROFIT MEASURES, APPROACH, AND STRATEGY

7.1 GENERAL

When a bridge truss is evaluated and found to be seismically deficient, the next step is to decide what retrofit strategy, if any, should be used to correct the deficiencies. Decision-making may be formalized by exploring different retrofit measures and comparing cost estimates with other implications for each measure. The identification of retrofit measures, development of a retrofit approach and selection of a retrofit strategy are presented below.

The *Bridge Retrofitting Manual* defines these three terms in Section 1.11 as follows:

A *Retrofit Measure* is the physical modification of a component in a bridge for the purpose of correcting seismic deficiencies of components or of improving the seismic performance of the bridge. Replacing lacing bars with perforated cover plates on a truss member to strengthen the member, or replacing a truss bearing with a more robust bearing that can transmit seismic lateral loads to the substructure, are examples of retrofit measures.

A *Retrofit Approach* is a method of improving the response of the bridge or correcting its seismic deficiencies. Strengthening and isolation are examples of retrofit approaches. One or more retrofit approaches may be employed in the seismic retrofit of a bridge.

A *Retrofit Strategy* is the overall plan for the seismic retrofit of a bridge. The plan can employ more than one retrofit approach and several different retrofit measures, as defined above. For example, the seismic retrofit strategy of the North Viaduct of the Golden Gate Bridge involved eight different types of retrofit measure which included replacing the expansion joints and the restrainers, replacing the existing support towers with new towers, replacing existing bearings with new isolation bearings and modification/strengthening of some connections and framings. The approach adopted in this case is the use of isolation bearings to reduce the seismic input to the superstructure. Some of the deficient members were also strengthened. However, the existing expansion joints and restrainers cannot accommodate the larger movement demand and they needed to be replaced at the same time. This completed retrofit strategy will ensure the safety of this structure in a major earthquake event.

In a typical seismic retrofit project for a bridge truss, the structure is screened and categorized as either a seismically standard (SS) or a seismically complex (SC) truss bridge, as defined in Section 2.2, and the importance of the bridge is determined, as presented in Section 2.5. The bridge is then subjected to global and local analyses, as presented in Chapter 4, for the two seismic hazard levels, as discussed in Section 2.4. After global analysis of the truss, the demand-to- capacity-ratios are determined for each member and are compared to member performances, as defined in Section 2.7. After determining the anticipated service life and service life categories as presented in Section 2.6, the process of formulating retrofit strategies to improve or correct seismic deficiencies can begin.

7.1.1 RETROFIT STRATEGIES

Determining a retrofit strategy for bridge trusses usually is not as simple as merely strengthening each deficient member. Especially in the case of a seismically complex bridge, a seismic retrofit, which is performed member-by-member, may sometimes result in not eliminating all of the deficiencies, or in forcing the deficiencies to other members of the truss. For example, retrofitting a member by strengthening may stiffen the member, which will then attract more force to that member, nullifying the effect of the retrofit or causing the connections to the member to become deficient. However, strengthening, if properly applied, can be a very effective retrofit strategy.

Adding energy dissipation devices, such as dampers, or using isolation techniques can be an effective alternative retrofit approach. However, energy dissipation devices can only function if connected between two points that have relative displacements in a seismic event. Isolation produces relatively large displacements of the bridge at the support points; the retrofit design must provide space at each abutment and support point to accommodate these large displacements.

Determining a retrofit strategy, particularly for a seismically complex truss, can be an exploration process with no standard formulas that the retrofit designer can routinely follow. Finding effective solutions for retrofit strategies depends largely on the experience and perseverance of the designers. As a suggested process, the retrofit designer could start the retrofit strategy from a global perspective, then move progressively to regional areas, and then to members and connections.

Designers should consider all possible retrofit approaches and retrofit measures and evaluate them analytically to determine the retrofit strategy. The designer needs to compare, for all possible retrofit strategies, cost, constructability, and construction impact to the bridge, to traffic, and to surrounding communities. If the construction of the retrofit blocks or interferes with traffic, then alternative detours must be developed to handle traffic, at added cost to the public. The designer must strive to achieve the lowest cost retrofit that is constructible and biddable by contractors, using available labor and equipment that has minimal impact to the surrounding community and the environment.

7.1.2 CONSTRUCTABILITY

Performing seismic upgrades on an existing truss is often more difficult than constructing a new bridge truss. The space is limited and there are existing members in the way where new members need to be added; there is localized demolition work for which the adjacent members need to be supported or protected; there is traffic on the bridge that in most cases needs to be maintained, as well as many other similar issues.

In most conventional new bridge design projects, the designer will allow contractors freedom to select their construction method. However, for seismic retrofit projects, the designer will provide better service to the owner if he/she does not follow conventional design practice. The retrofit designer should prove the acceptability of the design by developing at least one feasible construction method that a contractor can use. It is preferred to include a plan in the contract document that provides a detailed construction sequence. For example, when part of the structure is removed and the truss is temporarily supported at location(s) other than the original support location, there is a total change of the load path. Once the load path is changed, the member forces also change with it and sometimes a tension member might become a compression member. Some of the truss members were not designed for the reversed loads and temporary measures are needed to preserve structural integrity. Since this type of problem could be complex and the consequences could be drastic, the designer should prepare detailed instructions of construction for the contractor to follow. This does not mean that the designer actually designs the temporary supports for the contractor, but the detailed instructions need to indicate that a support must be provided at the location and a conservative load value given that the temporary support must be designed to carry.

This suggestion could reduce the possibility of contractors filing claims for a project which he/she bid, but for which he/she can demonstrate that the plans were not adequate nor buildable. A construction-evaluation process will eliminate retrofit approaches and measures that do not work well, are un-buildable, or are too expensive. A construction evaluation process will retain the best strategy for final selection.

7.1.3 THE "DO-NOTHING" RETROFIT, AND FULL-REPLACEMENT STRATEGY

In evaluating the retrofit strategy for a seismically deficient bridge, the designer should keep in mind the three performance objectives discussed in Section 2.3 of life-safety performance, limited performance, and full performance. These three retrofit performance objectives should be weighed against a "do-nothing" option, which can result in prolonged or permanent loss of bridge service after a seismic event, and which may result in higher total cost than for a full-service strategy option. The "do-nothing" option requires acceptance of damage to the bridge and to the possibility of total replacement of the bridge.

However, "do nothing" is not an acceptable seismic strategy unless analysis of the bridge has proven that under the action of an upper level earthquake, as defined in Section 2.4, no collapse will occur and life safety will be assured. The "do-nothing" option requires acceptance of damage to the bridge and to the possibility of total replacement of the bridge.

Foundation failures are unlikely to cause total span collapse (see Case Studies No. 4 Million Dollar Bridge), unless ground deformations are excessively large because of widespread liquefaction, or there is massive ground failure, such as from fault rupture. Fortunately, these events are rare. Nevertheless, judgment should be used when assessing bridge collapse potential.

If a truss bridge crosses a known active fault, the designer should attempt to retrofit the bridge to prevent total span collapse in the event that the fault should rupture during the life of the bridge. This is the lowest retrofit performance objective and the bridge may have to be replaced, but life safety will be preserved. Analysis may show that long span truss bridges may be able to withstand the ground deformation estimated to occur if the fault ruptures. Fault rupture, unstable slopes and liquefiable soil are considered and discussed in Section 2.4.

Full-replacement may be an option, particularly if the retrofit cost is of the same order of magnitude as the replacement cost of the bridge. Full replacement of either seismically standard or seismically complex trusses may be considered when the retrofit costs approach 70% to 80% of the cost of a new bridge. Full replacement may become even more attractive if the structure has non-seismic structural deficiencies, such as a narrow deck, and is functionally obsolete. The Cooper River Bridge in South Carolina is a good example. The existing twin bridges were functionally obsolete and reaching the end of their service lives. It is also very costly to seismic retrofit these old structures to the modern standard. The final retrofit strategy was to replace the existing bridges with a new cable-stay bridge. However, in making this recommendation, the designer should also consider, as part of the cost of the replacement alternative, the costs of demolition and the costs associated with the control and re-routing of traffic.

7.1.4 SEISMIC RETROFITTING DESIGN PROCESS

A complete bridge truss retrofitting process starts with data collection, a bridge inspection, developing a condition assessment report, establishing a computer model, computing member/system demands and capacities, comparing capacities to demands, determining the deficiencies of the structural system, evaluating various retrofitting measures and approaches, arriving at a retrofitting strategy and completing the retrofitting design. The seismic retrofitting design steps are discussed in Section 2.9. The following flowchart in Figure 7-1 illustrates steps 4 to 10 in more detail.

Chapter 4 of these *Guidelines* discussed various methods of analyzing the bridge truss structures. The evaluation of the member, connection and subsystem capacities is covered in Chapter 6. The last section of Chapter 6 addresses the Capacity/Demand analysis, which is a key step in the retrofitting procedure. The result of this analysis determines whether or not a truss bridge is deficient and whether seismic retrofitting is required. In the following sections discussion is focused on potential deficiencies of various truss components and on corresponding retrofit measures.

7.2 TRUSS MEMBERS

7.2.1 GENERAL

Two general measures are possible to improve the behavior and strength of members. These are the addition of stiffeners to increase the stiffness of any longitudinal plates that make up the cross-section of a member, and the addition of cover plates to increase the strength of a member.

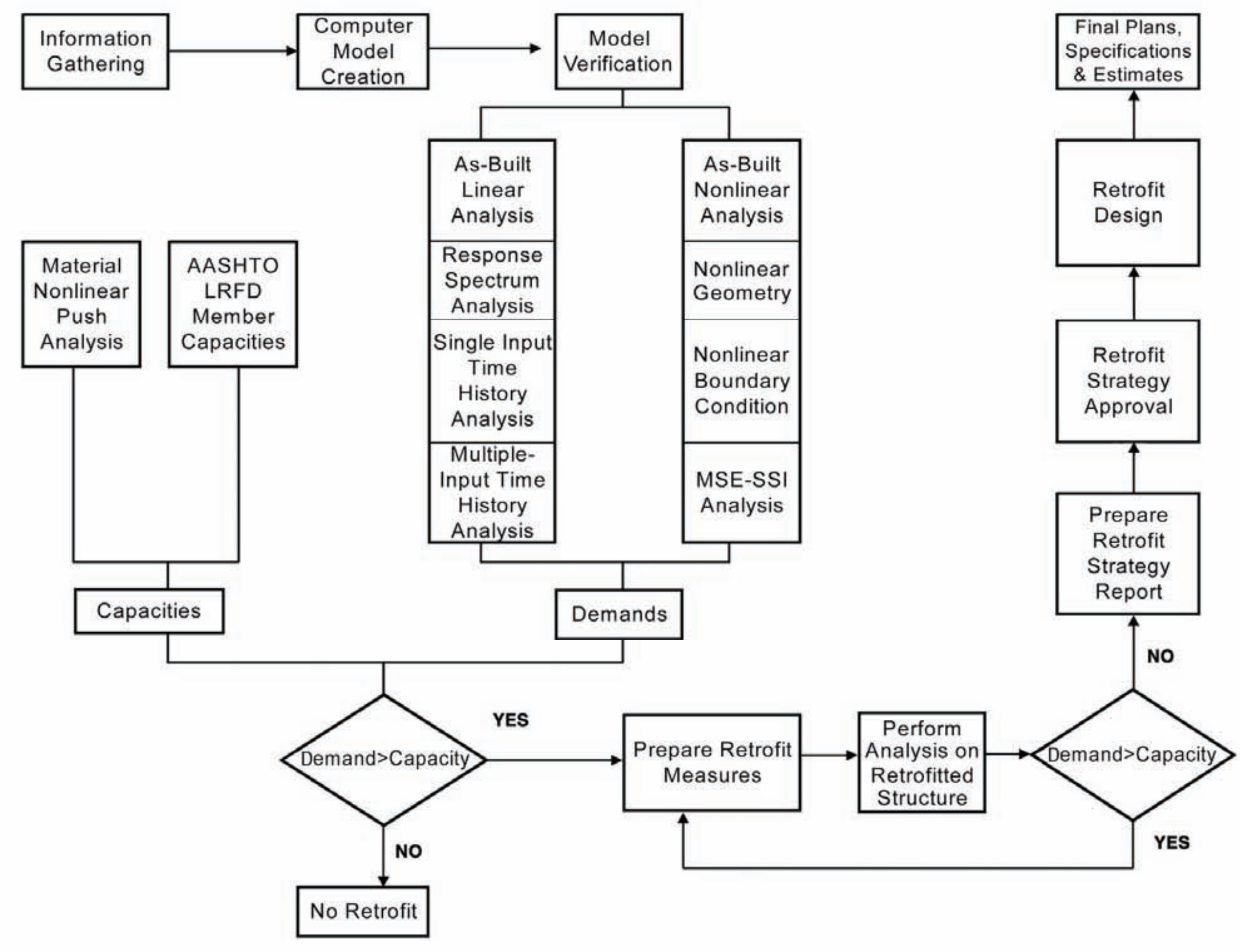

Figure 7-1. Flowchart of Seismic Retrofit Process

7.2.1.1. Stiffener Retrofit

If the plates that make up a member are slender, the strength of that member may be limited by local buckling of those plates. Also, the ductility of the member may be limited by early post-yield buckling. The addition of appropriately-designed stiffeners will reduce the width-to-thickness ratio of the plates to which they are applied. The critical stress of the retrofitted plate will be increased**,** as will the strength of the member in compression. Also, the ductility of the member may be dramatically improved. The relationship between plate slenderness and section ductility is discussed in Chapter 6.

An example of stiffening a member to improve its ductility is shown in Figure 7-2. Detailed design requirements for stiffeners are in [AASHTO 1998].

