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Structural Steel and Steel/Concrete Interface Details for Bridges

by

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Preface

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) is a national center of excellence in advanced technology applications that is dedicated to the reduction of earthquake losses nationwide. Headquartered at the State University of New York at Buffalo, the Center was originally established by the National Science Foundation in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission is to reduce earthquake losses through research and the application of advanced technologies that improve engineering, pre-earthquake planning and post-earthquake recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

MCEER's research is conducted under the sponsorship of two major federal agencies, the National Science Foundation (NSF) and the Federal Highway Administration (FHWA), and the State of New York. Significant support is also derived from the Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

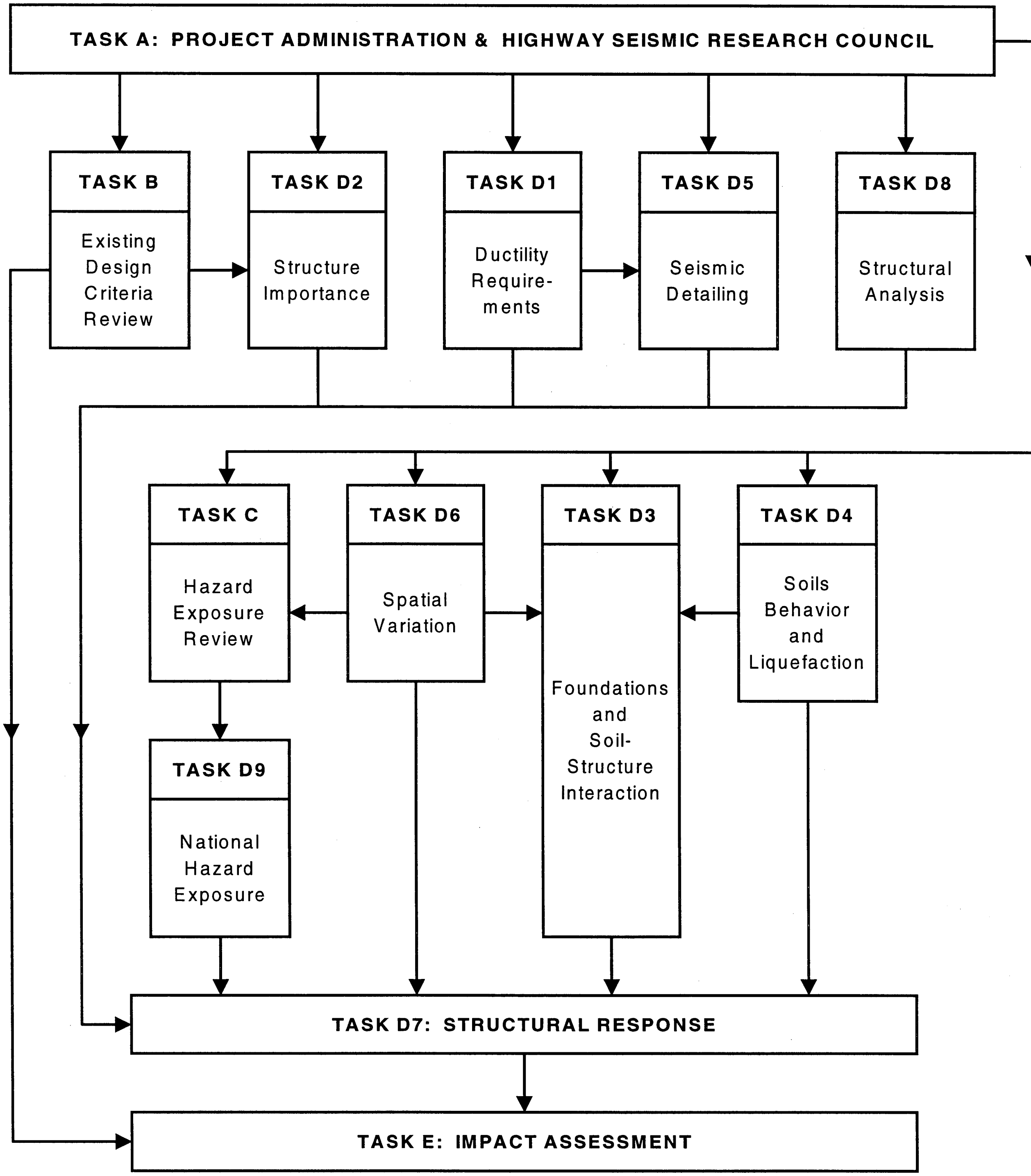
The Center's FHWA-sponsored Highway Project develops retrofit and evaluation methodologies for existing bridges and other highway structures (including tunnels, retaining structures, slopes, culverts, and pavements), and improved seismic design criteria and procedures for bridges and other highway structures. Specifically, tasks are being conducted to:

- assess the vulnerability of highway systems, structures and components;
- develop concepts for retrofitting vulnerable highway structures and components;
- develop improved design and analysis methodologies for bridges, tunnels, and retaining structures, which include consideration of soil-structure interaction mechanisms and their influence on structural response;
- review and recommend improved seismic design and performance criteria for new highway structures.

Highway Project research focuses on two distinct areas: the development of improved design criteria and philosophies for new or future highway construction, and the development of improved analysis and retrofitting methodologies for existing highway systems and structures. The research discussed in this report is a result of work conducted under the new highway structures project, and was performed within Task 112-D-5.4 "Structural Steel and Steel/ Concrete Interface Details" of that project as shown in the flowchart on the following page.

The overall objective of this task was to review and evaluate the performance of steel and composite steel/concrete highway bridge components during earthquakes and, where possible, to develop improved conceptual details that enhance structural response. This report examines the seismic performance of steel bridge towers that extend from a massive concrete substructure to the superstructure, and other steel sub- and superstructure details for new construction. Issues *addressed include identifying the most ductile cross-sections, investigating the application of eccentrically braced frames, exploring details to replace buckled plates or shapes following an earthquake, investigating anchor bolt performance under lateral and uplift loads, and developing economical moment connection details between steel superstructures and substructures.*

SEISMIC VULNERABILITY OF NEW HIGHWAY CONSTRUCTION
FHWA Contract DTFH61-92-C-00112



ABSTRACT

This report assesses the seismic performance of details associated with steel bridge towers extending from a massive concrete substructure to the superstructure, as well as the seismic performance of other steel sub- and superstructure details for new construction. Issues addressed are:

- Identifying the most ductile cross-sections.
- Investigating the application of eccentrically braced frames.
- Exploring details to replace buckled plates or shapes following an earthquake.
- Investigating anchor bolt performance under lateral and uplift loads.
- Developing economical moment connection details between steel superstructures and concrete substructures.

The most efficient member cross-sections were identified as tubular sections due to their self stiffening nature and optimal resistance to both local and overall buckling. Other shapes can provide adequate performance, however, and may be more economical due to savings in production and fabrication. For satisfactory cyclic performance, reduced b/t ratios may be in order, especially following the local buckling behavior of tubular steel piers in Kobe, and additional research in this area is warranted. While member ductility is important, the structural configuration can also greatly affect structural ductility.

Moment resisting frames (MRFs), concentrically braced frames (CBFs), and eccentrically braced frames (EBFs) can all supply adequate seismic resistance, providing that they are detailed correctly. Most bridge bracing systems consist of CBFs, since the open bays of MRFs are generally not required. CBFs are very effective, if the braces are detailed to ensure good post-buckling cyclic performance. There appears to be no reason why EBFs can not be applied to bridge lateral bracing systems, as long as they are detailed correctly.

With modern analysis methods and tools, the locations of damage can usually be predicted. The best way to design for post-earthquake repairs is to ensure that the damage occurs in secondary, non-gravity load bearing elements, such as bracing members. Members such as this are easily replaced, as no accommodations for alternative load paths need to be made. Damaged secondary members may not even need replacement, or replacement can often be delayed, as long as the load carrying capacity is not severely diminished.

Repairing members in-place is preferable to replacement, especially for gravity load bearing members. Heat or mechanical straightening are repair options for locally buckled steel members, although an alternative load path may need to be supplied. Reinforcing plates cut to shape or providing concrete infill/encasement are also repair options. The best way to replace damaged main gravity load bearing members is to provide the means for an alternative load path to be used, such as jacking locations and predrilled splice locations.

Anchor bolts embedded in concrete can be designed with sufficient depth and edge distance, or hairpin reinforcement, such that failure does not occur in the concrete. Brittle fractures of anchor bolts in the Northridge earthquake resulted in a recommendation that upset threads should be used. Shear keys can be used when anchor bolts do not provide sufficient shear resistance. Anchor bolts can either be designed to remain elastic or to yield. Allowing anchor bolts to yield can lead to

rocking of the structure, sometimes producing a beneficial elongation of the period.

The most economical moment connection between steel tower superstructures and concrete substructures is the embedded anchor bolt/base plate with stiffeners to transfer the force to the steel column. Embedding a steel member in concrete can also provide a moment connection, but this appears to be more promising as a repair technique, when the base of the column has suffered local buckling due to formation of a plastic hinge. The concept of integral construction has shown promise for improving the seismic resistance of steel girder bridges on concrete substructures.

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SECTION 1
INTRODUCTION

In the summer of 1993, the National Center for Earthquake Engineering Research (NCEER) initiated a research program with the goal of providing improved design and analysis procedures intended to minimize the seismic vulnerability of new highway infrastructure. The research is sponsored by the Federal Highway Administration (FHWA) of the U.S. Department of Transportation and consists of a series of studies, each focussed on the seismic design and analysis of particular highway system components and structural elements, and performed by researchers with expertise in that area of study.

This report summarizes the research conducted under Task 112-D-5.4, "Structural Steel and Steel/Concrete Interface Details." The objective of this task is to assess the seismic performance of details associated with steel bridge towers extending from a massive concrete substructure to the superstructure. Issues addressed include:

- Identifying the most ductile cross-sections.
- Investigating the application of eccentrically braced frames.
- Exploring details to replace buckled plates or shapes following an earthquake.
- Investigating anchor bolt performance under lateral and uplift loads.
- Developing economical moment connection details between steel superstructures and concrete substructures.

Due to good performance in past earthquakes, and general faith in the ductility of steel, steel portions of bridges have previously been assumed to be relatively safe, and seismic research has concentrated on concrete portions of bridges, especially concrete piers. This perception was changed by the performance of steel structures in recent earthquakes. The unseating and dropping of one of the spans of the Oakland-San Francisco Bay Bridge in the 1989 Loma Prieta earthquake, the fractures of bolted web-welded flange steel moment connections in buildings in the 1994 Northridge earthquake, and the poor performance of tubular welded steel bridge piers in the 1995 Kobe, Japan earthquake, have demonstrated that steel structures can be vulnerable to earthquakes. It should be pointed out that although steel structures suffered significant damage in the Northridge and Kobe events, most performed well from a life safety standpoint, with only a few collapses in Kobe.

There are generally two approaches to designing bridges to survive earthquake ground motions. One approach is to provide a load path to carry the inertial forces developed as a result of the motion to the ground. It is generally accepted that it would be uneconomical to design and construct all bridges in this manner to be strong enough to survive all potential earthquakes undamaged, since the return period of the maximum intensity earthquake to which a specific bridge site would be subjected may be considerably longer than the design life of the bridge, and this strength would most probably not be needed. This is especially true in the central and eastern portions of the U.S., large portions of which are regarded as areas of low seismicity. The second approach is to provide the structure with some means of isolation, damping, and/or energy dissipation, controlling the response, and limiting the forces to which the bridge is subjected. This can be accomplished with a controlled inelastic response of the structure itself, or by providing various "devices" designed to control the seismic response. It is this second method which this report addresses.

TABLE 1-1 1996 California Bridge Seismic Performance Criteria (Maroney, 1996)

GROUND MOTIONS	MINIMUM PERFORMANCE LEVEL	LIMITED PERFORMANCE LEVEL	FULL PERFORMANCE LEVEL
FUNCTIONAL EVALUATION EARTHQUAKE (FEE)	IMMEDIATE FULL SERVICE - Repairable within 90 days - Lane closures are allowable through off-peak hours - Minor concrete spalling - Buckling of secondary steel members - Joint damage allowable	IMMEDIATE FULL SERVICE - Repairable within 30 days - Repairs require minimum traffic interference - Buckling of secondary steel members - Joint damage allowable	IMMEDIATE FULL SERVICE - Essentially elastic - Minimal damage - Minor concrete cracking - Minor buckling of secondary steel members
SAFETY EVALUATION EARTHQUAKE (SEE)	NO COLLAPSE - Significant damage with a high probability of repairability - Maintain vertical load carrying capacity and a minimum lateral system capacity - No public access possible - Repairs will require complete evaluation	NO COLLAPSE - Immediate repairable damage - Light emergency vehicles within hours - Reduced public traffic lanes within days - Lateral system capacity is relatively reduced - Repairs within a year	IMMEDIATE FULL SERVICE - Minor repairable damage - Lateral system capacity slightly affected - Minor concrete spalling - Lane closures outside peak hours only - Repairs within 90 days
FAULT CROSSING (IE., RUPTURE EVALUATION)	NO COLLAPSE - Extensive damage with low probability of repair - Maintain residual capacity for probable vertical and lateral service loads only	NA	NA

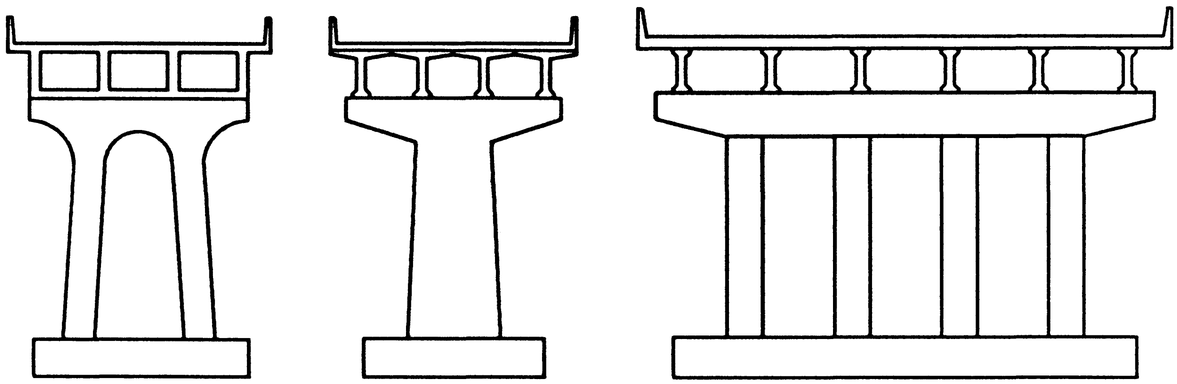
California's strategy in resisting earthquake forces is a two-tiered design approach. Bridges are designed for two earthquake events: The functional evaluation earthquake (FEE), and the safety evaluation earthquake (SEE). The FEE is an earthquake magnitude that has a good probability of occurring during the life of the bridge. The SEE is the maximum earthquake intensity to which the site is expected to be subjected. Depending on the assigned importance of the bridge, the post-earthquake performance level is assigned for each event as shown in table 1-1. For bridges in areas of low seismicity, but with the potential for a fairly high magnitude earthquake event, a similar two-tiered approach can be used, but the return periods of the FEE and SEE may need to be increased to account for the low probability of an earthquake occurring.

There are several isolation strategies for bridges. One method, using base isolation bearings, allows large movements at the bearings, shifting the period of the bridge, and reducing the dynamic forces resulting from the seismic input. Some form of damping and energy dissipation can also be provided with this approach. This strategy can be very effective in retrofits, and it is attractive due to its relative ease of construction. Isolation bearings are often used in conjunction with other schemes, as they can reduce the bridge response to a point where selective member strengthening is feasible. Bearings are not explored in this report, but many of the concepts that are investigated might effectively be coupled with isolation bearings to provide optimal solutions. Other NCEER reports are available on the application and performance of bearings (Mander, 1996).

Another approach, designing for other selected elements to provide a combination of isolation, damping, and/or energy dissipation, thereby limiting forces on the rest of the structure, is the main focus of this investigation. The inherent ductility of steel is well documented, making it an ideal material for this approach, but appropriate seismic design and detailing procedures must still be followed in order to take full advantage of steel's desirable properties. This report explores the methods of providing ductile behavior from not only the standpoint of economical first cost, but also from the perspective of ease of repair. Even if a bridge is expected to respond elastically to the maximum anticipated earthquake at the site, it still is sensible to provide a ductile ultimate failure mode, since earthquake loadings are so unpredictable.

