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Critical Seismic Issues for Existing Steel Bridges

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Technical Report MCEER-99-0013

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NOTICE

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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 University at Buffalo, State University of New York, 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 systems and 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 existing highway structures project, and was performed within Task 106-E-5.5, "Critical Seismic Issues for Steel Bridges" of that project as shown in the flowchart on the following page.

This task was a preliminary study conducted to identify critical issues in the seismic design and retrofit of steel bridges. The report provides an assessment of the performance of existing steel bridges in past earthquakes, including the 1989 Loma Prieta, 1994 Northridge and 1995 Kobe events. The components that were studied include steel columns, joints and connections, tower *bents and superstructure details. Vulnerable areas and details are identified, and recommendations for improved designs and retrofit techniques are made. Research needs are identified where existing knowledge is incomplete or lacking.*

ABSTRACT

This report assesses the performance of existing steel bridges in past earthquakes. Components studied include steel columns, steel joints and connections, steel tower bents, and steel superstructures. Vulnerable areas and details are identified, and recommendations for improved designs and retrofit techniques are made. Research needs are identified where existing knowledge is lacking.

While steel bridges performed very well from an ultimate safety standpoint, with few collapses in the recent Kobe and Northridge earthquakes, damage suffered by steel structures indicated that some steel bridge details are potentially vulnerable, and improved performance is possible. Tubular steel piers suffered more damage in Kobe than was anticipated, with widespread local and global buckling problems, generally occurring in the columns at section of abrupt changes in stiffness. Among the retrofit recommendations, one of the most promising is partially filling the hollow tubular piers with concrete, and installing a steel diaphragm directly above the concrete.

Connections continue to be a vulnerable area. Generally, it is desired that connections should be stronger than the connected members, such that inelastic behavior is confined to the gross section of members. In response to welded moment connection problems in Northridge, numerous concepts shifting inelastic behavior away from the column face were proposed including using welded cover plates/stiffeners, bolted stiffeners, and weakening the beam adjacent to the column (i.e., dogbone).

Many older bridges exist which employ riveted construction, often containing laced or latticed members. Replacing rivets with high strength bolts, and adding additional connection plates if necessary, are a widely accepted method of strengthening riveted connections, although some ductility may be sacrificed. Replacement of lacing bars with perforated plates is the current accepted retrofit technique for laced or latticed members found to be seismically vulnerable due to post-buckling strength degradation.

Global and local stability of steel compression members are the remaining major vulnerabilities. Global buckling is especially a problem in non-redundant structures or when fast strength degradation occurs. Both better post-buckling behavior and plastic hinge behavior are obtained when local buckling is postponed in steel members. Research into the use of lower width-to-thickness requirements in areas expected to experience inelastic behavior is warranted.

An examination of the three most common lateral force resisting systems, moment resisting frames (MRFs), concentrically braced frames (CBFs), and eccentrically braced frames (EBFs), indicates that properly proportioned and detailed, any of them can provide adequate seismic structural ductility. Despite several identified challenges, there appears to be no reason EBFs would not be an effective lateral resistance system for bridges. Existing X-braced and V-braced bridge piers are also analyzed to assess potential structural ductility.

Concrete slab-on-steel girder bridges have performed very well in earthquakes, with damage generally confined to the substructure-superstructure interfaces. Using the bearing cross-frames or diaphragms as locations of structural "fuses" to limit forces transferred to the substructure is a concept that has recently received attention. Research indicates that providing only one "ductile end diaphragm" at bearings is generally sufficient.

An alternative to ensuring that the structural members themselves possess sufficient ductility is to furnish "devices" that provide a combination of isolation, damping, and/or energy dissipation. Properly designed and installed in series or parallel with existing structural members, these devices can serve as structural "fuses," controlling the structural response, limiting forces, and protecting potentially vulnerable members. These devices fall generally into one of three categories, friction devices, viscous or viscoelastic devices, or material yield devices. A variety of devices are available, some proprietary, and several examples are presented.

The high strength to weight ratio and material ductility make steel an excellent material for earthquake resistance. Moreover, unlike concrete, where plastic hinging is limited to tops and bottoms of pier columns, the geometries of steel bridge structures are such that great flexibility is provided to the designer in how and where to retrofit in order to supply the isolation, damping and/or energy dissipation required to resist the maximum earthquake loading. As effective as steel can be, it is not foolproof, and steel bridges should not be overlooked when assessing seismic vulnerability.

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont'd)

LIST OF ILLUSTRATIONS

LIST OF ILLUSTRATIONS (Cont'd)

LIST OF TABLES

SECTION 1
INTRODUCTION

In the spring 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 the existing 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 vulnerability of particular highway system components and structural elements, and performed by researchers with expertise in that area of study.

Due to good performance in past earthquakes, and general faith in the material ductility of steel, steel bridges have previously been assumed to be relatively safe, and seismic research has concentrated on concrete. 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 Loma Prieta earthquake, the fractures of bolted web-welded flange steel moment connections in buildings in the Northridge earthquake, and finally the poor performance of tubular steel bridge piers in the Kobe earthquake, has demonstrated that existing steel structures can be vulnerable to earthquakes.

This does not mean we should avoid using steel in seismic zones because of these few problems. On the contrary, steel has properties that make it very attractive for use in seismic applications and it has several advantages over concrete when it comes to seismic resistance. Not only does steel possess an inherent material ductility, but steel also has a high strength-to-weight ratio. Steel superstructures weigh less than their concrete counterparts, resulting in lower inertial forces generated during earthquake motion. It may have been steel's apparent "foolproof" seismic behavior that lulled engineers into having unreasonable confidence in steel. While steel behavior is forgiving in many respects, good design and detailing practices are still required to achieve acceptable behavior. Furthermore, structures designed for adequate strength under typical lateral loads such as wind, centrifugal, or braking, may not possess the strength and/or structural ductility to withstand the cyclic deformation demands of earthquake loadings.

The main objective of this task is to identify the areas in existing steel bridge structures that are seismically vulnerable. Components to be examined include steel columns, steel tower bents, steel superstructures (particularly steel trusses), girder cross-frames, and steel connections. Where current knowledge is sufficient, preliminary recommendations will be made on methods to analyze and retrofit existing bridges in order to improve their seismic performance. Where knowledge is lacking, areas of required research will be identified.

Historically the philosophy in earthquake resistant design has focussed on life safety. Structures were designed with the goal of no collapse in the maximum credible earthquake expected to hit the site. In the Northridge and Kobe events, steel structures suffered significant damage, but most performed well from the life safety standpoint, with only a few collapses in Kobe. The cost of the damage experienced in these events was high, however, especially considering that Northridge was a moderate event. This led to a questioning of the "no collapse" philosophy, and whether this approach sufficiently limited damage in smaller earthquakes. In addition, the realization that emergency response could be hampered, and severe economic hardship could be suffered, due to the loss of service of certain critical structures, even if they did not collapse, prompted a re-evaluation of this philosophy.

Caltran's response has been to develop a two-tiered design strategy. 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 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 all bridges to be strong enough to survive all potential earthquakes undamaged, because the return period of the maximum intensity earthquake that a specific bridge site would be subjected to 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., that are regarded as areas of low seismicity. The other method 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 with which this report deals.

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

There are several "isolation" strategies for bridges, which can be used individually or in combination with one another. One method, base isolation, allows large movements at the bearings, shifting the period of the bridge, and reducing the dynamic forces resulting from the seismic input. Energy dissipation can also be provided. This can be a very effective strategy, and it is often used in conjunction with other retrofit schemes, because forces on the structure can be reduced to the point where selective strengthening of members becomes practical. Isolation 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.

Another approach, designing and retrofitting 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 material ductility of steel is well documented, making it an excellent 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 structural 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, as might be the case in the central and eastern U.S., it still is sensible to provide a ductile ultimate failure mode, because earthquake loadings are so unpredictable. A final problem with retrofits is that the most seismically vulnerable bridges are often older structures, and consequently, have historical significance. Architectural and aesthetic considerations can be overriding factors in the retrofit of these kinds of structures. Because the public does not want a "new looking" bridge, retrofit schemes that do not alter the appearance of a bridge are more attractive.

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 information exists on the seismic behavior of steel bridge components. A contributing factor to this situation is that the state which has been the leader in seismic resistant design, California, tends to construct bridges of monolithic concrete. The concrete philosophy is most reflected in the provision that limits plastic hinging regions to the top and bottom portions of pier columns. In concrete structures, this is the only feasible 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 (Astaneh-Asl, 1996a). There are also differences in the geometries 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 concrete and steel bridge towers. Among other differences, it is apparent that steel bridges employ secondary non-gravity load bearing elements, for bracing against lateral forces. These lateral bracing members are potential energy dissipation elements not present on concrete bridge structures (Astaneh-Asl, 1996a). Steel bridges can be designed to yield and provide energy dissipation at potentially many other locations, and in various modes, often in direct contrast to concrete philosophy, and should not be constrained by concrete design limitations. For instance, flexural failure modes are always desired in concrete, and brittle shear failure is avoided. In steel, often the shear yield mechanism is desired, because it

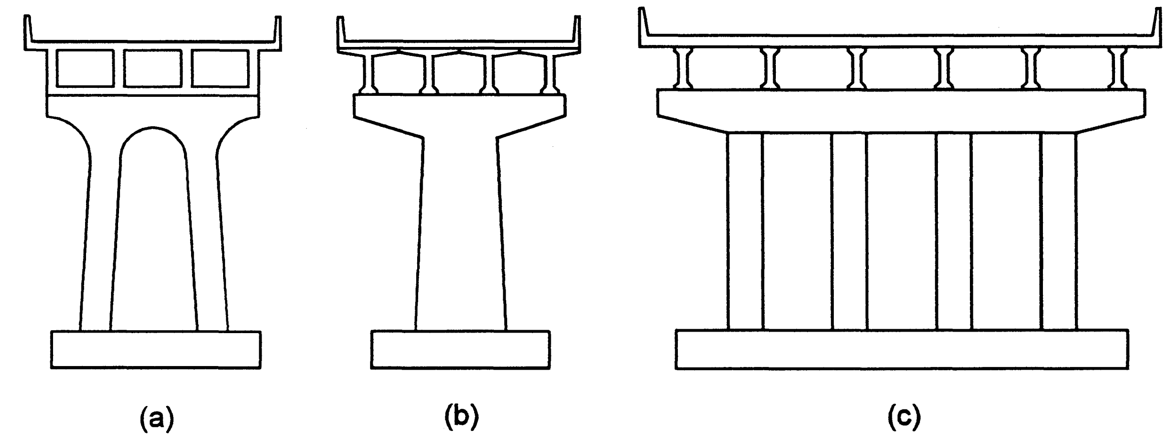


FIGURE 1-1 Several typical concrete sub- and superstructure configurations

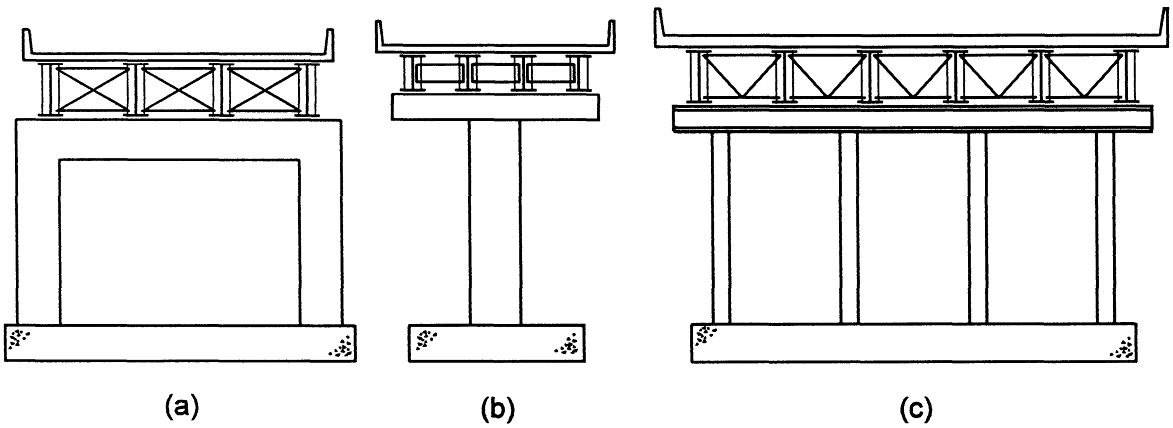


FIGURE 1-2 Several typical steel sub- and superstructure configurations

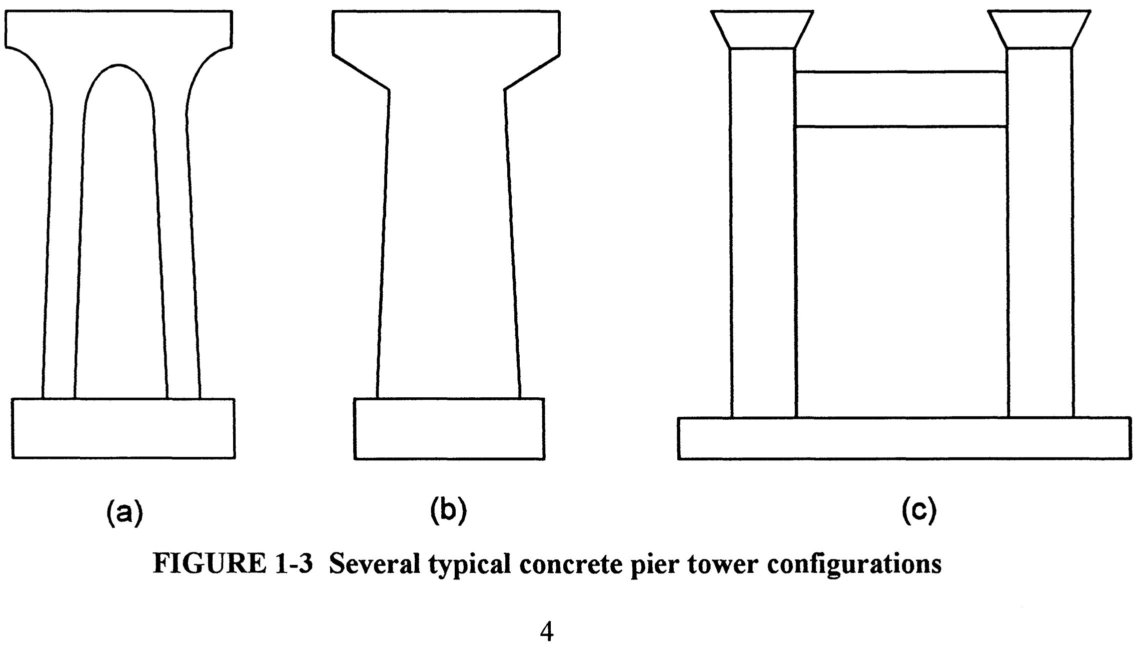


FIGURE 1-3 Several typical concrete pier tower configurations

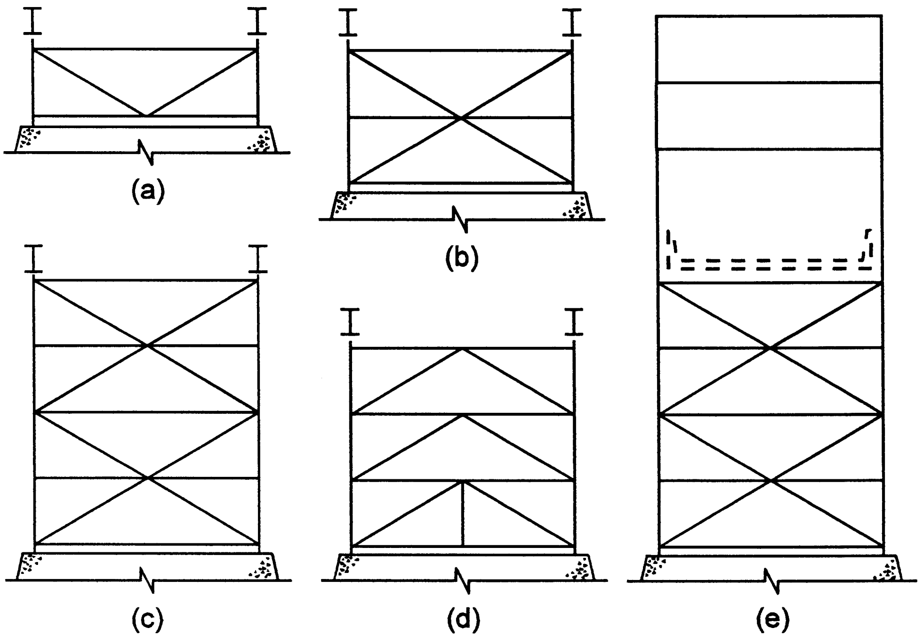


FIGURE 1-4 Several typical steel pier tower configurations

provides substantial stable energy dissipation. Steel bridge retrofit strategies need to be developed such that all potential effective approaches to seismic resistance of steel bridges are investigated, and the most appropriate method(s) selected.