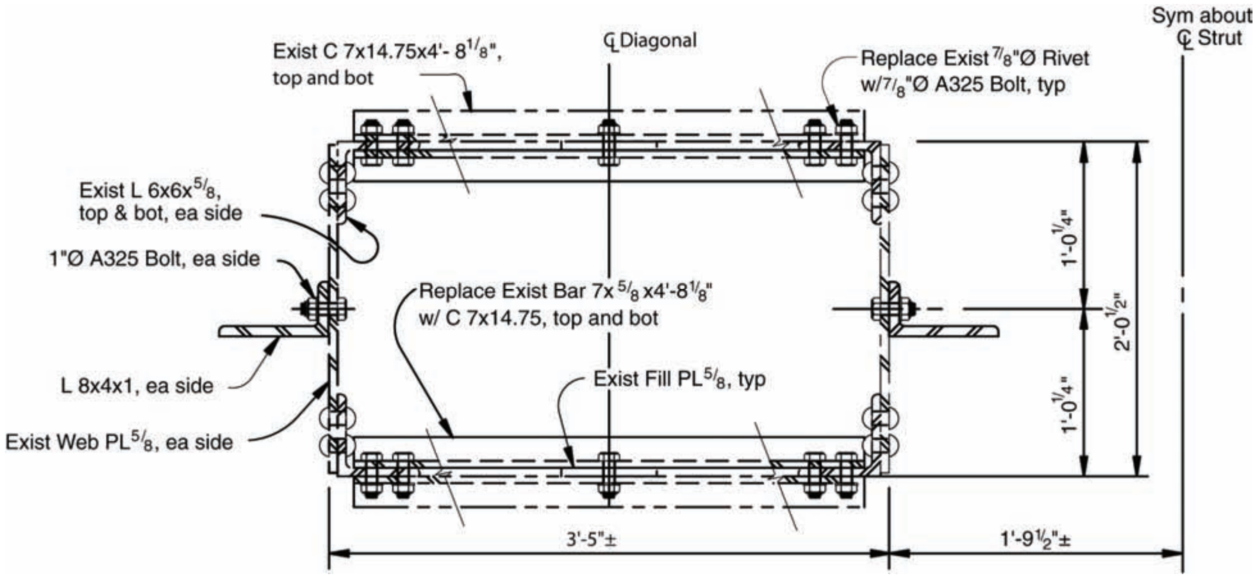

Figure 7-2. Plate Retrofit with Stiffeners

7.2.1.2. Cover Plate Retrofit

Cover plates may be added to strengthen a member in both tension and compression. An example is shown in Figure 7-3:

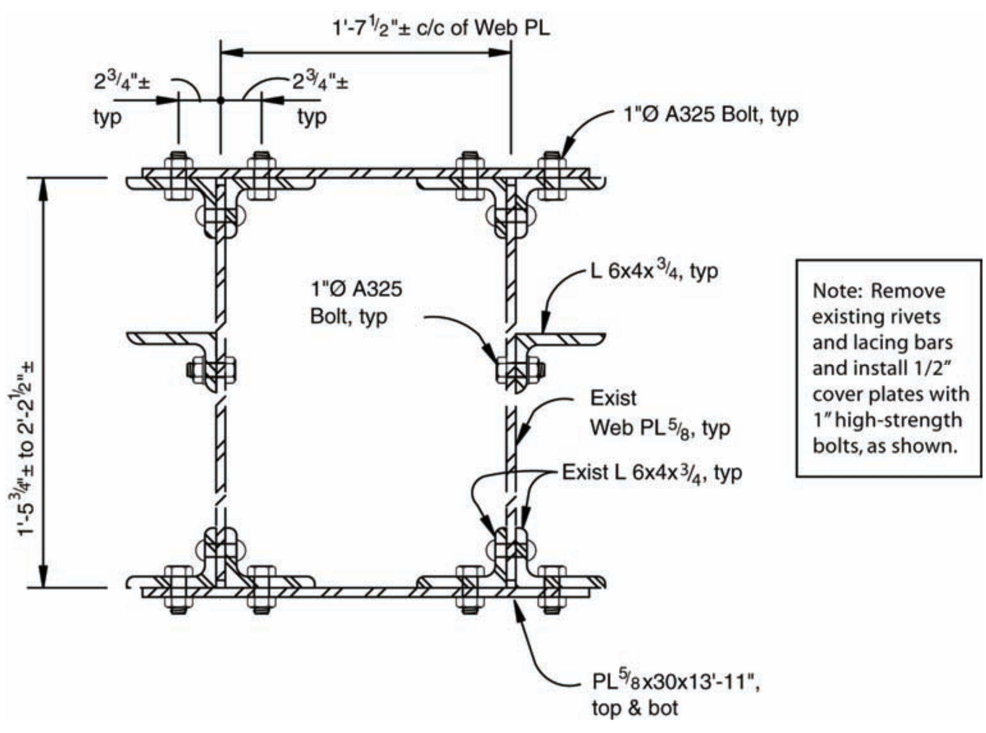

Figure 7-3. Cover Plate Retrofit

If a member is to be strengthened in this fashion, the strength of the connections to the member should be re-evaluated. It is likely that these connections will also need to be retrofitted to develop the strength of the improved member, or the strength of the new cover plates directly attached by the connections.

7.2.2 CHORDS

The chords are the main members of a steel truss bridge and they are typically the strongest and most robust members of all the truss components. The chords are usually designed to carry the axial forces of tension or compression resulting from dead and live loads. Wind loads will usually add only a small percentage to the axial loads resulting from the dead and live loads. For compression forces, chords are designed as built-up members from rolled sections, lacing bars or perforated plates, usually riveted together and occasionally fastened with high strength bolts. For tension forces, eyebars fastened with large diameter steel pins through the "eyes" at each end are usually employed because they are light weight, easily installed and, thereby, low cost. Built-up members are used as tension members as well.

In an earthquake event, the lateral bracing will primarily resist transverse seismic forces while the chords and the diagonal members of the main truss will resist the longitudinal seismic forces. For some multi-span truss bridges, longitudinal seismic forces are carried as added tension or compression forces along the lower chords, for several spans, to braced towers, which carry the seismic forces to the foundations.

Since the majority of the existing bridge trusses were designed before the intensity of seismic forces and displacements were fully understood, the only lateral load most of the trusses were designed for was the wind load. Therefore, many of these bridge trusses were under-designed for earthquake loadings.

The primary seismic vulnerability of eyebar tension chords is their inability to withstand added tension forces from dynamic seismic action and their inability to carry compression forces. In rare occurrences, under seismic dynamic excitation, the force reverses; the tension member is thrown into compression, which it cannot resist; and the eyebar plastically buckles. This action will dramatically change the dynamic resistance of the structure. As the force reverses again, the eyebar is thrown into tension, which has the potential of fracturing the eyebar.

Possible retrofit measures for eyebar chord members to increase their capacity are: to brace them to adjacent members; to replace the eyebars with sections capable of carrying the loads, which is an expensive procedure; or to change the dynamic response of the structure by isolation or by adding dampers; or to use these measures in combination.

The seismic vulnerability of compression chords is in their limited ductility. Under high seismic compression demands, the chord members can plastically buckle as a whole, or the angles, cover plates, and lattice plates which make up the built-up member can plastically buckle locally.

Possible retrofit measures for compression chords are to increase buckling-resistance by adding stiffeners or by replacing lacing bars or plates that have high b/t ratios with thicker plates, or by adding stiffeners as shown in Figures 7-2 and 7-3. When replacing lacing bars of a main compression element, care should be taken to insure that local and global stability will not be compromised.

7.2.3 PRIMARY DIAGONALS AND VERTICALS

Primary diagonal and vertical members, similar to chord members, act in tension or compression under dead, live, and wind loads depending on the truss configuration and the position of these members in the truss.

Their seismic vulnerabilities and potential retrofitting measures are very similar to those for chord members.

7.2.4 EYEBARS, HANGERS, AND OTHER TENSION-ONLY MEMBERS

Some truss members such as eye-bars acting as tension diagonals or tension verticals, and hangers supporting drop-in spans, were designed to carry tension forces only. These members are usually smaller than the eyebars used as tension chords and are connected with smaller steel pins through the "eyes" at each end. They act in tension only when the bridge truss is subjected to dead, live and wind loads.

Similarly to chord tension members, the seismic vulnerability of a smaller eyebar member may be its inability to withstand added tension force from dynamic seismic action and its inability to carry compression force. Under seismic dynamic excitation, the eyebar force may reverse and the tension member may be thrown into compression, which it cannot resist, and the eyebar may plastically buckle. As the force reverses again, the eyebar will be thrown into tension, which has the potential of fracturing the eyebar.

The most common retrofit measure for these "tension only" members is to reduce their slenderness ratio by clamping adjacent eyebars together at several points and bracing them at these points to adjacent members if possible. Check to ensure that no local buckling will occur. Bolting or clamping bars or plates along the length of the eyebars to act as lateral bracing may be effective. It is easier for a contractor to bolt or clamp bars and plates because the eyebars are under tension and often the existing eyebar materials are not weldable. If the eyebar steel is weldable, as demonstrated by testing, the eyebars could be detensioned for welding by transferring the load to temporary members placed along the sides of the eyebars. This is a difficult operation for contractors but may be the only retrofit measure available.

The hanger elements, at a hinge or supporting a drop-in span, may be a seismically vulnerable component of a truss bridge. These hangers are designed to carry vertical loads and may not able to resist the lateral forces and torsional demand of dynamic loading from an earthquake, if the shear locks fail. The most common retrofit measure of the hanger is to prevent the hangers from receiving lateral loads by reinforcing the shear locks to ensure that there is no failure of these devices. Often shear locks have to be modified to provide additional longitudinal displacements and they can be strengthened and reinforced at the same time. If retrofit of the hanger members becomes too difficult, a new load path can be created to reduce the forces in the hangers, as discussed in [Imbsen and Liu, 1993].

7.2.5 BRACING AND SECONDARY MEMBERS

Bracing and secondary members are similar to chords, lateral bracing, verticals, and diagonal members. They are not robust members and are usually assembled from a pair of angles riveted to lacing bars, forming an I shape, or riveted from four angles and lacing bars forming a rectangular shape. Their seismic vulnerabilities are their non-ductile type of behavior, and that they may not have capacity to resist large demands from an earthquake.

One retrofit measure to increase the ductility of these members is to replace the latticed member with plates forming a ductile box member, or to replace I-shaped members with new ductile members designed to provide larger sectional properties.

7.3 CONNECTIONS

Bridge trusses constructed before World War II used riveted connections between the truss members and the gusset plates. After the war, rivets continued to be used, but eventually high strength bolts were developed which could be tightened with an easy-to-use pneumatic wrench, so bolts gradually replaced rivets. By the 1960's, the art of riveting had ended and bolting has been used almost exclusively since that time. Very few bridge trusses have been constructed using fully welded connections.

7.3.1 EXISTING CONNECTIONS AND SPLICES

Existing connections and splices, between or within members having a ductility demand-to-capacity ratio greater than unity, should be strengthened so that their nominal capacity is at least 25% greater than the nominal capacity of the members they connect.

7.3.2 NEW CONNECTIONS AND SPLICES

New connections and splices, between or within members having a ductility demand greater than unity, should be designed to have a nominal capacity at least 25% greater than the nominal capacity of the members they connect.

7.3.3 RIVETED CONNECTIONS

Riveted connections may be readily strengthened by replacing rivets, one or two at a time, with high strength bolts, as illustrated in Figure 7-4. Friction bolts are recommended as a replacement for the rivets. By doing so, no significant redistribution of loads will occur within the connection.

The bolts may be of the same nominal size as the original rivets, or of a larger diameter, in which case the existing rivet holes must be reamed out to a larger diameter to accommodate the bolts.

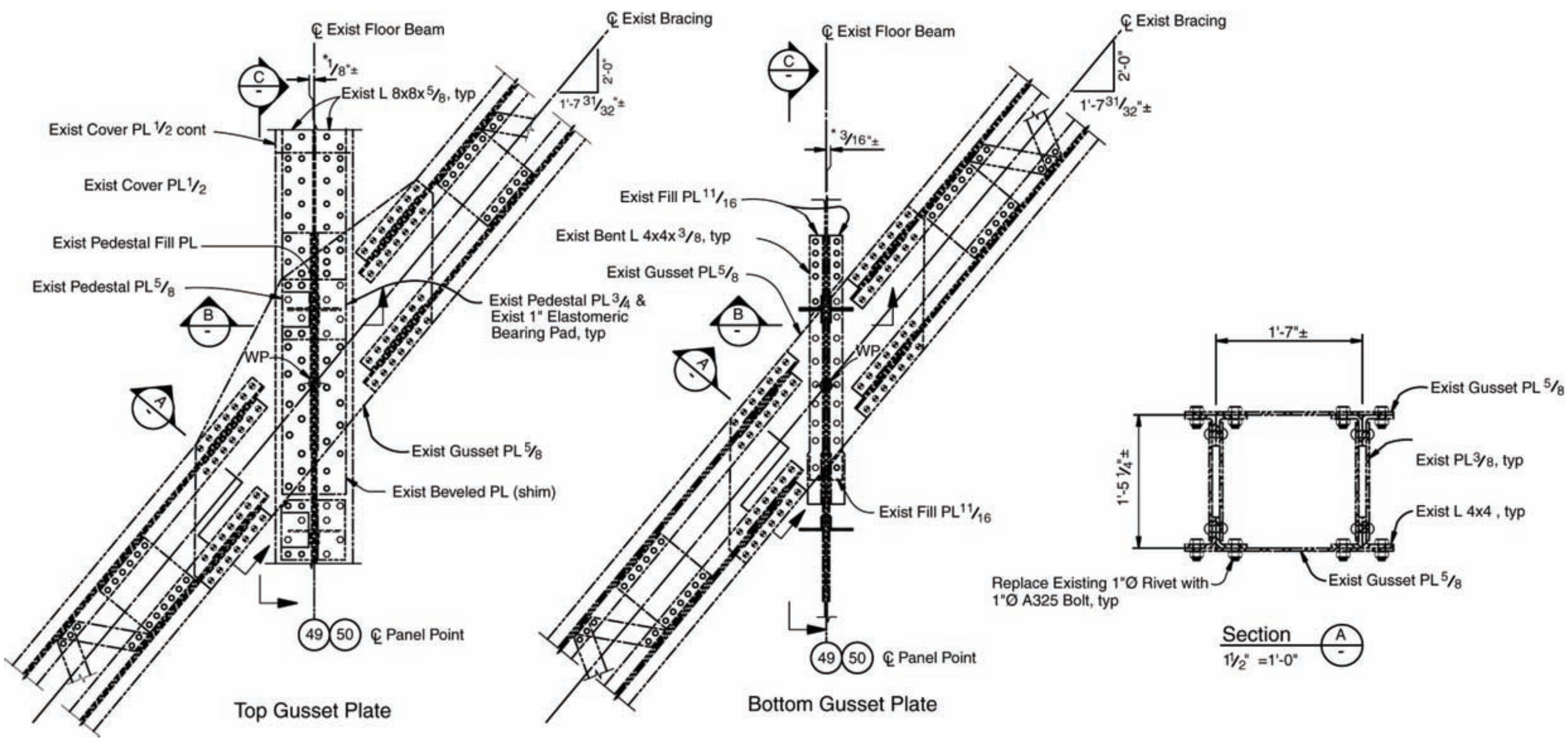

Figure 7-4. Strengthening Riveted Connections by Replacing Rivets with Bolts

7.3.4 BOLTED CONNECTIONS

In the latter part of the 20th century the use of rivets for new construction was discontinued in favor of A325 high-strength bolts. The A325 bolt is also commonly used for replacing rivets in retrofitting older bridges. The A490 bolts, compared to A325, are less ductile, more expensive, more difficult to tighten, and should be used only if required to provide higher capacity.

7.3.5 WELDED CONNECTIONS

As stated above, welded connections are rarely used in steel truss bridge construction. For seismic retrofitting, welding can be used in conjunction with high strength bolt replacement of riveted connections as a retrofit measure, if more capacity is required. It is advisable to perform chemical/metallurgical tests of the base metal if welded connections are being evaluated as a retrofit measure, because many of the older steels may not be suitable for welding.

Partial-penetration welds shall not be used in regions of members subject to inelastic deformation. Outside those regions, partial-penetration welds shall provide at least 150% of the strength required by calculation, and not less than 50% of the strength of the connected parts (regardless of the action of the weld). Potential fatigue problems have to be considered when welding is used in seismic retrofit projects.

7.3.6 PINNED CONNECTIONS

Some of the older steel bridge trusses have pinned connections in the plane of the truss. Often several members of the truss will converge to a common connection point in a chord, or at an intermediary point in a diagonal member. These converging members will have rounded cut-outs in their webs that fit on to a single common large-diameter pin. The pins themselves rarely need to be replaced because they are so enormous that they typically remain elastic even under large seismic forces.

Seismic vulnerability of pinned connections is usually associated with the members connecting to the pin. Some of the members may be overstressed because of reduction of the cross sectional area at the pin location, or they may buckle near the pin due to lack of stiffeners. The capacity of each connecting member at, or near, a pinned connection should be checked for compliance. The potential for stress reversal should also be evaluated for a pinned connection. Earthquake-induced stress reversal could be disastrous to the pinned connections near supports.