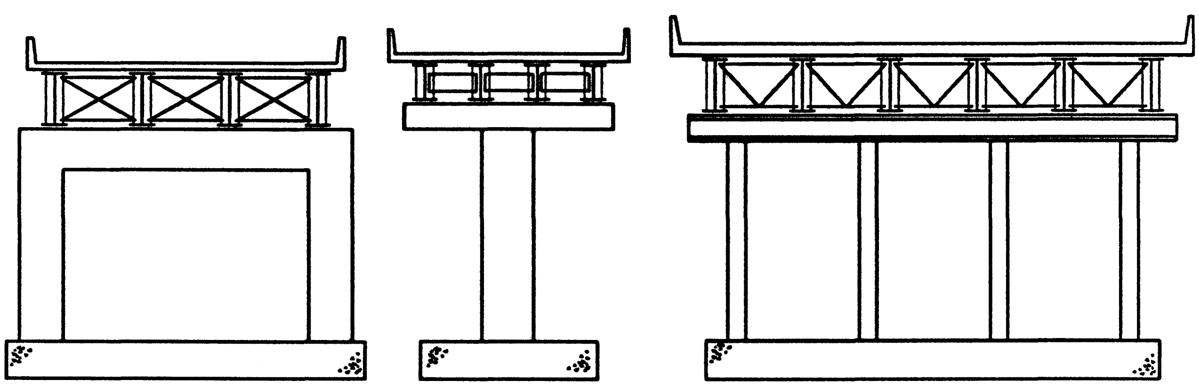
An examination of seismic provisions in bridge design codes results in an impression that seismic design philosophy was developed with concrete structures in mind, and then adapted for use in steel design as well. This is not surprising, given that little research has been performed on the seismic behavior of steel bridge components. The concrete philosophy is most reflected in the provision that limits plastic hinging regions to the top and bottom portions of pier columns. Considering the geometry of concrete structures, this is the best location for hinging, and the design and detailing methods for providing ductility at these locations is well understood.

Steel bridges, on the other hand, behave fundamentally differently from concrete bridges. Steel structures have differences in stiffness, ductility, mass, damping, and periods of vibration. There are also differences in the geometry of the structural systems of steel and concrete bridges. Figures 1-1 and 1-2 illustrate several typical concrete and steel bridge structural configurations. Figures 1-3 and 1-4 illustrate several typical steel and concrete bridge towers. Steel bridges can often provide for



(a) (b) (c)

FIGURE 1-1 Several typical concrete sub- and superstructure configurations



(a) (b) (c)

FIGURE 1-2 Several typical steel sub- and superstructure configurations

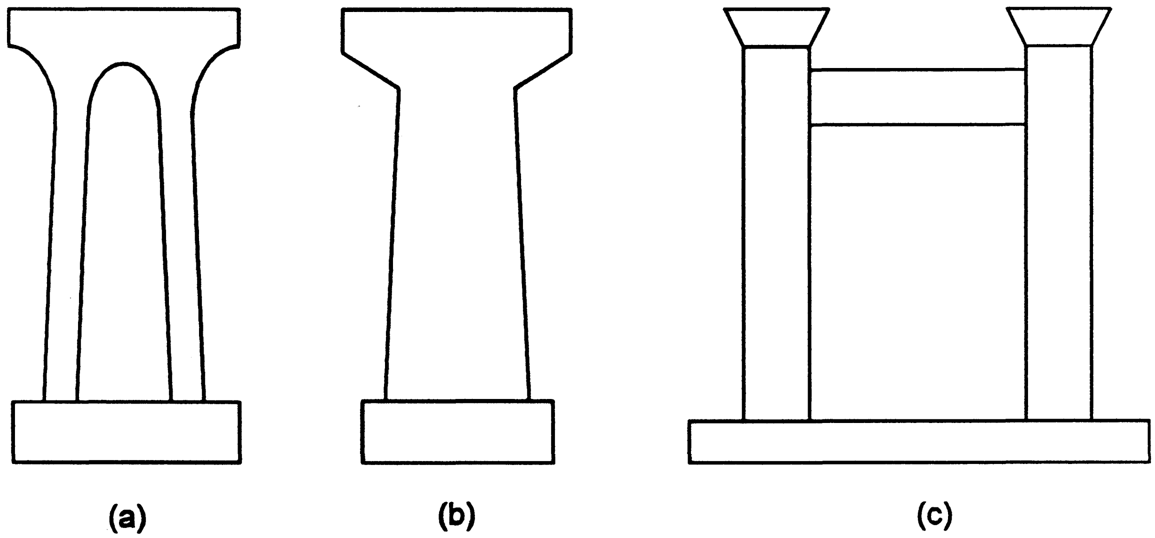


FIGURE 1-3 Several typical concrete pier tower configurations

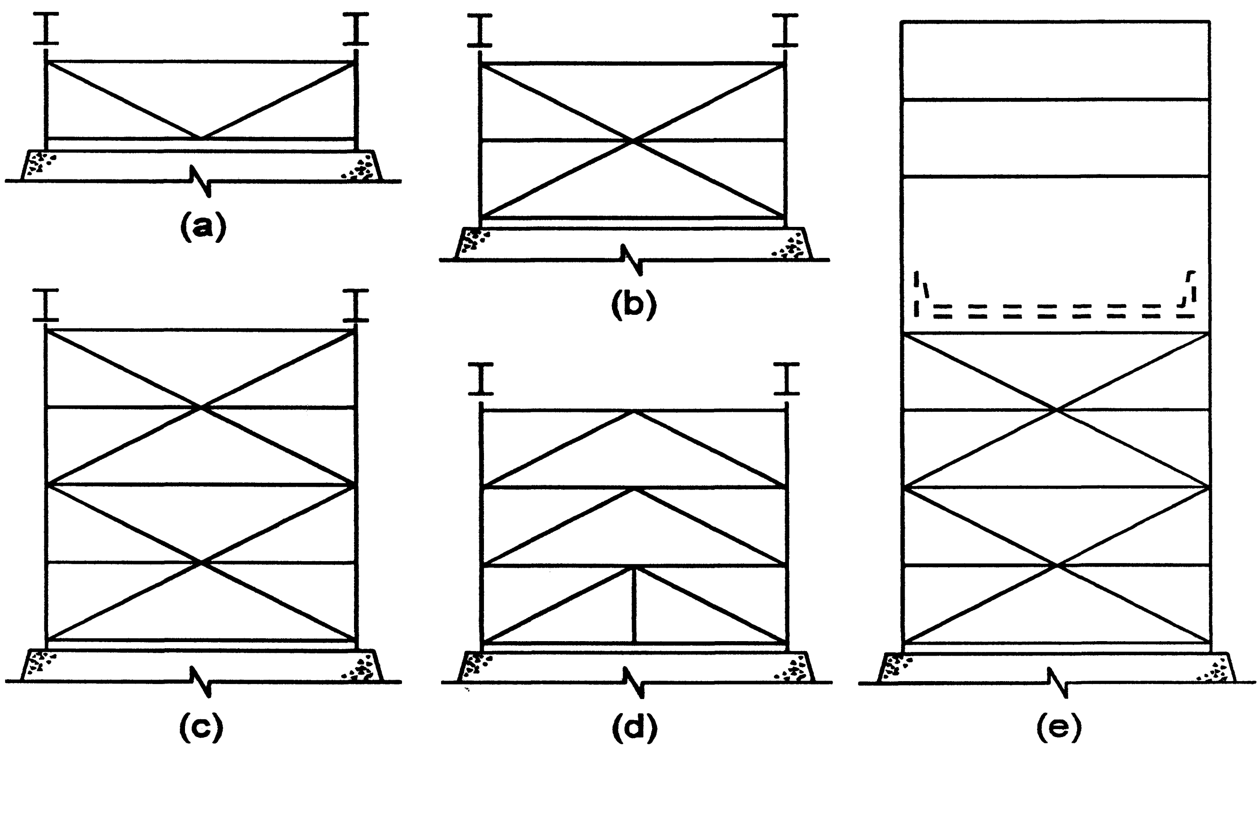


FIGURE 1-4 Several typical steel pier tower configurations

ductility and energy dissipation at multiple locations, and in various modes, often in direct contrast to concrete philosophy, and should not be constrained by concrete design limitations. One philosophical difference is that flexural failures are always desired in concrete and brittle shear failure is avoided. In steel, often the shear yield mechanism is desired, since it provides substantial stable energy dissipation. By comparing the various typical steel and concrete configurations in figures 1-1 to 1-4, it is apparent that the steel bridges can yield and dissipate energy at various locations such as bearings, diaphragms, lateral bracing, and pier caps/horizontal portal members, as well as the typical hinge location for concrete piers, the base and top of pier columns. Bridge design codes need to be modified such that all potential effective approaches to seismic resistance of steel bridges can be easily investigated, and the most appropriate method selected.

SECTION 2
DUCTILITY AND LATERAL FORCE RESISTING SYSTEMS

2.1 Ductile Cross-Sections

There are several factors that influence the "ductility" of members or structures. These include material properties, member cross-sectional shape, connection strength and geometry, structural frame geometry, and loading. These factors are all important in providing overall system ductility. As demonstrated in Northridge, using a ductile material, ductile member cross-sections, and a ductile frame geometry can all be negated by using a poorly detailed and constructed connection. In current earthquake design philosophy, the designer is expected to identify the areas where ductility is required, and detail these locations accordingly. Often, frames are provided with ductile "fuses," where preselected locations are designed for ductility, which limits forces on the remainder of the structure. It is important that the yield point of the steel be within a certain range in such designs, since the "fuse" is expected yield at a given load, prior to yielding or fracture of other elements.

Steel has traditionally been regarded as a ductile material. Evidence of this ductility is the typical stress-strain curve of A36 steel with the long yield plateau and the high strain to fracture (see figure 2-1). It is also generally accepted that ductility decreases as steel strength increases, because of the changing nature of the stress-strain behavior. Figure 2-1 also illustrates that as the yield strength increases, generally the yield plateau shortens, the strain to failure decreases, and the yield strength-to-tensile strength ratio increases. Design codes regard high yield-to-tensile ratios as undesirable. This is due to code provisions which depend on both the yield-to-tensile ratio and other post-yield

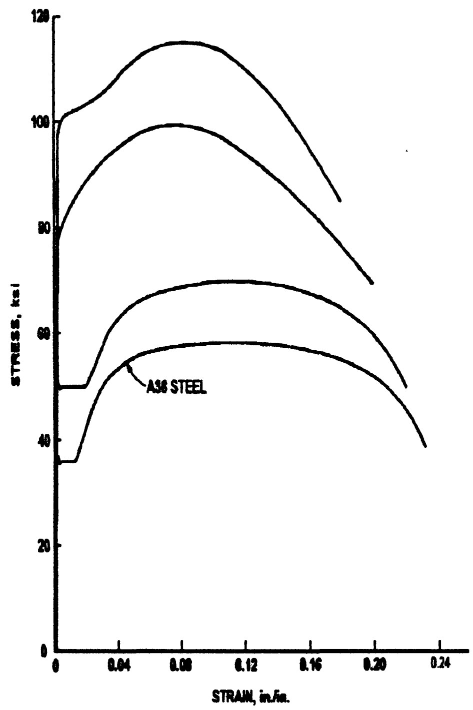


FIGURE 2-1 Typical stress-strain relationships of various grades of steel (adapted from Brockenbrough, 1974)

behaviors. Various post-yield material parameters are lumped under the yield-to-tensile ratio "umbrella" due to the difficulty in isolating individual effects. While a higher yield-to-tensile ratio might be acceptable, research would have to demonstrate the ductility capacity, and code provisions would have to be adjusted.

The 1996 AASHTO LRFD Bridge Design Specifications currently (Summer, 1997) limits utilization of the plastic moment capacity and certain moment redistribution provisions to members with yield strengths of 50 ksi (345 MPa) and less due to the lack of research data confirming the ability of higher yield steels to develop sufficient inelastic rotation. Recently developed High Performance Steels (HPS) designed to have improved properties, including ductility and toughness, have the potential to increase this yield strength limitation (Wright, 1996). Earthquake design requirements are typically more stringent than plastic design requirements, so it follows that research on higher yield steels, especially the new generation of HPS, is needed to document the rotation capacity of typical bridge members before their ductility is relied upon in earthquake resistant designs.

In order to quantify ductility, a ductility ratio is defined. There are several different definitions of ductility ratios, but they are generally based on a ratio of ultimate deformation to yield deformation or:

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{2-1}$$

where:
μ = Ductility ratio
Δ_u = ultimate deflection
Δ_y = yield deflection

There is not a standard definition of where yield and ultimate deformations occur, however, so care must be taken in comparing ductility values from different sources. Ductility ratios are used in several ways. By calculating the response of a structure to a given earthquake, a ductility demand can be determined and compared to the expected ductility, based on tests of the given system. Ductility ratios are also used to determine response modification factors, or R factors, which are used to reduce design forces based on the ability of a given structural configuration to yield and dissipate energy. While structural ductility can be much larger than individual member ductilities, or vice versa depending on member sizes, frame geometry, connection details, and loadings, increasing member ductility in the critical elements will increase structural ductility.

Member ductility of steel elements in tension is usually not a concern as long as failure is governed by yield on the gross section, or $\varphi_g A_g F_y \leq \varphi_n A_{ne} F_u$. Tension yielding can provide substantial ductility since instability is not a factor. It can be dangerous to rely on tension-only bracing, however, because one-sided response can lead to large deflections and early fracture. Member ductility is usually focused on elements or parts of cross-sections in compression, which is controlled by local or overall buckling. Sections with low slenderness ratios and low width-to-thickness ratios are therefore most attractive. From a cross-sectional point of view, tubular members fit these requirements best, and therefore are most efficient. Other member cross-sections can be effective also, but stiffeners or additional material (to decrease width-to-thickness ratios) may be required to postpone local buckling. The practicality and economy of fabrication and erection, such as making splices and connections, are also key considerations in selecting cross-sections to be used.

In order to determine the structural shapes most resistant to inelastic local buckling, one can examine the k factors given in the AASHTO LRFD Bridge Design Specifications (table 2-1) for use in the equation which is used for calculating limiting width-to-thickness ratios for members composed of plates:

$$\frac{b}{t} \leq \lambda_r = k\sqrt{\frac{E}{F_y}} \qquad (2\text{-}2)$$

where:
b = width of plate
t = thickness of plate
λ_r = limiting value of b/t to reach yield prior to local buckling
E = modulus of elasticity of steel
F_y = yield stress of steel
k = constant based on plate boundary conditions

The k factors give an indication of the efficiency of various member elements under uniform compression. Outstanding legs of single angles and separated double angles are the least efficient, followed closely by the flanges of I-sections, flanges of channels, and outstanding legs of pairs of angles in continuous contact. Stems of rolled T's are the most efficient element of the plates supported on one edge, most likely due to the relatively large flange providing rotational stability to the stem. Plates supported along two edges, such as box beam flanges and webs of rolled shapes, are much more efficient (about 2-3 times more efficient) than plates supported on one edge. Perforated plates have the highest k factor, because the factor is based on the total plate width, while the stress is based on the net width (total width less the hole width). The k-factors for tubular sections clearly show that these are the least susceptible to local buckling.

Compression elements which satisfy the width-to-thickness requirements calculated using the k factors given in table 2-1 will suffer local buckling soon after yield is reached. More stringent k factors are required if sufficient rotation capacity to reach a plastic hinge is needed. Seismic demands call for even more restrictive k factors than those for plastic hinges in order to provide stable cyclic ductility. Analogous to the k factors, k_p factors have been proposed for use in the equation:

$$\frac{b}{t} \leq \lambda_p = k_p\sqrt{\frac{E}{F_y}} \qquad (2\text{-}3)$$

where:
λ_p = limiting value of b/t to reach a plastic hinge prior to local buckling
k_p = constant based on plate boundary conditions

The k_p factors listed in table 2-1 have been proposed for designing members to reach plastic hinge rotational ductilities of 6 to 8 as required by current seismic codes (Astaneh-Asl, 1996). Of the sections for which k_p factors are proposed, box sections (plates supported along two edges) are the most efficient seismic sections (by 3-5 times) if local buckling is a controlling design factor. Tubular sections would be expected to be the most efficient with k_p factors expected to be about 1.12 for rectangular tubes and 1.30 for circular tubes in order to be in line with the listed values.