In order to identify vulnerable areas of bridges, the behavior of steel bridges and buildings during recent earthquakes, as recorded in published reports, was surveyed. Research into cyclic behavior of existing steel bridge components was examined as well. In addition, the literature was also searched for research into alternative details and retrofits of problems encountered. Because earthquake research on steel buildings has been much more extensive than on steel bridges, the data on steel buildings was examined for results that could be applied to steel bridges.

SECTION 2
SEISMIC RESISTANCE OF STEEL STRUCTURES AND BRIDGES: THE STATE-OF-PRACTICE AND THE STATE-OF-KNOWLEDGE

2.1 Scope

In this section, the performance of steel bridges as well as other steel structures in recent earthquakes is documented. Current practice and research relating to steel structure retrofits are also discussed.

2.2 Steel Bridges in Recent Earthquakes

2.2.1 Loma Prieta Earthquake

The worst and most visible damage suffered by a steel bridge in the 1989 Loma Prieta earthquake was the dropping of the upper and lower decks of one span of the San Francisco-Oakland Bay Bridge. This failure occurred at an anchor pier which served to react the longitudinal forces from seven spans. The seats for the spans were only 5" wide. The seismically induced longitudinal inertial forces were large enough to fail the shoe bolts, allowing the span to slip off the seats.

2.2.2 Northridge Earthquake

Some limited data on the performance of steel bridges is available from the Northridge earthquake. Steel bridges generally performed very well, although in California steel bridges are relatively rare; most bridges are constructed of monolithic concrete.

One report on the seismic performance of steel bridges in the Northridge earthquake described four bridges that were moderately damaged (Astaneh-Asl, 1994). None of these bridges had damage to the main load carrying members. Steel diaphragms, bearings, connections of the steel superstructure to the concrete substructure, and the concrete substructure itself, were the portions that sustained damage. Consequently, most recommendations pertain to the steel-concrete interface (for more information, see section 4.4)

It was determined that cable restrainers in skew bridges performed better when oriented perpendicular to the pier axis, rather than parallel to the girder. Offset diaphragms were also found to behave poorly in one skew bridge, and diaphragms oriented parallel to the piers were found to perform better than diaphragms oriented perpendicular to the girders. The yielding of the bearing diaphragms on several of the bridges, along with minimal damage to the remainder of the structure, led to a suggestion that this might be a good location for a ductile "fuse," which could limit forces and displacements transferred to the bearings and substructure, and provide energy dissipation (see section 5.3). This indicates that current seismic design recommendations in ATC-32 which call for diaphragms to remain elastic and transmit all lateral forces to the bearings, and restricts plastic hinging to the pier columns (ATC-32, 1992), may need reconsideration. Although the ATC-32 recommendations allow that other strategies of energy dissipation and response control can be used, they must be documented by “testing or rigorous analysis”.

2.2.3 Kobe Earthquake

The ground motions experienced in the Kobe earthquake were stronger than any measured in Japan previously. Generally, steel bridge structures performed extremely well from the standpoint of resistance to collapse, although damage was far more severe than contemplated by current seismic design.

As in Northridge, steel bridge superstructures suffered little primary damage. Most damage to superstructures was caused by failures of other components. Failures of reinforced concrete piers, restrainers, and bearings permitted horizontal movements and in some cases allowed spans to drop (Chodai, 1995; Matso, 1995). Damage to steel bearings included broken set bolts, fracture of the bearing itself, rollers knocked out of place, and anchor bolt failure (Iijima, 1995). Most of the damage to recently constructed bridges was concentrated in the steel bearings and the transverse restrainers, often the result of large substructure displacements caused by liquefaction of soft soils (Yashinsky, 1995; ODRA, 1995). Rubber bearing supports suffered far less damage (Iijima, 1995). As a result of brittle reinforced concrete pier failures and steel bearing (under steel superstructure) failures, Japanese bridge design may place a stronger emphasis on ductility (Rosenbaum, 1996a). According to the Earthquake Engineering Research Institute, the bearing failures could represent a "serious problem for modern bridges" (Matso, 1995). Bearings that can accommodate large displacements may be the solution.

Steel bridge piers experienced a variety of unanticipated problems as a result of the earthquake. Many tubular steel columns suffered damage due to local or overall wall buckling. Buckling often occurred in columns at sections of abrupt change in stiffness. In rectangular columns this occurred at bolted splices where longitudinal stiffeners were discontinued (Miki, 1996). Circular columns buckled at welded splices where wall thickness changed (Miki, 1996). Local buckling also occurred in steel pier columns between stiffeners.

Two rectangular steel box columns completely collapsed due to buckling at a splice that caused corner welds to tear, and then the corner welds unzipped down the columns (Miki, 1996). Although not known for certain, it was suggested 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 (Horikawa, 1996). The failure of the corner welds was also facilitated by the fact that as part of a mainly axially loaded column, they were designed as "stitch" welds only, and were never intended to transfer much load (Miki, 1996).

Many buckled pier columns suffered permanent out-of-plane deformations (leaning), but they did not collapse. Steel columns can be vulnerable to post-buckling strength degradation that occurs after only a few cycles (see also Sections 3.2 and 4.2 on steel piers). Buckling is also undesirable from the standpoint of difficulty of repair.

Several brittle fractures that occurred in steel pier columns were attributed to low cycle fatigue, or decreased fracture toughness due to large plastic strains (Miki, 1996). The brittle fractures were found in circular piers accompanied by lantern buckling, as well as in the interior corners and at the bases of rectangular section portal frames.

Another example of brittle fracture occurred in a centrifugally cast round steel pier shaft. The design thickness of the shaft was 40 mm, but the actual wall thickness was 68-72 mm, since a reinforcing portion was required to achieve the desired properties in the design portion. The design thickness portion of the pier satisfied specifications, but the excess portion of the wall thickness had very poor properties. Cracking initiated in microcavities in the reinforcing portion and propagated through the design thickness. If continued utilization of this type of fabrication is desired, then the challenge to researchers is to determine whether cracks initiating in the microcavities can be arrested before propagating into the design portion.

2.3 Other Steel Structure Damage in Recent Earthquakes

2.3.1 Northridge Earthquake

The most disturbing news that emerged from the Northridge earthquake was the extremely poor performance of welded moment connections in buildings. After initially appearing to have ridden out the Northridge earthquake with little damage, over 100 steel moment resisting frame (MRF) buildings were found to have cracked connections (Wright, 1994; Rosenbaum, 1995; Zarghamee, 1995). Probabilistic methods identified two global damage patterns. The first was a random pattern of rejectable weld conditions caused by poor welding, and likely pre-earthquake. The second pattern, superimposed on the first, was a set of serious fractures correlated to high local stresses (Bonowitz, 1995). Damage was limited to MRFs; concentric and eccentrically braced frames performed well (Chen, 1996). Most of the fractured connections had bolted webs (sometimes with supplementary welds) in conjunction with flange full penetration groove welds with backing bars left in place (see figure 2-1).

In most, if not all cases, the cracks initiated at the welds, often at the bottom flange. Although the cracks initiated at the toe of the weld, they propagated in various directions, not always resulting in complete fracture. Some resulted in a fracture of the beam flange only, occasionally removing "divots" from the column flange. Others propagated through the column flange and into the column web (Skiles, 1994).

Another less publicized problem mentioned in one article was the failure of a simply supported top-and-seat angle connection (Zarghamee, 1995). This was attributed to the connection being unable to undergo a negative rotation without damaging the bottom seat bolts. This failure illustrates that designers must ensure that compatible displacement of all components can occur even if the components are not being relied upon to carry loads.

2.3.2 Kobe Earthquake

As in the Northridge earthquake, steel buildings did not perform as well as expected in the Kobe earthquake. Half the damage to steel buildings was located at the welded beam-to-column connections (Horikawa, 1996). Many older buildings showed no evidence of any ductility, while other buildings displayed some evidence of yielding and energy absorption before fracture.

It should be pointed out that Japanese moment connection details differ from U.S. practice (Kircher, 1996). While U.S. practice generally uses I-shaped columns with welded moment connections framing in to the flanges only, Japanese practice utilizes rectangular tubular columns with welded

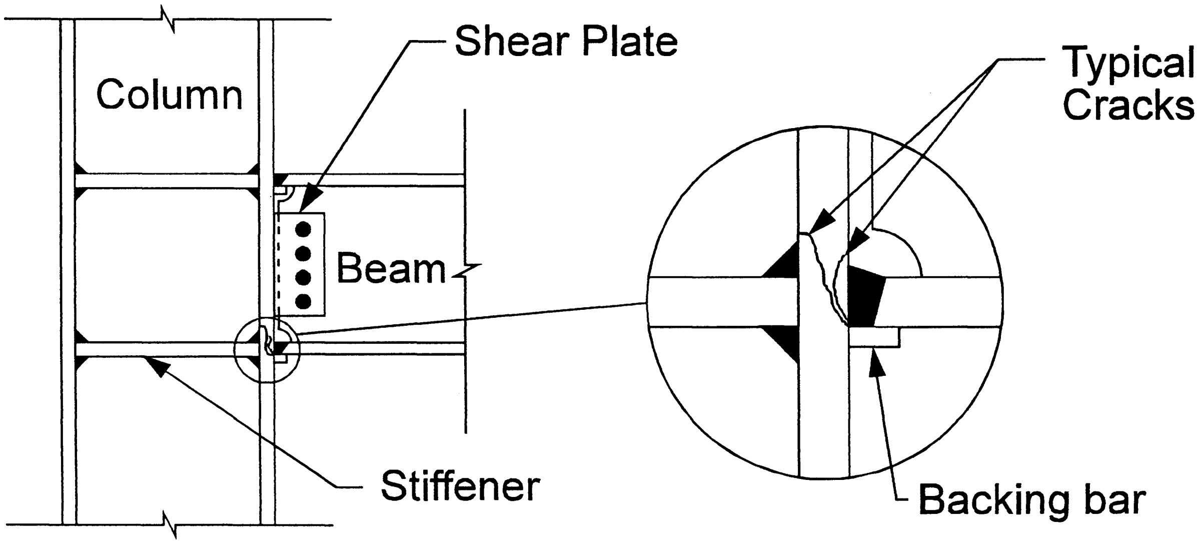


FIGURE 2-1 Typical cracks in welded MRF connection from the Northridge earthquake

moment connections framing in from both axes. Japanese practice cuts the tubular column at the connection in order to provide welded horizontal stiffeners at the levels of the flanges. While avoiding the through-thickness problems experienced in column flanges at Northridge, the increased welding present in the Japanese connections may make them more prone to weld related fractures. On the other hand, Japanese practice generally incorporates more redundancy than U.S. designs (Bonneville, 1996).

Another welded connection also exhibited problems. Buildings constructed with rigid truss framing systems having built-up chords with lateral and diagonal members more than 50 cm (19.7 in) deep demonstrated brittle fracture in 57 instances without evidence of plastic deformation (Horikawa, 1996). Although damage to steel buildings was extensive, more recently constructed buildings performed better, indicating that codes are improving (Normile, 1995).

2.4 State-of-the-Art Research and Its Applicability to Steel Bridges

2.4.1 Moment Connections

The important question is whether the problems with welded moment connections in buildings indicates that similar problems may be possible in bridges. Bridges are less susceptible to welded connection problems because they often must meet stricter requirements due to more demanding moving loadings. Fatigue and fracture concerns from cyclic loadings result in more careful attention to welded connections in bridges, because fatigue cracks often initiate at weld defects.

A major part of the problem with the welded connections in buildings was associated with the welds themselves. One problem identified was the low toughness of the weld metal. Tests at Lehigh University showed that if the pre-Northridge connections had been welded with higher toughness weld metal, continuity plates were provided on the columns, and the bottom flange backing bars were removed, the connections performed adequately under dynamic cyclic loads (Kaufman, 1996a, 1996b). The weld metal requirements in the current AASHTO specification references the Bridge Welding Code, that only specifies minimum toughness requirements for fracture critical members (AASHTO, 1996; AWS, 1995). Existing welded bridge details with low toughness weld metal may be seismically vulnerable if subjected to high cyclic loadings.

Another problem identified in the Northridge welds was quality control and inspection. This should be less of a concern in bridges, because many states avoid field welding, and most welds are performed in a shop environment, where quality control and inspection are easier. The presence of backing bars was also cited for providing a cracklike defect in the building connections and making inspection tougher. Removal of backing bars that are perpendicular to the applied stress is required in bridges, so this should not present a problem.

A study of past welded beam-to-column moment connection experiments was performed in order to try to determine the important parameters affecting flexural ductility (Roeder, 1996). Well over 120 experiments by various researchers were studied and 91 specimens were examined in detail. Considerable scatter in the results was attributed to the inherent variability in the tests themselves as well as fundamental variations in element and weld behavior. The conclusions indicate:

- Flexural ductility decreases as beam depth increases.
- Thicker flanges also seem to reduce ductility, although flange thickness is generally proportional to beam depth, and depth appears to be the primary influence.
- While panel zone yielding itself is a desirable ductile mechanism, experimental results indicate that flexural ductility is reduced when panel zone deformations occur.
- As the length-to-depth ratio of beams decreases, connection ductility also decreases.
- Weak axis beam-to-column connections have less flexural ductility than strong axis connections with no panel zone yielding, but more flexural ductility than those with panel zone yielding.

The failure of the bolted-web-welded-flange connections and the accompanying abandonment of that detail has resulted in a number of proposed connections to fill the void and permit the continued utilization of steel MRF's (Nelson, 1996; Fairweather, 1996; Richard, 1995; Iwankiw, 1996; Engelhardt, 1996). Proprietary solutions start with the MNH SMRF system that uses vertical side plates, fillet welds and no direct connection of the beams to the columns (figure 2-2). A second solution by the Allen Company looks to salvage the pre-Northridge connections by providing horizontal slots in the beam web, top and bottom, and vertical slots in the column web opposite the beam flanges, resulting in a "softening" of the connection, leading to reductions in stress concentration factors from 5 down to 1.2. AISC is exploring the use of "dogbone" connections, where the beam section is reduced a short distance from the column in order to weaken the beam at this location and draw the plastic hinge away from the column face (figure 2-3). Another reinforcing technique uses cut wide flange sections to move the hinge away from the column face. Other companies are looking at approaches using welded coverplates and vertical reinforcing plates. It should be noted that after initially performing very well, the “coverplate” solution 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.