7.3.7 GUSSET PLATES

Gusset plates shall be designed in accordance with the following requirements:

- Edges with l/t greater than $0.75 \cdot (E/F_y)^{1/2}$ shall be stiffened.

F_y	$0.75 \cdot (E/F_y)^{1/2}$
36	21
50	18
70	15
100	13

- The stiffener plus a width of gusset plate equal to ten times its thickness shall have an *l/r* ratio less than or equal to 40.
- The stiffener shall have an *l/r* ratio less than or equal to 40, between fasteners.
- The stiffener moment of inertia shall be greater than the larger of

$$I_s = 1.83 \cdot t^4 \sqrt{(b/t)^2 - 144}$$

$$I_s = 9.2 \cdot t^4$$

where:
I_s = the moment of inertia of the stiffener about its own centroid
b = the width of the gusset plate perpendicular to the edge
t = the thickness of the gusset plate
L = unbraced length

- The spacing of fasteners shall satisfy the *LRFD Bridge Design Specification* requirements for stitching and sealing bolts.
- If the attached member will buckle in the plane of the gusset, the gusset shall be stronger than the member.
- If the attached member will buckle out-of-plane, the gusset shall allow the member to rotate. This requirement need not be applied to box-shaped members connected at both flanges.

An example of a gusset plate retrofit with angles to stiffen the edge of the plate is shown in Figure 7-5.

7.4 DECK SYSTEMS

The predominant deck system for most bridge trusses is a reinforced concrete deck supported on longitudinal steel stringers connected to, or supported on, transverse floor beams. In deck trusses, the transverse floor beams connect to the top chords. In through trusses, the transverse floor beams connect to the bottom chords.

The floor beams are connected to the chords at the panel-point gusset plates by clip angles that are riveted or bolted to the web of the floor beam. This type of connection transmits shear and a little moment in the vertical plane and is completely flexible in the horizontal plane. The same type of connection connects the stringers to the floor beams in deck systems that have the stringers framing into the floor beams. For deck systems that have the stringers supported on the top flange of the floor beam, the connections are usually rivets or bolts connecting the bottom flange of the stringer directly to the top flange of the floor beam. A lateral bracing system is usually placed under the plane of the stringers and is framed into the intersections of the floor beam and the chord.

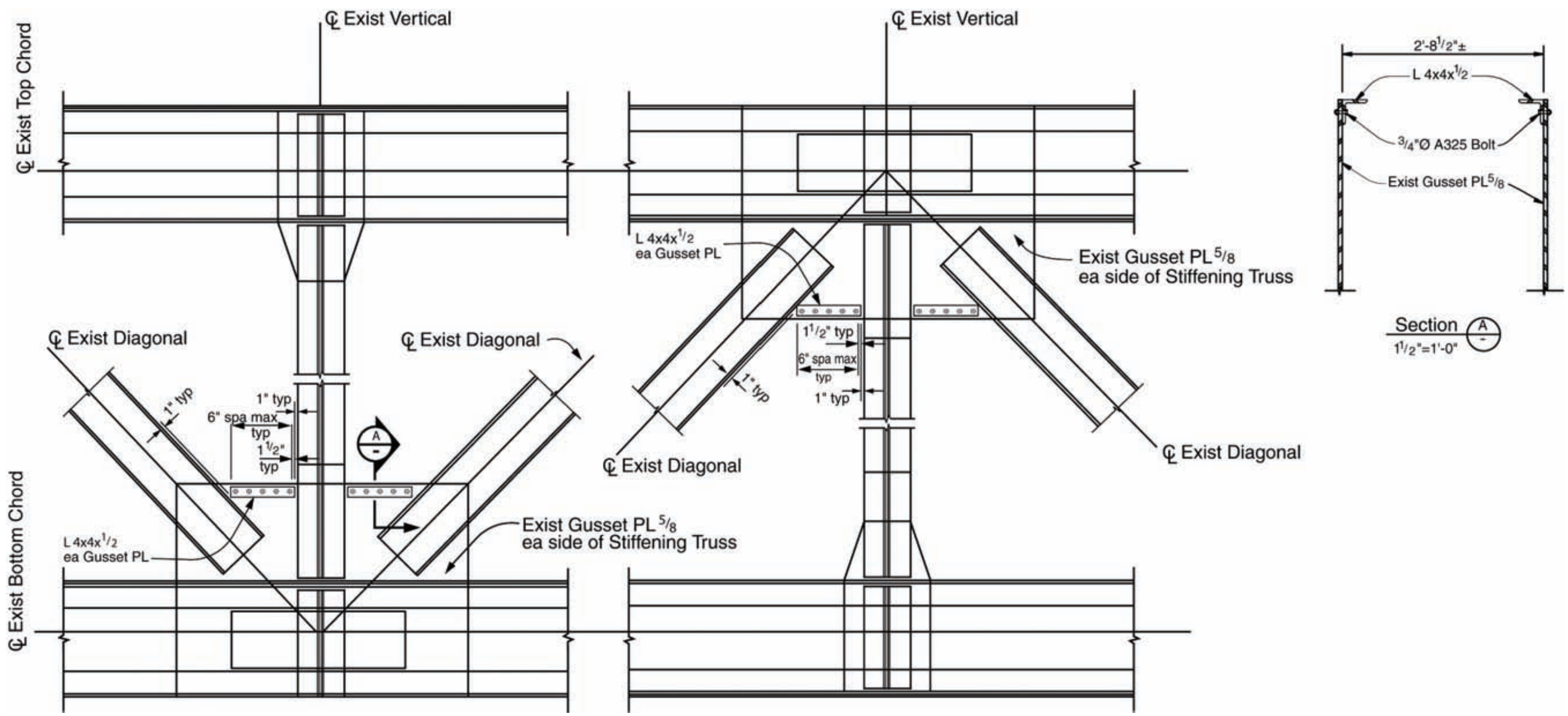

Figure 7-5. Gusset Plate Retrofit with Angles

The concrete deck is supported directly on the top flanges of the floor beams and the stringers, which are usually spaced four to six feet apart. The deck is generally not connected to the stringers with the intent of achieving composite action. Deck joints, placed over the floor beams, are spaced along the bridge-length from one to four floor beams. These deck joints are placed in the deck to eliminate any frictional composite-action participation from the differential expansion and contraction of the concrete deck with the chords of the truss. Usually the only connections holding the deck in place laterally are a concrete lip, or fillet, bearing against the top flanges of the stringers, and the cement bond of the concrete to the steel supports. Bonding of the concrete to the steel on the top flanges of the stringers will also provide partial composite action. However, this type of bonding is not intentional, and because the bond may break during service, should not be relied on for contribution toward strength or stiffness.

7.4.1 POTENTIAL VULNERABILITIES

The concrete deck is a heavy mass concentrated in the plane of the lower or upper chord along the length of the bridge. Under strong seismic excitation, the truss is deflected in a horizontal plane, with the amount of deflection controlled by the effectiveness of the lateral bracing system. The concrete deck is forced to follow these deflections and it structurally participates with the horizontal truss action until the concrete lips, or fillets, holding the deck in place, are sheared off. The concrete deck panels then float free and collide with one another and with the truss members. This may lead to cracking of the concrete deck, destruction of the deck joints, and damage to the truss members, leaving the deck displaced. The seismically-damaged deck may need extensive repair or complete replacement.

7.4.2 RETROFIT MEASURES

Connecting the concrete deck to the floor beams and stringers to hold the deck in position is one seismic retrofit option, with an added benefit that the horizontal stiffness of the concrete deck can be used to augment the resistance of the lateral bracing system. This retrofit measure may assist the steel lateral bracing system that may be deficient in resisting seismic lateral loads, as discussed in Section 7.5. The total length of the concrete deck can be made to act compositely by eliminating the deck joints at the floor beams; hence, the entire deck can be made joint-free.

Composite action can be achieved in the deck by coring through the concrete deck over the stringers and floor beams and welding shear studs to the steel top flanges. If the steel is not weldable, holes can be drilled and high strength bolts with two nuts can be installed to make a rigid connection for the shear studs. However, the concrete deck, stringers and floor beams, acting in composite action, will induce shear and horizontal bending moments with the chords at the floor beam connections, both from thermal movements and from seismic action. This type of retrofit would require significant strengthening of the floor beam-to-chord connections to resist large induced horizontal shears and moments. The added benefit from this retrofit is the significant additional horizontal stiffening induced by the full deck, acting compositely with the chords.

Replacing the existing lateral bracing system with a stiff and ductile system may also significantly reduce horizontal deflection. In this case, the deck can be held in place by connecting it only to the floor beams without changing the non-composite action of the stringers.

The width of the joints in the deck may need to be increased by saw cutting and removing concrete to allow room for the larger deck movements. These two retrofit methods work together and may be sufficient to reduce or prevent seismic damage to the concrete deck.

A more drastic retrofit measure is to replace the concrete bridge deck with a lighter-weight decking such as a steel orthotropic deck, or a fiber reinforced polymer (FRP) composite deck system which may result in a dramatic reduction of dead weight. Reducing the mass will reduce the seismic forces in the bridge trusses. Of course this is a more costly measure and has to be considered with other factors; however it may be worthwhile if there is a need for a major overhaul of the deck system.

7.5 LATERAL BRACING SYSTEMS

The lateral bracing system for most bridge trusses acts as the lateral load-resisting element. Usually designed in the form of cross frames or chevron frames, the lateral bracing system is placed near to, or in the planes of the top and bottom chords, although a few bridges do not have lateral bracing under the concrete deck.

For through trusses, the lateral bracing system placed under the deck in the plane of the lower chord is the principal longitudinal lateral load-resisting element. This system carries the lateral inertia loads of the deck and the wind loads on the lower portion of the truss longitudinally to the support bearings. The lateral bracing system placed in the plane of the upper chord is the secondary system that carries lateral wind loads on the upper portion of the truss longitudinally to the portal frames that are discussed in Section 7.6.

For deck trusses, the lateral bracing system placed under the deck near or in the plane of the upper chord is the principal longitudinal lateral load-resisting element. This system carries the lateral seismic inertia loads of the deck and the wind loads on the lower portion of the truss longitudinally to the portal frames that are discussed in Section 7.6. The lateral bracing system placed in the plane of the lower chord is the secondary system that carries lateral wind loads on the lower portion of the truss longitudinally to the truss supporting bearings.

Fortunately half-through highway trusses have not been used often, as these trusses are the most difficult to seismically retrofit and will need special study by retrofit designers. Lateral bracing systems are used in the planes of the upper and lower chords. However a third lateral bracing system under the deck may or may not be used. If it is not used, the lateral seismic inertia loads of the deck must be delivered to the vertical truss members, which produce out of plane bending in these members.

7.5.1 POTENTIAL VULNERABILITIES

The lateral bracing systems for most existing bridge trusses were designed to resist only wind loading, as the seismic loads that were specified at the time were too low and did not control the design of the lateral bracing systems. For areas with low wind-loads, minimum bracing requirements often governed sizing of lateral bracing members. For areas with high wind-loads, the wind-loading used to design the lateral bracing at that time is much less than the seismic loading recommended for seismic retrofitting in these *Guidelines*, except in areas of very low seismicity.

Most of the lateral bracing in bridges designed to older specifications will have non-ductile and slender bracing members, often using built-up sections from angles and lacing bars. Recent studies have demonstrated that the hysteretic energy dissipation of the slender bracing members can be very low. The members are designed for tension only and will buckle plastically under seismic compression loading. Under seismic tension loading, the members can easily fracture under a few cycles of stress reversals, under a phenomenon called *low cycle fatigue*. In some cases the connections may not survive since the riveted end connections are usually inadequate to carry the tension loading and can fail in rivet-shear.

The lateral bracing system (and the portal frames as discussed in Section 7.6) often will be the most seismically vulnerable portion of a bridge truss in moderate and high seismic areas. Retrofit measures warrant careful evaluation of these important and non-redundant subsystems.

7.5.2 RETROFIT MEASURES

A common retrofit measure for rectangular latticed bracing members is to replace lacing bars with two solid plates and two perforated plates (to allow access for interior painting) by removing the rivets and replacing them with high strength bolts. End connections may be strengthened by replacing the rivets with high strength bolts. Members with I-shapes are difficult to strengthen against buckling, and full replacement may be required. For half-through trusses without a lateral bracing system under the deck, the retrofit strategy is to focus on eliminating or reducing the bending produced in the vertical truss members. Adding a lateral bracing system under the deck may be one approach.

During the construction of these strengthening and replacing operations, the stability of the truss for static and construction wind loads and the stability of the members must be preserved at all times. Detailed evaluation and design requirements for retrofitting latticed members are discussed in [Caltrans 1995].

7.6 PORTAL BRACED FRAME SYSTEMS

Portal braced frames, as shown in Figures 7-6 and 7-7, are important bracing systems of the truss, but they function in several different forms which depend on their position and the type of truss of which they are a component. Portal braced frames usually have two components that form a system: (1) cross bracing to carry lateral loads connected to (2) two truss chord members within the plane of the trusses, which also act as chord-members for the cross bracing in the portal frame.

Portal braced frames are most visually prominent when they are positioned at the ends of the through- trusses, sloped toward, and connected to, the top chords. Often these sloped portals are designed as decorative entrances with the date of construction displayed in steel. The primary function of this type of portal braced frame is to carry the compressive forces in the top chords to the support bearings. For this function, the portal braced frame chords act effectively as an

Figure 7-6. Million Dollar Bridge, Alaska, Portal Frame

extension of the top chords of the truss. An important secondary function of the portal braced frame is to carry transverse lateral loads from the upper chord lateral bracing system through the portal braced frame chords to the support bearings. For this secondary function, the portals act as a moment resisting frame; hence the name, portal braced frames.

For cantilever and continuous through-trusses, portal braced frames are positioned at the intermediate support points at piers. Often these portal braced frames are placed on a diagonal, sloping from the lower chord support points to the upper chords. In some through-trusses, the sloping portal braced frame is not used and the portal braced frame location is shifted to a vertical position directly over the support point. For both of these positions, the portal braced frames are usually obscured and visually lost among the many other truss members.

For single spans, drop-in spans, cantilever, continuous deck and double deck trusses, the portal braced frame systems are required and are used in similar positions as in a through-truss, but they are often out of view to the bridge user.