TABLE 2-1 AASHTO-LRFD k factors and proposed k_p factors for limiting width-to-thickness ratios

Plates Supported Along One Edge	k	k_p	b
Flanges and Projecting Legs or Plates	0.56	0.22	• Half-flange width of I-sections • Full-flange width of channels • Distance between free edge and first line of bolts or welds in plates • Full-width of an outstanding leg for pairs of angles in continuous contact
Stems of Rolled Tees	0.75	0.29	• Full-depth of tee
Other Projecting Elements	0.45	0.18	• Full-width of outstanding leg for single angle strut or double angle strut with separator • Full projecting width for others
Plates Supported Along Two Edges	**k**	**k_p**	**b**
Box Flanges and Cover Plates	1.40	1.12	• Clear distance between webs minus inside corner radius on each side for box flanges • Distance between lines of welds or bolts for flange cover plates
Webs and Other Plate Elements	1.49	1.19	• Clear distance between flanges minus fillet radii for webs of rolled beams • Clear distance between edge supports for all others
Perforated Cover Plates	1.86	1.49	• Clear distance between edge supports
Webs in Flexural Compression	-	3.76	• Clear distance between edge supports
Tubular Sections	**k**	**k_p**	**b or D**
Rectangular Tubes	1.7	-	• Width of face
Circular Tubes	2.8	-	• Diameter of tube

The orientation of the member and the type of loading can also affect the efficiency of a member. Members loaded axially will be limited by the element which has the lowest resistance to local buckling, while members stressed in bending will be controlled by the resistance of the elements in compression. For instance, an I-section loaded in bending about its strong axis will be controlled by either the compression flange or the depth of the web in compression, as compared to the same I-section in bending about the weak axis, where the controlling elements will be the projecting flanges under compression.

The second consideration when determining the ductility of members in compression is overall buckling. Studies of the cyclic post-buckling behavior of members is limited, and more work needs to be performed in this area. Studies performed to date (Astaneh-Asl, 1996) indicate that buckling ductility is directly related to the slenderness of the member, λ, taken as:

$$\lambda = \left(\frac{kL}{\pi r}\right)^2 \left(\frac{F_y}{E}\right) \tag{2-4}$$

where:
k = effective length factor
L = length of the member
r = radius of gyration
F_y = yield strength of the steel
E = modulus of elasticity of steel

For slenderness parameter values, λ, of less than 1.0, a post-buckling ductility of 4 is considered reasonable (Astaneh-Asl, 1996). For λ values of 2.25 or greater, a ductility of 1.0 can be assumed, with a linear interpolation between the values proposed (Astaneh-Asl, 1996)

As mentioned previously, tubular members are the most efficient shapes under compressive forces due to their self-stiffening nature and their optimization of resistance to both local and overall buckling. Steel tubular members for bridge piers have been used in Japan due to their minimal cross-sectional area and their fast erection time. Some of these tubular steel piers experienced a variety of unexpected problems in the Kobe earthquake, however.

Two rectangular steel box columns completely collapsed due to buckling at a splice causing corner welds to tear, after which the welds "unzipped". Although not known for certain, one report speculated that the columns were damaged in the quake, and then failed subsequently due to adjacent concrete pier failures shedding dead load into the steel columns (Iijima, 1995). One contributing factor was that the welds at the corners of these columns were never meant to transfer load, they were only designed as partial penetration stitch welds (Miki, 1996).

In Kobe, many round and rectangular steel tubular columns suffered damage due to local or overall wall buckling, sometimes resulting in permanent deformations (leaning), but they did not collapse, thus performing well from a life safety standpoint. Circular piers suffered from "lantern buckling" in which the walls of the piers bulged outward. This buckling occurred very close to welded joints where the plate thickness changed. In two piers, brittle fractures occurred along the outward plastic deformation, a previously undocumented phenomenon of compressive strain embrittlement resulting in fracture (Miki, 1996).

An investigation of a brittle fracture at an interior corner of a portal frame bridge pier indicated that the crack was the result of greatly decreased fracture toughness due to large plastic deformations at the corner (Miki, 1986). Several other cracks at the corners of portal frames and the bases of columns were attributed to low cycle fatigue, with local buckling accompanied by tensile cracking (Watanabe, 1996).

A final example of brittle fracture occurred in a centrifugally cast steel pier shaft. Although the design thickness of the tubular shaft was only 40 mm, a thickness of 68-72 mm was actually provided since a reinforcing portion was required to achieve the desired properties in the design portion. Cracks initiated in microcavities in the poor quality reinforcing portion, and propagated through the design thickness. The challenge to researchers is to determine whether these cracks can be arrested before they enter the design thickness of the column, or whether this type of fabrication process will have to be avoided in seismic zones.

The Japanese have performed some research on tubular steel box piers (Kawashima, 1989, 1992a; Usami, 1992; Nishimura, 1992; Yamada, 1992; MacRae, 1992), and additional work is being performed due to problems experienced in the Kobe quake. Although few complete failures occurred, existing steel columns could be vulnerable due to the post-buckling strength degradation which occurs after only a few cycles. Buckling is also undesirable from the standpoint of difficulty of repair. One lesson learned was that abrupt changes in stiffness, such as that occurring at a bolted or welded splice, can lead to local buckling, and should be avoided (Miki, 1996). Limiting the axial compressive stress to 1/10 of yield to minimize P-delta effects and reduce progressive shortening was also proposed, as well as eliminating fatigue prone details. Use of stiffeners which are three times as rigid as the minimum needed to prevent buckling was also advised, and a qualitative recommendation was made that maximum width-to-thickness ratios specified in the Japanese bridge design code be reduced to limit local buckling and ensure good post-yield cyclic ductility (Watanabe, 1996). Subsequent evaluation of width-to-thickness requirements resulted in the previous values being retained.

According to ATC-32 (ATC-32, 1992), AISC width-to-thickness requirements for tubes in seismic applications are already more stringent than the Japanese requirements prior to Kobe (see table 2-2). Nevertheless, after the performance of tubular piers in Kobe, width-to-thickness requirements for tubes as well as I-shapes and other compression elements in seismic applications should be re-evaluated, especially for members under primarily axial compression. The challenge to researchers is to determine how much cyclic rotation capacity is required, and set the width-to-thickness ratios accordingly. Comparisons of various existing code requirements for width-to-thickness ratios for 235 Mpa (34 ksi) steel compiled by Fukumoto (Fukumoto, 1992), and ATC-32 (ATC-32, 1992) are reproduced here in table 2-2. There are indications in the literature that some of the values in table 2-1 are high, considering the value of 18.9 for rectangular tubes in seismic applications in table 2-2 corresponds to a k value of about 0.65. The results of the current Japanese research on piers should be evaluated for additional insight.

TABLE 2-2 Limiting width-to-thickness ratios (b/t or d/t) for assessment of local ductility of rectangular and circular tubular members

F_y = 235 Mpa (34 ksi)			seismic		non-seismic		Remarks
			beam	column	beam	column	
Rectang	AISC LRFD - EQ (1992)		18.9		33		b/t ratio for square hollow or box sections
	EC 8 (1988 ed.)		66	33	66	33	for plate elements alpha=0.5 for beam
	AIJ LSD (draft 1990)	hot-frm, welded	41	32	48	39	b/t ratio for square hollow or box sections
		cold-formed	30	24	35	28	
Circ	AISC LRFD - EQ (1992)		38		38		Table B5.1
	EC 8 (1988 ed.)			50		50	class A
	AIJ LSD (draft 1990)		38	36	56	54	S-1 or S-2

As discussed previously, additional research will have to be performed to determine the seismic suitability and appropriate width-to-thickness ratios for the new generation of high performance steels (HPS) which are beginning to be applied. While HPS may have many desirable properties including high strength, high toughness, and weldability, the shape of the stress-strain curve and the post-yield behavior is not the same as the mild steels with which the average designer is familiar. The ductility and local stability of these steels need to be verified before they are considered for seismic applications.

Relatively recently the concept of hysteretic shear yielding in steel webs has been introduced as a technique for providing isolation and energy dissipation in buildings. This is the concept upon which eccentrically braced frames (EBFs) are based. I-sections are more suited to EBFs in buildings because of the ease of providing vertical stiffeners, and the availability of the deck and floorbeams to provide lateral-torsional bracing. Because of the need for bracing, EBFs in bridge pier applications will either have to be supplied in pairs, or will have to utilize torsionally stiff cross-sections, such as rectangular boxes. For more information on EBFs, see Section 2.4.

A second location where shear yielding can occur is the panel zone at beam-to-column connections. In thin-wall rectangular box portal frame bridge piers tests, a shear hinge in the corner panel resulted in more deformation capacity than moment hinges in the adjacent members (Nishimura, 1992). This paper also cautioned against stiffening the corner web to maintain elastic behavior, as it could lead to fractures at the corner web to flange weld (Nishimura, 1992). Furthermore, examination of steel beam-to-column connection research has indicated that panel zone yielding decreases beam flexural ductility (Roeder, 1996). Based on this research, the beam-column panel zone should be regarded as a desirable location for yielding, as long as flexural yielding of the beam at the column is not also required.

Encasing or filling steel members with concrete can postpone local buckling and is often used as a seismic retrofit (Sundstrom, 1996; Watanabe, 1996). This generally increases member strength and stiffness, but whether this results in increased ductility depends on the individual case (Usami, 1992; Kitada, 1992; Kawashima, 1992b; Takanashi, 1992). After the Kobe earthquake, one retrofit advocated for existing steel tubular piers was to fill them with concrete (Watanabe, 1996). A study by Caltrans proposed encasing many of the east span laced steel tower members of the San Francisco-Oakland Bay Bridge in concrete, as shown in figure 2-2 , in order to stiffen the towers (Sundstrom, 1996; Maroney, 1996). A second benefit of encasement in retrofit applications is that any lead paint present is also enclosed within the concrete, and flaking or removal is no longer an environmental concern. A proposed retrofit scheme for the Richmond-San Rafael bridge calls for concrete filling in the bottom portions of the steel tower legs to prevent local buckling in the plastic hinge zone. To avoid stiffening the legs the concrete will be placed in layers separated by a compressible joint material (Vincent, 1996). Concrete can also be placed inside tower legs to shift the plastic hinge zone away from the base as is proposed in the retrofit scheme for the steel towers of the San Mateo-Hayward bridge (Prucz, 1996). This moves the plastic zone away from the base plate connection area, as well as moving it away from the anchor bolt connections.

Using concrete or some other material (e.g., fiber-reinforced composites) to stiffen thin-wall steel members to increase strength and/or ductility is also an option for new construction (Note: No research projects or applications of fiber-reinforced composites in this manner are known). Such jacketing not only improves strength and/or ductility of steel members by postponing local and/or

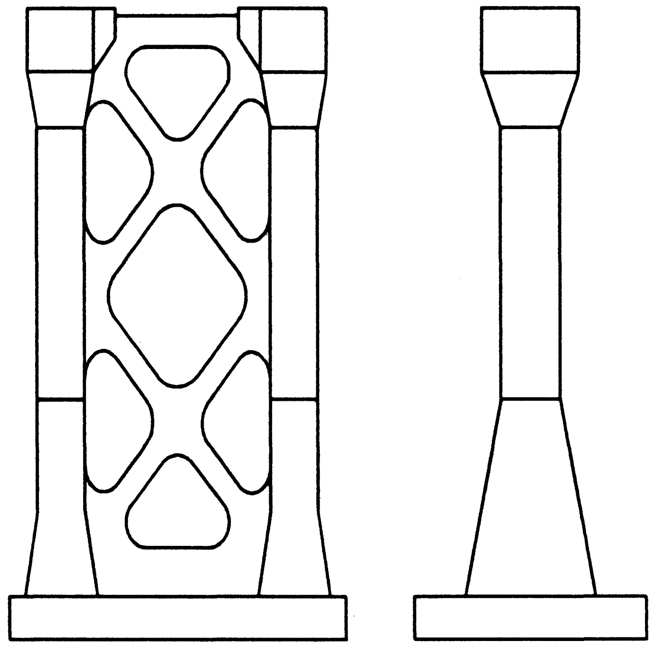


FIGURE 2-2 Appearance of pier after steel is encased in concrete (adapted from Maroney, 1996)

overall buckling, but it also provides protection from the elements. Spalling of concrete would be a concern, especially in potential plastic hinge locations, and steps such as providing mild reinforcement in these areas will have to be taken to prevent it.

2.2 Moment Resisting Frames

As stated previously, structural ductility is not only dependent on member ductilities, but also on structural geometry. Moment resisting frames (MRFs) can provide large ductilities, but they also suffer from relatively low lateral stiffness, and are susceptible to P-delta effects. MRFs are often used due to architectural requirements of open bays in buildings, which is not generally a requirement in steel tower piers, except at the roadway level. Vierendiel bracing is sometimes used on steel bridge towers for aesthetic reasons, the Golden Gate bridge towers being the most famous example. As mentioned previously, a study of box beam-to-column connections concluded that portal frames designed to yield in shear in the panel zone displayed more ductility than those designed to yield in the beam or the column (Nishimura, 1992).

Welded moment resisting frames (MRFs) in buildings performed poorly in the recent Northridge and Kobe earthquakes, with cracking occurring at the welded beam-to-column connections, as illustrated in figure 2-3 (Chen, 1996; Bonneville, 1996; Horikawa, 1996). While many of the problems encountered in buildings are not applicable to bridges due to differing construction practices, the knowledge gained from the welded moment connection performance and subsequent research could be beneficial to bridge designers. Of particular interest should be the field welding quality control practices and inspection procedures that are recommended, since field welding of bridges may become popular again if the new generation of high performance steels lives up to its promises.

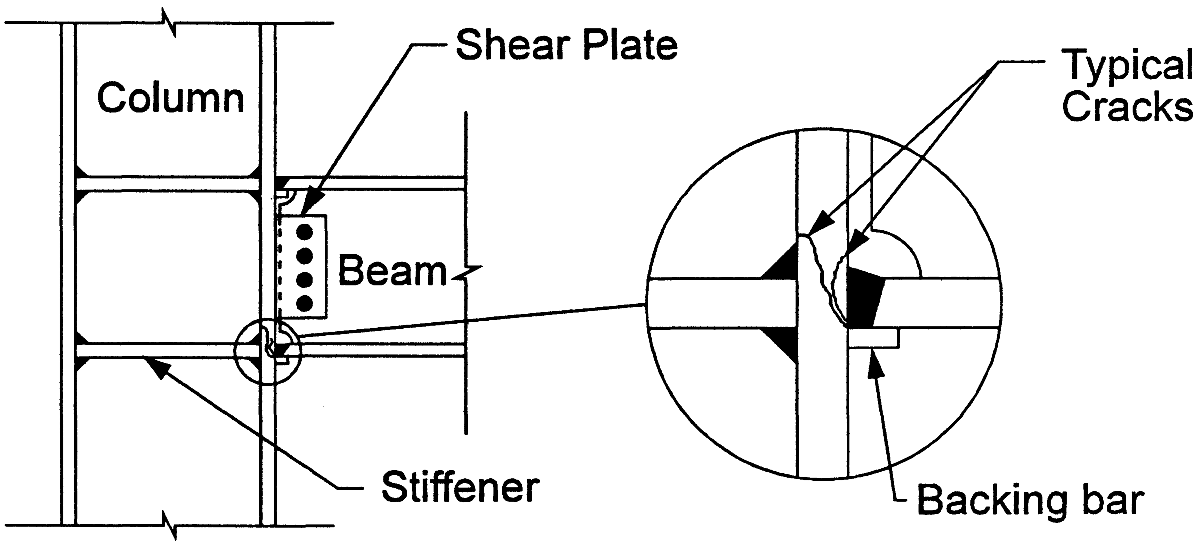


FIGURE 2-3 Typical cracks in welded MRF from the Northridge earthquake

The problems in Northridge could not be blamed on one single factor. The material, fabrication, and design all contributed somewhat to the problems. The contributing material factor was the use of low toughness weld electrodes. The weld procedures and inspection were also inadequate. The design which left backing bars in place and extrapolated on outdated static connection research results was also partly at fault. The combination of these and other factors led to the poor seismic performance of the connections. The problems also highlight the fact that seismic vulnerabilities of steel structures can often be traced to the connections.

One of the most important philosophies of seismic design is that connection failures should be avoided. This is because joint failures are usually brittle, and even though they do not necessarily result in complete loss of load carrying ability of the member, strength and stiffness losses can have serious consequences. Since often the highest forces are located at connections, and geometry changes and discontinuities at the joints result in stress concentrations, it can be difficult to achieve this goal. Some of the concepts proposed in the aftermath of the Northridge failures that may be useful in bridge connection designs include methods of reinforcing connections or weakening members in order to shift inelastic action away from the connection locations and into main members (Fairweather, 1996).