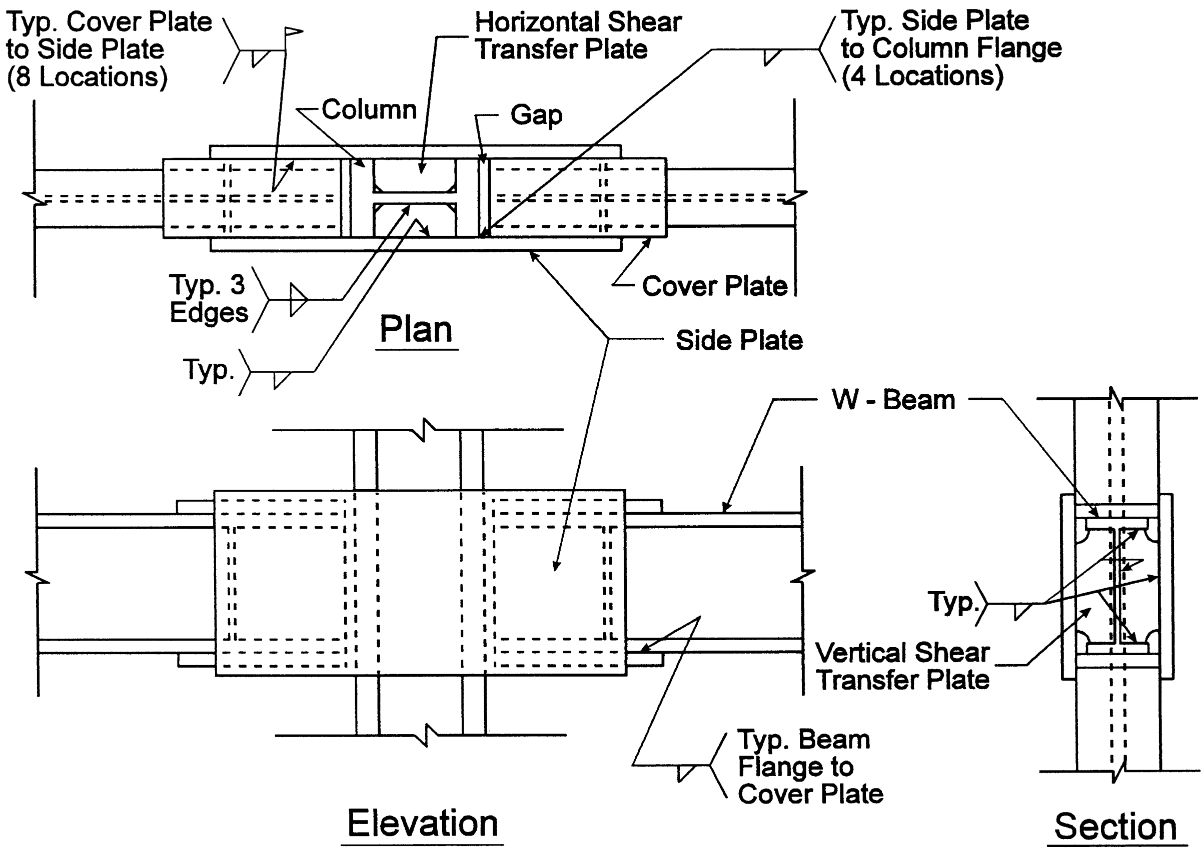


FIGURE 2-2 MNH-SMRF connection

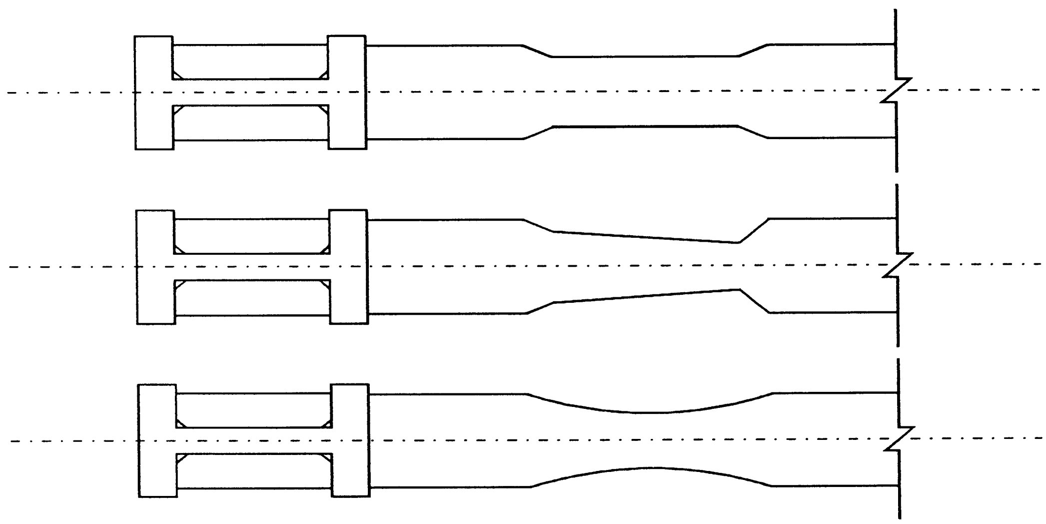


FIGURE 2-3 Several "dogbone" connection configurations

ICF Kaiser and Lehigh University have teamed together to develop a bolted moment connection suitable for repair, retrofit, or new construction (Kasai, 1997). Tests to date on these connections, illustrated in figure 2-4, have been very successful. Two aspects of these connections that have improved their net section performance are the washer plates and locating the bolts as close to the beam web as possible. Bolted connections are attractive because they do not rely on the skill of a welder. Bolted connections are also more desirable for bridge applications, where field welding is often avoided because of fatigue and quality control concerns.

While not necessarily directly applicable to bridges, because construction techniques differ between buildings and bridges, many of the concepts used in the proposed new connection designs and retrofits may be useful in bridge designs and retrofits. One of the main tenets of earthquake design is that connections should not fail, because joint failures are usually brittle fractures, and structural integrity is often lost. Because the highest forces are often located at connections, and geometry changes and discontinuities at the joints result in stress concentrations, it can be difficult to achieve this goal.

Many of these new concepts include methods of reinforcing connections or weakening members in order to shift inelastic action away from connections or other undesirable locations. The most promising concepts for seismic retrofit of bridges are the "dogbone," and the "Kaiser/Lehigh Bolted Connections." Bridge connections with large gusset plates might benefit from the same approach used in the MNH-SMRF connection, with welded or bolted reinforcing plates forcing inelastic action into the main member. Bolted solutions, such as the Kaiser/Lehigh connections, are particularly attractive for bridge applications, due to the reluctance of many states to use field welding.

2.4.2 Lateral Bracing Systems

While little research has been performed on the response of steel bridges to earthquakes, a substantial amount of research into the response of steel building frames has been performed. Three types of lateral load resisting systems are typically used in buildings: moment resisting frames (MRFs), concentrically braced frames (CBFs), and eccentrically braced frames (EBFs). Hybrid systems such as an MRF superimposed on a CBF or EBF for redundancy are sometimes used also. Any of these three systems, properly proportioned and detailed, has been demonstrated to be effective in resisting seismic forces. The philosophy in seismic design of buildings would seem to be completely opposite that of bridges, however. In buildings, a strong column - weak beam philosophy is used, while in bridge piers (at least for typical concrete column bent piers) a strong beam - weak column philosophy is used.

Due to the limited seismic research data available on steel bridges, the seismic provisions developed by ATC-32 for bridge design were forced in many cases to combine the results of seismic research on steel buildings with the design philosophy of concrete bridges. This results in provisions essentially identical to those for beam-column moment connections in buildings, except that the plastic hinge region is specified for the column instead of the beam. Provisions for the design of concentric bracing are given, but not for eccentric bracing. Bracing is to remain elastic, however, as inelastic action is limited to the pier columns.

The limitation of plastic hinging to the pier columns in concrete bridge designs is reasonable, considering the structural configuration and that designing for ductile behavior in concrete columns

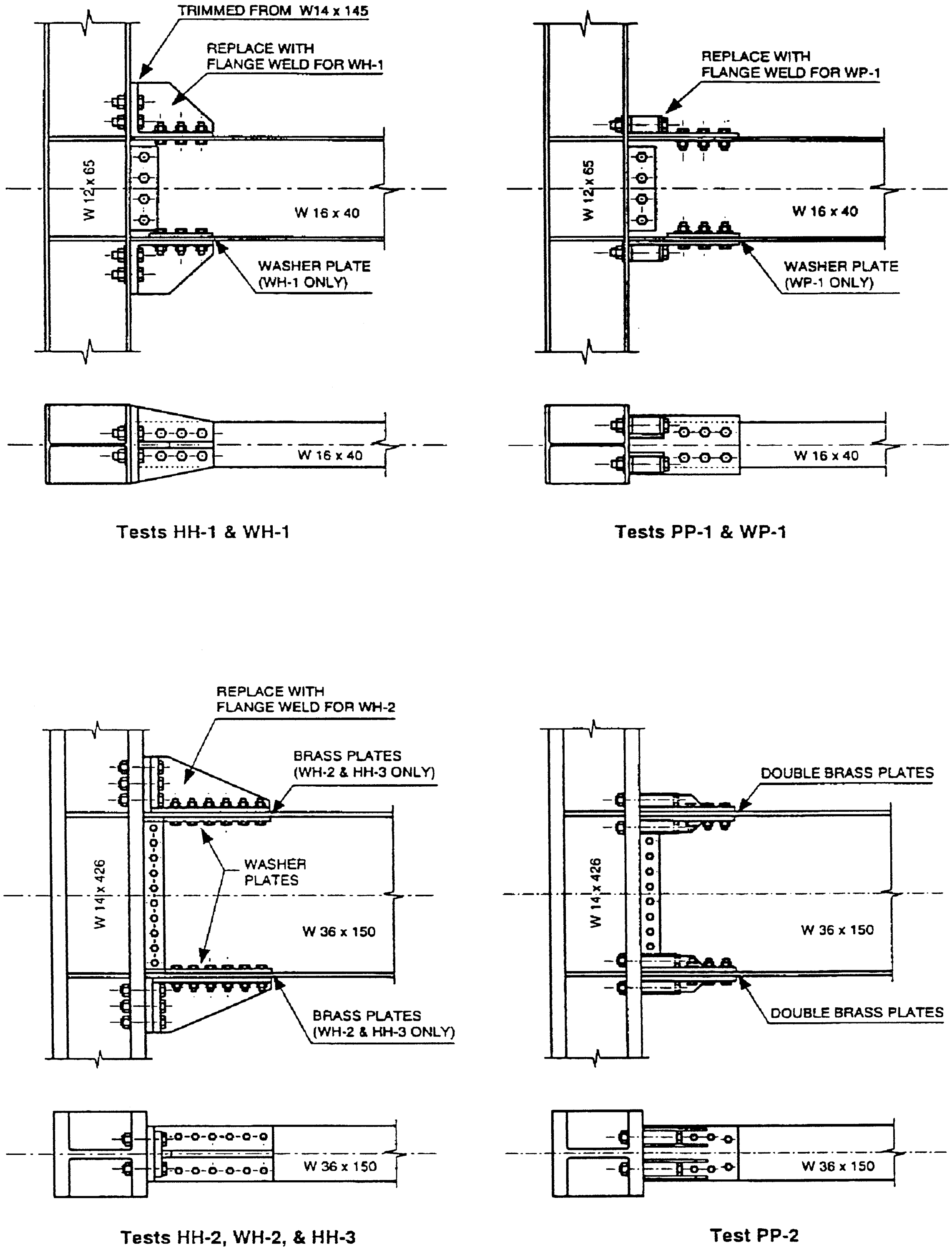


FIGURE 2-4 Kaiser/Lehigh Bolted Connections (Kasai, 1997)

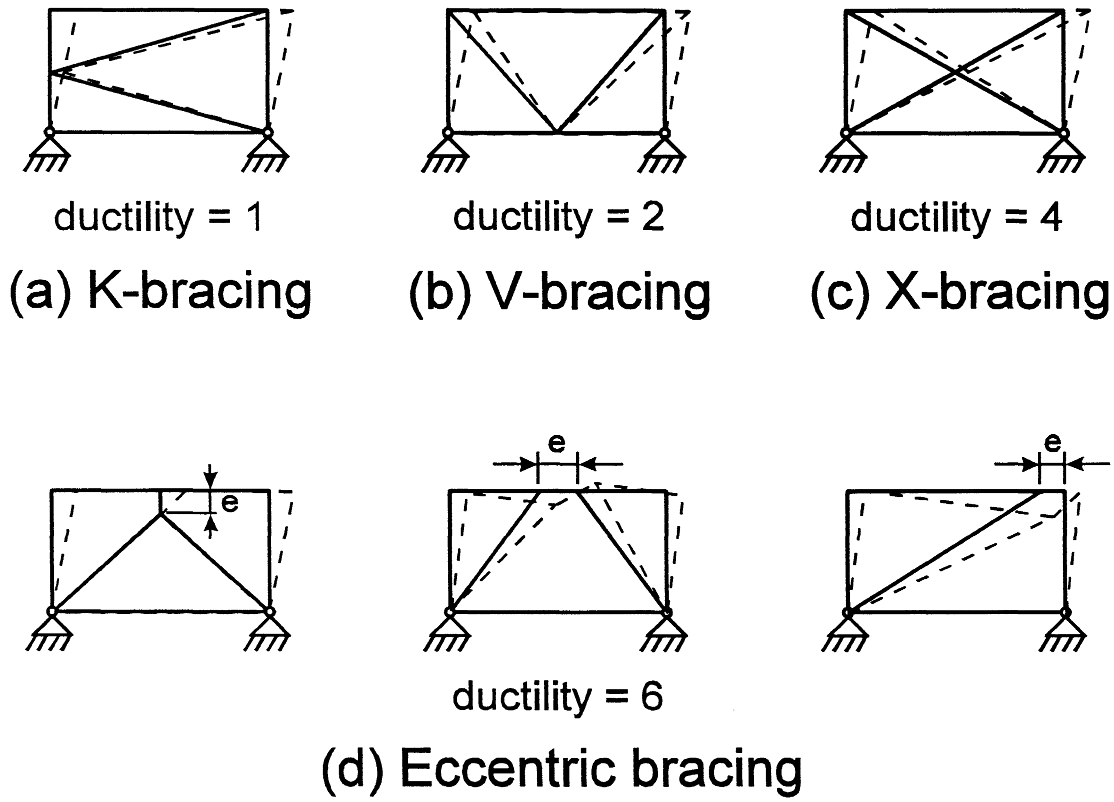


FIGURE 2-5 Various lateral bracing schemes (deflected shapes in dashed lines) and their approximate relative ductilities

is well understood. As mentioned previously, limiting steel bridges to the same philosophy is overly restrictive, because the inelastic behavior in ductile steel components can be accommodated at a variety of different locations. There are often secondary components in steel bridge structures, such as bracing or cross-frames, that can serve as ductile energy dissipators, while also limiting forces developed in other components. These secondary members can also often be easily repaired or replaced, as they are not the main gravity load carrying members. Furthermore, the design of tall steel tower bents has much in common with steel building frame design, and similar design philosophies should be effective.

One obstacle to applying research on building lateral load resisting systems to bridges appears to be the use of bolted connections. Bolted web-welded flange connections in EBFs did not perform as well as fully welded connections due to bolt slip at high load (Malley, 1983), and concern over loss of clamping force in slip-critical connections has been expressed in ATC-32 (ATC-32, 1992). It is apparent that fully bolted connections can be adequate, but additional research should be performed in this area.

Figure 2-5 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, because 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. 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 ductility of each configuration is listed on the figures (ECCS, 1991).

2.4.2.1 Moment Resisting Frames (MRF)

Moment resisting frames can provide excellent ductility and energy dissipation capacity, but can also suffer from large deflections and as a result high P-delta forces. Additionally, as demonstrated in the Northridge earthquake, the moment resisting connections are the critical component of the design. Great care must be taken to insure that connections can develop the required strength to form plastic hinges in the adjacent members (or in the connection itself).

2.4.2.2 Concentrically Braced Frames (CBF)

The use of concentric bracing in frames is a very efficient method of resisting lateral forces due to the true truss action present. Current designs generally use braces that carry both tension and compression loads. In the past, X-bracing that carried loads in tension only was used, but this practice has been largely abandoned in buildings, and its use is strongly discouraged.

Poor behavior of concentric bracing in several recent moderate to strong earthquakes resulted in increased design force requirements in the building codes. Several researchers have asserted that increasing the design forces to try to compensate for a lack of ductility is not the solution (Astaneh-Asl, 1992; Goel, 1992a; 1992b). They contend that providing members and connections with sufficient strength, ductility, and energy dissipation capacity to survive the large cyclic deformations of a severe earthquake is a more rational approach.

Most practical braces in buildings are of intermediate slenderness, and will inelastically buckle in compression during a severe earthquake. Depending on the size of the structure, bracing in bridges can be stockier than buildings, but braces are still prone to fail by inelastic buckling. In order to utilize concentric bracing systems, not only must braces be constructed to survive these large post-buckling deformations, but the designer must be able to model the complex hysteretic behavior of the braces, and use this to predict overall frame response.

Recent research has shown that concentric bracing can be ductile. Strength and ductility are highly dependent on the details and connections, however (Astaneh-Asl, 1984; 1994; Goel, 1992a). The connections must be able to accommodate the deformations and forces induced by inelastic buckling. Built-up members consisting of two shapes must be sufficiently tied together to act as a single unit (Goel, 1992a). Local buckling has been found to be especially deleterious to braces and large b/t ratios are recommended to limit post-buckling strength degradation. Post-buckling strength degradation was found to occur faster as the slenderness of the struts increased (Popov, 1981).

The AISC method of calculating 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 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 are recommended for braces (Goel, 1992a), when using the AASHTO dimesionless equation:

$$\frac{b}{t} = k\sqrt{\frac{E}{F_y}} \tag{2-1}$$

where:
b = width of the plate
t = thickness of the plate
E = modulus of elasticity of steel
F_y = yield point of steel

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). Stitches subjected to post-buckling shear should be designed to carry a force of $A*F_y/4$ from one element of the member to the other where A is the total area of the member (Astaneh-Asl 1986).

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:

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

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). The end connections of the bracing members should not only be designed for an axial force of $A*F_y$, but also for an axial force of $A*F_y/2$ in conjunction with a moment of $2.5*M_y$ (Astaneh-Asl, 1986), in order to accommodate post-buckling forces. 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):

$$\frac{L_{fg}}{t} < 0.75\sqrt{\frac{E}{F_y}} \qquad \textbf{(2-3)}$$

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.

Several methods are available for modeling the complex hysteretic behavior of braces as part of a frame (Ikeda, 1986). The most promising of the methods for design is the hinge model, which falls into the category of physical theory models (Ikeda, 1986; Nakashima, 1992). Physical theory models only require material properties and common geometric or derived engineering properties of a member, which make them easily applicable in all 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). 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 (Ikeda, 1986; Soroushian, 1990).