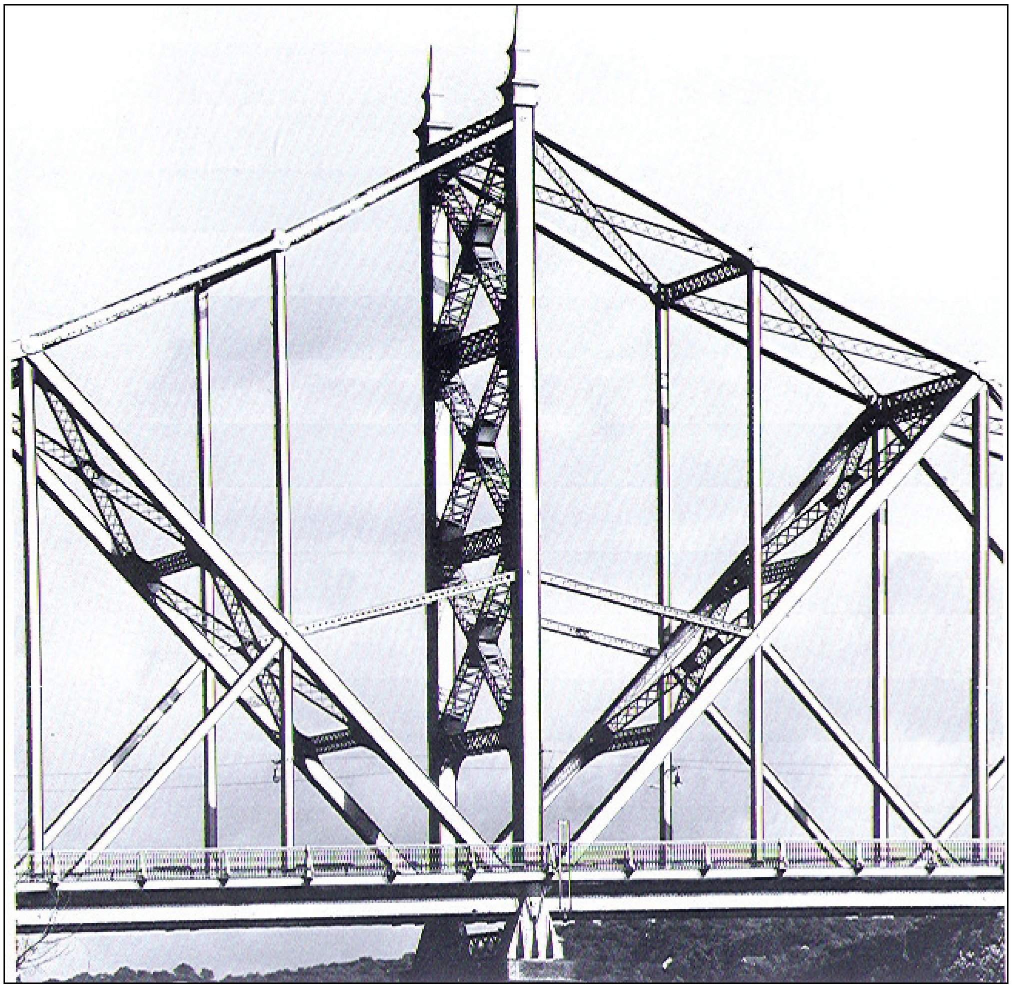

Figure 7-7. Portal Braced Frames at Intermediate Pier Support of a Through Truss (Plowden)

7.6.1 POTENTIAL VULNERABILITIES

Portal braced frame systems are the primary transverse lateral load-resisting elements in a truss system and act as extensions of the longitudinal lateral bracing systems discussed in Section 7.5. Similar to the longitudinal lateral bracing system, the portal braced frame systems of most existing bridge trusses were designed to resist only wind loading, as the seismic loads that were specified at the time were too low and did not control the design of the portal braced frame systems. Also, similar to the longitudinal lateral bracing system, most of the lateral bracing members in portal braced frames, designed to older specifications, will have non-ductile and slender bracing members with inadequate connections. The most likely failures are plastic buckling of the bracing members, shearing or tearing-out, of the riveted connections of the bracing members, or the complete out-of-plane buckling of all the bracing members.

The most important potential seismic vulnerability of the portal braced frame system is failure of the in-plane sloping chord members of the frame. These chord members are subjected to a combination of compression forces from the truss compression chords, and to bending and shear forces from the lateral loads on the bracing system acting to overturn the portal frame. The most likely failure is plastic buckling of the chords or plastic lateral displacement of the frame.

7.6.2 RETROFIT MEASURES

Options for deficient portal braced frame systems are to accept some inelastic action, or to strengthen or replace existing members. Some strengthening retrofit measures are:

- Replacing rivets with high strength bolts.
- Replacing the lattice bars on the portal chord members with perforated plates.
- Adding section area to the portal chord members by bolting on steel plates or small rolled sections.
- Strengthening the moment-connections from the portal chord members to the floor beam connections.

Seismic isolation is also a retrofit measure that reduces demand on the portal braced frame system. Isolation essentially de-couples the structure from potentially damaging earthquake-induced ground motions by replacing existing bearings with energy-dissipation bearings. This de-coupling is achieved by increasing the flexibility of the system because the longer the vibration period, the lower the seismic demand on the structural elements.

There are several new retrofit measures for portal based frame systems, using ductile energy dissipation devices. Energy-dissipating devices and framing configurations can be placed in deck trusses as retrofitting measures for portal braced framing, as shown in Figure 7-8:

1. **Eccentrically Braced Frames** (EBF - Figure 7-8 a) [Sarraf and Bruneau, 2002]. In this portal frame retrofit, a link is created by offsetting the diagonal braces by an eccentricity, “e”. The link is designed to behave elastically when the structural frame is subjected to wind load, but the link will yield and will dissipate energy in a ductile manner, under large seismic forces.
2. **Vertical shear link** (VSL - Figure 7-8 b) [Sarraf and Bruneau, 2002]. A shear link is added between the chevron bracing frame and the horizontal beam. This link is designed to yield under large earthquake forces so as to protect the rest of the bracing system. The main advantage of this system is that the yielding of the link will happen outside the horizontal member that typically supports the floor system. This will make replacement of the yield link after a major earthquake easier and more economical.
3. **Triangular Added Damping and Stiffness** (TADAS - Figure 7-8 c) [Sarraf and Bruneau, 2002]. In this device, a stack of short triangular-shaped steel cantilever plates is placed between the chevron bracing frame and the horizontal beam. Under lateral loading, the plates, acting together as a stack of cantilever beams, are subjected to a bending moment, which produces a yielding plastic moment simultaneously along the entire length of all the cantilever beams due to their triangular shape. Under reverse lateral loading, the cantilever beams are bent in the opposite direction. This action produces very open hysteretic curves indicating very efficient energy dissipation.

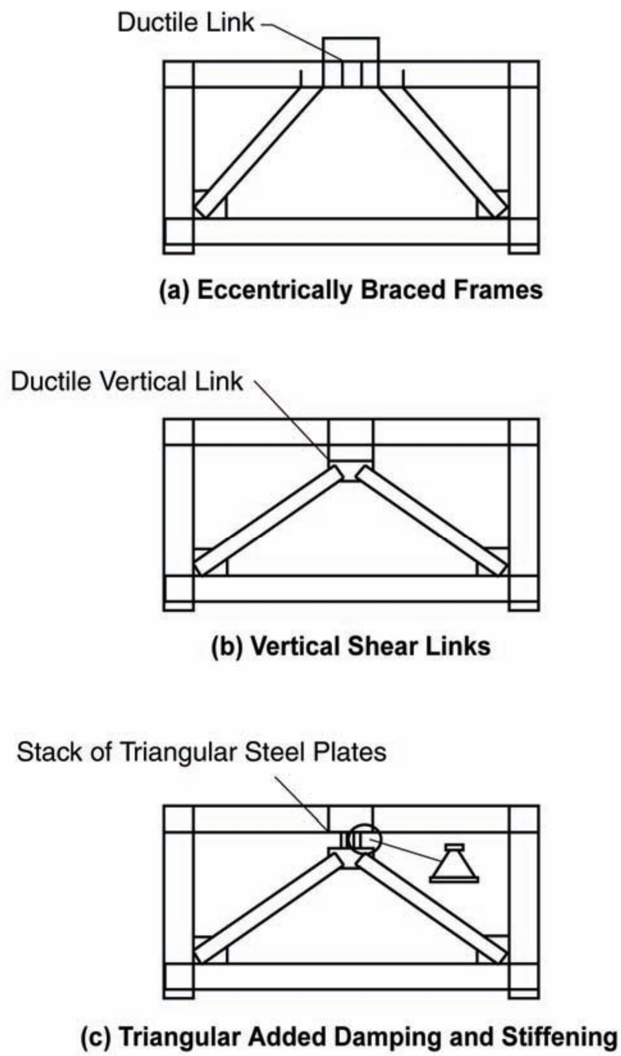

Figure 7-8. Ductile Steel Energy Dissipation Devices

Post-tensioning tendons can be arranged in innovative configurations as retrofit measures for portal braced framing by replacing portal bracing with post-tensioning tendons [Matson and Buckland, 1995] as shown in Figure 7-9. The same tendons that are used in post-tension concrete construction can be installed in the end portals of deck trusses to provide comparable lateral load capacity, as in the un-retrofitted structure. This system will modify the lateral force load-path and will transfer less force to critical members, such as the support bearings. The tendon elements can also be designed to behave inelastically, under severe ground shaking, to dissipate seismic energy.

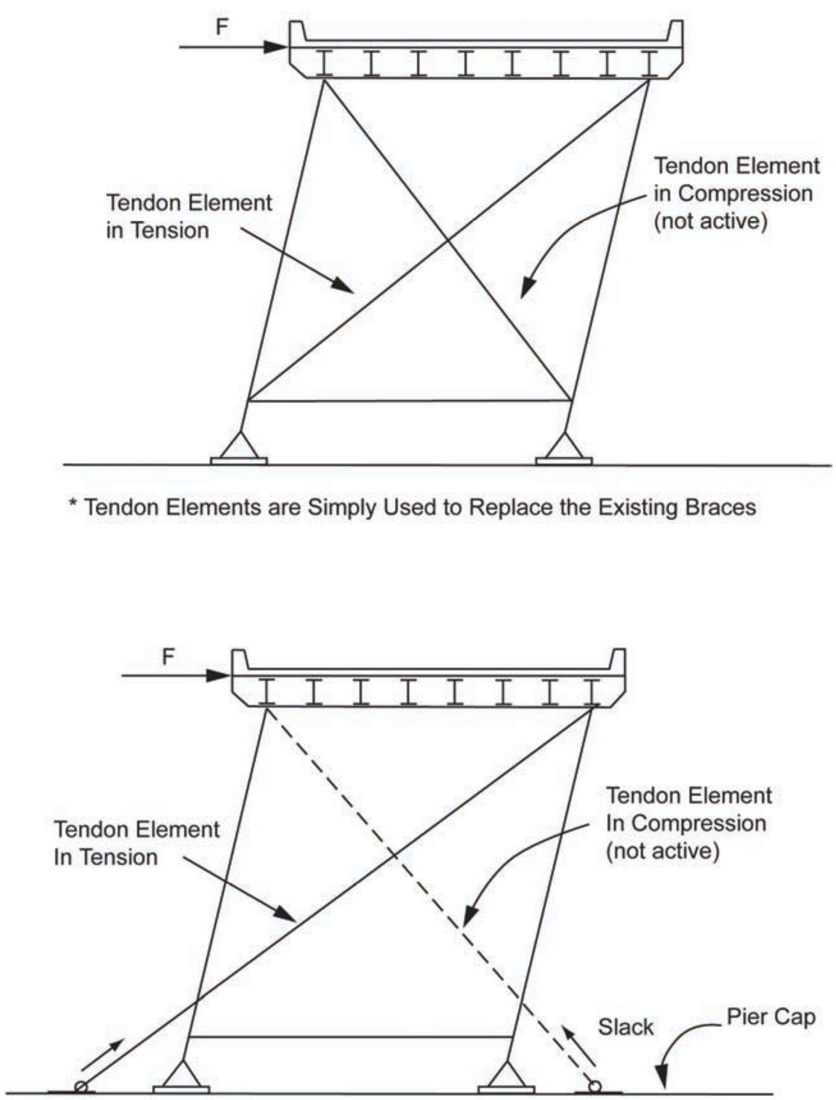

Figure 7-9. Post-Tensioning Tendon Seismic Retrofit

7.7 SWAY FRAMES

Sway frames are transverse bracing systems placed at each panel point that function, as their name implies, to resist side sway distortion of the truss cross section. Sway frames were used to retain the rectangularity of the truss cross section against distortions produced by wind loads. Often, sway frames are incorporated as the transverse component of the lateral bracing system. They are designed as moment-resisting members framed into the truss vertical members, and the moments developed at their ends produce some bending in the verticals.

For through-trusses, sway frames take various forms depending on the depth of the trusses. For shallow-depth trusses, the sway frame can take the form of shallow warren trusses; for deeper trusses, they usually take the form of a chevron truss or of several chevron trusses stacked one above the other.

For deck trusses, the depth of the sway frames is not restricted and often full-depth-and-width cross bracing is used. For double deck trusses, the floor beams supporting both decks develop partial end moments that function as moment-resisting members to resist transverse distortions.

7.7.1. POTENTIAL VULNERABILITIES

The seismic vulnerabilities of sway frames, similar to the longitudinal lateral bracing systems, were designed to older specifications and may have non-ductile and slender bracing members with inadequate connections. The most likely failures are plastic buckling or tension fracturing of the bracing members, the riveted connections of the bracing members shearing or tearing-out at the end connections to the truss vertical members, or the complete out-of-plane buckling of the sway frame.

7.7.2 RETROFIT MEASURES

Seismic retrofit measures for sway frames must be developed in conjunction with the seismic retrofit measures of the lateral bracing system. Bending moments developed in the truss verticals by the sway frames must be included in the retrofit strategies of the main trusses.

Strengthening members to improve resistance to fracturing, or adding sectional area to improve resistance to buckling, are common retrofit measures. The end connections of the sway frame to the verticals may be strengthened by replacing rivets with high strength bolts. If this is not sufficient, the entire sway frame may need to be strengthened or replaced. Sway fames using truss systems with I-shaped cross sections are difficult to strengthen against buckling and full replacement may be required.

7.8 CONCRETE PIER SHAFTS, CELLULAR STEEL SHAFTS, AND STEEL BRACED TOWERS

Steel highway bridge trusses are usually supported on concrete pier shafts, on steel shafts of cellular construction, on steel braced frames, and, occasionally, on unreinforced masonry piers.

Concrete pier shafts are usually fixed to the concrete foundations by embedded reinforcing bars that often were spliced at the base of the pier shafts, for ease of construction. The shafts were designed to resist lateral wind loads only, are lightly reinforced, and many have a concrete cover over the reinforcing bars that is too thin by today’s standards.

Steel shafts of cellular construction or steel braced frames are usually attached to the concrete pile caps, or to the caissons, by steel anchor bolts. Towers of steel cellular construction usually are shop connected in older structures by riveting and in newer structures by shop welding. Field connections are typically riveted for older structures and are bolted with high strength bolts for newer bridges. Welded field connections have rarely been used. Steel-braced frames are usually made up of rolled sections that are field-riveted. These steel supports were designed also to resist lateral wind loads only.

7.8.1 POTENTIAL VULNERABILITIES

The reinforcing bars, splices, and stirrups in concrete pier shafts are inadequate by current seismic standards; they are confined with small diameters and widely spaced stirrups**.** Concrete pier shafts constructed in the first half of the 20th Century used reinforcing bars with a low profile or no deformation-profile that had a lower bond stress than the bars that are used today and they had a yield point of 40,000 psi. One of the most seismically vulnerable points in a concrete shaft is at the pier base where the vertical reinforcing bars in the shaft are spliced to the reinforcing bars embedded in the concrete footing. The stirrups are inadequate to provide confinement to the concrete, and the thin concrete cover may allow reinforcing bars to corrode.

Steel braced frame members are usually slender and inadequately braced, with potential for buckling under seismic lateral loads. Typically, steel braced frames and cellular steel towers are inadequately anchored to the substructure and cannot resist seismic shear and uplift. The main supporting legs of the steel braced frames and the panels in cellular steel towers can buckle under seismic over-turning forces.

7.8.2 RETROFIT MEASURES

The *Bridge Retrofitting Manual* offers many retrofit measures for concrete piers, shafts, and columns.