Several of these connection concepts are proprietary, including the MNH-SMRF connection (figure 2-4), which uses side plates and fillet welds to make the moment connection (Nelson, 1996). Looking at this connection, one cannot help but think of it as a slightly more complicated welded gusset plate connection. A second proprietary solution supplies horizontal slots in the beam web top and bottom, and vertical slots in the column web opposite the beam flanges (Richard, 1995). These holes serve to "soften" the stiff areas of the connection. Analysis and testing have shown that this method reduces stress concentration factors from 5 down to 1.2 (Fairweather, 1996). A solution that uses

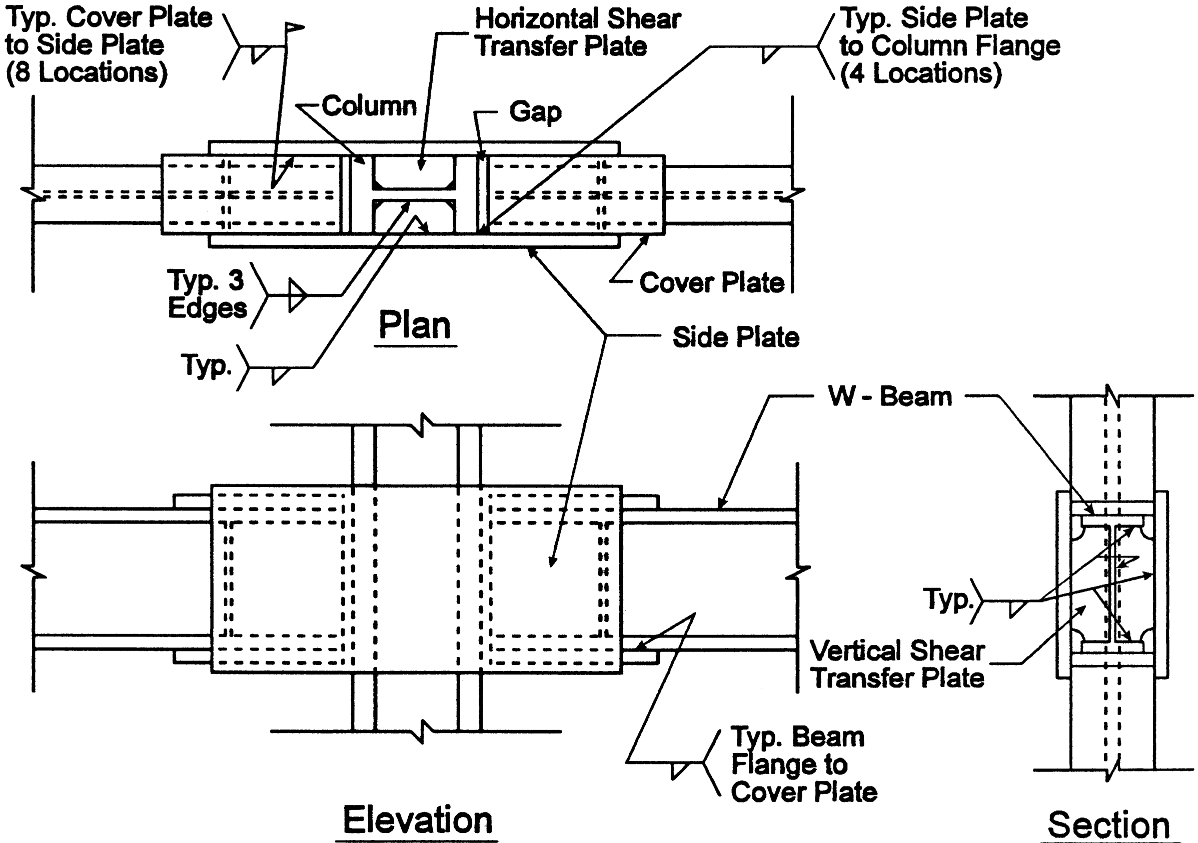


FIGURE 2-4 MNH-SMRF connection

bolts and does away with field welding altogether is being tested at Lehigh University, and has performed well. The bolted connections are supposedly just as stiff and more ductile than their welded counterparts (Fairweather, 1996).

Several non-proprietary Northridge solutions include strengthening the connection zone with cover plates and/or stiffening plates (figure 2-5) and the "dogbone" connection (figure 2-6), which removes material from the beam flange a short distance from the column (Engelhardt, 1995; Engelhardt, 1996). Both of these connections shift the plastic hinge away from the column face and into the beam member. The circular arc "dogbone" connection configuration is part of the proposed retrofit of the Richmond-San Rafael bridge (Vincent, 1996). Another solution uses cut wide flange sections to reinforce connections and shift the plastic hinge away from the column (see figure 2-7).

Most of the Northridge solutions still entail field welding, although some of the ideas may be applicable in conjunction with bolted moment connections also. All of the connection concepts listed can be used in new construction, while the reinforcing plate system, the "dogbone", and providing the proprietary slotted solutions are most suited for retrofit. It should be noted that after initially performing very well, the "coverplate" solution (figure 2-5) has achieved mixed results in subsequent testing (FEMA, 1997), and care should be taken in applying this detail. For a more comprehensive look at connection alternatives, see SAC,1997.

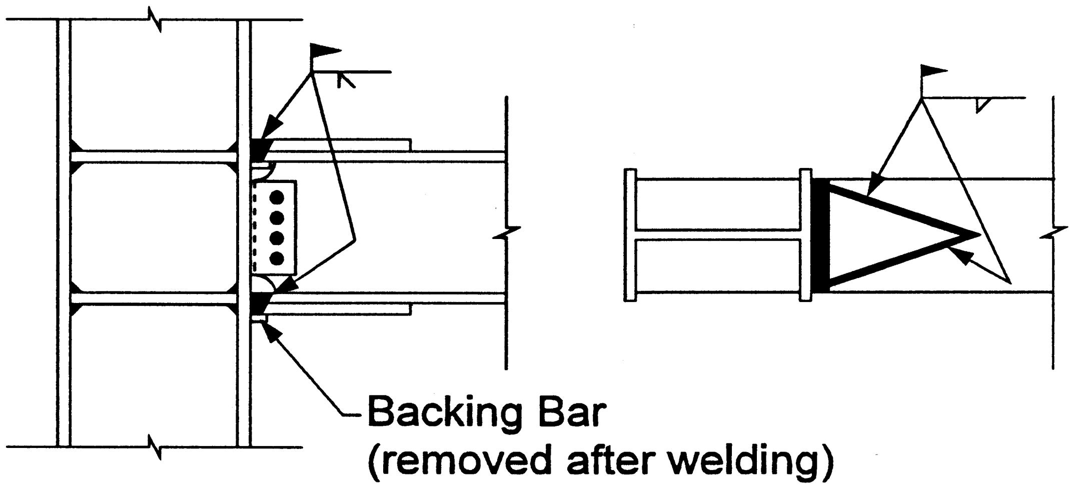


FIGURE 2-5 Connection reinforced with cover plates at the column

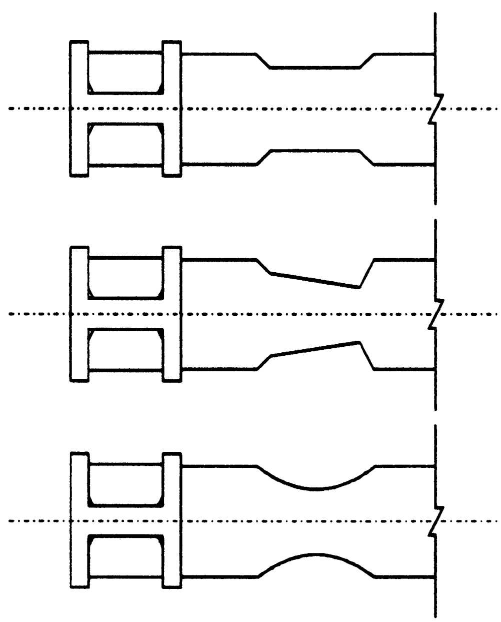


FIGURE 2-6 Various "dogbone" connection configurations

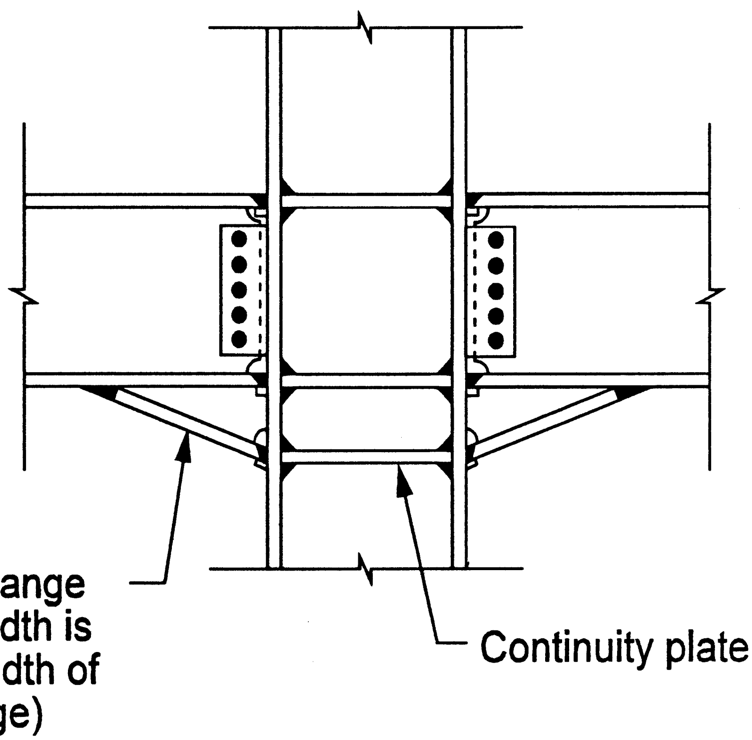


FIGURE 2-7 Post-Northridge connection using wide flange sections to move plastic zone away from column face

2.3 Concentrically Braced Frames

One of the most common lateral force resisting systems in steel bridges is concentric bracing. Braced frames provide good lateral stiffness, but different bracing schemes can provide very different structural ductilities. While X bracing appears to be most common, V and inverted V or chevron bracing is also prevalent (see figure 2-8). Concentrically braced building frames have a reputation for limited ductility, due to the historical cyclic performance of compression braces. Reduced capacity due to overall buckling and low cycles to failure mainly because of severe local buckling led to seismic code requirements that braces be designed for 1.5 times the design loads. Recent research has indicated that the approach of increasing the design load to compensate for the lack of ductility is not an effective solution, and that a more rational approach would be to detail the members and connections for better cyclic post-buckling performance (Goel, 1992a, 1992b). Using lower width-to-thickness ratios to limit local buckling and designing the connections for the post-buckling forces and deformations greatly improve brace performance (Goel, 1992a). Post-buckling strength degradation was found to occur faster as the slenderness of the struts increased (Popov, 1981).

Several analytical methods have been developed to deal with the complicated post-buckling cyclic behavior of bracing members (Maison, 1980; Ikeda, 1986; Soroushian, 1990, Nakashima, 1992). The most promising of the methods for design is the hinge model, which falls into the category of physical theory models. Physical theory models only require material properties and common geometric or derived engineering properties of a member, which make them easily applicable in all

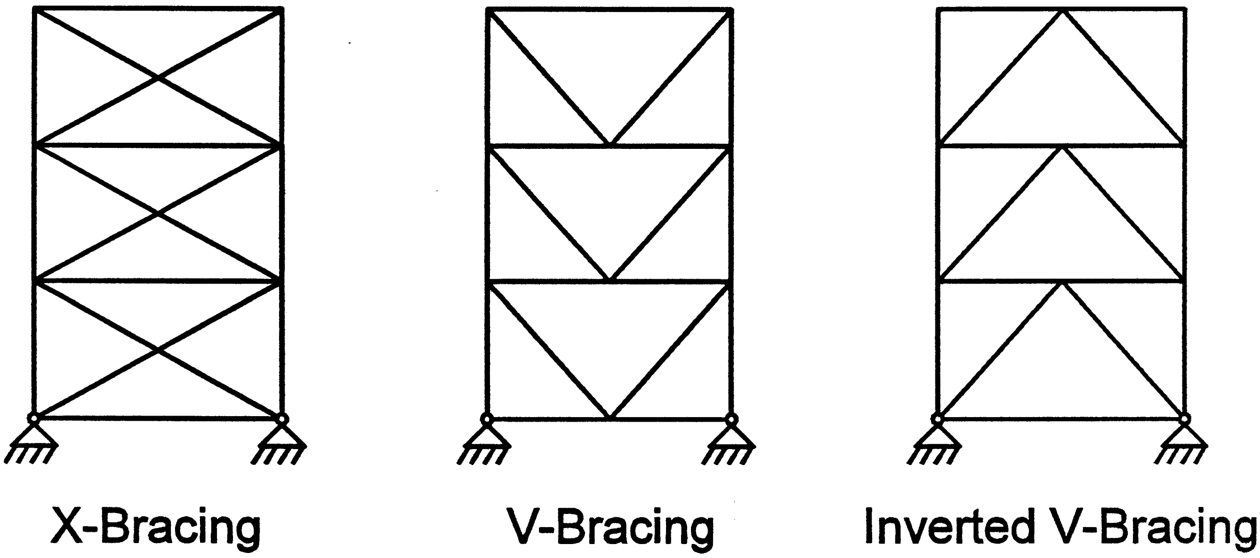


FIGURE 2-8 Typical concentric bracing configurations

cases. Other classes of models are finite element models, which are computationally intensive, and phenomenological models, which require calibration to experimental results.

The hinge model assumes a pin ended member with a plastic hinge at midspan. The hinge model can be extended to fixed end braces by using an effective length of 0.5L (Nakashima, 1992). For X-braces with a connection at midspan, the effective length factor varies from 0.4 to 0.65 (Nakashima, 1992). Several refined hinge models have been developed that empirically incorporate such factors as strength degradation in subsequent compressive buckling cycles and the axial force-rotation relationship of the plastic hinge. Strain-hardening and the Bauschinger effect are the main factors that result in a reduction of the compressive resistance under load reversals (Nakashima, 1992).

Figure 2-9 shows several bracing schemes and their deflected shapes. By examining the lateral load paths, a qualitative idea of the relative ductilities of the various schemes can be determined (ECCS,1991), based on experience with buildings. K-bracing, (a), provides the least ductility, since very large demands are placed on the columns when the compression brace buckles. For this reason the use of K-bracing is not recommended. Chevron bracing, (b), while not quite as bad as K-bracing, suffers similarly in that the beam must be extremely strong and stiff if it is to pick up the load once the compression brace buckles. Chevron braces are relatively common in bridge towers and top chord lateral bracing in trusses and arches. X-bracing, (c), is the best of the concentric bracing schemes due to the redundancy provided by the configuration. Even after the compression brace buckles, the tension brace provides a load path for the lateral forces. As mentioned previously, this should not be taken to mean that tension-tension bracing is appropriate. On the contrary, the challenge in designing X-braces is to provide enough compression ductility in the brace member and its connections such that it can survive the load reversals that accompany the cyclic seismic loading. Eccentric bracing, (d), which has been used primarily in buildings to date, would be expected to have better ductility than any of the concentric bracing schemes. The estimated relative ductilities of each configuration is listed on the figures (ECCS, 1991).