Concentric bracing is an efficient system for resisting lateral forces. The high stiffness provided by braced frames can limit damage by limiting lateral deflections and the accompanying P-delta induced moments. When properly designed and detailed for the deformations of severe cyclic buckling, concentric braces have the ductility and energy dissipation capacity to perform satisfactorily during earthquakes. Relatively easy to apply hinge models are available to predict brace behavior as part of a frame.

2.4.2.3 Eccentrically Braced Frames (EBF)

The eccentric braced frame (EBF) is a hybrid system that aims to combine the stiffness of a concentrically braced frame (CBF) with the ductility and energy dissipation capacity of a moment resisting frame (MRF). As the name suggests, in an EBF the bracing is given an intentional eccentricity, in order to furnish a beam segment (also known as an active link) that will yield under extreme seismic forces, providing a structural "fuse" that limits forces and contributes energy dissipation. 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, 1993; Popov, 1989, 1992; Foutch, 1989; Lee, 1989; Goel, 1989c; Engelhardt, 1989, 1992a, 1992b; Ricles, 1994). In well proportioned EBFs, yielding is limited to the links, with the rest of the structure remaining elastic.

Eccentric braces can be provided in a number of different geometries, resulting in considerable architectural flexibility. 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 (AISC-LRFD uses a cutoff of 2.6), with a transition length in between (Engelhardt, 1992b). Short (shear yielding) active links are more attractive structurally because of the advantages of strength, stiffness, and inelastic deformation capacity, although architectural and functional constraints calling for large openings in braced bays have created an impetus for using longer (moment yielding) links (Engelhardt, 1992b).

As link length increases, the types of local instability change from those related to shear (shear buckling of the link web) to those related to flexure (flange buckling and lateral-torsional buckling). Web shear buckling is delayed by the addition of equally spaced vertical stiffeners in shear links. In flexural links, full depth stiffeners placed $1.5*b_f$ from the ends of links do not prevent flange buckling, but effectively limit the strength loss. Transitional length links should have both types of stiffeners. Flange buckling outside the link in the brace panel can be mitigated by adding a 75% depth vertical stiffener $1.5*b_f$ beyond the end of the link (Engelhardt, 1992b).

Lateral braces are required at both ends of an active link, to avoid lateral-torsional buckling failures prior to developing the full strength of the link. Research has shown that the lateral bracing can develop significant forces (Popov, 1989). In multiple bay buildings this is not a major concern, because the bays can be braced against one another, and floorbeams and the floor slab can be used to provide bracing. In a single bay bridge tower bent, there is nothing to brace against. In the proposal for the seismic retrofit of the Richmond-San Rafael bridge, the bracing problem was solved by replacing the single plane bent with a dual EBF bent consisting of two EBFs placed very close together and braced against each other (Vincent, 1996). Another solution to this problem may be to use torsionally stiff tubular sections for the beam containing the active link, although research would be needed to verify the suitability of this solution.

The geometry containing the active link in the middle of the beam is preferred, because it does not require any moment resisting beam-to-column connections. During the early development of EBFs, links were often located at both ends of a brace. Subsequent studies showed that only one of the links was active as an energy dissipator. Therefore, unless dictated by other requirements, it is recommended that a less expensive concentric connection be used at one end of the brace (Popov, 1989).

The predominant failure mode in the tests of long links connected at the column was fracture of the link flange-to-column connection due to the high link rotation demands. For shear links, all-welded link-column connections can develop the full capacity. However, it is recommended that links welded to columns should not be used until connection details that can be used with confidence are identified (Engelhardt, 1989). Bolted web-welded flange connections may be inadequate due to bolt slippage at high shears (Malley, 1983).

To proportion the links in a building, the design shear force in a link is approximated with the equation $V_{link} = V_{story} * (h/L)$, where V_{story} is the story shear, h is the floor height, and L is the span length (Popov, 1992). The link capacity, V_p, must be greater than or equal to the V_{link} or $V_p / V_{link} \equiv \alpha \geq 1.0$ where α is the strength index. Designs should strive to achieve a uniform value of α, in order that links will yield simultaneously, and distribute the inelastic behavior as much as possible. EBF designs with overstrength in the higher stories may result in soft story mechanisms in the lower floors. Similarly, bridge bents should also be designed such that all links yield as close to simultaneously as possible.

The brace and beam outside of the link should be designed to resist the maximum forces generated by the link. The design shear yield strength of the link (AISC-LRFD, 1994) is given by:

$$V_p = 0.6\ F_y\left(d - 2t_f\right)t_w \tag{2-4}$$

where:
d = depth of the beam
t_f = thickness of the flange
t_w = thickness of the web

In the literature, it is common for an overstrength factor of 1.5 to be applied to the design shear yield strength of the link to account for strain hardening and actual yield strengths exceeding nominal yield strengths. The AISC-LRFD seismic provisions for steel buildings specifies that the beam and brace be designed for 1.25 times the nominal strength of the link. This approximately corresponds to multiplying the overstrength factor by the φ factor (= 0.85 to 0.90), or $\varphi * 1.5$.

The EBF is recognized as an effective design concept for seismic design, and equations for EBFs have been codified for the design of buildings (AISC-LRFD, 1994). There is no reason why the EBF concept should not be applied to bridge structures. As mentioned previously, the proposed Richmond-San Rafael bridge retrofit in California replaces the existing chevron braced steel towers with welded EBFs to provide favorable elastic stiffness as well as a ductile "fuse," limiting forces on the superstructure and substructure. The lengths and depths of the built-up active links are larger than the limits for rolled shape shear links in buildings, even though the desired behavior is shear yielding. Analytical studies have predicted the desired behavior, and physical testing has been performed to verify the performance.

A preliminary report on the behavior of the shear links is contained in Itani, 1998. This paper reports that the current code specifications are applicable in the design of built-up shear links, although the overstrength factor needs to be re-examined. While the plastic rotation exceeded the design value of 9%, the strength exceeded the predicted 1.5 overstrength by over 40%. Further examination of building specifications indicates that to calculate the strength of beam-column panel zones, V_n, the following equation is used (AISC-LRFD, 1994):

$$V_n = 0.6\ F_y D_c t_w \left[1 + \frac{3 b_{cf} t_{cf}^2}{D_b D_c t_w}\right] \tag{2-5}$$

where:
F_y = yield strength of steel
D_c = total depth of column section
t_w = thickness of web
b_{cf} = width of column flange
t_{cf} = thickness of column flange
D_b = total depth of beam section

Although this equation may not be specifically appropriate for calculating shear link overstrength, the term with t_{cf}^2/t_w suggests that the overstrength factor is greatly increased when a thick flange is accompanied by a thin web. The EBF is a relatively new concept, however, and much remains to be learned about their behavior under severe earthquakes, especially their behavior as part of bridge structures.

SECTION 3
SEISMIC EVALUATION OF STEEL BRIDGES

3.1 Scope

Although U.S. researchers have developed and codified very good methods for predicting the static strength of most structural members in bridges, there are not any widely available techniques to easily calculate the cyclic yielding, energy dissipation, and strength degradation characteristics of yielded members, let alone their effects on overall structure behavior. The only way to calculate these values at this time is to model the member or structure using an advanced computer analysis program which incorporates nonlinear behavior. This is beyond the capabilities of most design offices. This section presents methods available at this time to evaluate the seismic behavior of bridge members and structures.

3.2 Strength and Ductility Evaluation of Steel Bridge Piers

Two methods for evaluating tubular steel piers developed in Japan are presented here. A relatively simple method for designing and estimating the deformation capacity of steel box bridge piers has been developed by MacRae and Kawashima. This work was used to incorporate design rules for steel box piers in ATC-32 (MacRae, 1992; Kawashima, 1992b; ATC-32, 1992). These rules were developed pre-Kobe and may need to be updated as a result of current research being performed in Japan in response to tubular pier performance in Kobe. A second more complicated, but more general method of modeling the inelastic cyclic behavior of steel box piers, the Evolutionary-Degrading Hysteretic Model, was developed by Usami et. al. (post-Kobe)(Usami, 1992; Kumar, 1996a, 1996b).

3.2.1 MacRae-Kawashima Method

The Japanese bridge design code requirement for width-to-thickness ratios of stiffened rectangular box columns (illustrated in figure 3-1) can be reasonably approximated by the general expression (Kawashima, 1992b):

$$\frac{b}{t \cdot n \cdot f} < \frac{1047.7}{\sqrt{F_{ya}}} \qquad \textbf{(3-1)}$$

where:
b = main plate width
t = main plate thickness
F_{ya} = the specified allowable steel stress (kgf/cm^2) - (about 0.6 F_y)
F_y = yield strength of steel
n = number of panels divided by vertical stiffeners
f = coefficient related to the stress gradient given as:

$$f = 0.65\left(\frac{\varphi}{n}\right)^2 + 0.13\left(\frac{\varphi}{n}\right) + 1.0 \qquad \textbf{(3-2)}$$

where:

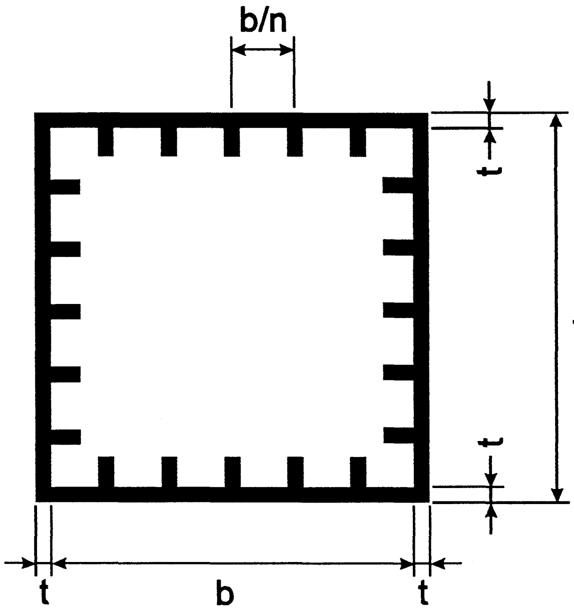


FIGURE 3-1 Rectangular stiffened box column

φ = stress gradient given as:

$$\varphi = \frac{(\sigma_1 - \sigma_2)}{\sigma_1} \tag{3-3}$$

where:
σ_1, σ_2, are the stresses at each edge of the plate, $\sigma_1 > \sigma_2$, and compressive stress is defined as positive

For compact sections expected to yield, f can be taken conservatively as 1.0. Putting this equation into the dimensionless AASHTO format of:

$$\frac{b}{t \cdot n \cdot f} = k\sqrt{\frac{E}{F_y}} \tag{3-4}$$

where:
E = modulus of elasticity of steel
k = constant based on plate boundary conditions

results in a calculated k of about 0.95. Kawashima, 1992b, indicates that a constant of 831, rather than 1047.7 should be used, as research has not been performed supporting a higher limit. The 831 value corresponds to a k of about 0.75. This value is not quite as conservative as the AISC LRFD, 1994 value for rectangular tubular bracing members (corresponding k = 0.65), but more conservative than the value recommended by Astaneh-Asl, 1996 for plates supported along two edges (k = 1.12). For the longitudinal stiffeners, an allowable width-to-thickness ratio for outstanding plates corresponding to a k of 0.45 is recommended (Kawashima, 1992b). This value corresponds to the k value for non-compact outstanding elements in the AASHTO-LRFD, 1994.

In order to determine the adequacy of a longitudinally stiffened box column such as the one pictured in figure 3-1, use the following procedure:

Calculate the slenderness parameter, γ_l:

$$\gamma_l = \frac{11 \cdot I_l}{bt^3} \tag{3-5}$$

where:
I_l = longitudinal stiffener second moment of inertia taken about an axis located along the inside face of the flange plate

Calculate the aspect ratio α and critical aspect ratio α_o:

$$\alpha = \frac{a}{b} \tag{3-6}$$

$$\alpha_o = \sqrt[4]{1 + n\gamma_l} \tag{3-7}$$

where:
a = lateral stiffener spacing (if present)

Calculate the actual, d_l, and required, $d_{l,req}$, ratio of the cross-sectional area of the longitudinal stiffener

$$d_l = \frac{A_l}{bt} \tag{3-8}$$

where:
A_l = area of longitudinal stiffener

$$d_{l,req} = \frac{0.10}{n} \tag{3-9}$$

Calculate the minimum plate thickness for full load carrying capacity, t_o:

$$t_o = \frac{b\sqrt{F_{ya}}}{831nf} = \frac{1.33b}{nf}\sqrt{\frac{F_y}{E}} \tag{3-10}$$

When the aspect ratio of the panels, α, is less than the critical aspect ratio α_o, and the second moment of inertia of the transverse stiffener, I_t, taken about an axis located along the inside face of the flange plate satisfies the following equation:

$$I_t \geq \frac{bt^3}{11} \cdot \frac{1 + n\gamma_{l,req}}{4\alpha^3} \tag{3-11}$$

then $\gamma_{l,req}$ is calculated using the following equations:

$$\begin{aligned} \gamma_{l,req} &= 4\alpha^2 n \left(\frac{t_o}{t}\right)^2 (1 + nd_l) - \frac{(\alpha^2 + 1)^2}{n} & (t \geq t_o) \\ \gamma_{l,req} &= 4\alpha^2 n (1 + nd_l) - \frac{(\alpha^2 + 1)^2}{n} & (t < t_o) \end{aligned} \tag{3-12}$$

In all other cases, such as when the effect of transverse stiffeners is insignificant and is ignored, $\gamma_{l,req}$ is given by:

$$\begin{aligned} \gamma_{l,req} &= \frac{1}{n}\left[\left\{2n^2\left(\frac{t_o}{t}\right)^2 (1 + nd_l) - 1\right\}^2 - 1\right] & (t \geq t_o) \\ \gamma_{l,req} &= \frac{1}{n}\left[\left\{2n^2 (1 + nd_l) - 1\right\}^2 - 1\right] & (t < t_o) \end{aligned} \tag{3-13}$$

If the column being checked meets the following:

$$\frac{\gamma_l}{\gamma_{l,req}} \geq 1.0 \tag{3-14}$$

$$\frac{d_l}{d_{l,req}} \geq 1.0 \tag{3-15}$$

$$\frac{t}{t_o} \geq 1.0 \tag{3-16}$$

then the slenderness criteria have been met. If the requirements are not met, remedial action should be taken in order to meet the criteria. Adding longitudinal stiffeners or increasing width-to-thickness ratios by adding reinforcing plates to the column faces are two possible options. Filling the column with concrete is a third option, which increases strength and stiffness, but the ductility of the column may decrease.

When a steel column meets the slenderness criteria above, the following procedure based on the equal energy method may be used to estimate the seismic resistance, R_e, and the effective ductility, μ_e (Kawashima, 1992b).

Calculate the slenderness parameter γ_l*:

$$\gamma_l^* = 4\alpha^2 n(1 + nd_l) - \frac{(\alpha^2 + 1)^2}{n} \qquad (\alpha \le \alpha_o)$$

$$\gamma_l^* = \frac{1}{n}\left[\left\{2n^2(1 + nd_l) - 1\right\}^2 - 1\right] \qquad (\alpha > \alpha_o) \tag{3-17}$$

The ratio of γ_l/γ_l^* can be used to predict whether local panel buckling or wall buckling would be the initial buckling mode. If $\gamma_l/\gamma_l^* > 1$ then local panel buckling would be expected. If $\gamma_l/\gamma_l^* < 1$ then wall buckling would be expected. This is represented graphically in figure 3-2. Experiments have indicated that even though the γ_l/γ_l^* ratio may predict the initial buckling mode, actual column failure is governed by wall buckling at ratios significantly greater than 1.0 (Kawashima, 1992b).