The panels in cellular steel shafts may be vulnerable to shear buckling from lateral seismic loads and may require reinforcement by bolting on steel stiffeners or bolting on steel plates. The steel panels at the shaft base are vulnerable to buckling under compression from lateral seismic loads and may also require reinforcement by bolting on steel stiffeners or bolting on steel plates. Stiffeners preferably should be installed on the inside of the cells because installing stiffeners on the exterior is unsightly and may produce objections from the owner or from members of the community.

Field-welding of stiffeners or plates is not recommended unless the steel in the cells is tested for weldability. If the steel is weldable, the designer should consider that the cellular walls are under dead-load stress and the effect of the welding on the steel under stress must be evaluated; the steel cells may be too small to admit a welder who can comfortably weld in all the positions required to weld a stiffener onto the cell walls; steel plates may too large to insert into a small cell and can only be used if a method is developed to insert, lift, and hold the heavy plate in position for welding.

The anchor bolts, and the anchorage plates and stiffeners to which the anchor bolts are attached, may be inadequate under lateral seismic loads requiring the addition of more anchor bolts. Additional anchor bolts may be added by anchoring the bolts in new holes cored into the concrete footing. Adding the new anchorage plates and stiffeners by bolting is preferred but restricted space may force the welding of these items.

In braced steel towers, the legs of the tower are primary members carrying vertical compression loads and may be vulnerable to axial buckling. These members may be strengthened by adding steel plates and stiffened by adding stiffeners. If stiffeners are required they can be shop welded to the strengthening plates and these plates can be bolted onto the existing legs of the tower.

Adding steel plates to the legs that are often rolled angles or H-sections can be a difficult design problem, particularly in finding configurations for drilling bolt holes and adding the plates. The legs are under dead load stress, which makes them difficult to weld, or to remove and replace from the bracing system.

The anchor bolts at the base of the tower legs and the anchorage plates and stiffeners to which the anchor bolts are attached, may be inadequate under lateral seismic loads. These components may require replacing with larger anchor bolts or the addition of more anchor bolts. Replacing the anchor bolts is difficult as it requires cutting the bolt off, coring the concrete around the bolt, coring through the steel anchor grillage (if any), and removing the concrete core with the embedded anchor bolt. Additional anchor bolts may be added by anchoring the bolts in new holes cored into the concrete footing and providing new steel collars around the legs to anchor the new bolts. Securing the new collar may also be difficult to accomplish because of restricted space and the small size of the leg member.

The bracing members under vertical loading are secondary members and act to laterally support the primary leg members and increase their resistance to buckling. Under lateral wind and seismic loading, the bracing members can be considered as primary members for carrying these lateral forces. Under seismic lateral loading, the bracing members are vulnerable to buckling under compression forces, fracturing under tension forces, and the riveted connections of the bracing members are subject to shearing or tearing-out at the end connections.

Replacing deficient bracing members is usually less costly than repairing, and bracing can usually be easily replaced with stronger or stiffer members. Deficient end connections can be strengthened by replacing rivets with high strength bolts. Gusset plates connecting bracing members can be easily replaced, and will usually need replacement if the members framing into it are replaced.

Another retrofit approach is to provide damping to absorb energy, and isolation to lengthen the period of the tower. This retrofit approach was successfully applied to the 1958 Carquinez Straits Truss Bridge [CH2MHill/HNTB 1996], shown in Figure 7-10.

The braced tower was allowed to rock up and down under seismic loading, but was restrained by the flexing of two thin, ductile steel box beams; the rocking lengthened the fundamental period of the tower to lower seismic input forces and the ductile flexing of the steel box beams absorbs energy that reduces seismic displacement.

Figure 7-10. 1958 Carquinez Straits Truss Bridge showing Rocking Steel Box Beams that Provide Isolation and Damping of the Steel Braced Towers

7.9 SUPPORT BEARINGS

Bridge bearings function to support the vertical load of the bridge at piers and abutments, restrain or allow longitudinal translation for temperature movements and restrain transverse movements while allowing live load rotations. The bearings restrain horizontal loads from traffic, from wind, and from earthquakes. Bridge bearings are designed to transmit these forces to the substructure, to provide for these complex articulations, and to last the life of the structure. They are the most important element in a truss bridge and a failure jeopardizes the function and life of the bridge.

Bridge support bearings installed on existing truss bridges are usually fabricated structural steel devices. Designers of older structures often designed the bridge bearings and detailed them on the design drawings as part of the design process. Often these support bearing designs reflected the training or personal preferences of the bridge designer or, occasionally, were standard designs developed by owners of the bridges, such as state and toll bridge authorities.

Many of the bearings installed in older bridges were designed as steel castings or riveted steel plate designs. After the World War II, bearings were often designed as welded plates. Most bearings for truss bridges have large diameter pins that provide for live load rotations while transmitting horizontal forces. Some bearings that allow longitudinal translation are supported on steel rollers. These pins and rollers can lock-up from corrosion. Corrosion is the usually the most frequent cause of bearings not functioning as intended.

Bridge designers today do not have to resort to designing and detailing bridge bearings for new truss bridges, or for retrofitting existing truss bridges except in special cases, because a number of bridge bearing types and devices are available today from bearing manufacturers. The catalogues of bridge bearing manufacturers display a large number of types, shapes, sizes, and functions. A bridge owner may direct the bridge designer to use a certain type of bearing for the retrofit construction because of the owner's previous good experience with the performance of the device.

For the retrofit of existing truss bridges, the designer may be constrained in the selection of bearing devices because of space restrictions or large seismic forces or displacements. For these conditions, the designer must be creative and devise a bearing design that can be fabricated and installed and that will accommodate the forces and displacements. This situation occurred during the design of the new San Francisco-Oakland Bay Bridge East Bay replacement span. The design seismic forces are many times larger than any used in previous seismic designs and standard products of bearing manufacturers would not work. The designer of the new bridge worked with a foundry capable of very large steel castings and developed what could be the largest seismic bearing ever used.

The primary function of fixed bridge-bearings for both new bridges and for retrofitted bridges is to transfer vertical gravity loads, vertical loads from wind and seismic overturning, and wind, traffic and seismic lateral loads from the truss superstructure to the bridge piers. While performing these functions, the fixed bearings must accommodate rotation movements from live loads and resist horizontal loads from traffic, wind, earthquake, and temperature. Moving bearings that slide or roll on rollers also transfer gravity loads and, in addition, accommodate rotation movements from live loads and allow horizontal movement from temperature effects with little resistance. The bearing design often requires sliding and rolling bearings to accommodate movement in one direction, usually longitudinally, and to restrain movement in the other direction.

Most bridge bearings are designed to function for the design-life of the bridge. Some bearings on existing bridges may be in excellent condition and can perform their gravity and thermal functions, but a seismic retrofit analysis of the bearings may demonstrate they cannot carry seismic loading. This was the classic failure mode of the famous Pier E9 of the East Bay span of the San Francisco-Oakland Bay Bridge in the 1989 Loma Prieta Earthquake. Some bearings may have had little or no maintenance and are so corroded that they are "locked-up" and do not move, or move only slightly under temperature changes. If they are not to be replaced, these locked-up bearings must be cleaned and lubricated to restore their original function.

A seismic retrofit analysis may demonstrate that the existing bearings supporting the truss cannot carry the seismic loading or displacement, and the bearings will require replacement. The new bearing devices often will not be as high as the old ones. The space above the new bearings can be filled with a fabricated steel spacer, or a cast concrete block can be placed below each bearing, but the concrete block must be anchored to the pier cap to resist seismic lateral loading and overturning forces.

7.9.1. POTENTIAL VULNERABILITIES

Under strong seismic excitations, the existing bearings may become unstable under seismic shear forces or large seismic displacements and topple over, or jam or become permanently displaced. The truss may not collapse, but because the deck is offset from the abutment or adjacent span deck the truss may not be capable of its function of carrying traffic after the event. On multi-span trusses, one displaced truss falling off its bearings, can displace adjacent spans, possibly causing failure to a series of truss spans. These types of dynamic failures of the bearings could produce large, three-dimensional displacements, damage the pier caps, and distort the truss members.

The most likely failure is the shearing off of the anchor bolts holding the bearing in place. This type of failure will allow the truss to be displaced, the deck offset, and the truss may be incapable of functioning to carry traffic after the event

If the bearing cannot accommodate seismic force or displacement, the bearing may fracture, or the anchor bolts most likely will shear off. This dynamic failure of the bearings could produce large, three-dimensional displacements, damage the pier caps, and distort the truss members.

7.9.2 RETROFIT MEASURES

One retrofit measure is to reuse the existing bearings by correcting their deficiencies in place. Strengthening the bearings by bolting or welding on plates, adding more anchor bolts, or extending rotational or rolling capacity by adding length to the base plates are retrofit measures that may allow the use of the modified bearings. Although these retrofit measures may be difficult and costly to install in the field, they may be the only resort if the existing bearings cannot be removed for modification because of bridge weight or other restrictions preventing removal [Mander et al., 1996].

Seismic shear keys can be constructed to limit bearing displacement to a level that the existing bearings can tolerate. Another possible retrofit measure that has not been widely used or researched, is installing dampers of sufficient capacity to reduce forces or displacements to a level that the existing bearings can tolerate. Part of the reason the dampers have not been used is that dampers cost more than just a simple shear key and adding dampers will make the structure much more complex to analyze.

Another retrofit measure is to replace deficient bearings with specially designed structural steel bearings, or with tested and proven proprietary bearings, see Figure 7-11. New bearing designs may need to be laboratory proof-tested. The replacement of deficient bearings with tested and proven proprietary bearings offers selection from a variety of types of bearings, and proprietary bearings may cost less than specially-fabricated bearings.

Pot and disk bearings are considered to be “modern” bearings. Any bridge truss equipped with these bearings is either a modern design or is an older bridge that has had the original bearings replaced. However, large pot and disk bearings of sufficient capacity to support long-span trusses or heavy, wide, or double-deck trusses have not been used extensively, but they are being designed to carry larger loads and may be considered as a retrofit measure for replacing bearings.

Isolating trusses, by using isolation bearings, is another retrofit measure that has proven economical for several seismic retrofit projects. Replacing of deficient bearings with isolation bearings also offers retrofit solutions for vulnerable areas of the truss superstructure, substructure, and foundations. Isolation, as a retrofit measure, may have some restrictions in its application to long-span trusses, or to heavy, wide, or double deck trusses because of the difficulty and the expense of lifting a heavy bridge at the supports to remove and replace the bearings.

Isolation is not covered in these *Guidelines*. Reference can be found in the *Seismic Isolation Manual for Highway Bridges* [Buckle et al., 2006] as well as other documents that cover in detail the technology of isolation [Priestley et al., 1996].

Figure 7-11. Rocker Bearing Replacement

7.10 SHEAR LOCKS (WIND TONGUES)

A wind tongue is a device that transfers the transverse wind load from each end of the suspended span to the cantilever arms of cantilever or Gerber truss bridges. The device is usually located at the lower chord level and allows the truss to expand and contract to temperature in the longitudinal direction while transferring loads in the transverse direction. The device can take many forms but in concept it consists of a steel plate or steel casting, the tongue, sliding in a groove or tube that allows the tongue to slide as it transfers shear loads from wind; it will also transmit seismic loads up to its capacity. Perhaps the more descriptive title is “shear lock”, as it is often called.

The shear locks are connected to the lateral bracing system that collects the wind and seismic transverse loads at each end of the suspended span. The shear locks then transfer the transverse load to the lateral bracing system of the cantilever arms, which then transfer the load to the supporting bearings connected to the substructure.

7.10.1 POTENTIAL VULNERABILITIES

Potential seismic vulnerabilities are inadequate connections of the device to the lateral bracing system and inability of the shear lock to transfer the seismic load and the longitudinal seismic displacements.

7.10.2 RETROFIT MEASURES

Retrofit measures for deficient shear locks are to rebuild or to replace the shear locks with sufficient capacity to carry seismic loads and displacements [Imbsen and Amini, 2002].

7.11 EXPANSION JOINT DEVICES

As noted in Section 1.3.1, "bridge expansion joints" is the popular expression for devices placed at each end of the truss, or installed in the concrete deck within the spans. The devices allow thermal expansion and contraction movements and provide for the safe, smooth, and, optimally, quiet passage of vehicles using the bridge. The proper name of these devices is thermal movement devices or thermal joint devices. These devices should also be designed for seismic displacement.

Expansion joints, placed at each end of the truss, bridge the gap created to allow thermal movement between the end of the deck and the approach slab. The gap distance increases with the span length of the truss. For short spans with narrow gaps, a steel sliding plate is often used, with one side of the slider plate anchored into the concrete deck of the truss and the other side resting on a steel plate imbedded in the approach slab. For longer spans with large gaps, two intermeshing fingerplates span the gap. One fingerplate is anchored into the concrete deck of the truss and the other is fixed to the approach slab. Under thermal movement, the fingers of one plate intermesh with the fingers on the other plate, providing full support for the passage of vehicle wheels.

Many other types of expansion joints have been used over the years by bridge designers who often detailed them on the design drawings as part of the design process. Occasionally the original expansion joints have worn out under millions of wheel-load passages and have been replaced with modern modular expansion joints that are available today from several manufacturers of expansion joints.

Often expansion joints are placed in the concrete within the length of the truss span to break up participation by composite action between the concrete deck and the chords of the truss as noted in Section 7.4. These expansion joints are usually narrow gaps in the concrete deck and may be protected against wear with imbedded steel angles anchored into the concrete deck.

7.11.1 POTENTIAL VULNERABILITIES

The expansion joints are among the most vulnerable elements of long-span bridges. Nearly all of the existing bridge trusses have expansion joints that were designed before modern earthquake design criteria were developed. Seismic excitations may cause large three-dimensional global movement between the truss and the abutments or between the adjacent spans. These movements can easily exceed the capacity of the existing expansion joints. Damage of the expansion joints during a seismic event is highly likely and the damage may prevent the safe passage of vehicles on the bridge during an earthquake or the use of the bridge after the event, even if the rest of the bridge survives with minor damage.

7.11.2 RETROFIT MEASURES

The Do-Nothing alternative is to allow the expansion joints to be damaged or destroyed in a seismic event. The bridge will probably be closed to traffic until maintenance crews can be assembled, the debris removed, and steel plates temporarily placed to bridge the gaps. After a seismic event, steel plates may be hard to find and transport quickly to the bridge site. As a possible preventive solution to this problem, the owner or engineer could require that steel plates be supplied and stored in an easily accessible area near the bridge.

As was discussed in Section 7.4, one seismic retrofit measure is to eliminate the joints in the deck. This will allow the deck to participate in the lateral stiffness of the truss, which will reduce the seismic response of the truss.

A retrofit measure for bridges that require immediate access after a seismic event is to remove vulnerable expansion joints and to replace them with proprietary devices designed and tested for three-dimensional seismic movement. Several manufacturers offer these seismic joints which will allow movements up to about 40” longitudinally, 20” transversely, and 4” vertically. These devices are more costly than the standard joints; the bridge deck must be revised to allow space for placing of the comparatively large fabricated units; and the placement of the seismic joints in the deck will require closing the bridge to traffic. These devices will be required if isolation is selected as a retrofit measure.