The AISC equation for the first buckling load of braces has been found to be very accurate (Astaneh-Asl, 1984). Additionally, research has shown that concentric braces can be detailed to achieve

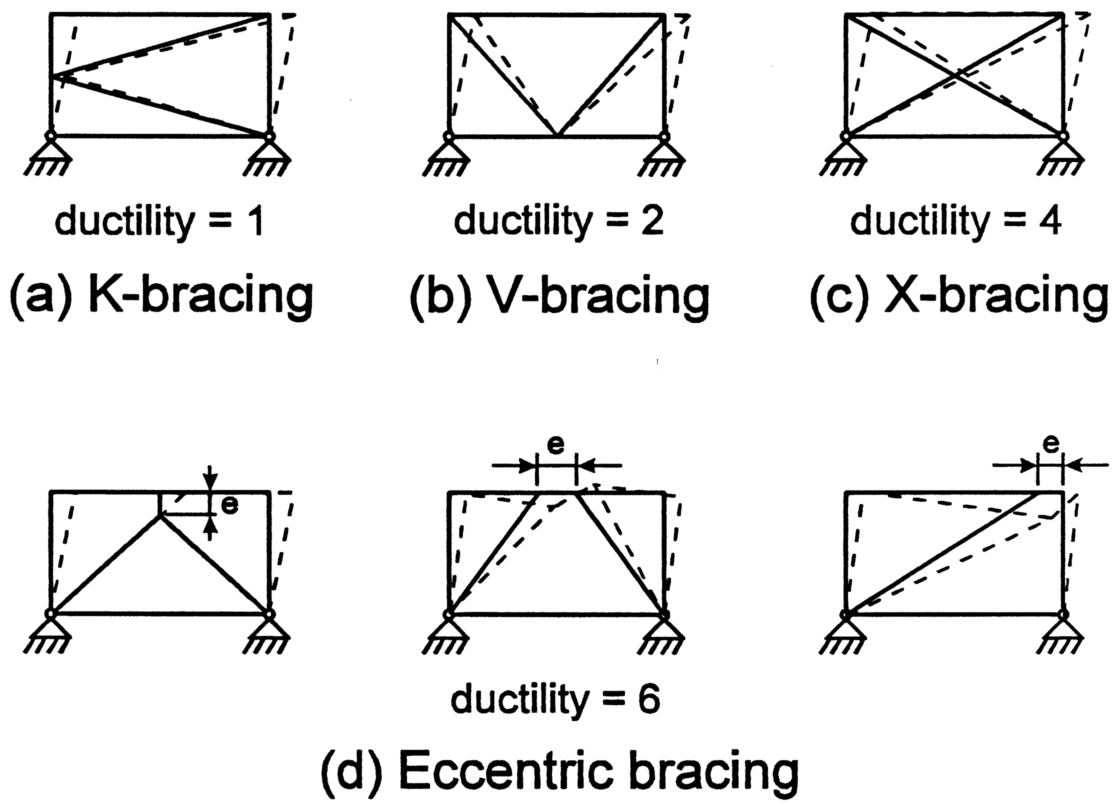


FIGURE 2-9 Various lateral bracing schemes (deflected shapes in dashed lines) and their approximate relative ductilities

adequate cyclic ductility (Astaneh-Asl, 1984, 1986; Goel, 1989a, 1989b). Width-to-thickness ratios corresponding to k factors of about 0.20 for outstanding legs of angles and 0.56 for rectangular tubes (refer to table 2-1) are recommended for braces (Goel, 1992a). Concrete infilling doubled the allowable width-to-thickness ratio (Goel, 1992a).

For built-up members with stitches, the slenderness ratio, L/r, of the individual elements between stitches should not exceed 0.4 times the governing slenderness ratio of the built-up member for stitches subjected to post-buckling shear, and 0.75 times the governing slenderness ratio of the built-up member for stitches not subjected to post-buckling shear (ATC-32, 1992). Bolted stitches should not be placed at locations of plastic hinge formation (usually midspan) (Astaneh-Asl, 1984, 1986). Based on actual measurements of post-buckling forces, stitches between double angle braces buckling out-of-plane and subjected to post-buckling shear should be designed to carry a force of $A*F_y/4$ acting at the centroid of one angle from one element of the member to the other where A is the total area of the member (Astaneh-Asl 1986; AISC, 1994).

Connections are one of the most critical areas when detailing for seismic resistance. The required strength of bracing joints should be the least of:

- The design axial tension strength of the member.
- The maximum force that can be transferred to the brace by the system.

For bolted braces, in order to ensure that the failure is not brittle fracture through the net section, the minimum ratio of effective net section area to gross section area should be limited by:

$$\frac{A_e}{A_g} \geq \frac{1.2\alpha P_u^*}{\varphi_t P_n} \tag{2-5}$$

where:
A_e = effective net area
A_g = gross area
α = fraction of the member force that is transferred across a particular net section
P_u^* = required strength of the brace as defined above
φ_t = tension strength resistance factor
P_n = nominal tension strength of member

In order to obtain better cyclic performance of bolted connections, the plastic hinge region can be shifted into the gross area of the brace by welding reinforcing at the first bolt hole such that the net section becomes stronger than the gross section (Astaneh-Asl, 1984). Experiments on double angle bracing members showed that for design purposes, using the point on the plastic M-P interaction diagram corresponding to an axial force of $0.5*P_y$ ($A*F_y/2$) in conjunction with a moment of $2.5*M_y$ (Astaneh-Asl, 1986), very conservatively accommodated post-buckling forces for all double angle members in the AISC Manual (for other sections these values may differ). In addition, connections should be checked for an axial force of P_y ($A*F_y$). Welded or bolted connections should also be designed to carry the moment caused by the eccentricity of the centroids of the member elements to the centroid(s) of the connecting elements.

For braces that can buckle out-of-plane, the brace should terminate at a minimum of two times the gusset thickness from a line about which the gusset plate can bend unrestrained by the column or beam joint to allow for plastic hinge formation (Astaneh-Asl, 1986). The gusset plate should be designed to carry the compressive design strength of the brace member without local buckling. In order to prevent premature buckling of gusset plate edges, the following limit is recommended (Astaneh-Asl, 1992):

$$\frac{L_{fg}}{t} < 0.75\sqrt{\frac{E}{F_y}} \tag{2-6}$$

where:
L_{fg} = length of free edge of gusset plate
t = thickness of gusset plate

Chevron (inverted V) braced buildings designed with a force reduction factor R = 10 and ductile braces were found to exhibit very satisfactory behavior (Goel, 1992b). It should be noted that much of the concentric bracing research was performed on light section members representative of building braces, but the concepts of detailing for cyclic post-buckling behavior of compression braces can still apply to steel bridge lateral bracing members, even though bridge members are not necessarily the same size and cross-section as building braces. Some research has also been performed on tubular bracing members representative of offshore construction (Popov, 1980), but additional research to confirm the cyclic ductility of typical bridge-sized bracing members and connections should be performed.

2.4 Eccentrically Braced Frames

The initial concept of eccentrically bracing building frames in seismic applications was developed at Berkeley in the late 1970's (Roeder, 1978). Eccentrically braced frames (EBFs) were found to provide strength and stiffness approaching that of a concentrically braced frame, along with the ductility and energy dissipation capacity approaching that of a moment resisting frame (Roeder, 1978). As the name suggests, in an EBF the bracing is given an intentional eccentricity, e (see figure 2-9(d)), in order to furnish a beam segment (also known as an active link) which will yield under extreme seismic forces, providing a structural "fuse" which limits seismic forces and contributes energy dissipation. In well proportioned EBF's, all of the links yield almost simultaneously, with the remainder of the structure remaining elastic.

Much research on eccentric links has been performed, demonstrating the validity of the concept, and providing design and detailing rules (Hjelmstad, 1983, 1984; Manheim, 1983; Malley, 1983; Kasai, 1986a, 1986b, 1992; Popov, 1989, 1992; Foutch, 1989; Lee, 1989; Goel, 1989c; Engelhardt, 1989, 1992a; Ricles, 1994). While initially designed and detailed with small eccentricities to achieve yield in shear (shear links), longer links which develop plastic moment hinges (flexural links) have also been tested. There is a concern that differential rotations at the ends of the active links could lead to additional out-of-straightness in the braced compression members. This does not appear to be addressed in the literature on building frames, so it may not be a problem, but if this concern is addressed, there is no reason that the EBF concept can not be applied to bracing of bridges.

Structurally, shear links are superior to flexural links due to their higher strength, stiffness, and inelastic deformation capacity. Flexural links should only be used where functional constraints dictate that large openings are required (Engelhardt, 1992b; Kasai, 1993). Research on building frames has shown that link lengths $\leq 1.6*M_p/V_p$ experience shear dominated behavior, while link lengths $\geq 3.0*M_p/V_p$ experience predominantly flexural behavior (Engelhardt, 1992b).

Figure 2-9(d) shows some of the various geometries that eccentric bracing can take, including one where the active link is either the gusset plates connecting the V-bracing to the strut (Astaneh-Asl, 1992) or some other type of shear link system.

Since steel bridge towers are generally only one "bay" wide, the bracing geometries of figure 2-9(d) with the link at the center of the strut would appear to be the most promising. These provide a symmetrical one bay bracing geometry, while also having the advantage of moving the plastic hinge location away from the beam-column connection. Although eccentric links located at the beam-column connection can be detailed to perform well, after the problems at Northridge and Kobe with beam-column moment connections (Chen, 1996; Bonneville, 1996; Horikawa, 1996), some engineers might be hesitant to use them. As stated previously, EBF's used in bridge towers will have to have either torsionally stiff members containing the active links, or be applied in pairs, in order to provide out-of-plane bracing at the plastic hinge locations, since no floor system is present to supply bracing.

2.5 Structural "Fuse" Concepts and Innovative Systems

There are quite a few concepts and systems that have been recently advocated for increasing seismic resistance. One of the most popular new concepts is to provide a controlled response through use of specific devices providing one or more of the following: Isolation, damping, and energy

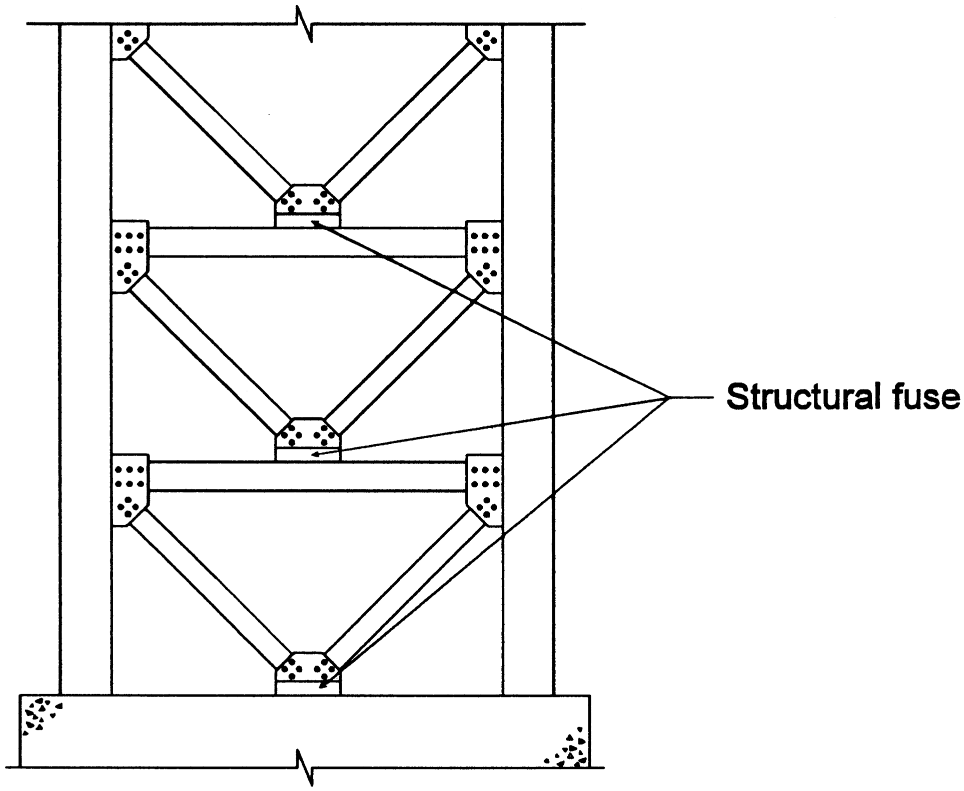


FIGURE 2-10 Concentric bracing with structural "fuse" at connection

dissipation. Devices utilizing these characteristics can function as structural "fuses," providing sacrificial elements which protect the remainder of the structure from damage. The devices fall generally into three categories: Friction devices, viscous or viscoelastic devices, and material yield devices. These "devices" can either utilize a part of the structural frame, as with the active link in an EBF, or be a separate apparatus placed in series or parallel with a portion of the structural frame.

Often the bearings of bridges are locations of choice for isolation, damping, and energy dissipation devices, but many of these devices can also be used in conjunction with structural frames. Used in series with concentric bracing for instance, as illustrated in figure 2-10, brace forces can be kept below buckling loads, confining any damage to the fuse element. While many of the devices were developed for building applications, there is no reason why they would not be effective for bridges also, as long as accommodations are made for environmental exposure, when necessary. Several promising systems are described below, although there are many other systems, some proprietary, which use these three categories of seismic protection.

2.5.1 Slotted Friction Connections

The slotted bolted connection (SBC) shows promise in providing a device which is low cost, easy to design, and easily repairable. The SBC can be implemented wherever lap-joint or butt-splices are feasible. One potential location would be in the connections of concentric lateral bracing systems of steel bridge substructures or superstructures.

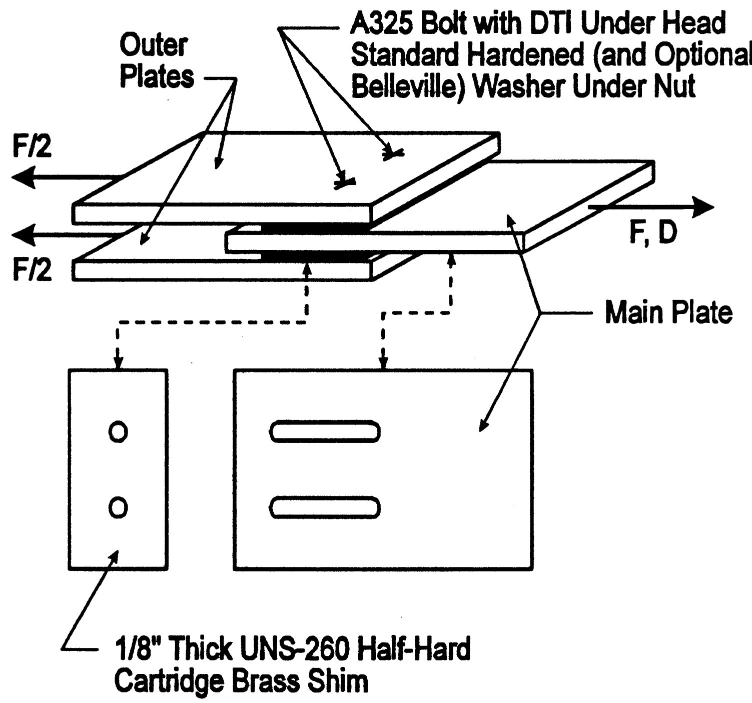


FIGURE 2-11 Slotted bolted connection

The concept of the device is simple. A steel plate with slotted holes parallel to the direction of loading is bolted between two other steel plates with brass insert plates separating them as illustrated in figure 2-11 (Grigorian, 1994). The bolts connecting the plates are tensioned to a specified amount in order to provide sufficient friction such that no slip occurs under working loads. Under extreme event loads, the friction force is overcome and the connection slips under constant load. Application of cyclic displacements results in almost rectangular hysteresis loops (essentially elastic-perfectly plastic behavior), which provides consistent, repeatable energy dissipation. After slipping, the bolts and brass plates can be replaced, and the connection retensioned.

Shake table experiments have verified the performance of the device in buildings, but the long term performance as part of a member exposed to the environment should be confirmed before applying this concept to the lateral bracing systems of bridges.

2.5.2 Knee-Brace-Frame

Another emerging lateral bracing scheme which has received little attention to date is the Disposable Knee-Brace (DKB) (Aristizabal-Ochoa, 1986) or Knee-Brace-Frame (KBF) (Balendra, 1993). Like the EBF, the KBF combines the stiffness of a CBF with the ductility of an MRF. In this system one end of the concentric diagonal brace is connected to a knee anchor as illustrated in figure 2-12. Rigid connections are supplied between the knee anchor and the beam and column. The system provides good stiffness while remaining elastic for wind loads and smaller earthquakes. For large earthquakes, the knee brace yields, providing a structural "fuse," supplying energy dissipation and limiting forces on the remainder of the structure. Flexural yielding in the knee brace was initially studied, although shear yield has also been investigated, and both have performed well.