Calculate the inelastic rotation capacity θ_i:

$$\theta_i = \frac{2a}{D}\frac{\varsigma}{\kappa}\frac{\gamma_l}{\gamma_{l,req}} \tag{3-18}$$

where:
D = depth of steel section
ς = critical deformation factor, empirical constant based on experiments, equal to the maximum measured strains divided by the gamma ratio, taken as 0.006
κ = factor used to limit the value of the $\gamma_l/\gamma_{l,req}$ ratio to 1.5 when the γ_l/γ_l^* ratio exceeds 2.0 in order that the deformation capacity is not overestimated - additional research is required to deal with this discontinuity in the equation:

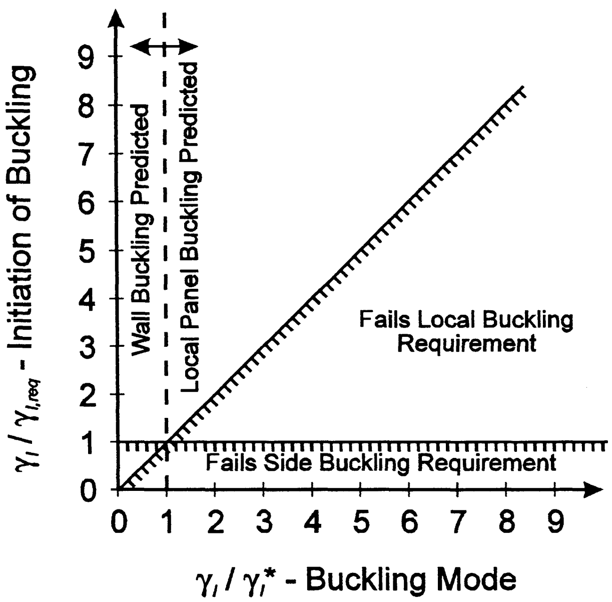


FIGURE 3-2 Allowable pier design parameters (adapted from Kawashima, 1992b)

$$\kappa = 1 \qquad \left(\frac{\gamma_1}{\gamma_1^*} \le 2\right)$$

$$\kappa = \frac{\gamma_1}{1.5 \cdot \gamma_{1,req}} \qquad \left(\frac{\gamma_1}{\gamma_1^*} > 2\right) \tag{3-19}$$

Calculate the maximum deflection capacity of the pier, δ_u:

$$\delta_u = \delta_y + \delta_i \tag{3-20}$$

The deflection at yield, δ_y, is calculated from the bending and the shear deformations using:

$$\delta_y = \frac{H_y L^3}{3EI} + \frac{H_y L}{GA_s} \tag{3-21}$$

where:
I = moment of inertia of the pier
L = distance from point of horizontal load application to base of pier
A_s = shear area of the pier - approximated by Area/2.4 for square thin-walled box sections
G = shear modulus of steel
H_y = the horizontal force that causes first yield at the hinge and is equal to:

$$H_y = \frac{M_y}{L} = \frac{1}{L}\left(F_y - \frac{P}{A}\right)S \tag{3-22}$$

where:
M_y = moment that causes first yield in the pier
P = axial force present in pier
A = cross-sectional area of pier
S = section modulus of pier

The inelastic deflection, δ_i, is equal to:

$$\delta_i = \theta_i \cdot L \tag{3-23}$$

The ductility factor, μ, can now be calculated:

$$\mu = \frac{\delta_u}{\delta_y} = \frac{L\theta_i + \delta_y}{\delta_y} = \frac{L\dfrac{2a}{D}\dfrac{\varsigma}{\kappa}\dfrac{\gamma_1}{\gamma_{1,req}} + \delta_y}{\delta_y} \tag{3-24}$$

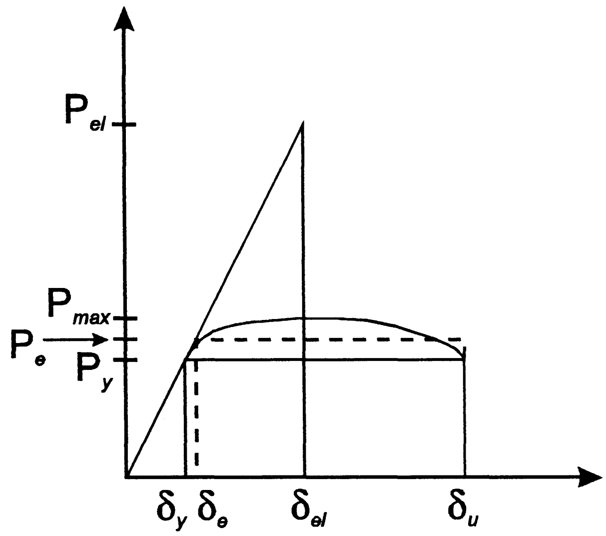


FIGURE 3-3 Equal energy method for ductility assessment (adapted from Kawashima, 1992b)

To simplify the analysis, the actual nonlinear load-deflection curve is transformed into an equivalent "effective" linear elastic-perfectly plastic curve using the equal energy method. The equal energy method sets the areas under the P-δ curves to be equal, as illustrated in figure 3-3, and assuming the overstrength curve can be approximated by a triangle and the effective yield, δ_e, is equal to the actual yield, δ_y, the effective overstrength factor, OSF_e, is:

$$OSF_e = \frac{(OSF + 1)}{2} \quad \textbf{(3-25)}$$

where the overstrength factor, OSF, is equal to M_{max}/M_y, and is taken to be equal to 1.40. The effective ductility, μ_e, is calculated as:

$$\mu_e = \frac{\mu}{OSF_e} \quad \textbf{(3-26)}$$

From this the resistance due to ductility, Q_e, can be calculated from:

$$Q_e = \sqrt{2\mu_e - 1} \quad \textbf{(3-27)}$$

The effective resistance factor, R_e, is then equal to:

$$R_e = OSF_e \cdot Q_e \quad \textbf{(3-28)}$$

If the required resistance factor, R, is greater than the effective resistance factor, R_e, then the effective pier resistance must be increased.

In the previous method the slenderness ratio equations to limit buckling were based on linear elastic principles. This means that they only insure that yield is reached prior to buckling. Increasing longitudinal stiffener rigidity to three times the optimum according to linear elastic theory or:

$$\frac{\gamma_l}{\gamma_{l,req}} \geq 3 \tag{3-29}$$

was found to greatly improve cyclic inelastic behavior (Usami, 1992). Increases beyond 3 times resulted in no noticeable improvement.

3.2.2 Evolutionary-Degrading Hysteretic Model

This model incorporates the various effects involved in the inelastic response of a structure into a single variable, denoted the damage index, D:

$$D = (1-\beta)\sum_{j=1}^{N_1}\left(\frac{\delta_{max,j}-\delta_y}{\delta_u-\delta_y}\right)^c + \beta\sum_{i=1}^{N}\left(\frac{E_i}{H_y(\delta_u-\delta_y)}\right)^c \tag{3-30}$$

where:
δ_y = yield displacement
δ_u = ultimate displacement under monotonic loading
$\delta_{max,j}$ = maximum displacement produced for the j-th time
E_i = hysteretic energy dissipated in the i-th half cycle
H_y = minimum of the yield, local buckling, and instability loads
N = total number of half-cycles
N_1 = number of cycles producing $\delta_{max,j}$ for the first time such that $\delta_{max,j} > \delta_{max,j-1} + \delta_y$
β, c = empirically derived constants

The damage index takes deformation damage into account with the first term, and low-cycle fatigue damage into account with the second term. The index has been normalized to reach a value of one when the residual strength degrades back to a value of H_y, which has been defined as the "collapse load." This value of the collapse load has been found to set a rational limit to the damage in box columns and at the same time takes into account the influence of various material and geometric properties.

Although the experimental database for determining the values of c and β is limited, tests indicate that values of β from 0.1 to 0.2 and values of c from 1.0 to 2.0 are reasonable. An initial value of $\beta = 0.11$ is recommended because for this value of β the scatter in c was a minimum. If experimental results are available for calibration, c can be calculated. Otherwise, the mean value of c was found to be 1.3 for unstiffened columns and 1.8 for stiffened columns. The minimum recommended value for c is 1.0.

Slenderness parameters based on the width-to-thickness ratio and on column stability are used to determine the ultimate displacement δ_u by calculating the ductility μ.

The width-to-thickness ratio parameter, R_f, is equal to:

$$R_f = \frac{b}{t}\sqrt{\frac{F_y}{E}\frac{12(1-\nu)}{\pi^2 k}} \tag{3-31}$$

where:
b = flange width
t = flange thickness
E = Young's modulus
F_y = steel yield strength
ν = Poisson's ratio
k = buckling coefficient

The column slenderness parameter, $\bar{\lambda}$,

$$\bar{\lambda} = \frac{2h}{\pi r}\sqrt{\frac{F_y}{E}} \tag{3-32}$$

where:
h = column height
r = radius of gyration

It was found that μ was more influenced by R_f than $\bar{\lambda}$, so $R_f * \sqrt{\bar{\lambda}}$ was selected as the combination parameter. Since very low values of R_f and $\bar{\lambda}$ should result in a perfectly plastic hinge with values of μ going to infinity, an equation of the following form was chosen:

$$\ln\mu = a_1 \ln\left(R_f\sqrt{\bar{\lambda}}\right) + a_2 \tag{3-33}$$

where:
a_1, a_2 = empirical constants

For unstiffened columns, a_1 and a_2 were found to be -2.31 and 0.44 respectively. For stiffened columns the values were found to be -2.10 and -0.45 respectively.

The presence of axial load affects the yield load, H_y, the maximum strength, H_{max}, and the ultimate deformation, δ_u. The yield load including local buckling can be found using the following interaction formulas and setting H_y equal to the smaller value of H (Usami, 1992):

$$\frac{P}{P_u} + \frac{0.85 \cdot H \cdot h}{Q_B \cdot M_y\left(1 - \frac{P}{P_E}\right)} = 1.0 \tag{3-34}$$

$$\frac{P}{Q \cdot P_y} + \frac{H \cdot h}{Q_b \cdot M_y} = 1.0 \tag{3-35}$$

where:
P_E = Euler buckling load
and

$$Q = \frac{\sum A_e}{\sum A} \leq 1.0 \qquad \textbf{(3-36)}$$

$$Q_B = \left(\frac{A_e}{A}\right)_{\text{Flange}} \leq 1.0 \qquad \textbf{(3-37)}$$

where:
A = area of a component plate
A_e = effective area of a component plate

The relationship between A and A_e is given by:

$$\frac{A_e}{A} = 1.24 - 0.54 \cdot R_f \leq 1.0 \qquad \textbf{(3-38)}$$

The relationship between P_u and P_y is given by:

$$\frac{P_u}{Q \cdot P_y} = 1.0 \qquad (\lambda' \leq 0.2)$$

$$\frac{P_u}{Q \cdot P_y} = 1.109 - 0.545 \cdot \lambda' \qquad (0.2 \leq \lambda' \leq 1.0) \qquad \textbf{(3-39)}$$

$$\frac{P_u}{Q \cdot P_y} = \frac{1}{0.773 + \lambda'^2} \qquad (\lambda' \geq 1.0)$$

where:
P_y = axial yield force
and:

$$\lambda' = \sqrt{Q} \cdot \lambda \qquad \textbf{(3-40)}$$

Based on several assumptions, the axial load ratio effect on the maximum column strength, H_{max}, is given as:

$$H_{max,2} \simeq H_{max,1} + \frac{(P_1 - P_2)\delta'}{h} \qquad (P_2 < P_1) \qquad \textbf{(3-41)}$$

where:
δ' = deflection corresponding to the maximum load

Similarly, the axial load effect on the maximum deflection, δ_u, is given by:

$$\delta_{u,2} = \left(\frac{P_1}{P_2}\right)\delta_{u,1} \qquad (P_2 < P_1) \tag{3-42}$$

The residual strength of the column, H, was found to degrade exponentially with the damage index, so an exponential equation was chosen to calculate H. The equation was calibrated such that in the undamaged state (D=0) the strength is a maximum, and at the defined collapse point (D=1) it is equal to the yield strength (H_y):

$$H = H_{in}\, e^{-\left[\ln\left(\frac{H_{in}}{H_y}\right)\right]D} \tag{3-43}$$

where:
H_{in} = hypothetical initial undamaged column strength taking strain hardening into account - calculated by substituting the maximum column strength H_{max} and the corresponding damage index D_1 in the above equation. When experimental data for H_{max} is not available, a value of H_{in} = $1.70{*}H_y$ is recommended.

The stiffness degradation of the column can be expressed in a similar manner, where K is the residual elastic stiffness:

$$K = K_E\, e^{-\left[\ln\left(\frac{H_{in}}{H_y}\right)\right]D} \tag{3-44}$$

where:
K_E = initial elastic stiffness

Both of the methods outlined above are empirical, and require experimental data to determine constants. The Evolutionary-Degrading Hysteretic Model appears to be the better of the two. Although more complicated, it is also a more rational general approach, and it has been demonstrated to give good results when calibrated to experimental tests. While developed for tubular steel piers, the approach of this model is applicable to other structural shapes and forms as well. A substantial research effort would be required to determine and verify the parameters most suitable for general design and analysis.

3.3 Strength and Ductility Evaluation of Connections

While there are numerous approaches to resist seismic loadings, most have a common requirement that failure should not occur in the connections. The reason for this is that connections failures are generally brittle fractures, and often they result in a loss of structural integrity. There are exceptions to this rule, however, if the connection can be demonstrated to have a ductile failure mode. Brittle failures can result when the connection itself is understrength relative to the adjacent member, or when holes in the connected materials reduce the net section to a point where net section fracture governs over gross section yielding, expressed by (AASHTO, 1994):

$$\varphi_y F_y A_g < \varphi_u F_u A_n \qquad \textbf{(3-45)}$$

where:
φ_y = resistance factor for yield in tension = 0.95
F_y = yield strength of steel
A_g = gross area
φ_u = resistance factor for fracture in tension = 0.80
F_u = ultimate strength of steel
A_n = net area

Even connections meeting the criteria of yield in the gross section can be vulnerable if plastification of the section results in additional secondary moments generated on the connection, or cyclic loading results in plastic hinging, local buckling, and low cycle fatigue in the vicinity of the connection.

Connection behavior is also important from the standpoint of structural analysis. In structures composed of many individual members, the connections perform a critical role in dictating the performance of the structure at both service and ultimate limit states. In seismic analyses, as opposed to conventional design or rating, it is important to determine actual behavior as closely as possible. It is not sufficient to make a conservative assumption and provide overstrength. In typical structural designs, connections are usually assumed to behave as simple (no rotational fixity - shear connections) or fixed (fully restrained against rotation - moment connections). In reality, while these ideal conditions can be approached, all connections fall somewhere in between, in the partially restrained realm. In seismic analyses, it is important to model the moment-rotation behavior of the connections as accurately as possible in order to calculate force distribution in the structure reasonably well.

3.3.1 Welded Connections

Welded connections are attractive to the designer because they closely approximate a fully fixed connection for analysis, and as noted previously, properly detailed welded connections can perform very well under cyclic earthquake loadings. In addition, the strength of a properly detailed welded connection can be easily calculated as the plastic moment of the section. On the other hand, inadequate welded connections can demonstrate very brittle behavior, as evidenced in the Northridge earthquake. While there are some relative guidelines assessing ductility of welded connections, such as those listed in section 2.4.1, there are few specific evaluation guides. Because of the complex inelastic cyclic behavior of welded connections, the most reliable method of determining the ductility of a weldment is to perform cyclic testing of realistic sized specimens (Astaneh-Asl, 1996a).

Unstiffened box beam-to-column connections in portal frames, illustrated in figure 3-4, were found to behave better if designed to yield in shear in the corner panel rather than in bending in the beam or column (Nishimura, 1992). Decreasing width-to-thickness ratios of the beam and column flanges can prevent local buckling prior to formation of the shear hinge. A width-to-thickness value of 30 for steel with a yield strength of 2400 kg/cm^2 (235 MPa, 34 ksi) was used in this work. The shear strength of the corner web was reasonably well predicted by:

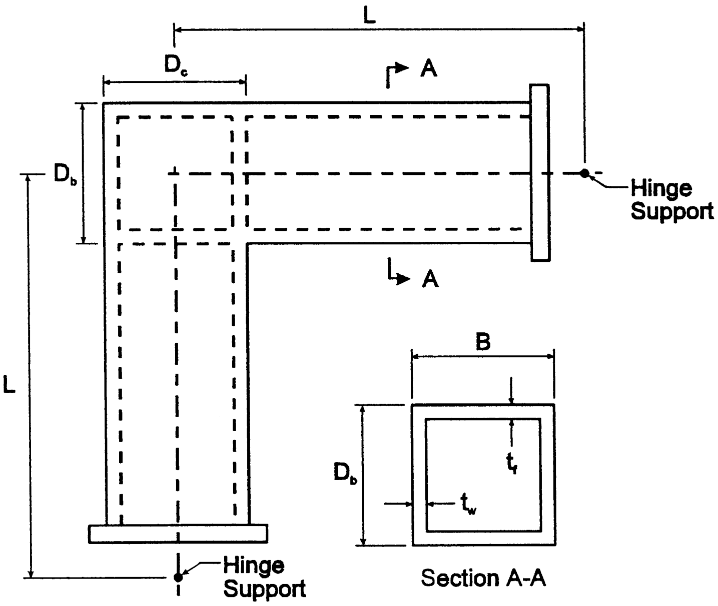


FIGURE 3-4 Box beam-column joint geometry

$$V_y = \frac{1.04 F_y t_w D_b D_c}{L\left[1 - \frac{(D_b + D_c)}{2L}\right]} \qquad \textbf{(3-46)}$$

where:
F_y = steel yield strength (kg/cm^2)
t_w = web thickness (cm)
D_b = beam depth (cm)
D_c = column width (cm)
L = distance from the intersection of the beam and column longitudinal centerlines (meeting at the box joint) to the point of zero moment (base of hinged column or inflection point in beam or fixed column) (cm)

Equation 3-46 represents the lateral (or vertical) shear force on the bent when the corner yields in shear. The panel zone shear due to the bending moment is included using the length to the hinge (L). The choice of L in the equation (if the column and beam L values are different) gives the value of shear in that member which results in yield. For instance, using the column length gives the yield shear in the column. The value of shear in the opposite member can be calculated by using the other L value, or using equilibrium principles. This research indicated that shear yielding in beam-to-column connections of frames may be a preferred mechanism for providing ductility and energy dissipation.