A sacrificial seismic retrofit measure for expansion joints has been developed as a non-proprietary concept for installation between the ends of the truss and the abutments [Hinman et al., 1998]. This retrofit can be altered to fit different configurations of the two abutments and the two ends of the truss. Wheel loads will be supported by thick steel plates, by a concrete-filled grid deck, or by a concrete slab unit that is wide enough to span the gap at its widest seismic opening. The truss end of the special span unit is supported on a sacrificial wall or on a series of columns. A proprietary joint large enough to accommodate temperature and live load moments is connected from the end of the special span unit to the end floor-beam of the truss. The abutment end of the special span unit is supported on a smooth concrete slab with space provided for the span unit to slide in an earthquake. The slide space is filled with ordinary asphalt.

During a large seismic event, the first movements of the truss toward the abutment will destroy the sacrificial wall or the columns. The special span unit will fall a few centimeters and will be caught by special seats on the floor beam. As the truss continues to move toward the abutment, the truss will push the special span unit toward the asphalt fill, which will be forced out, allowing the truss full, unrestricted movement. The truss may then move unrestricted in the other direction for the full design-distance. After the event, vehicles can slowly cross over the displaced asphalt. Within a short time, the asphalt surface can be smoothed out.

There are many other possibilities for the seismic retrofitting of expansion joints. The retrofit measure selected will need to be a balance between long-term service performance under daily traffic and seismic performance of the joint after an earthquake, and the initial cost for the retrofitting of the joint.

7.12 SEISMIC ISOLATION

A glance at any typical acceleration response spectrum (ARS) curve will indicate that a structure with a low fundamental period of vibration (typically less than 1 second) will attract a higher spectral acceleration than will a structure with a higher period. This means larger seismic input forces will be transmitted into the low period structure. Therefore, a very attractive way to retrofit a bridge is by artificially increasing the fundamental period of vibration of the structure to lower its seismic input forces.

On the other hand, the equation of motion of a linear system shows that the higher the viscous damping, the lower the forces to be resisted by the structure. Viscous damping is actually a way to dissipate energy. Similar to isolation, energy dissipation is a way of changing a fundamental characteristic of the structure, which will reduce seismic input forces and the damage potential.

The masses of most steel bridge trusses are concentrated at the deck level, and the decks are usually designed to remain elastic under seismic action. When a seismically complex truss is supported on bearings at the top of piers, the bearings can be replaced by isolation bearings that contain an energy-dissipation device. The stiffness, yielding, and elongation capacity of these devices can be selected as a function of the desired protection and of the seismic intensity expected at the bridge site.

Many types of isolation devices are available on the market today, such as lead-rubber bearings and friction pendulum bearings. For a description of these systems and for the modeling, design and the application of these systems, refer to [Priestley et al., 1996] and *Seismic Isolation Manual for Highway Bridges* [Buckle et al., 2006].

The most common problem associated with using isolation devices is the larger displacement of the superstructure. Larger deck displacement will require larger expansion joints, and perhaps wider pier shafts to accommodate the bearings. All of these issues need careful study before an isolation retrofit strategy is adopted.

7.13 HISTORIC PRESERVATION

The rehabilitation and retrofit of historic bridges must be done in a way that respects the historic nature of the structure. The Historic Preservation Act of 1966 and "The Secretary of the Interior's Standards for Rehabilitation," and the "*Guidelines for Rehabilitating Historic Buildings*" also apply to historic bridges. The first step in the rehabilitation of a historic building is to identify, retain, and preserve the materials and features that define the historic character of the building. Rehabilitation repair then abides by the following hierarchy:

1. Protect and maintain.
2. Replace.
3. Design for missing historic features.
4. Alterations/additions to historic buildings.
5. Consideration of: energy efficiency, accessibility, and health and safety.

For different elements of a building and for the different levels of the rehabilitation hierarchy, the *Guidelines for Rehabilitating Historic Buildings* recommend certain rehabilitation methods and recommend against others. For example, in reference to the repair of structural systems, their *Guidelines for Rehabilitating Historic Buildings* recommend:

> **“**Repairing the structural system by augmenting or upgrading individual parts or features. For example, weakened structural members such as floor framing can be paired with a new member, braced, or otherwise supplemented and reinforced.”

The *Guidelines for Rehabilitating Historic Buildings* recommend against:

> “Upgrading the building structurally in a manner that diminishes the historic character of the exterior, such as installing strapping channels or removing a decorative cornice; or damaging to interior features or spaces.”
> “Replacing a structural member or other feature of the structural system when it could be augmented and retained.”

The rehabilitation and retrofit of a historic bridge shall be in general accordance with the above, although rehabilitation and retrofit must first satisfy the requirements of the bridge design codes listed herein.

Rehabilitation and retrofit measures shall have minimal visual impact on the bridge. The preferred course of action is, in order of increasing impact:

1. Strengthen an existing component without altering its general appearance.
2. Replace an existing component without altering its general appearance.
3. Introduce a new component that is visually compatible with existing elements.
4. Strengthen an existing component, but altering its general appearance.
5. Replace an existing component, but altering its general appearance.
6. Remove an existing component.
7. Introduce a new component visually incompatible with existing elements.

Guidelines for the rehabilitation and retrofit of specific bridge elements are given below.

7.13.1 LACED MEMBERS

Strengthening of a laced member should be the minimum required in accordance with following the hierarchy of measures:

1. Replace the laces with new, similar laces.
2. Add additional angles or plates.

3. Replace the laces with a perforated cover plate. Preferably, a plasma-cut plate that mimics the original laces should be used, as is shown on the left in Figure 7-11. A conventional cover plate, like that shown on the right in Figure 7-11 may be used if necessary.

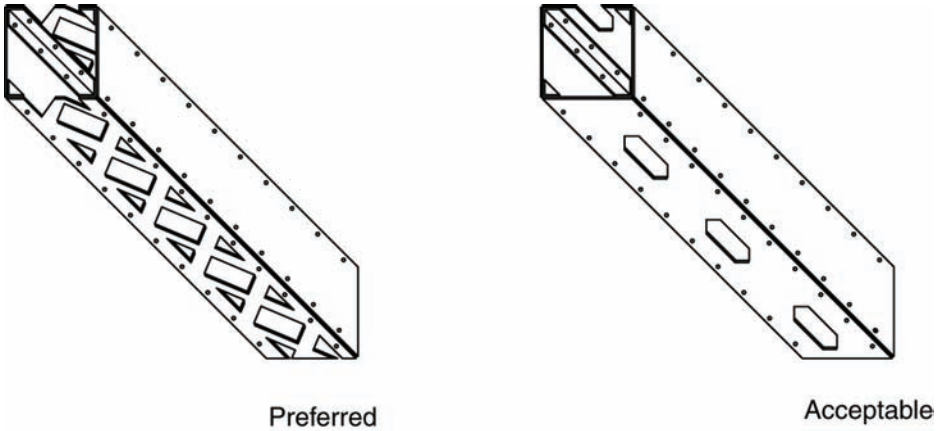

Figure 7-12. Preferred and Acceptable Cover Plate Retrofits

If a member cannot be strengthened, a new member may replace it:

1. Preferably, the new member should be a welded box section made from plasma-cut plates that mimic the existing laces, as is shown on the left in Figure 7-12.
2. Otherwise, a "solid" welded box section is acceptable. If an access hole is visible, it should mimic the existing laces, as is shown on the right in Figure 7-12.

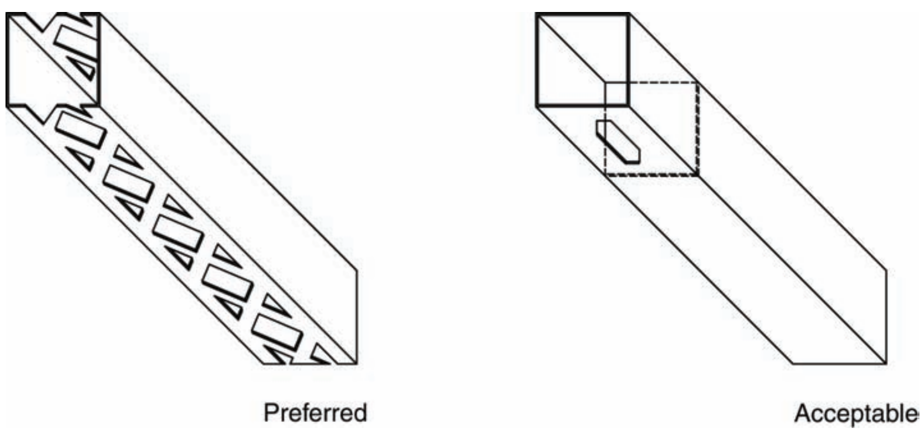

Figure 7-13. Preferred and Acceptable Box Section Retrofits

7.13.2 GUSSET PLATES

Gusset plates should be replaced with plates having the same overall shape. The new plates may be thicker and larger than the existing plates to accommodate demands which exceed the capacity of the existing plates.

7.13.3 REPLACEMENT OF EXISTING, VISIBLE CONCRETE

Replacement of existing, visible concrete with new concrete shall be performed in accordance with the following steps:

1. Document the condition of the concrete before removing it. The documentation shall include the surface texture, color, and finish of the concrete, and the type and pattern of the formwork used to cast the concrete.
2. Remove the concrete. If the concrete has a flat surface, use a straight saw-cut to remove it. If the surface is faceted, cut it in a zigzag fashion, following the faceting.
3. Make the new concrete from aggregate similar to that used in the original concrete, and match the original color.
4. Cast the new concrete in formwork made from the same size and type of lumber, with the same surface texture as that used to cast the original concrete. Match the form lines in the original concrete.
5. After removing the formwork, sandblast the surface of the new concrete to expose enough aggregate to match any surrounding, existing surface, or the surface texture of the original concrete.

7.13.4 NEW, VISIBLE CONCRETE

New, visible concrete shall be harmonized with the original design of the bridge, and should have angles, bevels, etc. similar to those of existing concrete elements. Materials, formwork, and finishing shall be in accordance with steps 3, 4, and 5 above.

7.14 COST CONSIDERATIONS

Cost is a major consideration in seismic retrofitting. The average construction cost of seismic retrofitting is much higher than the incremental cost of adding seismic resistance at the time of new construction. Engineering costs for retrofit evaluation and design are also higher than the engineering costs for new construction. It is realistic to expect that the costs of seismically retrofitting a truss bridge may be two or three times the cost required for providing seismic resistance in a new bridge of similar value. Since seismically complex bridges are unique, they often require customized retrofit strategies. The standardization of retrofit details is, therefore, difficult to achieve. In addition, detailed seismic evaluation of a bridge and the identification of the most appropriate retrofit strategy is a time-consuming process because it may involve detailed dynamic analysis and many trials in the investigation of possible strategies.

The life of the seismically retrofitted components need not exceed the remaining service life of the bridge**.** The annualized cost from seismic retrofitting an existing bridge is higher because of the annualized cost of adding seismic resistance to a new bridge with a longer service life. These facts should be considered during the selection of the most appropriate retrofit strategy for either standard or SC bridges.

CHAPTER 8: CASE STUDIES

1. Carquinez Bridge (1958 span), California
2. Benicia-Martinez Bridge, California
3. Richmond-San Rafael Bridge, California
4. Million Dollar Bridge, Alaska
5. Hernando DeSoto Bridge, Tennessee
6. Aurora Bridge, Washington
7. Cooper River Bridge, South Carolina

Case Study No. 1
Carquinez Bridge (1958 span)

Original Bridge Data

Figure 8-1. Carquinez Bridge (1958 span), California

Location: Carquinez Strait, between Solano and Contra Costa Counties, California; Interstate 80
Year Built: 1958
Year of Retrofit: 1998
Structural Type: Tandem cantilever truss
Span Lengths: 500 ft –1100 ft - 1100 ft – 500 ft
Width: 80 ft
Piers: Steel A-frames with chevron bracings for side spans. Rectangular frame with chevron bracing for center support
Soil Condition: 50 ft of bay mud, shale bedrock
Foundation: hollow concrete caissons for main spans, steel H-piles for side spans
Steel Types: High-strength A514 steel (100 ksi yield), A-242 steel (50 ksi yield)

Truss tension members are welded H shapes. Compression members are box sections with perforated plates. Most connections use A325 high-strength bolts. At the expansion joints between the suspended and cantilever spans, hydraulic lock-up devices were previously installed.

Retrofit Criteria
No collapse under Safety Evaluation Earthquake with an expected mean return period of 1,000 to 2,000 years. Damage and closure of bridge allowed. Essentially elastic response under lower level Functional Evaluation Earthquake.

Structural Analysis
Linear and non-linear time-history analyses with multiple-support inputs, gap elements at expansion joints, bi-linear springs and dashpots for rocking of concrete caisson foundations. Pushover analysis for braced towers.

Seismic Evaluation
Existing Truss superstructure has design capacity for one-fifth of the demand imposed by the Safety Evaluation Earthquake. Many existing members determined to have limited ductility and expected to yield, buckle, or rupture. Due to its robust bracings, yielding of main tower would not significantly reduce the demands in the superstructure.

Retrofit Measures

- Slender truss members strengthened with bolted stiffeners. AISC LRFD slenderness ratio criterion adopted.
- Steel plates were added on to bottom chords of suspended spans to resolve capacity deficiencies.
- Lateral bracing members with weak connections were strengthened by replacing gusset plates and increasing number of A325 bolts.
- Flexible restrainer beams were installed at base of towers that are designed to yield, allowing the tower to rock for energy dissipation but limiting uplift.
- Anchor beams installed at main span tower base supports. Beam are held down by post-tensioned strands anchored to thicked caisson cap thickened by 14-ft to anchor strands.

Figure 8-2. Seismic Retrofit of A-Frame Tower Rocking Supports

Retrofit Cost

Approximately $60 million or $230/SF**.**

References

Hinman, J., Toan, V. and Thoman, S., (1998), “Seismic Retrofit of 1958 Carquinez Bridge,” Transportation Research Record 1624, Paper No. 98-0543, pp. 54-63.

Thoman, S., Toan, V., Holloway, L., Hinman, J. and Jones, M., (1996), “Overview of the I-80 Carquinez Strait Bridges Seismic Retrofit Design,” Second U.S. Seminar on the Seismic Design, Evaluation, and Retrofit of Steel Bridges, Nov. 20-21, pp.183-213.

Case Study No. 2
Benicia-Martinez Bridge

Original Bridge Data

Location: Carquinez Strait, Benicia, California; Interstate 680
Year Built: 1962
Year of Retrofit: 2002
Structural Type: Parallel-chords deck trusses, with drop-in truss spans forming Gerber framing systems.
Span Lengths: Seven 528-ft spans, two 429-ft spans, one 330-ft span
Width: 42 ft
Piers: Concrete cellular columns
Soil Condition: bay mud, variable bedrock of, claystone, siltstone, sandstone to 120 ft below sea level. Local scouring detected.
Foundation: Steel caissons designed for scouring

Figure 8-3. Benicia-Martinez Bridge, California, 1960; Gerber Framing, 528-ft truss Spans (1932 RR Camel-Back Truss in the Background)

Truss chords and diagonals are H-type cross sections. High-strength bolts in gusset plate connections. Trusses sit on rocker bearings, 3.5- ft high, on massive concrete pier shafts.

Retrofit Criteria
As a lifeline bridge, to provide normal traffic following a Safety Evaluation Earthquake (1 in 1000-year event). Elastic response under Functional Evaluation Earthquake (1 in 300-year event).

Structural Analysis
Non-linear time-history analyses. Three set of ground motions generated based on a near-source earthquake event, 2-miles away, 13-sec. duration, and 6.75-magnitude.

Seismic Evaluation
The rocker bearings were unstable. Concrete cellular pier shafts were non-ductile and lack sufficient capacity. Connection of footing to steel caisson not adequate to transfer base shear or tension.