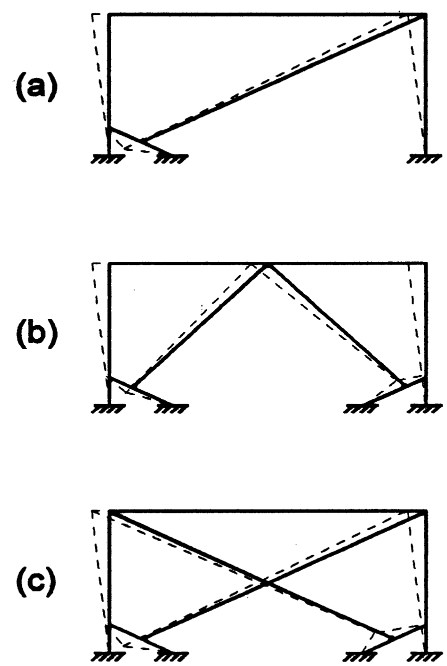


FIGURE 2-12 Disposable knee-brace (DKB) or knee-brace-frame (KBF) (deflected shape dashed)

The main advantage espoused for the KBF compared to other seismically resistant frame configurations is the potential ease of repair following a large earthquake. To return a KBF to original pre-quake form, one must only bolt replacement knee brace anchors into locations which yielded in the quake. The difficulty of jacking the structure back to plumb, if necessary, may make this task somewhat more difficult, but other conventional bracing systems share this problem. Since the diagonal braces are not part of the gravity load bearing frame, immediate replacement is not necessary, and no shoring would be required. MRFs and EBFs both use members that are integral parts of the frame (beams) as the energy dissipation elements. This results in more difficult repairs following a major earthquake. CBFs require that the entire lateral bracing system be replaced.

The main disadvantage of the KBF is that bracing members would have several extra connections each, which is a very labor intensive task in both fabrication and erection. Designers should also be aware that as of this writing (1998), while the limited laboratory investigations have indicated some potential, this system has had no field-proven applications. Although the KBF is on the right track in attempting to provide for ease of post-earthquake repair, some of the other schemes presented in this report would seem to perform similarly while being more economical.

2.5.3 Steel Plate Added Damping and Stiffness (ADAS) Devices

Steel plate added damping and stiffness (ADAS) devices, as illustrated in figure 2-13, fall into the material yield device category. Devices such as this are attractive due to their consistent response at any temperature, and their ordinary maintenance requirements, since they are made of the same material as the structure they are part of. By utilizing ADAS devices the energy dissipation capacity of a building was shown to be substantially increased while the demand on the framing members of

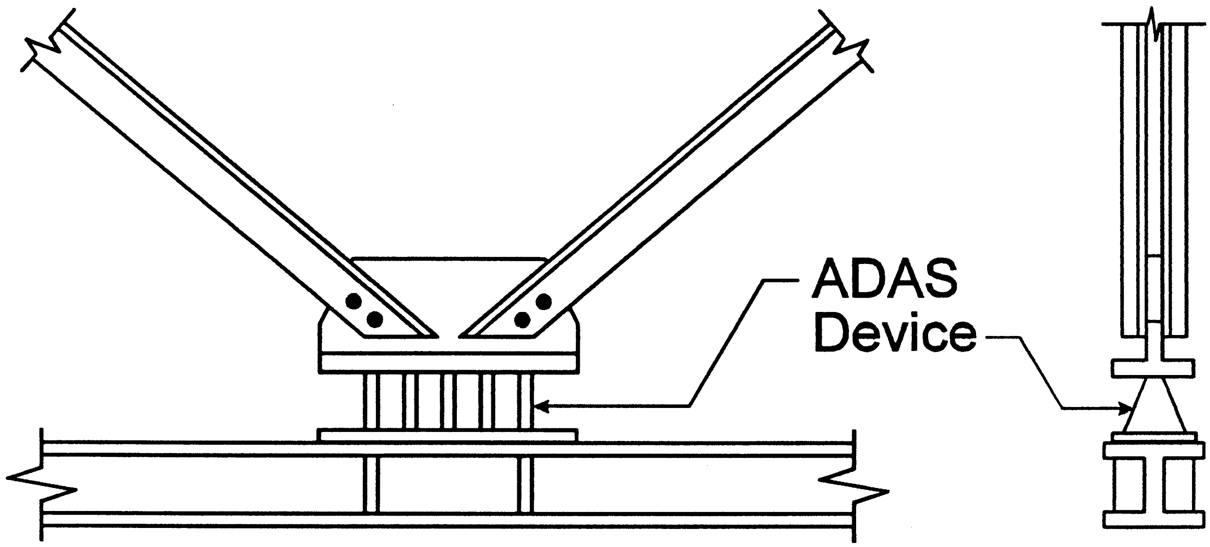


FIGURE 2-13 Added Damping and Stiffness (ADAS) device

the structure was reduced (Xia, 1992). The behavioral characteristic of the device must be well known, including post-yield behavior, as the yield force, yield displacement, strain-hardening ratio, ratio of the device stiffness to the brace stiffness, and the ratio of the device stiffness to the structural story stiffness without the device in place have been identified as the most important parameters (Xia, 1992).

2.5.4 Shear Panel Systems (SPS)

Shear panel systems work on the same principle as active links in EBFs. Essentially, instead of using a portion of one of the structural members as the active link, the active link is isolated as a separate device, and then located where the structural fuse is desired. One suitable location would be at the locations of bracing member connections.

2.5.5 Viscoelastic Dampers

Viscoelastic dampers are devices that are designed to absorb and dissipate, as heat, the vibrational energy that passes through them. These devices have been successfully applied to buildings to damp wind motions. A typical viscoelastic damper sandwiches a steel plate between two steel tees with a thin layer of viscoelastic material between the tee flanges and the plate. Behavior of the dampers has been found to be dependent not only on the loading, but on the temperature of the device. Damping decreases as temperature increases. It has been shown (Tsai, 1993) that viscoelastic dampers can significantly reduce the response of a bridge to earthquake loading.

2.6 Slab-On-Steel Girder Systems

Several slab-on-steel girder bridges suffered damage to their bearing diaphragms or cross-braces in the Northridge earthquake, while the accompanying superstructures and substructures (including the bearings) were relatively undamaged. Essentially identical bearing locations where the bearing

(Astaneh-Asl, 1994). This behavior resulted in a recommendation that the bearing diaphragm locations would be good positions to allow yielding to occur, limiting forces on the remainder of the structure, and providing energy dissipation. The original suggestion for the "ductile end diaphragm" was to provide initially bent bearing diaphragms or cross-frames, whose behavior was well defined, in order to furnish a controlled response (Astaneh-Asl, 1994). Subsequent researchers have proposed other methods of providing controlled response and energy dissipation at bearing cross-frame locations.

Researchers at the University of Ottawa are investigating several ductile end diaphragm approaches. The proposed systems, illustrated in figure 2-14, include shear panel systems (SPS), eccentrically braced frames (EBF), and triangular plates added damping and stiffness devices (TADAS) (Zahrai, 1996). A simplified 2-D calculation procedure has been proposed, and is outlined in Zahrai, 1996, which essentially reduces the problem to a 1 degree of freedom system by calculating generalized mass and stiffness parameters for the deck and combining it with the bi- or tri-linear behavior of the end diaphragm. Preliminary results indicate that only one ductile end diaphragm panel is required, regardless of the number of girders.

This approach can be used on girder bridges without a bottom flange lateral bracing system, since the presence of a bottom flange lateral bracing system provides an alternative load path to bypass the ductile end diaphragm. It may be possible to adopt a similar approach for bottom flange lateral bracing, and have them work in parallel with the ductile end diaphragms, however. The ductile end diaphragm approach is only effective for transverse loadings; longitudinal forces and movements must be dealt with separately.

2.7 Proposed Values of Response Modification Factors for Steel Bridges

Response modification factors (R-factors) are commonly used in the seismic design of common short and medium span bridges. R-factors are used to reduce the forces calculated assuming an elastic response to obtain the design forces. R is defined as the ratio of the maximum deflection due to the earthquake loading assuming a completely elastic response to the yield deflection or:

$$R = \frac{F_{el}}{F_{inel}} = \frac{\Delta_e}{\Delta_y} \tag{2-7}$$

where:
R = Response modification factor (or R-factor)
F_{el} = Force corresponding to a completely elastic response
F_{inel} = Inelastic response force
Δ_e = Displacement corresponding to a completely elastic response
Δ_y = Yield displacement

Based on the equivalent energy method, the response modification factor can be related to the ductility factor and the period of vibration of the single degree of freedom system (Astaneh-Asl, 1996):

$$R = 1.0 \qquad \text{for } 0 \le T \le 0.1 \text{ sec.} \tag{2-8}$$

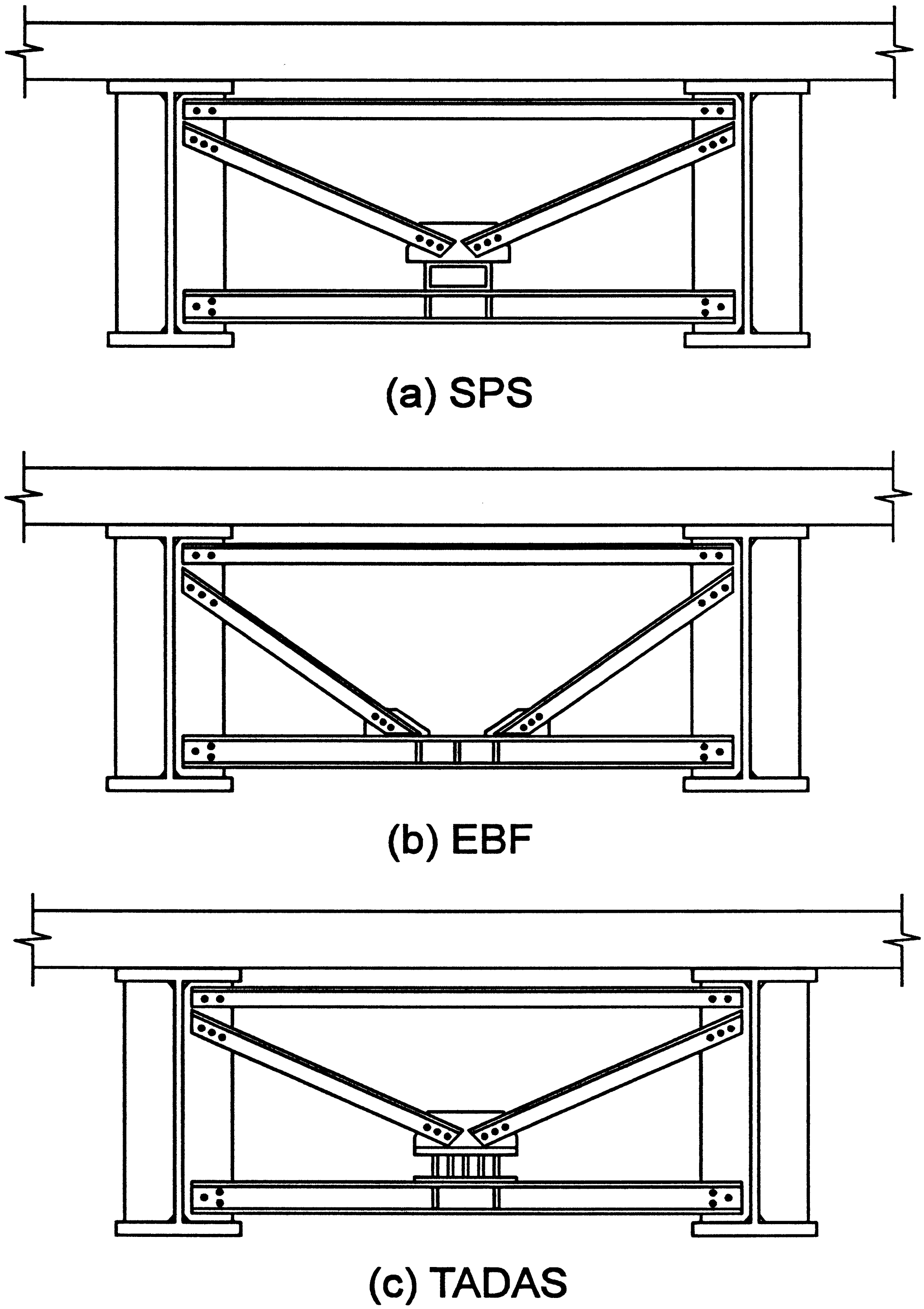


FIGURE 2-14 Ductile end diaphragms

$$R = 1.0 + (\mu - 1.0)\left(\frac{T - 1.0}{T_o - 1.0}\right) \qquad \text{for } 0.1 \le T \le T_o \tag{2-9}$$

$$R = \mu = \frac{\Delta_u}{\Delta_y} \qquad \text{for } T_o \le T \tag{2-10}$$

where:
T = Period of single degree of freedom system.
T_o = Parameter based on the soil type at the site.

The parameter T_o in the above equation is given by:

$$\begin{aligned} &\text{For soil type } S_1,\ T_o = 0.3 \\ &\text{For soil type } S_2,\ T_o = 0.4 \\ &\text{For soil type } S_3,\ T_o = 0.6 \\ &\text{For soil type } S_4,\ T_o = 1.1 \end{aligned} \tag{2-11}$$

The definitions of the four soil types can be found in the AASHTO-LRFD Bridge Design Specification (AASHTO, 1994). Steel bridges, in most cases, will have a fundamental period of vibration greater than T_o seconds, which justifies the use of μ in determining the R-factors (Astaneh-Asl, 1996).

R-factors for concrete bridges are well established based on extensive research and the limited locations of plastic behavior, usually the top and bottom of pier columns. Due to the limited amount of research on cyclic behavior of steel bridges, many steel bridge components use the same R-factors as their concrete counterparts. Because of differences in the structural systems and behavior of concrete and steel, their ductility and R-factors should be different. For example, in addition to the top and bottom of the pier columns, steel bridges can accommodate yielding in the bearings and cross-bracings of both the substructure (if steel) and superstructure. Based on the behavior of steel bridges in recent earthquakes, the available research data and analysis results, and engineering judgement, the R-factors listed in table 2-3 have been proposed (Astaneh-Asl, 1996). These values represent a good starting point, but additional research should be performed to verify some of these values before including them in design specifications. Subsequent research into the behavior of steel bridges may result in different proposals.

TABLE 2-3 Proposed Response Modification Factors for steel bridges (Astaneh-Asl, 1996)

Structure	Importance Category		
	Critical	Essential	Other
Steel girders on wall-type piers:			
• with skew angle less than 15 degrees	3	4	5
• with skew angle more than 15 degrees	2	3	4
Steel girders on reinforced concrete pile bents with:			
• vertical piles only	2	3	4
• batter piles	1	1.2	1.5
Steel girders on steel single column tee bents and steel or composite piles	3	4	5
Steel multiple-column bents on steel or composite piles with:			
• vertical piles only	4	5	6
• batter piles	1.5	2	3
Steel truss superstructures			
• in longitudinal direction	1	2	4
• in transverse direction	3	4	6
Steel girders on base isolation bearings and:			
• on steel bents	2.5*	3.5*	5*
• on reinforced concrete bents	2*	3*	4*
Steel girders with energy dissipating end cross-bracings and:			
• on steel bents	4	5	6
• on reinforced concrete bents	3	4	5

* Tentatively proposed

Connections	Importance Category		
	Critical	Essential	Other
Base plate anchor bolt assembly:			
• in single column bents	2	3	4
• in multiple column bents	3	4	5
Bearing assemblies with:			
• roller bearings in transverse direction	1	2	3
• pin bearings in transverse and longitudinal directions	1	1.2	1.5

SECTION 3
POST-EARTHQUAKE REPAIRS

3.1 Repair of Earthquake Damaged Members

Current earthquake design philosophy acknowledges that bridge damage is likely to occur as a result of earthquakes, and in many cases, relies on the damage to limit forces on the remaining structure. In the case of small earthquakes which result in minimal damage, structures should remain serviceable, and repairs should be able to be completed within a reasonable amount of time. When subjected to large earthquakes, structures are required to survive without collapse, and damage should be easily repaired where possible (AASHTO, 1996).

Steel bridges provide the designer with the advantage of being able to limit damage to one or more locations, with multiple areas available to yield and dissipate energy. This allows the designer to not only select the yield location(s) which are structurally optimal, but also to choose locations and details which result in easier damage repair. Sacrificial elements which are intended to fail can act as "fuses" which limit inertial forces generated in large earthquakes and protect other more important structural elements. Steel elements also provide the advantage of easy member replacement or reinforcement. Bolted members can be easily removed and replaced, and steel reinforcing can be easily bolted or welded on. Welded members are tougher to replace, but it is still possible to cut out the damaged portion and weld in a replacement.

While there are many factors which affect the economy of steel bridge repair, some general conclusions can be drawn. Damage with the following attributes is preferable:

- Secondary (non-gravity load bearing) members damaged.
- Repairs can be performed under traffic.
- Damaged members are easily accessible.
- Minimize the amount of material to be repaired/replaced.
- Minimize the amount of field work to be performed.