A more general form of the panel zone shear yield capacity, regardless of the source of shear loading, is calculated from (AISC-LRFD, 1994):

$$V_y = 0.6\ F_y D_c t_w \tag{3-47}$$

An additional multiplier used to take strain hardening into account is used in seismic applications where a panel zone yielding mechanism is expected to occur in seismic design resulting in a shear resistance of (AISC-LRFD, 1994):

$$V_n = 0.6\ F_y D_c t_w \left[1 + \frac{3 b_{cf} t_{cf}^2}{D_b D_c t_w}\right] \tag{3-48}$$

where:
b_{cf} = width of the column flange
t_{cf} = thickness of the column flange

3.3.2 Riveted and Bolted Connections

As stated previously, use of realistic cyclic moment-rotation relationships at connections is important in determining seismic response. The stiffness of a structure is a primary factor in determining the fundamental period of a structure, which greatly affects the seismic response. Depending on the geometry of bolted (or riveted) connections, practically the entire range of connection fixity can be achieved. The connections that cannot be adequately approximated with pinned or fixed conditions are referred to as semi-rigid, as illustrated in figure 3-5 (Leon, 1994). Some recent research has advocated the use of semi-rigid connections in seismic applications, and design and analysis methods have been proposed for several specific connections.

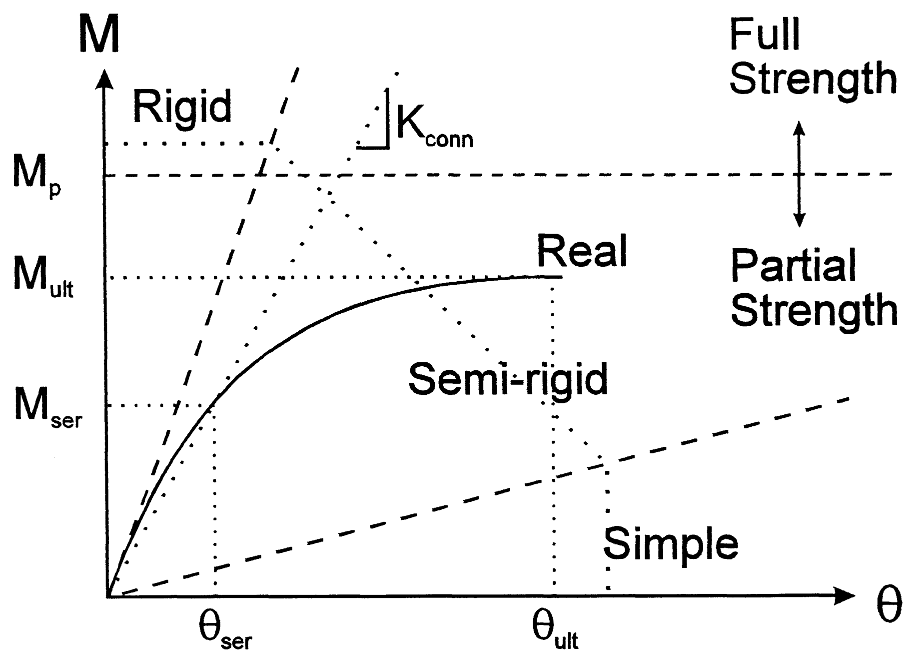


FIGURE 3-5 Moment-rotation curve for a typical connection (adapted from Leon, 1994)

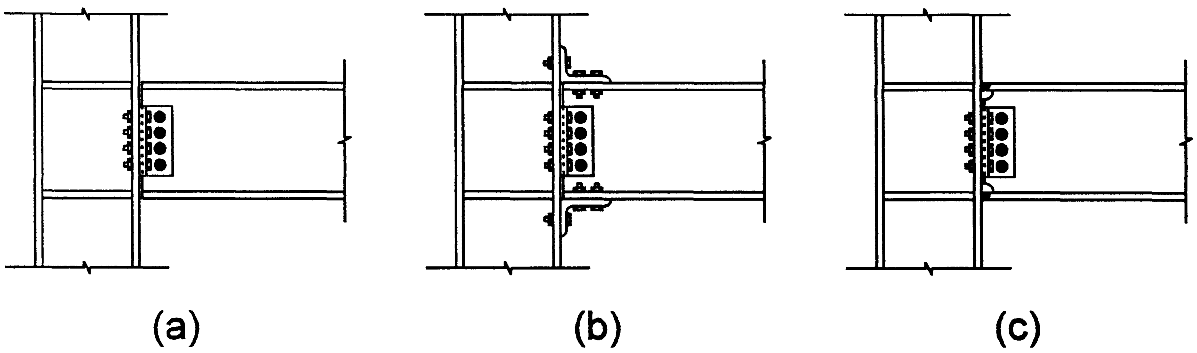


FIGURE 3-6 Flexible, semi-rigid, and rigid connections (adapted from Nader, 1996)

An investigation of the shaking table behavior of a one story, one bay steel structure with flexible, semi-rigid, and finally rigid beam-column connections was performed to study the effect of connection rigidity on frame response (Nader, 1996). The connections, illustrated in figure 3-6, show that the flexible connection consists of a double angle shear connection (figure 3-6a). To make the connection semi-rigid, top-and-seat angles were added (figure 3-6b). Finally, the top-and-seat angles were removed and the flanges of the beam were welded to the column with full penetration welds to form the rigid connections (figure 3-6c).

The semi-rigid connections demonstrated a three phase type of behavior. During the first phase, the connections have sufficient elastic stiffness to keep the structure stable under service loads and moderate earthquakes with almost no structural damage. As the inertial earthquake forces increase, the connections yield, reducing the stiffness and increasing the damping of the structure, thus reducing seismic forces. In the third phase, as lateral deflections become very large, kinematic hardening and development of catenary forces in the top and seat angles of the connection serve to increase the stiffness of the connections, thus preventing excessive drift. The semi-rigid connections were also found to be significantly more ductile than the rigid connections. The behavior of the steel frame was predicted reasonably accurately using the DRAIN-2D computer program with a model incorporating a semi-rigid element with a bilinear moment rotation relation. This research indicates semi-rigid connections may enhance the seismic behavior of certain steel building structures. Because bolted steel bridge connections have many similarities to these semi-rigid connections, further research should explore whether typical bolted steel bridge connections provide similar improved seismic resistance.

One of the obstacles to implementing semi-rigid connection design has been the lack of computer analysis tools to deal with the connection behavior. These tools are beginning to become more available and as they become common, designs incorporating semi-rigid connections should increase. Two connection models are utilized by the DRAIN-2D program. The first, the widely used bilinear rotational spring, requires only one variable per connection (figure 3-7a). The second, intended for top-and-seat angle connections, uses two bilinear hysteretic axial elements to model the two angles in tension and requires two variables per connection (figure 3-7b). The elements function as pins when the angles bear against the column. The static moment-rotation curves for these models, shown in figure 3-8a, are identical, although the cyclic hysteretic moment-rotation differ greatly, as illustrated in figure 3-8b. A study of these elements compared to test results of top-and-seat angle connections concluded that the rotational spring underestimated energy dissipation, but that it was

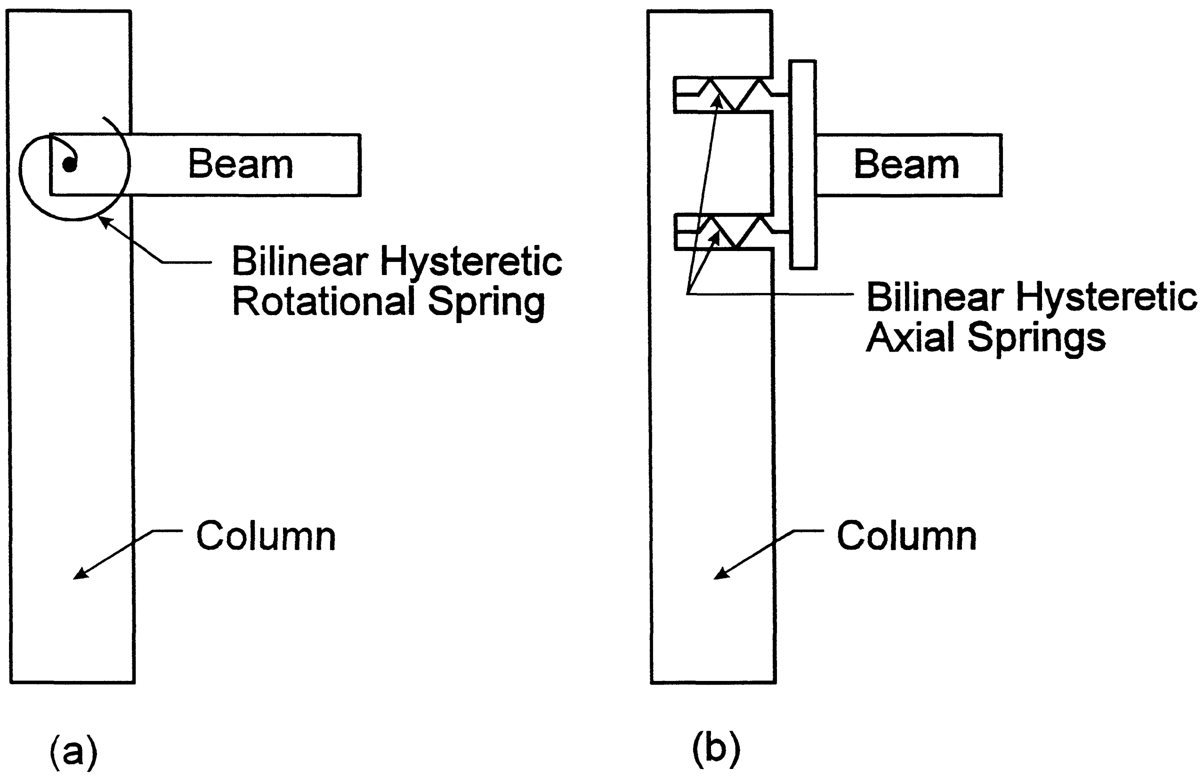


FIGURE 3-7 Analytical models of connections

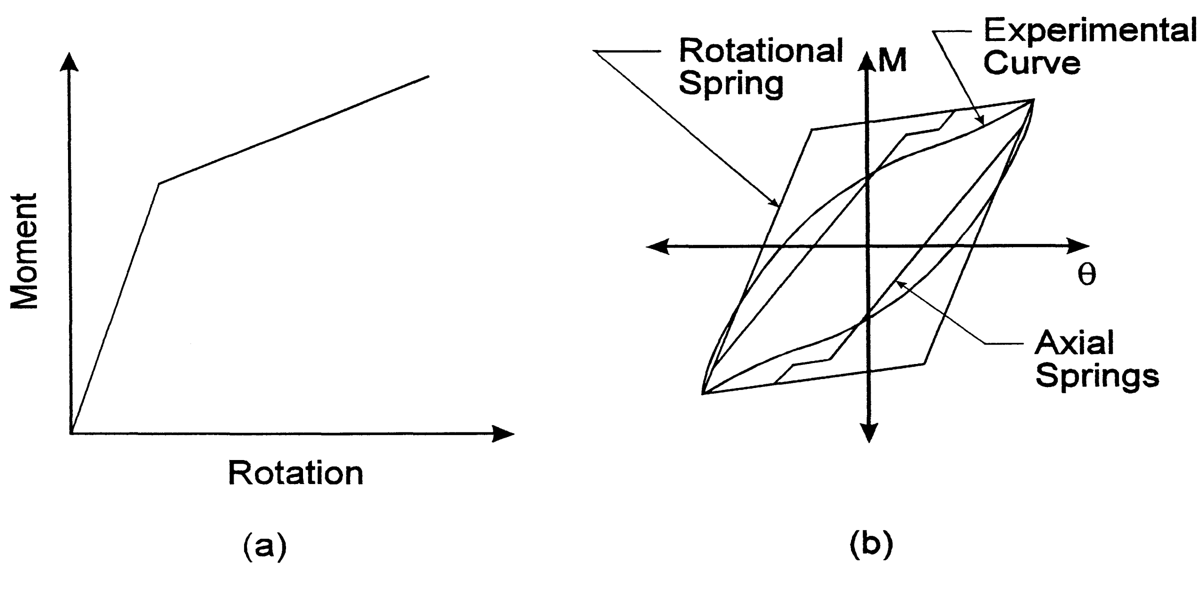


FIGURE 3-8 Comparisons of moment-rotation hysteresis

sufficient for modelling entire structures. The two axial elements overestimated energy dissipation, but did a better job of modelling individual connection behavior (Dicorso, 1989).

As far as seismic analysis is concerned, the main obstacle is the lack of dynamic cyclic moment-rotation relationships available to model structures. Most research data to date has been obtained using static monotonically applied loads. The resulting analytical representations of the moment-rotation curves, usually bilinear curves, approximate the behavior of static tests very well, and are more than adequate for standard design, but seismic designs require better models. Several analytical methods exist to calculate initial (elastic) stiffness (Azizinamini, 1987; Youssef-Agha, 1989; Kishi, 1986 from Mander 1994). The method by Youssef-Agha, while very general, is also fairly complicated. The method developed by Kishi and Chen, and reproduced in Mander, 1994, is simpler but assumes a center of rotation at m_1 as shown in figure 3-9b. The resulting equation for elastic stiffness, K_{je} is given as:

$$K_{je} = \frac{4EI_s}{l_{so}} + \frac{3EI_t}{1.0 + 0.78 \cdot \left(t_t^2 / g_2^2\right)} \left(\frac{d_1^2}{g_2^3} \right) \tag{3-49}$$

where:
I_s = moment of inertia of the seat angle
I_t = moment of inertia of the top angle
l_{so} = distance from the center of rotation to the tip of the angle leg on the beam flange
t_t = thickness of the top angle
d_1 = distance between centerlines of angle legs on beam flanges
g_2 = distance from top of beam to plastic hinge m_3 in top angle

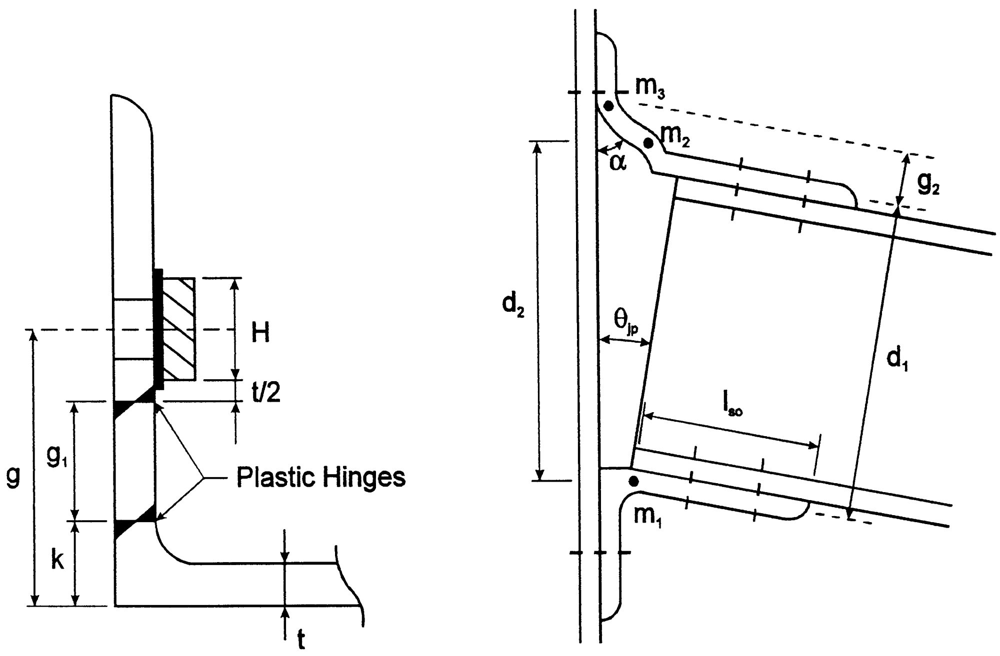


FIGURE 3-9 Definition of parameters for top-and-seat angle connections

Assuming plastic hinging in the connection elements as shown in figure 3-9b, the plastic moment capacity can be calculated using virtual work principles (Mander, 1994). If the top and bottom angles are the same, $m_1 = m_2 = m_3 = m = F_y bt^2/4$, and the plastic moment capacity, M_{jp}, is equal to:

$$M_{jp} = 2m\left(1 + \frac{d_2}{g_1}\right) \qquad \textbf{(3-50)}$$

where:
d_2 = distance between plastic hinges m_1 and m_2
g_1 = distance between plastic hinges m_2 and m_3

Note that in the above equation, the plastic moment capacity is heavily influenced by the value of g_1. The sensitivity of g_1 is such that bolt spacing, type of washer, and even orientation of the bolt head can affect the plastic moment.