Retrofit Measures
- Replacement of rocker bearing with friction pendulum isolation bearings.
- Deficient truss members and connections replaced or strengthened with bolted on cover plates.
- New transverse shear locks at truss hinges to control relative movement.
- New expansion joints to minimize deck damage.

- Anchorage of existing foundation by adding new steel caissons, core drilled 150-ft down to bedrock.
- Existing caissons retrofitted with pipe tie-downs, core-drilled to bedrock.
- Foundation footings strengthened with two, 10-ft deep internal reinforced concrete cap beams.
- Land pier footings enlarged and keyed into the rock.
- Concrete pier shafts strengthened with interior stiffener walls and external sloping walls.

Figure 8-4. Friction Pendulum Bearing

Retrofit Cost
Approximately $90 million or $440/SF.

Reference

Imbsen, R. A., (1996), “Seismic Evaluation and Retrofit Design of the Benicia-Martinez Bridge,” Second U.S. Seminar on the Seismic Design, Evaluation, and Retrofit of Steel Bridges, Nov. 20-21, pp. 255-273.

Case Study No. 3
Richmond-San Rafael Bridge

Original Bridge Data

Location: In San Francisco Bay, between Richmond and San Rafael, California
Year Built: 1956
Year of Retrofit: 1995
Structural Type: Two sets of double-deck cantilever main spans, double-deck simple span trusses
Span Lengths: 4.5 miles per deck.
Width: 40 ft
Piers: Dual or quadruple-shaft concrete piers with steel chevron-braced towers
Soil Condition: Bay mud, softy clayey silt, silty sand, and soft silty clay. Depth to bedrock less than 200 ft
Foundation: Concrete belled pier foundations on steel H-piles up to 160-ft in length

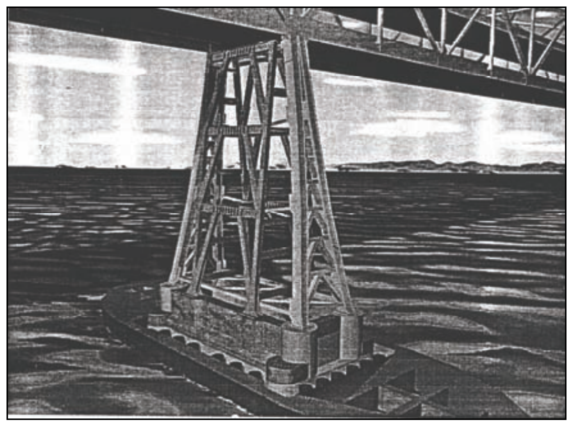

Figure 8-5. Richmond-San Rafael Bridge, California, 1956
Main Span Tower Seismic Retrofit

Retrofit Criteria
Limited service level after a maximum credible earthquake with average return period of 1500 years. Significant damage is accepted but with minimum risk of collapse.

Structural Analysis
Design earthquake events: (1) San Andreas Event: $M_w = 8$, R =16.5 km, 40-sec. duration. (2) Hayward Event: $M_w = 7.25$, R = 7 km, 40-sec. For both events, multi-support, three-component rock motion time histories were generated for the belled pier foundations. Peak mud-line spectral accelerations ranged from 1g to 3g. Peak vertical accelerations ranged from 2g to 2.8g. Global nonlinear modeling captured the response of the rocking “A-frame” towers, hydraulic viscous dampers, and hydrodynamic added mass effect of the bay water surrounding the piers. Upper- and lower-bound curves of towers determined from pushover analyses.

Seismic Evaluation
Pushover analyses indicated that demand on non-compact H-piles exceed capacity. A 6-inch drift criterion was adopted to prevent fracture and was substantiated by laboratory cyclic testing. Steel towers deficient for transverse drift. Half-scale specimen of an eccentrically braced tower leg specimen was carried out to verify retrofit method and the spread of plasticity. Yielding

confirmed for the truss shoe connections angles at the top of the tower legs. Demand-capacity ratios below 1.0 for truss compression chords and diagonals. Truss tension chords capable of achieving full yielding. Truss posts have sufficient post-yield drift capacities to withstand lateral racking.

Retrofit Measures

- Installations of large diameter (8- to 13-ft) concrete-filled steel tubular "shear" piles to limit drifts imposed on existing H-piles to within 6 inches.
- Micro piles installed -- 8-inch pipe driven, drilled, and socketed into rock.
- Grouted steel casings installed around the foundation bells to provide confinement.
- Precast concrete jacket installed around concrete pier shaft to prevent shear failure and unzipping of the diaphragm wall.
- Additional vertical reinforcement in the form of rock bolts drilled into the concrete pier shafts.
- Removal of chevron bracing from the tall steel towers. Installation of dual eccentrically braced frames.
- Special moment resisting frames installed in the shorter towers with scalloped beam detailing to move plastic hinging away from column face.
- New eccentrically braced frame towers for split-bent locations with hinged yokes to isolate flexural and torsional moment from existing truss chords.
- Installation of special four-legged ductile frame for towers supporting the main cantilever structures. Existing tower leg anchorage modified to allow controlled uplift and rocking.
- Approximately 25% of primary truss member required modification: Lacing bars replaced, cover plates added, rivets replaced with high-strength bolts.
- New bracings to reduce racking or prevent instability.
- Connections strengthened between floor beams and vertical truss posts.
- Truss spans separated from the plate girder approach structures with seismic isolation joints (3-ft movement).
- Displacement-dependent hydraulic viscous dampers for control of longitudinal forces.
- Lead-core rubber isolation bearings at piers without steel towers to control forces transmitted to the superstructure.
- Velocity-dependent hydraulic viscous dampers installed at five intermediate expansion joints to prevent hammering.

Retrofit Cost

Approximately $750 million or $395/SF.

Reference

Vincent, J., (1996), "Seismic Retrofit of the Richmond-San Rafael Bridge," Second U.S. Seminar on the Seismic Design, Evaluation, and Retrofit of Steel Bridges, Nov. 20-21, pp.215-232.

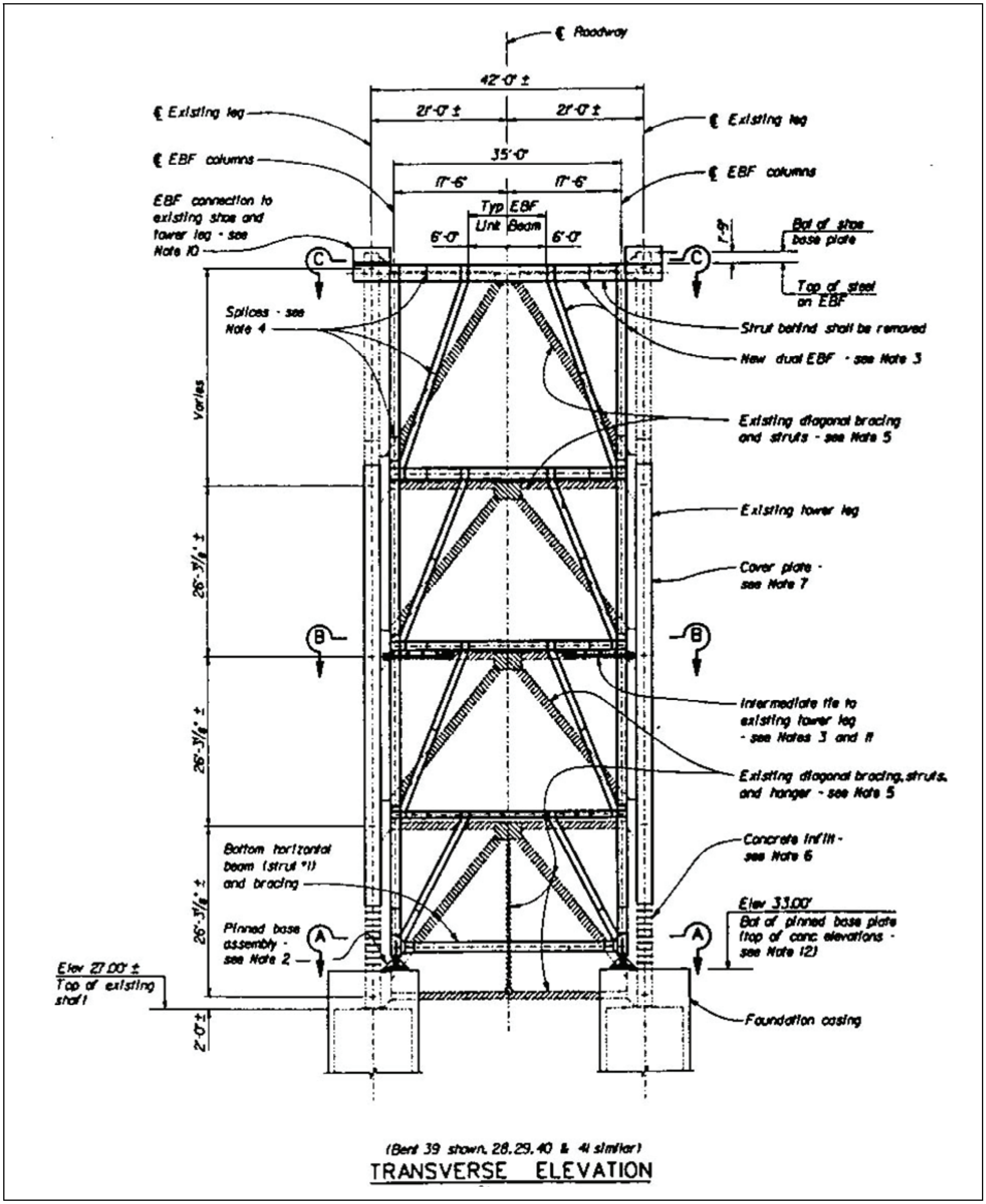

Figure 8-6. Dual Eccentrically Braced Frame Retrofit for Regular Towers

Case Study No. 4
Million Dollar Bridge

Original Bridge Data
Location: Copper River, Alaska
Year Built: 1910
Year of Retrofit: 2004
Structural Type: Pratt truss
Span Lengths: Four spans measuring 400 ft, 300 ft, 450 ft, and 400 ft.
Width: 24 ft
Piers: Unreinforced concrete piers.
Soil Condition: Estimated ultimate bearing capacity under piers and abutments 50 ksf to 150 ksf. Liquefiable soils underlying Pier 3.
Foundation: Concrete caissons.

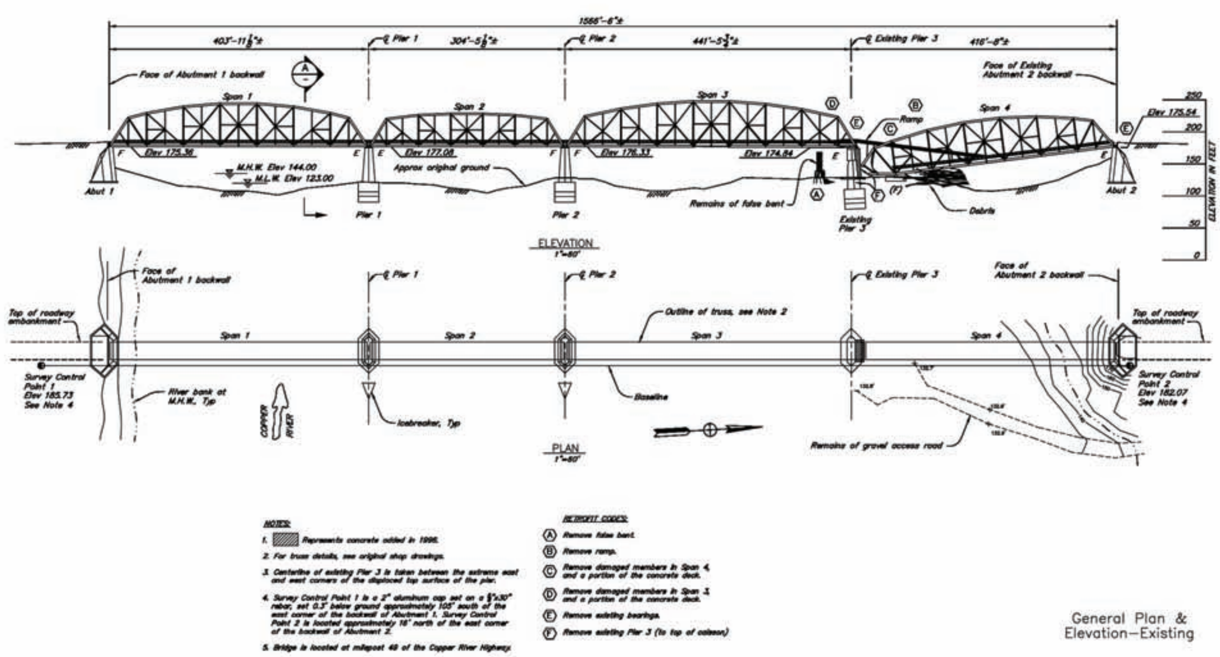

Figure 8-7. Million Dollar Bridge, Copper River, Alaska

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Retrofit Criteria
Raise the fallen Span 4. Bridge retrofit to resist a design earthquake with a return period of 475 years. As bridge is listed on the National Register of Historic Places, rehabilitation to meet guidelines of the Historic Preservation Act.

Structural Analysis
USGS 475-year hazard rock spectrum, corresponding to Magnitude 8+ event, was extrapolated to long periods. Two seed ground motions were made compatible with the target rock spectrum: (1) Caleta De Campo, 1985 Mexico City Earthquake, and (2) Vina del Mar, 1985 Chile Earthquake. Demands on the structure were computed using nonlinear time-history analysis. Ground displacements were applied to soil springs support the bridge model. Independent, uncorrelated displacements were applied simultaneously in the longitudinal, transverse, and vertical directions. A response spectrum analysis of a stand-alone model of Span 1 was used to check the time history analysis methodology. Friction pendulum bearings were modeled using a contact surface model.

Seismic Evaluation
From the Demand/Capacity evaluation, Span 3 bottom chord members and diagonal eye bars were found to be overloaded. Chord splices satisfy capacity design provisions. The rivets connecting the bottom lateral gusset plates to the bottom chords are not able to transfer the resultant force from the adjacent laterals. The main vulnerability of the piers is in flexure. The sliding potential of the abutments were found to be small. Computed caisson bearing pressures are low.

Retrofit Measures
- Replacement of damaged members in Spans 3 & 4.
- Lifting of Span 4.
- New pier and foundation for Pier 3.
- Installation of friction pendulum bearings.
- Relocation of abutment back walls.
- Retrofit of Piers 1 & 2 with reinforced concrete jackets.

Retrofit Cost
Phase 1, approximately $9 million or $240/SF.

Reference

Copper River Highway: Million Dollar Bridge Strategy Report, Project BH-0851(62)/60803, T.Y. Lin International, Dec. 2001a.

Case Study No. 5
Hernando DeSoto Bridge

Original Bridge Data
Location: I-40 over Mississippi River, Memphis, Tennessee.
Year Built: 1969
Year of Retrofit: 2005
Structural Type: Truss arch made of built-up steel box sections.
Span Lengths: Two 900-ft arches.
Piers: 126-ft-tall tapered concrete columns connected by a 6-ft thick web wall.
Soil Condition:
Foundation: Distributed blocks supported by concrete footings on top of concrete-filled steel caissons.