Damage which does not result in bridge closure is always beneficial. This is why limiting damage to the lateral load carrying system may be desirable. Being secondary members not required for gravity loads, these members would not need to be repaired immediately, and gravity loads would not require auxiliary supports while these members are repaired or removed and replaced. Life-cycle economics of a given system are difficult enough to quantify without adding the additional uncertainty of major earthquake occurrence, but initial cost of an earthquake protective design must be balanced against the cost of repair at some future date. In areas of low seismicity this is especially important, as a future high cost repair may be a worthwhile gamble if the initial savings is great enough, as the design earthquake may not even occur during the life of the structure.

Damage to conventional frame systems can be repaired, but certain configurations are more difficult to repair than others. In MRFs, depending on the location of the plastic hinge, either the beam or the column will require repair. Concentric X-bracing can be replaced, although the connections at the ends might also be damaged if braces are permitted to buckle, making repairs more difficult. EBFs suffer from a problem similar to MRFs, in that the active link is an integral part of the brace beam, and the entire beam most likely requires replacing. If the EBF is configured such that the active link is isolated (much like the shear panel system), then repair becomes much easier.

Any properly functioning structural "fuse" systems, including those listed in section 2.5, should result in a relatively easy repair. After the earthquake, all damaged structural fuses can be easily removed and replaced.

3.2 Details to Replace Buckled Plates or Shapes

The ease of repair or replacement of a buckled plate or shape depends mainly on the gravity dead load forces present in the members. Secondary members which carry no gravity loads, such as lateral bracing are easily removed and replaced, without the need for extensive temporary bracing or shoring. Primary load bearing members, on the other hand, present much more of a challenge, due to the necessity of providing an alternative path for the loads while repair or replacement is performed.

The degree of damage, and the post-buckling behavior of the member, also are factors in the repair scheme chosen. The do-nothing option may be the best choice, especially for secondary members, if the strength of the post-buckled member is still adequate, and the aesthetics of the bridge are not adversely affected.

Repair strategies which do not require member replacement are more attractive. One such approach is to use heat and/or mechanical means to straighten bent plates. These methods may require that the member have some or all of the load in it removed at the time of repair, however. Specialized training is also called for in performing heat or mechanical straightening, especially with today's modern steels, which can be degraded with improper application of heat. Once members are straightened, additional strengthening can be easily performed, however.

Another repair strategy is to use concrete infill and/or concrete encasement with or without steel shells to stiffen and/or strengthen areas which have buckled. This technique may be particularly attractive in cases where plastic hinges and local buckles have developed at the base of steel columns as shown in figure 3-1(a). This would not only repair the damaged locations, but would also provide a new location further up the column for subsequent hinge formation. Alternatively, steel stiffeners or reinforcing plates can be cut or bent into the buckled shape, then bolted or welded in place to strengthen a buckled area.

If the member to be repaired/replaced is primarily a tension member, relieving the load can often be achieved fairly easily using cables or rods. Much more difficult is the removal of load from a main compression element, since buckling of the alternative load path becomes the main concern. With the analytical tools available to designers, the probable locations of plastic hinge formation can be identified, and details can be added to make repair/replacement much easier.

Jacking of columns in order to repair/replace plastic hinge zones at the base can be performed if columns are detailed as illustrated in figure 3-1(b). Sufficient bearing area should be supplied on piers and abutments to permit jacking of the column. A splice location should also be provided well outside the plastic hinge zone for both the members used to jack the original column, and the replacement member to be spliced on. This concept with a similar detail can be used at other locations where only the plastic hinge regions at the ends of members are to be replaced.

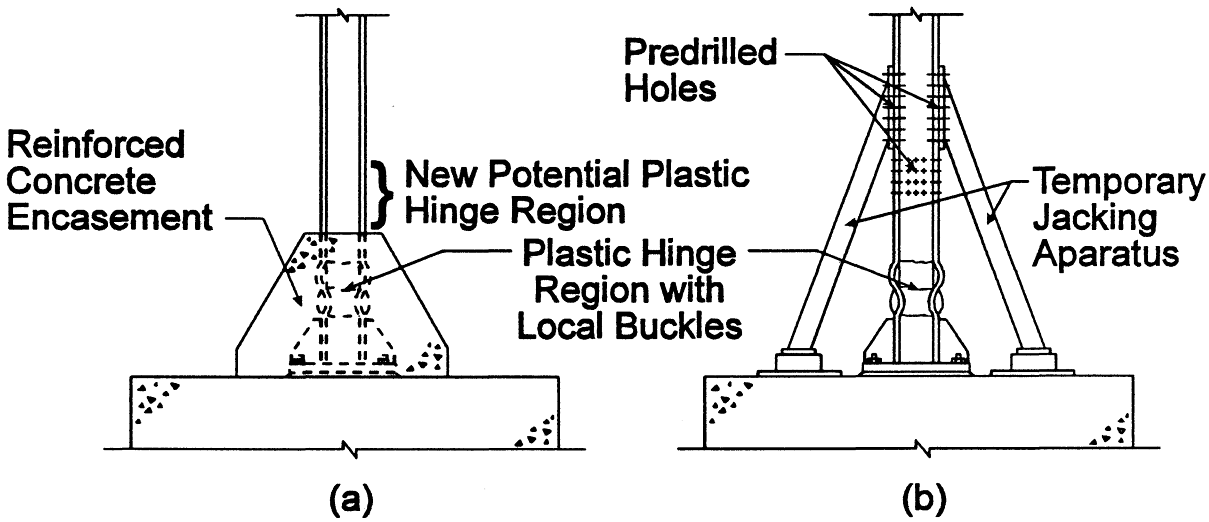


FIGURE 3-1 Potential column base plastic hinge repair/replacement strategies

SECTION 4
STEEL-CONCRETE INTERFACE

4.1 Anchor Bolts

One of the most common devices used to connect steel structures to concrete structures is the threaded steel anchor bolt. Anchor bolts are either cast-in-place, or adhesively bonded into drilled holes after the concrete has cured. Both of these methods of construction have provided satisfactory performance. For the best ductility in the anchor bolt itself, A307 anchor bolts should be used, although higher strengths are acceptable, especially when protected by yielding elsewhere in the structure.

Anchor bolts can develop their full tensile strength if sufficient embedment and/or mechanical anchorage is provided. Full shear strength can be developed if sufficient embedment and edge distance or reinforcement is provided. Many empirical equations are available to assure that adequate anchorage strength is provided in plain concrete, such that ductile anchor bolt failures will occur, and brittle concrete failures can be avoided (ACI 349, 1978; Cannon, 1981; Klingner, 1982; DeWolf, 1990). All of the equations may give reasonable results, but the equations in ACI 349 are one of several that have a rational basis in observed experimental conical failure modes, and will be presented here.

The pullout strength U_p of concrete for anchor bolts in tension is calculated based on a uniform stress given by:

$$4\varphi\sqrt{f_c'} \tag{4-1}$$

where:
φ = resistance factor for an embedded anchor
f_c' = specified compressive strength of concrete, psi

This stress acts on an effective area defined by the projected area of stress cones radiating toward the attachment from the bearing edge of the anchors. The effective area is limited by overlapping stress cones, by the intersection of the cones with concrete surfaces, by the bearing area of the anchor heads, and the overall thickness of the concrete, as illustrated in figure 4-1 (ACI 349, 1978). The inclination angle for calculating projected areas is taken as 45 degrees. As bolts approach the edge of the embedment concrete, the effective stress area decreases. Therefore, minimum edge distances, m, for anchor bolts under tension only and under tension and shear are limited to, respectively:

$$m = D\sqrt{\frac{f_{ut}}{56\sqrt{f_c'}}} \tag{4-2}$$

$$m = D\sqrt{\frac{f_{ut}}{7.5\sqrt{f_c'}}} \tag{4-3}$$

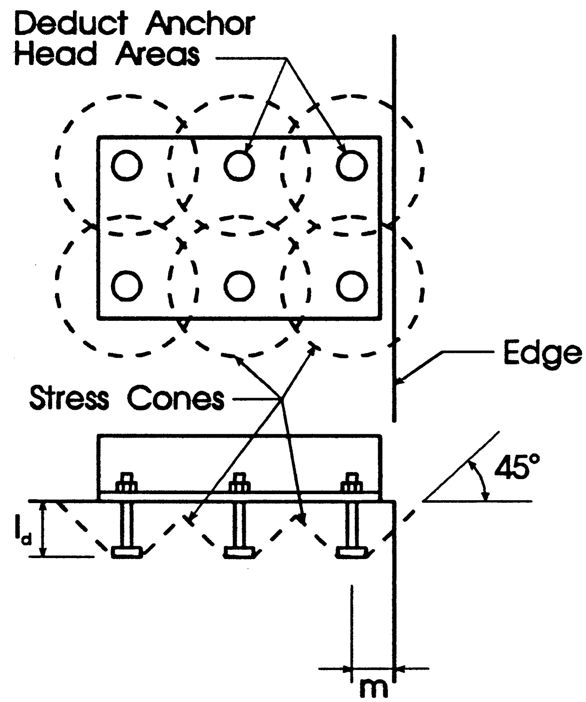


FIGURE 4-1 Failure cone for embedded anchor bolts (adapted from ACI 349, 1978)

where:
D = nominal anchor bolt diameter
f_{ut} = minimum specified tensile strength of anchor steel, psi

These equations have been found to provide a lower bound solution for single anchor bolts, but may be unconservative for multiple anchor bolt assemblages, due to unequal load sharing. Tests have demonstrated that pairs of anchor bolts may only exceed the strength of a single bolt by 40-60% (Ueda, 1990).

To calculate the strength of steel anchor bolts under combined shear and tension, the use of a straight line interaction has been proposed (Scacco, 1992). While this would be conservative for the anchor bolt, using the elliptical interaction equations developed for high strength bolts (Fisher, 1974; AISC, 1989), or AISC trilinear approximations of the elliptical interaction equations for bolts (AISC, 1994), would be better for designing the anchorage. The three types of curves are shown in figure 4-2.

When reinforcement is used in the embedment concrete, it should allow the anchor bolt to develop its full shear strength. 180° hairpins around the anchor bolt at the base and as close to the concrete surface as practical (maximize the moment arm) were found to be effective at resisting cyclic loads when bolts were closer than the critical distance to the concrete edge (Klingner, 1982). Confinement reinforcing for anchor bolts similar to concrete column reinforcing could also be effective, if embedment is sufficient.

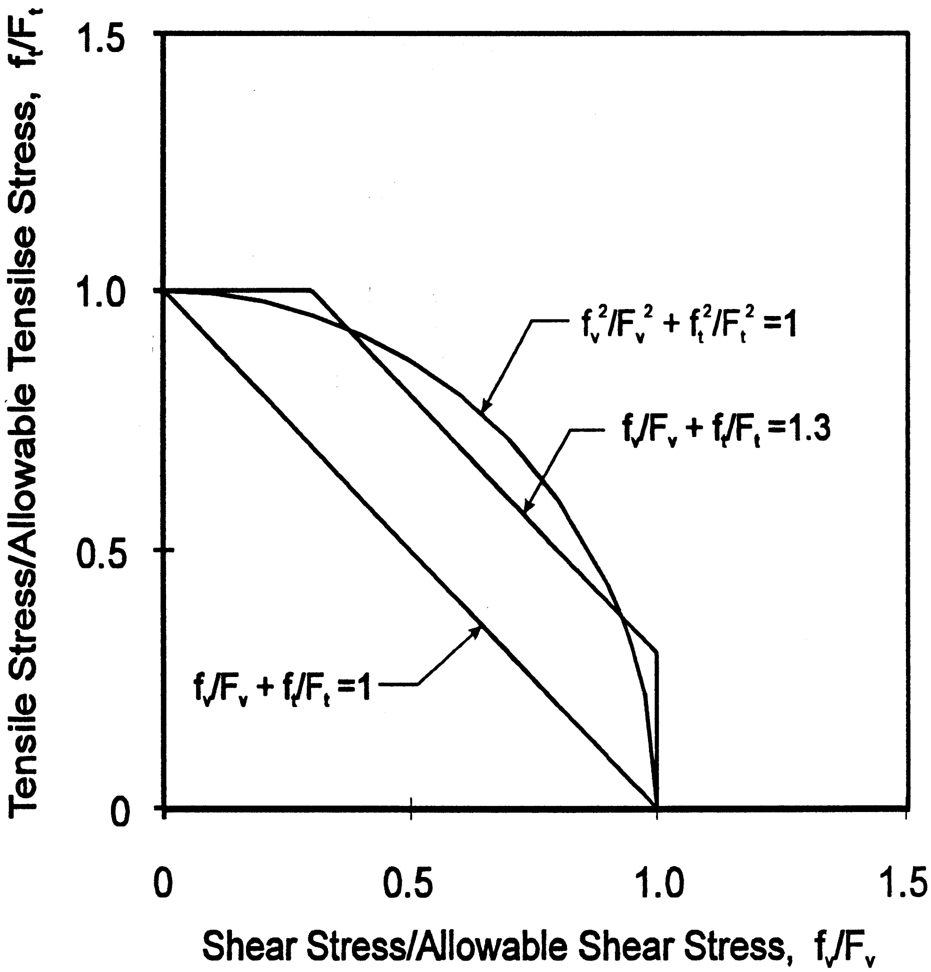


FIGURE 4-2 Tension-shear interaction curves for bolts

In the Northridge earthquake, some bridges suffered brittle fracture of anchor bolts through the threads (Astaneh-Asl, 1994). To avoid this type of failure, the threads should be excluded from the failure plane. One method which accomplishes this is to use upset anchor bolts (see figure 4-3), where the shank is machined to about 3/4 the diameter of the threaded portion, which ensures yield and elongation occurs in the shank instead of fracture through the threads (Astaneh-Asl, 1994). Another possible solution is to fill the anchor bolt sleeves with a more flexible material or leave them unfilled, allowing some slight bending of the resulting long anchor bolts, permitting some lateral displacement (Astaneh-Asl, 1994). Sleeved anchor bolts have been used in the past, in order to equalize loads and provide ductility.

For seismic response, anchor bolts may be sized to deform inelastically and act as a structural "fuse," or they may be sized to perform elastically and inelastic behavior may be directed elsewhere in the structure. Allowing plastic deformation in anchor bolts may lead to a rocking of the structure, which can produce a beneficial elongation of the period, and lead to a reduction in forces generated (Astaneh-Asl, 1993a; Midorikawa, 1993). Alternatively, an innovative design featuring sleeves around the anchor bolts as illustrated in figure 4-4, prevents compressive buckling of the anchor bolts,

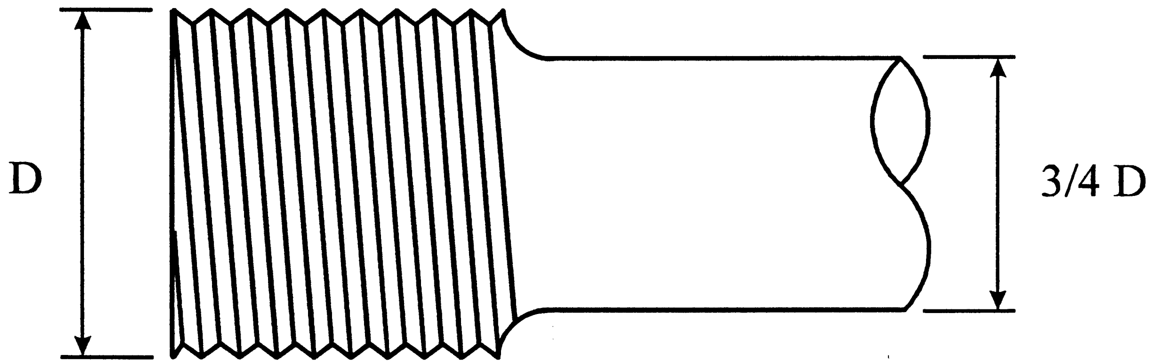


FIGURE 4-3 Bolt with upset threads

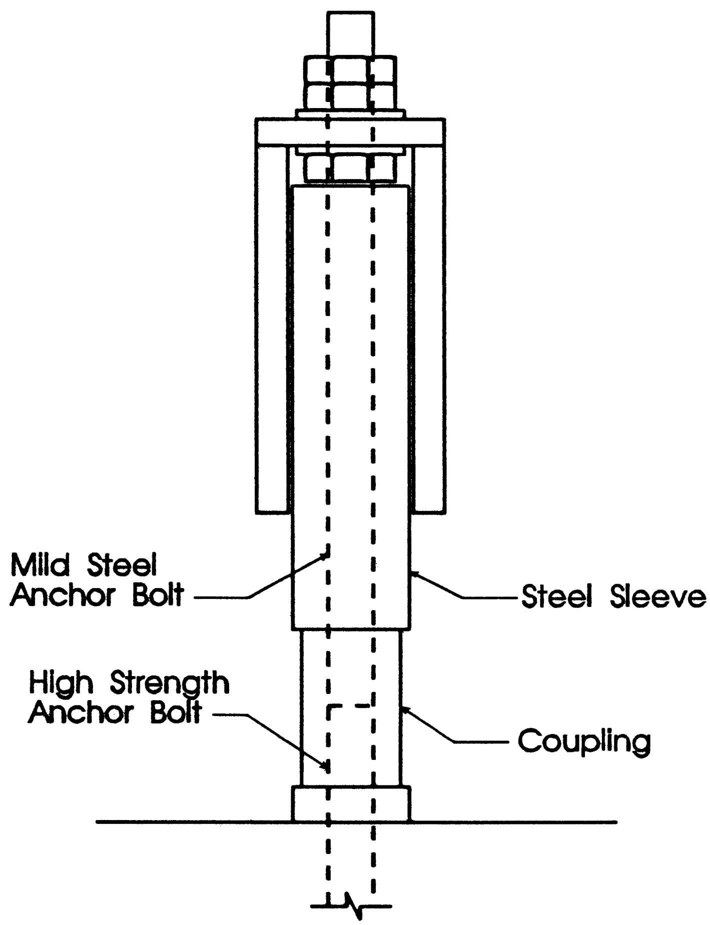


FIGURE 4-4 Anchor bolt with sleeve to permit compression yielding (adapted from Prucz, 1996)

and permits compressive yielding (Prucz, 1996). The addition of a threaded coupler to the anchor bolt assembly allows for easy replacement following a seismic event.