The upper bound for the strength occurs when one of the apexes of the bolt head is pointing down, and the distance g_1 is a minimum with the yield line forming directly beneath the hardened washer. In the limit as g_1 approaches 0, shear capacity rather than flexural capacity of the angle controls the plastic moment capacity, resulting in the following equation:

$$M_{jp} = 2m\left(1 + \frac{h_b + k}{t}\right) \qquad \textbf{(3-51)}$$

where:
h_b = depth of the beam

The lower bound strength is obtained by assuming the yield line is able to penetrate up underneath the bolt, approaching the center of the bolt hole. This can occur if hardened washers are not present, such as when load indicator washers are used. In this case, $g_1 = g - k$.

Examining the results of tests reveals that in most cases a radial fan yield line forms beneath the bolt head and washer connecting the angles to the column, as illustrated in figure 3-9a. The nominal plastic moment capacity can be calculated by assuming the radius of the yield line around the bolts is equal to the washer radius plus t/2 and calculating an effective g_1, $g_{1,eff}$, (for 2 bolts) as:

$$g_{1,eff} = g_1 - \frac{\pi r^2}{b_{angle}} \qquad \textbf{(3-52)}$$

where:
r = washer radius plus half of the angle thickness
b_{angle} = width of the connection angle

To model the monotonic moment-rotation behavior of the top-and-seat angle connections, Mander et. al. utilized the Menegotto-Pinto (M-P) equation, which can accurately describe a curve with two tangents and a variable radius of curvature at the intersection (Mander, 1994). The basic form of the equation, depicted graphically in figure 3-10, is:

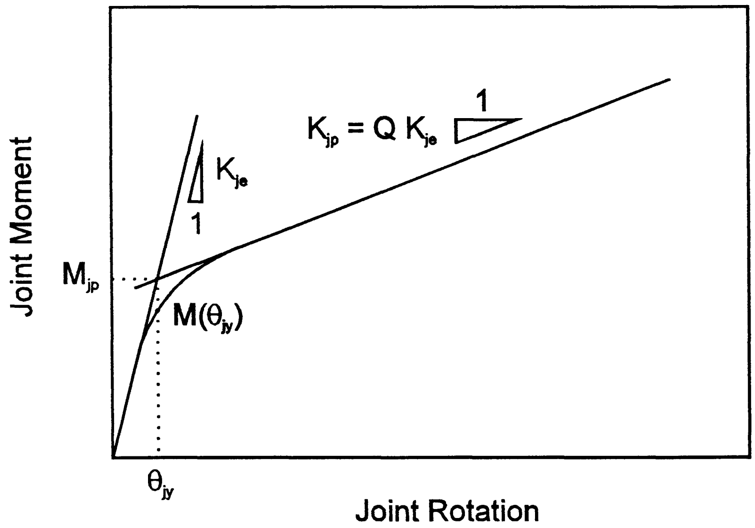


FIGURE 3-10 Menegotto-Pinto idealization (adapted from Mander, 1994)

$$M = K_{je}\theta_j \left[Q + \frac{1-Q}{\left[1 + \left(\frac{K_{je}\theta_j}{M_{jp}}\right)^R\right]^{\frac{1}{R}}} \right] \tag{3-53}$$

where:
Q = ratio of the plastic to the elastic stiffness of the connection, K_{jp} / K_{je}
R = a curvature parameter that can vary between 1 and 25, the higher value giving a bilinear curve, and calculated here as:

$$R = \frac{\ln 2}{\ln\left[\frac{1-Q}{\left(M\left(\theta_{jy}\right)/M_{jp}\right) - Q}\right]} \tag{3-54}$$

where:
K_{je}, K_{jp}, θ_{jy}, M_{jp}, and $M(\theta_{jy})$ are defined graphically in figure 4-6

This model does not account for the cyclic yielding, energy dissipation, and strength degradation characteristics, however.

Some testing and analysis has been performed on dynamic cyclic behavior of semi-rigid connections, and some general analytical models have been developed, but more research is required. An analysis method for composite semi-rigid connections has been developed by Leon (Leon, 1994), but this type of connection has little application to bridges. Most of the additional research on semi-rigid connections deals with top-and-seat angle connections, with or without web angle connections, such as that pictured in figure 3-11. While not necessarily exactly representative of bridge connections, results from tests on connections such as these can give insights into the behavior of some types of bridge connections made using bolted angles.

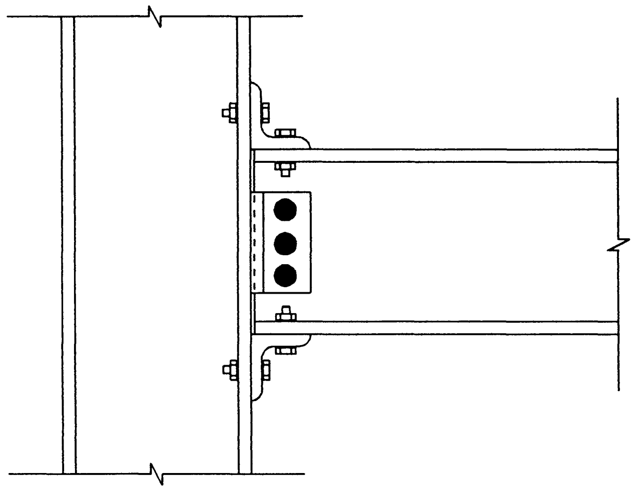


FIGURE 3-11 Typical top-and-seat angle connection with web angle connections

Research into the low-cycle fatigue resistance of top-and-seat angle connections concluded that the seismic fatigue limited plastic rotation capacity was on the order of 3%, which is well above the typical maximum demands of buildings, where drifts rarely exceed 2% (Mander, 1994). This research also developed some general equations for predicting low cycle fatigue using plastic connection rotation (analogous to the commonly applied strain) along with number of cycles, in applying the standard Manson-Coffin type of fatigue model. The plastic strain amplitude (θ_{jp}) can thus be related to the number of cycles, N_f, by:

$$\theta_{jp} = \theta'_{Nf} \left(2N_f\right)^c \tag{3-55}$$

where:
θ'_{Nf} and c are constants unique to a particular connection geometry
$2N_f$ = number of reversals to failure

The average cyclic energy per cycle, ΔW_f, which is measured as the integral of one hysteresis loop at midlife ($N_f/2$), can be related to the fatigue life by:

$$\Delta W_f = W'_{Nf} M_{jp} \left(2N_f\right)^d \tag{3-56}$$

Similarly the total hysteretic energy can be represented by:

$$W_f = W''_{Nf} M_{jp} \left(2N_f\right)^e \tag{3-57}$$

Furthermore, equations 3-55 and 3-57 can be combined to relate the total work to the constant plastic amplitude:

$$W_f = W'_{\theta f} M_{jp} \theta_{jp}^{f} \tag{3-58}$$

where:

W_{Nf}', W_{Nf}'', $W_{\theta f}'$, d, e, and f are constants unique to the connection geometry
M_{jp} is employed to keep the fatigue relationships independent of size (geometry and material strength)

While these equations can be calibrated for use with any connection geometry, a substantial research effort for each geometry would have to be made in order to develop a database large enough to establish constants for every case.

3.4 Strength and Ductility Evaluation of Bridge Superstructures

3.4.1 Trusses

Statically determinate truss superstructures are obviously vulnerable to earthquakes. Many older truss structures exist that were constructed at a time when little thought was given to redundancy, and the loss of one member could result in a catastrophic failure. Additionally, while determining the actual forces in all members is important, an analysis of structural behavior during a seismic event is difficult because members rarely respond as true truss elements due to connection fixity typically provided by large gusset plates. The feasibility and cost of any retrofit strategy must also be taken into account, as these are main load-bearing members, and traffic must generally be maintained during work.

3.4.2 Laced Members

Many older bridges were built using laced (also referred to as latticed or battened) built-up members. The lacing was used to increase the moment of inertia of the sections in order to prevent global buckling, while minimizing the amount of material required. There are a wide range of laced members, and several geometries of lacing, some of which are illustrated in figure 3-12. Two common geometries of lacing shown in figure 3-12b and 3-12c are X and V. From simple statics one can conclude that X lacing is more redundant than V lacing, and X lacing with perpendicular laces (figure 3-12a) is more redundant still. Often members have lacing in only one plane, with solid plates in the other plane, but some members are laced in both planes, with angles at the corners. Webs of I-shaped members, as well as those of box members, can also be laced.

The expected plane of buckling is important when analyzing laced members. Buckling out of the plane of the lacing results in little additional force on the lacing member, while buckling in the plane of lacing results in shear forces that must be transmitted by the laces. These shear forces can cause the laces themselves to buckle, contributing to strength degradation.

Until recently, the information on the cyclic behavior of laced members was very limited, especially the axial force - bending moment interaction. A static compression test of a truss chord from the Golden Gate Bridge (as illustrated in figure 3-13) indicated that while laced members can achieve their design strength, they may behave in a brittle-like manner in the post-buckling range (Bowen, 1996). For this reason there was great concern over the seismic vulnerability of laced bridge members, and consequently, a ductility factor of 1 was usually assigned to laced members in compression. This meant that a laced member was assumed to have failed if the seismic demand exceeded the calculated capacity of the member.

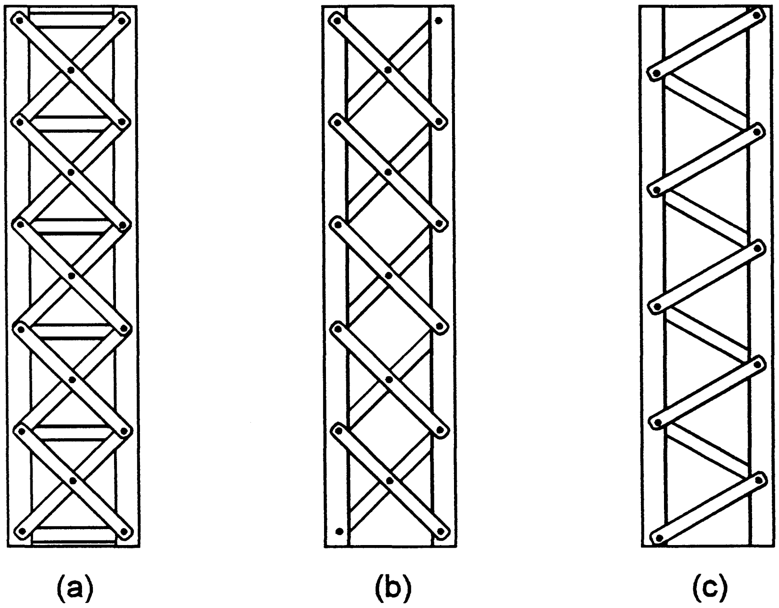


FIGURE 3-12 Several laced member configurations

A research program was initiated by Caltrans to develop a better understanding of cyclic hysteretic behavior of laced members (Astaneh-Asl, 1998). The axial force-moment interaction, as well as the cyclic post-buckling strength degradation, were two areas of focus of the research effort. A result of one of the tests is graphed in figure 3-14. The solid portion of the curve represents pre-buckling performance, the dashed portion of the curve is the post-buckling behavior. It is apparent that the compression capacity degrades quickly after buckling, but the tensile capacity degrades much more slowly. This indicates that laced members serving as redundant X-bracing may behave adequately as tension-tension bracing during seismic loadings, although the testing also revealed that differential distortion caused by individual buckling of the latticed member could reduce the member tensile capacity by up to 50%.

Otherwise, the testing found that the existing equations in the LRFD Specifications correlated reasonably well with the test results for both compressive strength and M-P interaction. The gusset plate end connections were found to be the weakest link in the system, with edge buckling and out-of-plane buckling of the gussets resulting in a relatively brittle failure mode. The equations in the LRFD Specifications were found to adequately predict these cases as well, however.

Computer finite element models were found to over-predict strength and stiffness slightly, and were unable to predict panel buckling locations due to local buckling randomness. Because finite element analyses are labor and computer time intensive, it would be impractical to perform such analyses for routine design, although they are useful as a complement to testing or for running parametric studies. Development of a generalized beam element which incorporates low cycle fatigue and net section properties would be sufficient to model member behavior for a nonlinear, global structural analysis.

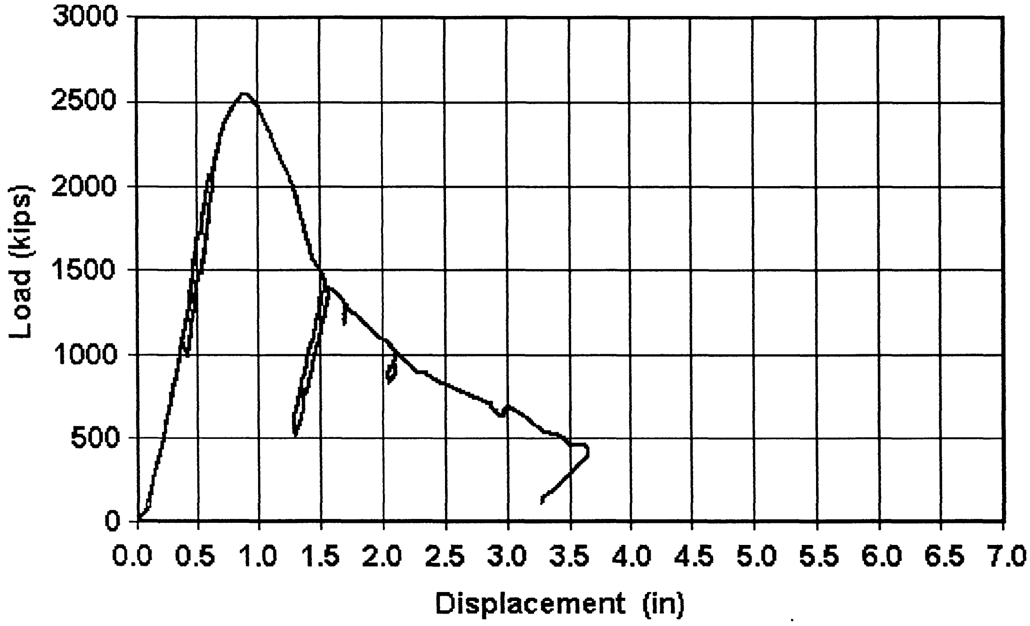


FIGURE 3-13 Axial load - axial deformation curve for laced column specimen (from Bowen, 1996)

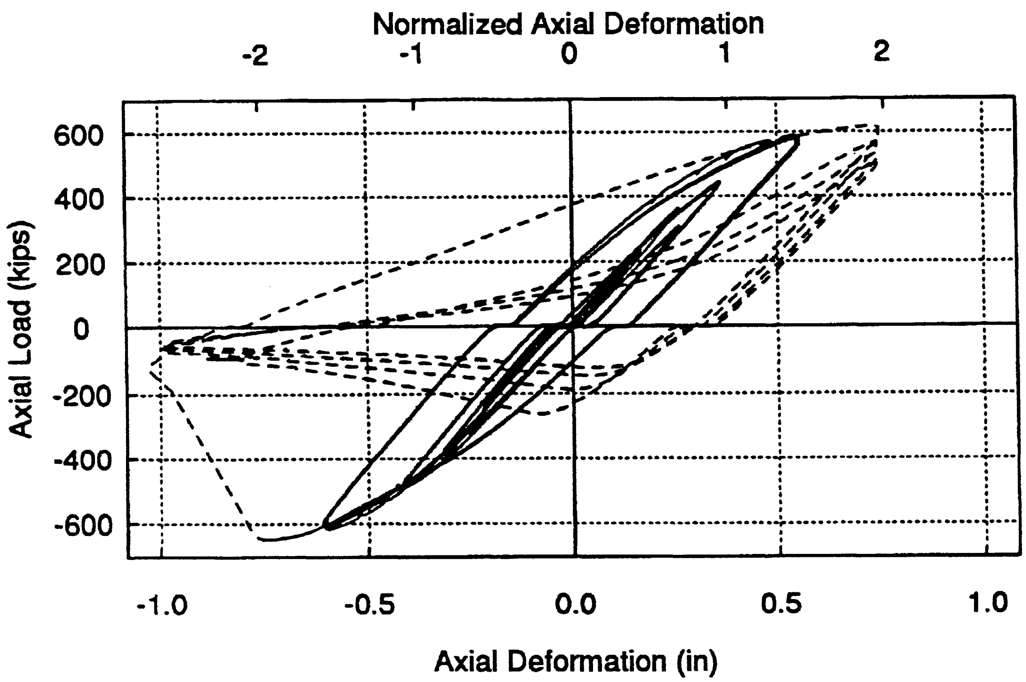


FIGURE 3-14 Axial load - axial deformation graph for laced member under concentric axial load (from Uang, 1997)

3.4.3 Slab-on-Girder Systems

The superstructures of slab-on-girder bridges have performed extremely well in recent earthquakes. As mentioned previously in Section 2.2.2, main load carrying members were not damaged in girder bridges. The steel diaphragms (or cross-frames), bearings, connections of the steel superstructure to the concrete substructure, and the concrete substructure itself, were the portions that sustained damage. Obviously the bearings and the anchorage to the concrete substructure are locations of vulnerability. These areas should ideally remain elastic and they would benefit from strengthening. The damage suffered by bearing diaphragms or cross-frames, rather than exposing a vulnerability, indicated that this may be a location where allowing yielding could supply a ductile "fuse," providing energy dissipation and limiting forces on, and preventing damage to, the remainder of the structure (Astaneh-Asl, 1994).