Figure 8-8. Hernando DeSoto Bridge

Retrofit Criteria
Bridge must remain serviceable after the maximum probably contingency-level earthquake in a 2,500 return period (or a 2 percent probability of exceedance in 50 years).
Closure of the bridge after an earthquake limited to 2-3 days.

Structural Analysis/Seismic Evaluation
A three-dimensional nonlinear analysis showed overstressed truss members and connections, insufficient resistance in the deck in both transverse and longitudinal directions, excessive plastic hinges with poor confinement at the base of pier columns and web walls, inadequate amount of reinforcing steel in the footings to resist rocking. Existing expansion finger joints insufficient for seismic displacements.

Retrofit Measures
- Friction pendulum bearings with vertical capacity of 12.6 million pounds.
- Lead rubber bearings with lateral displacement capacity of 22.5 inches.
- Existing expansion finger joint replaced with a seismic swivel joint.
- Extension of column caps. Web walls extended to column caps.
- Replace cross frame and lateral bracing members.
- Install direct connections between the deck and box girder.
- Stiffening connections between stringers and floor beams.
- Footing strengthened by concrete encasement.
- Pier columns retrofitted by jacketing.

Figure 8-9. Installation of Friction Pendulum Bearing

Retrofit Cost
Approximately $27 million or $180/SF.

Reference:

Jaramilla, B., Sharp, P. and Wasserman, E., "A Vital Embrace for the Big One, FHWA Resource Center, RC Success Stories, March 2003, www.fhwa.dot.gov/resourcecenter/success/successstories/vol2iss02.cfm

Case Study No. 6
Aurora Avenue Bridge

Original Bridge Data
Location: Lake Washington Ship canal, Seattle, Washington.
Year Built: 1931
Year of Retrofit: 2005
Structural Type: Cantilever truss with drop in span.
Span Lengths: 800-ft main span with 150-ft drop-in span
Width: 70 ft.
Piers: Reinforced concrete
Soil Condition: Glacially overridden soil. Liquefiable soil, 30-ft deep, present under three piers.
Foundation: Piles

Figure 8-10. Aurora Avenue Bridge, Seattle

Retrofit Criteria
475-year earthquake (10% probability of exceedance in 50 years). Estimated peak ground acceleration 0.3g.

Structural Analysis/Seismic Evaluation
Results of response spectrum analysis and 2-D pushover analyses found the lightly reinforced main piers deficient in shear and flexure. Nonlinear time history analyses performed for friction pendulum retrofit alternative.
Three time histories were used representing crustal, intraplate and subduction earthquakes.

Retrofit Measures
- Installation of four 6.75-ft diameter friction pendulum isolation bearings.
  Reduction in seismic load estimated to be 72% in the longitudinal direction and 87% transversely.
- Strengthening of pier cap with post-tensioning cables.
- New expansion joints.

Figure 8-11. Friction Pendulum Bearing Installed

Retrofit Cost
Approximately $10 million or $180/SF.

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T.Y.Lin International, (2001b), “Washington State D.O.T. SR99-Aurora Avenue Bridge: Friction Pendulum Seismic Retrofit: Preliminary Design Report,” Project No. 69105.12, Sept. 26.

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Case Study No. 7
Cooper River Bridges

Figure 8-12. Cooper River Truss, Charleston, South Carolina, 1963
Main Tower Bearings and Vertical Lateral Bracing, Upper Chords Lateral Bracing System, and Safety Hand Railing on Upper Chords

Original Bridge Data
Location: Charleston, South Carolina.
Year Built: 1929 (Grace Memorial Bridge) and 1966 (Pearman Bridge)
Year of Replacement: 2005
Structural Type: Cantilever truss.
Span Lengths: 1050-ft main span, 2.71 miles total length.
Width: 20 ft
Vertical Clearance: 150 ft.
Piers: Reinforced concrete

Retrofit Criteria
Functional Evaluation Earthquake (500-year return period) – immediate access and essentially elastic response. Safety Evaluation Earthquake (2500-year) – Functional to emergency vehicle and repairable damage.

Retrofit Measure
Retrofit was deemed not to be feasible. The old bridges were obsolete (narrow 10-ft traffic lanes with no shoulders) and reaching the end of their service life. It would not be cost effective to retrofit the existing bridges to meet the required traffic capacity, structural safety, and navigational clearance. The bridges were replaced by a cable-stayed bridge.

Replacement Cost
The total bridge cost (including approaches) was approximately $522,000,000. The unit cost of the main span was approximately $411 per square foot (according to an email from Lucero Mesa, SC DOT to J. O'Connor, MCEER on 7/18/06).

Reference

Abrahams, M.J., Wang, J-N, Wahl, P.M., and Bryson, J.A., "Seismic Design of the Ravanel Bridge, Charleston, South Carolina," *Proceedings* of the Third National Seismic Conference and Workshop on Bridges and Highways, FHWA, May 2002, pp. 85-94.

Figure 8-13. Bearings and Lateral Bracing System

Figure 8-14. Typical Chevron Lateral Bracing Under the Deck of a Through Truss, Floor Beams and Stringers Support a Concrete Slab, the Three Rails Support a Maintenance Traveler

Figure 8-15. Replacement of the Cooper River Bridge, Charleston, South Carolina

CHAPTER 9: GLOSSARY

Anticipated Service Life (ASL) – An estimate of the remaining life of a bridge based on current age, structural condition, specification used for design, and capacity to handle current and future traffic.

Beam – A structural member whose primary function is to transmit loads to the support primarily through flexure and shear. Generally, this term is used when the component is made of rolled shapes. See "Girder."

Block Shear Rupture – Failure of a bolted web connection of coped beams or any tension connection by the tearing out of a portion of a plate along the perimeter of the connecting bolts.

Bolt Assembly – The bolt, nut(s), and washer(s).

Bowstring Truss -- A bowstring truss is a *pony truss* with the top chord shaped in the form of a bow. The top chord at each panel is not inclined but is an extension to the curve of the bow.

Bracing Member – A member intended to brace a main member or part thereof against lateral movement.

Charpy V-Notch Impact Requirement – The minimum energy required to be absorbed in a Charpy V-Notch Test conducted at a specified temperature.

Charpy V-Notch Test -- An impact test complying with AASHTO T243M (ASTM A 673M).

Clear Distance of Bolts – The distance between edges of adjacent bolt holes.

Clear End Distance of Bolts – The distance between the edges of a bolt hole and the end of a member.

Collapse – A major change in the geometry of the bridge rendering it unfit for use.

Collapse Load – That load that can be borne by a structural member or structure just before failure becomes apparent.

Compact Section – A section that is capable of developing the fully plastic stress distribution in flexure; the rotation capacity required to comply with analysis assumptions used in various articles of this section is provided by satisfying various flange, web slenderness, and bracing elements.

Component – Either a discreet element of the bridge or a combination of elements requiring individual design consideration; a constituent or part of a structure.

Composite Beam – A steel beam connected to a deck so that they respond to force effects as a unit.

Composite Column – A structural compression member consisting of either structural shapes embedded in concrete or a steel tube filled with concrete designed to respond to force effects as a unit.

Constant Amplitude Fatigue Threshold – The nominal stress range below which a particular detail can withstand an infinite number of repetitions without fatigue failure.

Controlling Flange – Top or bottom flange for the smaller section at a point of splice, whichever flange has the maximum ration of the elastic flexural stress at its midthickness due to the factored loads to its flexural resistance.

Cross-Frame – A transverse truss framework connecting adjacent longitudinal flexural components.

Deck Truss – A truss system in which the roadway is at or above the level of the top chord of the truss.

Design life – Period of time on which the statistical derivation of transient loads is based.

Detail Category – A grouping of components and details having essentially the same fatigue resistance.

Diaphragm – A transverse flexural component connecting adjacent longitudinal flexural components.

Distortion-Induced Fatigue – Fatigue effects due to secondary stresses not normally quantified in the typical analysis and design of a bridge.

Ductility – Property of a component or connection that allows inelastic response.

Edge Distance of Bolts – The distance perpendicular to the line of force between the center of a hole and the edge of the component.

End Distance of Bolts -- The distance along the line of force between the center of a hole and the end of the component.

End Panel – The end section of a truss or girder.

Extreme Event Limit States – Limit states relating to events such as earthquakes, ice load, and vehicle and vessel collision with return periods in excess of the design life of the bridge.

Eyebar – A tension member typically has a rectangular section and enlarged ends for a pin connection.

Fatigue – The initiation and/or propagation of cracks due to a repeated variation of normal stress with a tensile component.

Fatigue Design Life – The number of years that a detail is expected to resist the assumed traffic loads without fatigue cracking.

Fatigue Life – The number of repeated stress cycles that results in fatigue failure of a detail.

Fatigue Resistance – The maximum stress range that can be sustained without failure of the detail for a specified number of cycles.

Finite Fatigue Life – The number of cycles to failure of a detail when the maximum probable stress range exceeds the constant amplitude fatigue threshold.

Fixed bridge – A bridge with a constant vehicular or navigational clearance. See “Moveable Bridge.”

Force Effect – A deformation, stress, or stress resultant (i.e., axial force, shear force, torsional, or flexural moment) caused by applied loads, imposed deformations, or volumetric changes.

Fracture Toughness – A measure of the ability of a structural material or element to absorb energy without fracture. It is generally determined by the Charpy V-Notch Test.

Fracture-Critical Member (FCM) – Component in tension whose failure is expected to result in the collapse of the bridge or the inability of the bridge to perform its function.

Gage of Bolts – the distance between adjacent lines of bolts; the distance from the back of an angle or other shape to the first line of bolts.

Girder – A structural component whose primary function is to resist loads in flexure and shear. Generally, this term is used for fabricated sections. See "Beam."

Grip – Distance between the nut and the bolt head.

Gusset Plate – Plate material used to interconnect vertical, diagonal, and horizontal truss members at a panel point.

Half Through-Truss Spans – A truss system with the roadway located somewhere between the top and bottom chords. It precludes the use of a top lateral system.

Hybrid Girder – A fabricated steel girder with web that has a specified minimum yield strength lower than one or both flanges.

Inelastic Action – A condition in which deformation is not fully recovered upon removal of the load that produced it.

Inelastic Redistribution – The redistribution of internal force effects in a component or structure caused by inelastic deformations at one or more sections.

Interior Panel – The interior section of a truss or girder component.

Lacing – Plates or bars to connect components of a built up truss member.

Lateral Bracing Component – A component utilized individually or as part of a lateral bracing system to prevent buckling of components and/or to resist lateral loads.

Level – That portion of a rigid frame that includes one horizontal member and all columns between that member and the base of the frame or the next lower horizontal member.

Limit State – A condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed.

Load Factor – A factor accounting primarily for the variability of loads, the lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads, and also related to the statistics of resistance through the calibration process.

Load Modifier – A factor accounting for ductility, redundancy, and the operational importance of the bridge.

Load Path – A succession of components and joints through which a load is transmitted from its origin to its destination.

Load-Induced Fatigue – Fatigue effects due to the in-plane stresses for which components and details are explicitly designed.

Longitudinally Loaded Weld – Weld with applied stress parallel to the longitudinal axis of the weld.

Low Cycle Fatigue – When the stress is high enough for plastic deformation to occur, fatigue will happen with a low number of load cycles.

Main Member – Any member designed to carry the loads applied to the structure.

Model – An idealization of a structure for the purpose of analysis.

Movable Bridge – A bridge with a variable vehicular or navigational clearance.

Multiple-Load-Path Structure – A structure capable of supporting the specified loads following the loss of a main load-carrying component or connection.

Net Tensile Stress – The algebraic sum of two or more stresses in which the total is tension.

Nominal Resistance – Resistance of a component or connection to force effects, as indicated by the dimensions specified in the contract documents and by permissible stresses, deformations, or specified strength of materials.

Noncompact Section – A section that may develop the yield strength in compression elements before the onset of local buckling but that cannot resist inelastic local buckling at strain levels required for a fully plastic stress distribution.

Noncontrolling Flange – The flange at the point of splice opposite the controlling flange.

Orthotropic Deck – A deck made of a steel plate stiffened with open or closed steel ribs welded to the underside of it.

Overstrength -- Strength that takes into account all the possible factors that my cause a strength higher than the specified yield strength

Permanent Deflection – A type of inelastic action in which a deflection remains in a component or system after the load is removed.

Pitch of Bolts – The distance along the line of force between the centers of adjacent holes.

Plate – A flat rolled steel product whose thickness exceeds ¼ ". See "Sheet."

Pony Truss -- A truss with the deck in the plane of the lower chord, without a lateral bracing system in the plane of the upper chord. The top chord at each end is sharply inclined.

Portal Frames – End transverse truss bracing or Vierendeel bracing to provide for stability and to resist wind or seismic loads.

Redistribution Moment – An internal moment caused by yielding in a continuous span bending component and held in equilibrium by external reactions.

Redistribution of Moments – A process that results from formation of inelastic deformations in continuous structures.

Redistribution Stress – The bending stress resulting from formation of inelastic deformations in continuous structures.

Redundancy – The duplicate system(s) or member(s) of a bridge that enables it to perform its function in a damaged state.

Redundant Member – A member whose failure does not cause failure of the bridge.

Regular Service – Condition excluding the presence of special permit vehicles, wind exceeding 55 mph, including scour and other extreme events.

Rehabilitation – A process in which the resistance of the bridge to load is either restored or increased.

Required Fatigue Life – A product of the single-lane average daily truck traffic, the number of cycles per truck passage, and the design life in days.

Resistance Factor – A factor accounting primarily for variability of material properties, structural dimensions and workmanship, and uncertainty in the prediction of resistance, but also related to the statistics of the loads through the calibration process.

Secondary Member – Member not designed to carry primary loads.

Service Life – The period of time that the bridge is expected to be in operation.

Service Limit States – Limit states relating to stress, deformation, and cracking.

Sheet -- A flat rolled steel product whose thickness is between 35 gauge and 1/4". See "Plate."

St. Venant Torsion – A torsional moment producing pure shear stresses on a cross-section that remains plane.

Stress Range – The algebraic difference between extreme stresses resulting from the passage of a load.

Subpanel – A stiffened web panel divided by one or more longitudinal stiffeners.

Sway Bracing – Transverse vertical bracing between truss members.

Through-Girder Spans – A girder system where the roadway is below the top flange.

Through-Truss Spans – A truss system where the roadway is located near the bottom chord and where a top chord lateral system is provided.

Tie plates – Plates used to connect components of a member.

Tied Arch – An arch in which the horizontal thrust of the arch rib is resisted by a horizontal tie.

Transversely Loaded Weld – Weld with applied stress perpendicular to the longitudinal axis of the weld.

Trough-Type Box Section – A U-shaped section without a common top flange.

True Arch – An arch in which the horizontal component of the force in the arch rib is resisted by an external force supplied by its foundation.

Unbraced Length – The distance between brace points resisting the mode of buckling or distortion under consideration; generally, the distance between panel points or brace locations.

Warping Torsion – A twisting moment producing shear stresses and normal stresses and under which the cross-section does not remain plane.

Yield Strength – The stress at which a material exhibits a specified lifting deviation from the proportionality of stress to strain.

CHAPTER 10: REFERENCES AND BIBLIOGRAPHY

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EARTHQUAKE ENGINEERING TO EXTREME EVENTS
University at Buffalo, The State University of New York
Red Jacket Quadrangle ▪ Buffalo, New York 14261
Phone: (716) 645-3391 ▪ Fax: (716) 645-3399
E-mail: mceer@buffalo.edu ▪ WWW Site http://mceer.buffalo.edu
University at Buffalo The State University of New York