Another innovative scheme uses a device called a ring spring to provide isolation and energy dissipation at anchor bolt locations. Ring springs consist of stacks of concentric inner and outer rings with interactive taper surfaces which slide across each other (Erasmus, 1988). Under axial forces, the wedge action of the taper expands the outer rings and contracts the inner rings, allowing axial deflection. It is also suggested that ring springs would be effective in series with bracing or as horizontal thrusters.

4.2 Steel-Concrete Moment Connections

Anchor bolts can be used to provide a moment connection between steel and concrete. While a simple base plate welded to the bottom of the column can be designed to carry considerable moment, additional stiffeners are usually provided in bridges such as illustrated in figure 4-5 in order to transmit high loads directly to the column flange and avoid using a very thick base plate (Ricker, 1989, DeWolf, 1990). Six different failure modes have been identified for columns with steel base plates attached to concrete footings using anchor bolts, as illustrated in figure 4-6 (Astaneh-Asl, 1993b). The preferred modes are plastic hinge formation at the base of the column, or yielding of the base plate in bending, as they result in stable hysteresis. Tensile elongation of the anchor bolts, as mentioned previously, is acceptable as it does not result in catastrophic failure, but repair is difficult unless it is specifically designed for, with couplings provided to allow removal and replacement of the bolts. Undesirable brittle failure modes include pull-out of the anchor bolts, compression crushing of the grout under the base plate, or fracture of the column to base plate welds.

Moment resistance can be increased by either increasing the size or number of anchor bolts, or increasing the spacing (moment arm) between them. Axial resistance to uplift can only be increased

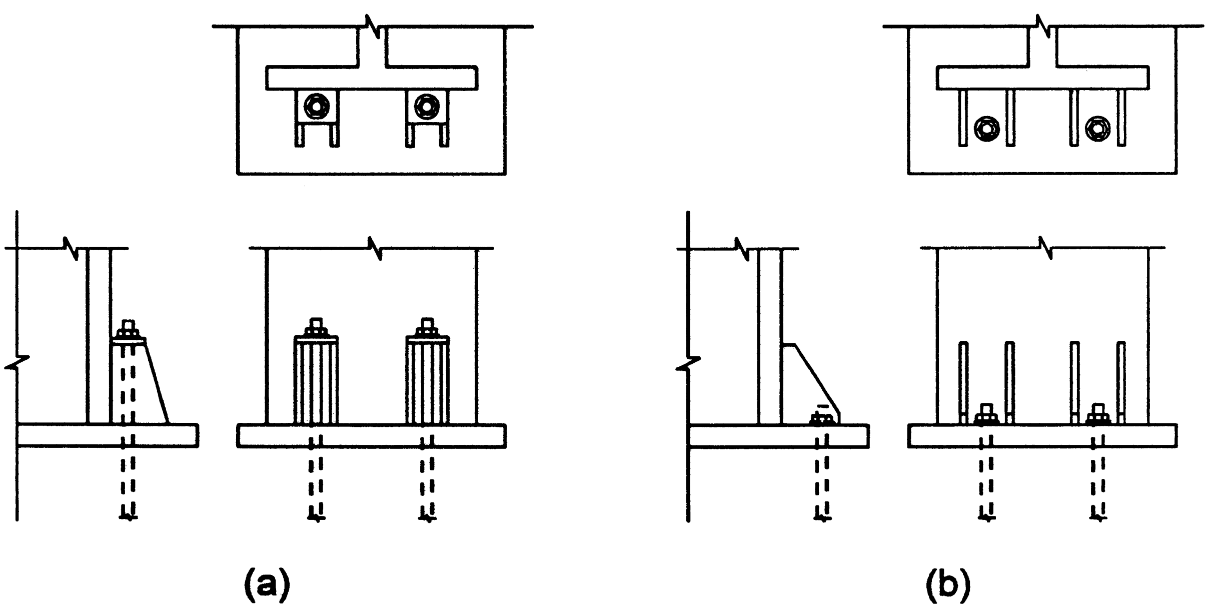


FIGURE 4-5 Two examples of base plates with stiffeners to aid in load transfer

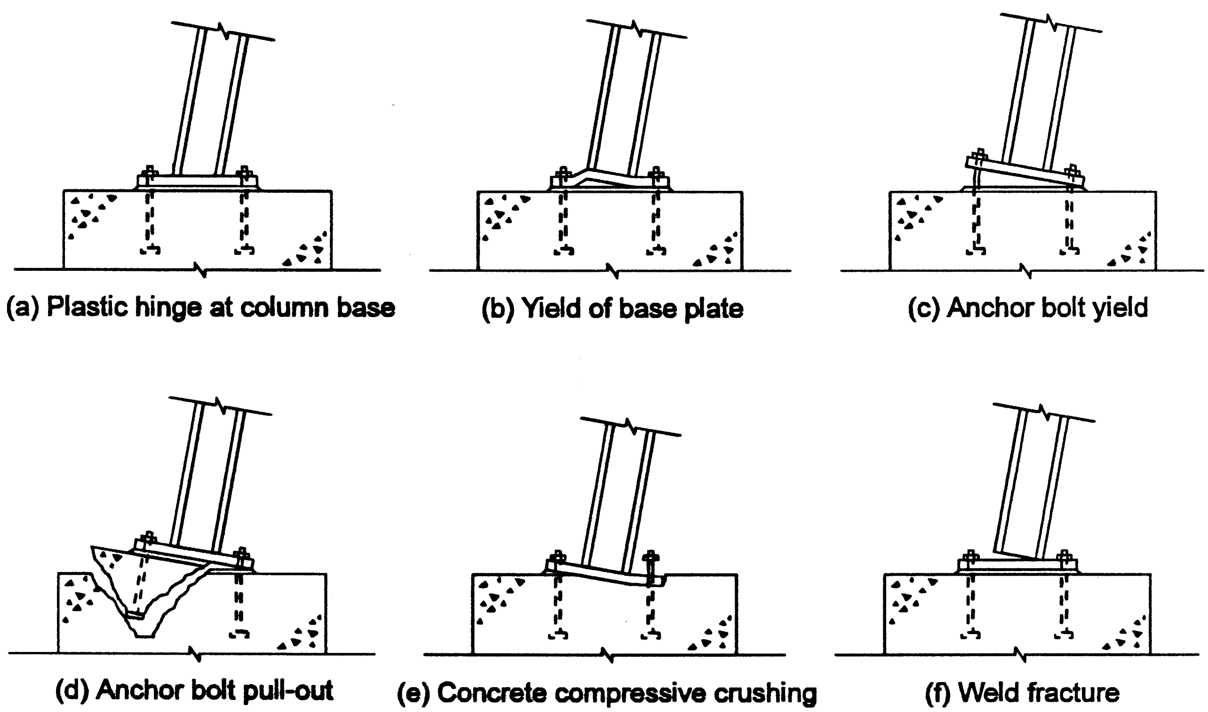


FIGURE 4-6 Various failure modes of columns with steel base plates attached to concrete with anchor bolts

by increasing the size or number of anchor bolts. Shear keys can be provided at the steel-concrete interface to increase the shear resistance in either direction.

A second technique of connecting a steel member to a concrete member is to embed the member sufficiently to transmit the loads (see figure 4-7). According to Morita, 1989, when the concrete embedment depth, d, and front cover, t, exceed the empirical critical values:

$$d_c = 0.6 \sqrt{\frac{B_c * {}_c t_f * \sigma_y}{\tau_c}} \tag{4-4}$$

$$t_c = 0.75 \sqrt{\frac{B_c * H_c * {}_c t_f * \sigma_y}{d * \tau_c}} - 0.5 B_c \tag{4-5}$$

where:
B_c = width of column section
Hc = depth of column section
${}_c t_f$ = flange thickness of column section
σ_y = specified minimum yield strength of steel
d = embedded depth of column
τ_c = cone type shear strength of footing concrete

punching shear failure of the footing concrete is avoided under seismic loading, and ductile yielding

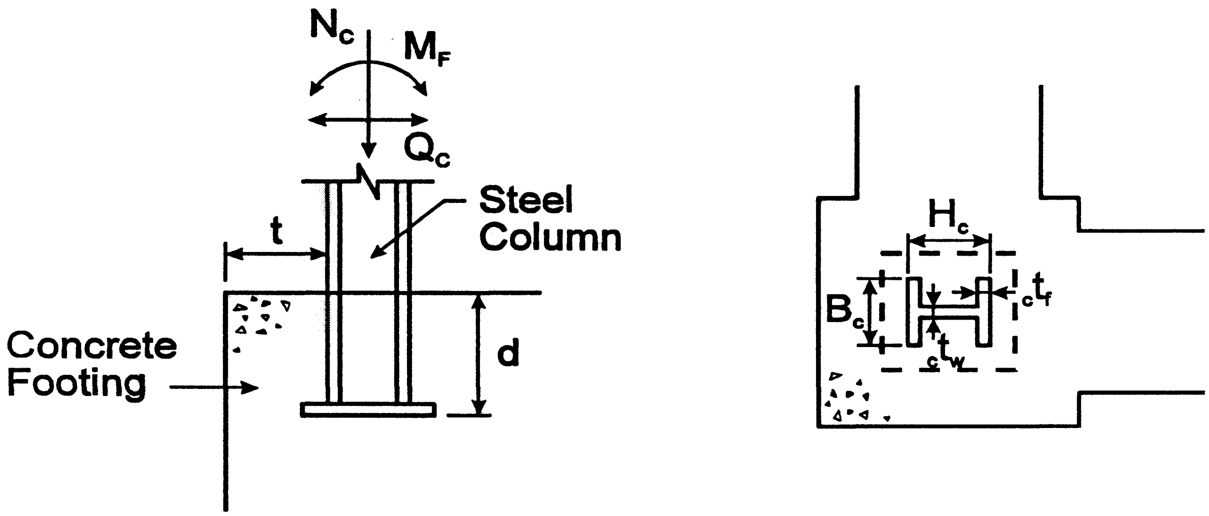


FIGURE 4-7 Embedded steel column (adapted from Morita, 1989)

of the steel column is assured (Morita, 1989). Mechanical anchorage can also be provided and the concrete can be reinforced if insufficient concrete cover is available to meet these criteria.

4.3 Integral Construction

Integral construction of steel girder bridges with concrete substructures has recently received attention for both seismic and non-seismic advantages. Integral abutments are being increasingly utilized in order to eliminate expansion joints at the ends of bridges. More recently, integral piers have been advocated as not only better for seismic resistance, but also for additional economic benefits.

The elimination of steel bearings as well as making pier caps integral results in savings in cost in both seismic and non-seismic situations, while having the added benefit of eliminating a potential "weak link" between girders and substructure in seismic designs. Reducing the mass at the top of the pier by elimination of the typical hammerhead pier cap is of some advantage, but the main benefit comes from the increased fixity of the top of the pier columns in the longitudinal direction. By changing the boundary condition of the top of the columns from a pinned or a free condition to a fixed condition, the column dimensions can be reduced while still maintaining stability. Although limited information on integral construction, especially integral piers, is currently available in the literature, National Cooperative Highway Research Project (NCHRP) 1254 is currently underway to investigate the potential of integral construction.

SECTION 5
CONCLUSIONS

Steel is an ideal material for earthquake resistance. Its high strength to mass ratio and good ductility have resulted in superior seismic performance for many years. Good practice in design and detailing will result in a structure that responds very well even under extreme earthquake loads. Advances in design and analysis have resulted in techniques which permit the designer to control the response of a bridge structure, protecting potential brittle elements, and directing the forces and displacements to locations that can accommodate them.

Local buckling continues to be a problem. Recent research indicates that achieving stable seismic cyclic hinging may require more stringent width-to-thickness ratios than currently specified for plastic design. Additional research in this area is warranted.

Connections and splices remain the most seismically susceptible locations in bridges. Reinforcing connections such that plastic hinges form in the gross section of members and not in the connection elements is one effective way of approaching this problem. Another approach is to weaken the member away from the connection, ensuring that the plastic hinge forms at the weakened location. Any of the lateral force resisting frames commonly in use, moment resisting frames (MRFs), concentrically braced frames (CBFs), or eccentrically braced frames (EBFs), can be effective if the connections are detailed correctly and designed for the post-yield/post-buckling forces. A relative newcomer to the building frame scene, there is no reason that the EBF concept could not be effective for bridge towers.

Another effective design approach is to supply the structure with some type of structural "fuses" which provide some form of isolation, damping, and/or energy dissipation, controlling the response, and limiting the forces to which the remainder of the bridge is subjected, thus protecting the members of the structure. These fuses can take the form of friction devices, viscous or viscoelastic devices, or material yield devices. These can be very effective when placed in series or parallel with the lateral bracing systems of bridge towers or superstructures. Material yield devices, such as the added damping and stiffness (ADAS) device, are most attractive for bridges, since environmental exposure is not a problem, and response is predictable over time.

Member repair and replacement schemes should be designed into steel bridge structures. With the analysis tools available, designers should be able to design the structure such that damage occurs in preselected locations. The best locations for damage are secondary (non-gravity load bearing) members, since these are more easily removed and replaced, without the need for elaborate temporary supports. When it is expected that main compression members will suffer damage, providing connection details and jacking locations for temporary support members during design is the best solution.

Embedded anchor bolts continue to be the best method of providing a moment connection between steel towers and concrete substructures. Along with steel base plates and stiffeners to transfer the forces into the steel member, anchor bolts should be designed such that a ductile yield failure occurs in the main member, the base plate, or the anchor bolts, and not a brittle concrete fracture. Supplementing the shear resistance of the anchor bolts by providing shear keys may be required under the high lateral seismic forces. Upset threads or sleeved anchor bolts have been proposed as techniques to avoid brittle fracture through anchor bolt threads. Embedding a steel member in concrete can also provide a moment connection, but this appears to be more promising as a repair technique, when the base of the column has suffered local buckling due to formation of a plastic hinge. Integral construction for girder bridges shows great promise as a technique for not only increasing seismic resistance, but providing additional economic benefits as well.

Steel bridges are not only materially different from concrete bridges, but geometrically different also. Steel bridges provide many different locations where inelastic action can be permitted, while in concrete bridges hinging is generally limited to bases and tops of columns. These additional locations in steel bridges can be used to great advantage in constructing more economical, easily repairable, seismically resistant steel bridges. It is important that steel bridge structures are not overlooked when designing highway structures for seismic resistance. Steel members should receive the same attention paid to concrete components when detailing for ductile behavior.

SECTION 6
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