This proposal would seem to be at odds with the recommended provisions of ATC-32, where Article 10.20.4.3 Design Criteria specifies that "... Diaphragms and cross frames and their connections which are identified by the designer as part of the load path carrying seismic forces from the superstructure to the bearings shall be designed and detailed to remain elastic under all design earthquakes..." The suspicion is that this provision reflects the concrete seismic design philosophy that ductility be provided only at the top and bottom of pier columns and the rest of the structure remain elastic.

Recent research has confirmed that the bearing diaphragms are indeed the critical links and intermediate diaphragms have an insignificant effect on the seismic performance of bridges (Zahrai,1998). This same research indicates that supplying ductile end diaphragms in steel bridges can also be an effective approach for seismic resistance. Design codes should be updated to reflect these findings. It is unduly restrictive to subject steel bridges to the same provisions as concrete bridges because they have structural differences that can be reflected in different seismic resistance philosophies.

It would be convenient if "typical" existing bearing diaphragms and cross-frames possess the strength and ductility characteristics required to function as "fuses" with little or no retrofit required. Obviously, diaphragms from different bridges must be analyzed individually, but some general observations can be made about some "typical" diaphragms and cross-frames. End diaphragms and cross-frames from the Federal Highway Administration Standard Bridge Plans are reproduced in

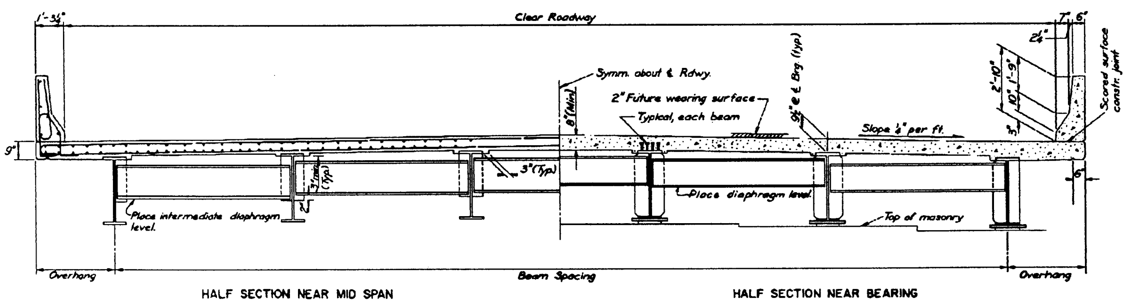


FIGURE 3-15 FHWA typical diaphragms

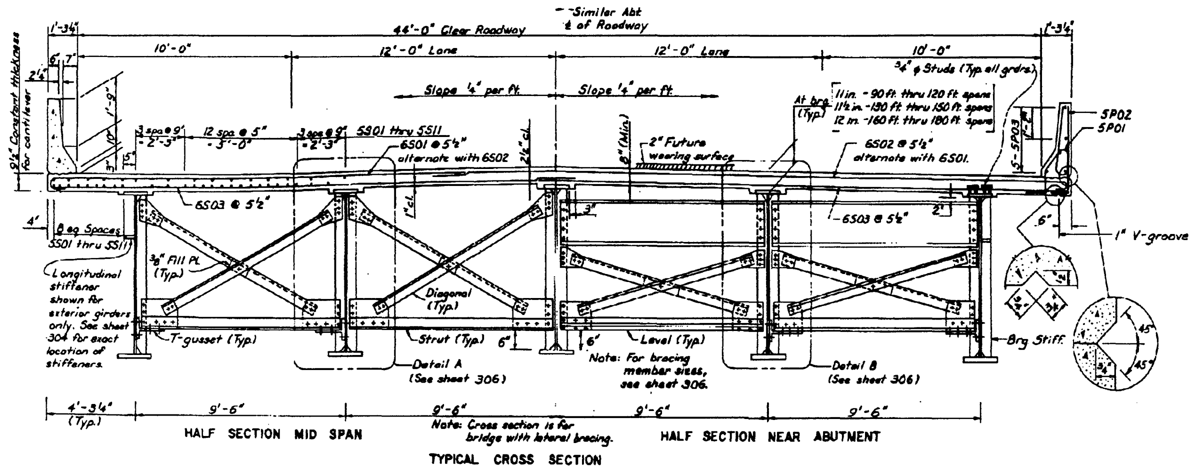


FIGURE 3-16 FHWA typical cross-frames

figures 3-15 through 3-18. Additionally, standard details from several various states are contained in Appendix A.

Inspection of the standard cross-frames details reveals that generally the top flange strut consists of a channel or wide flange section. The bottom flange strut is often a tee section or an angle section. The diagonals are generally angles. X and V-bracing configurations are most common, although other more elaborate configurations are used also. X-bracing is more ductile due to the redundancy of the system. The ductility of the V-bracing is dependent on the bending resistance of the strut when the compression leg buckles. It follows that V-bracing that connects to the top strut, i.e., inverted V-bracing would behave better than V-bracing framed into the bottom strut. See Sections 2.4.2 and 3.5 for more on the relative ductilities of X and V-bracing.

In order to take full advantage of the ductility of the bracing members, the connections must be adequate. In some standard cross-brace designs, only the minimum two bolts are supplied for connections. For most cases, two bolts will be inadequate to develop the full tensile strength of the braces, and more bolts should be provided. The eccentricity of the angle connection to the gusset should be taken into account as well. Removing the material for a single line of bolts on an angle should not be enough to cause fracture in the net section, however, and yielding in the gross section should continue to be the mode of tensile failure.

Stability will control the resistance of the diagonal braces in compression. Global buckling will occur before yielding in compression. In addition, width-to-thickness ratios of angle legs are often not quite sufficient to reach yield before local buckling occurs. It follows that the brace will buckle globally with local buckling occurring at the hinge location(s). Based on research of angle braces in buildings, local buckling greatly decreases cyclic ductility of angle bracing (Goel, 1992a). This indicates that existing "typical" bearing cross-frames will behave poorly under cyclic loadings. Improved cyclic performance would be gained by increasing width-to-thickness ratios for the bracing angles. For more on the ductility of angle bracing see Section 2.4.2.2 on concentrically braced frames.

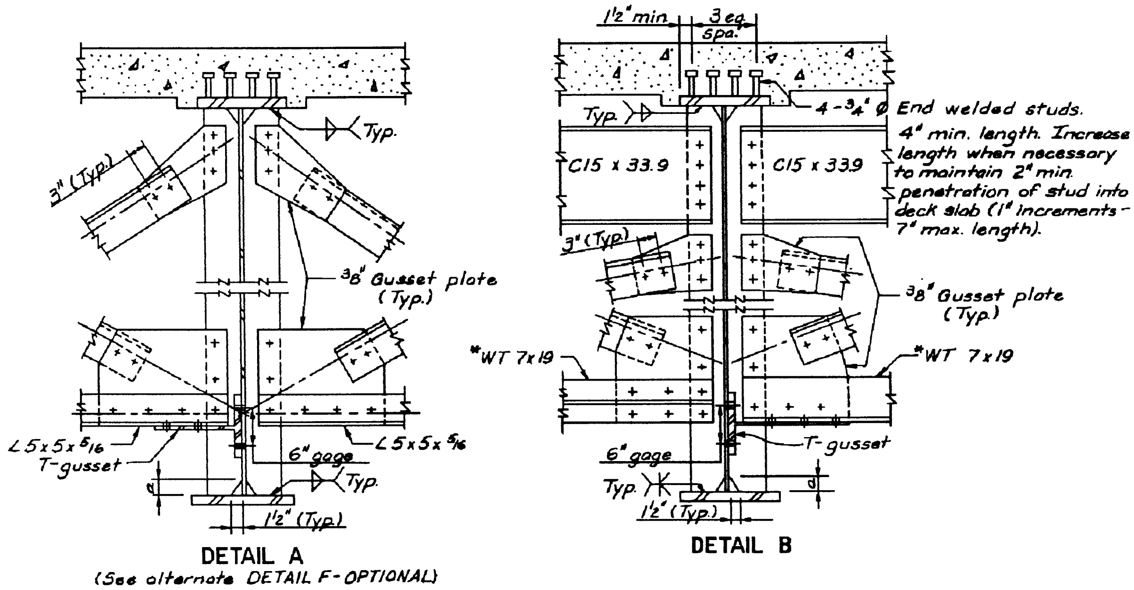


FIGURE 3-17 FHWA typical cross-frame details

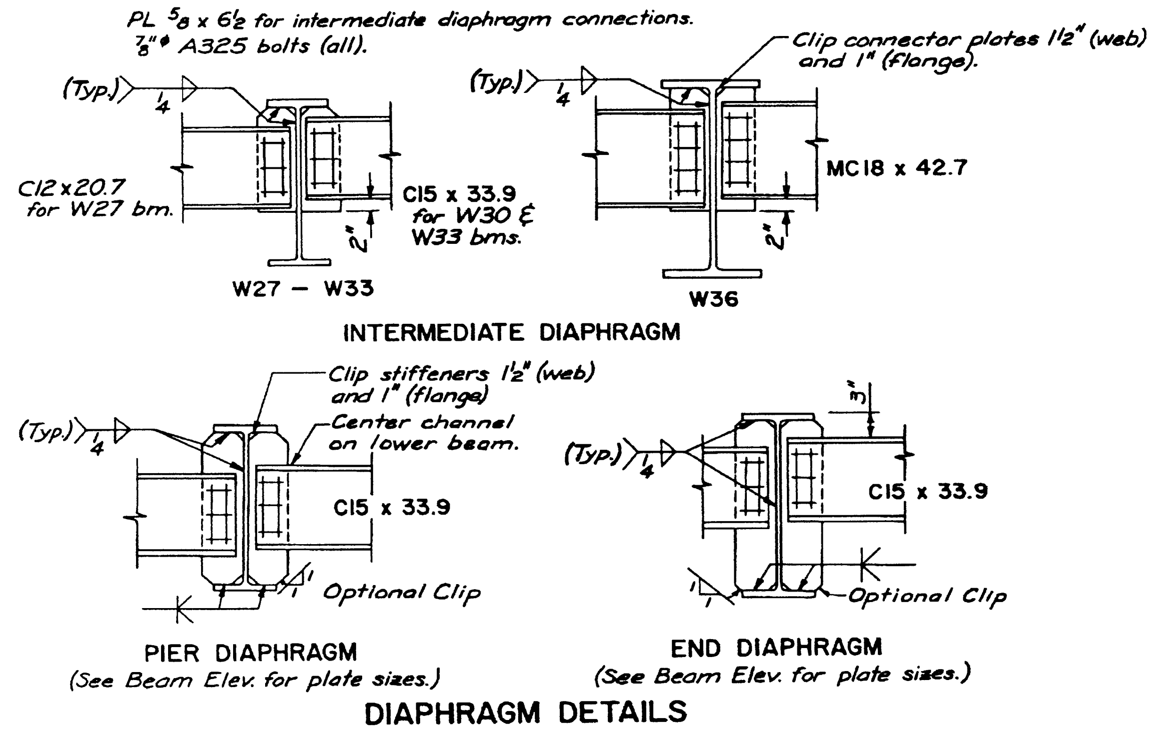


FIGURE 3-18 FHWA typical diaphragm details

3.5 Strength and Ductility Evaluation of Steel Towers and Tower Bents

There are many existing bridges whose substructures consist of steel towers or steel bents extending from concrete foundations up to a steel superstructure. These towers or tower bents may be constructed of single or multicellular steel boxes or open shapes such as wide flange sections and built-up latticed members. The types of members and the geometry of the tower are the main factors in determining the structure ductility, assuming the connections are adequate.

One fairly simple method of determining structure ductility is the pushover analysis. In this inelastic analysis method, a structure is subjected to ever increasing displacements until it can no longer support its gravity loads. This method is particularly appropriate for steel bridge structures, since the bulk of the mass of the bridge, and consequently the seismic force response, is concentrated at the concrete deck level. Thus, by pushing at the level of the deck, the ductility of the tower can be found. A two-dimensional lateral pushover analysis was used to examine two existing steel tower bents with two common bracing configurations.

Steel tower bents from two bridges were examined. The first was an X-braced bent from the First Blue Water (BW1) bridge over the St. Clair River, connecting Port Huron, Michigan and Point Edward, Ontario, shown schematically in figure 3-19a. The second was a chevron or V-braced bent from the Greater New Orleans No. 1 (GNO1) bridge over the Mississippi, shown schematically in figure 3-19b. Both of these bents were constructed using wide flange shapes for the columns and built-up latticed members for the bracing members.

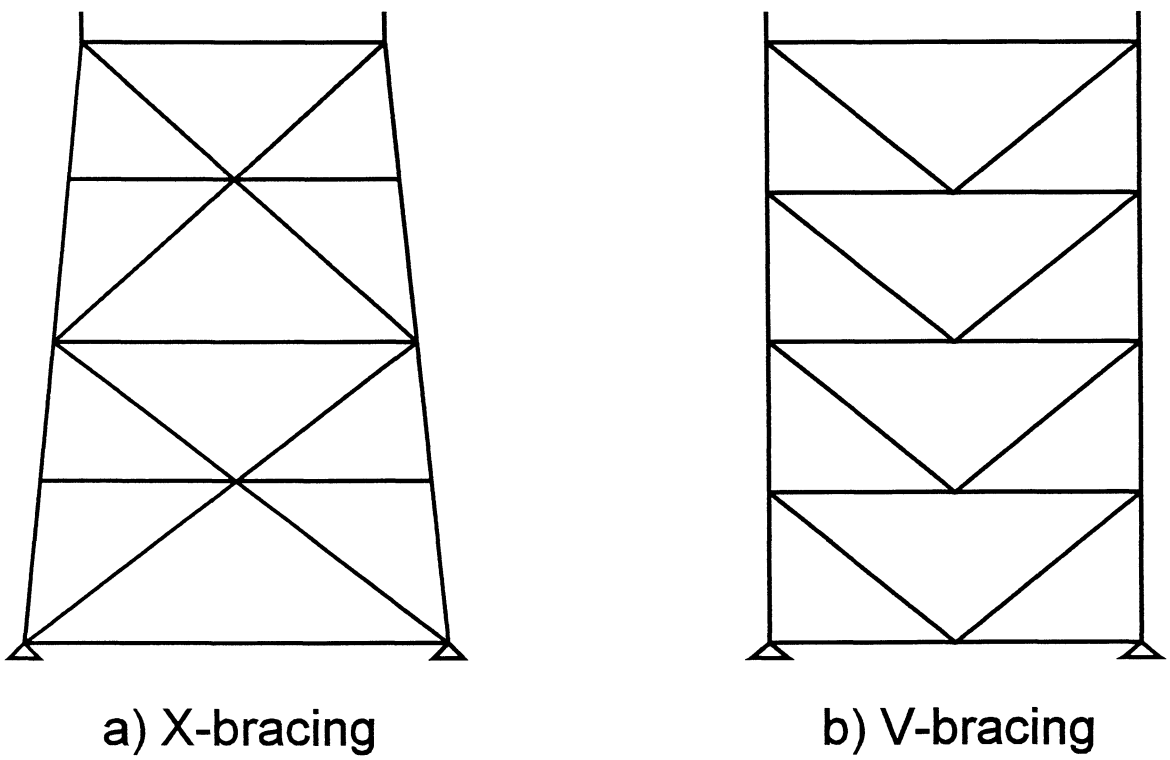


FIGURE 3-19 Tower bent geometries used in pushover analyses (not to scale)

The force-deflection behavior of the top of the bents is graphed in figures 3-20 and 3-21 for BW1 and GNO1, respectively. The inelastic analyses were conducted using the DRAIN-2DX program. The X-braced bent results show separate curves for cases with and without P-delta effects. For the V-braced bent, the difference between the two curves was insignificant. The elastic degrading analysis was conducted using SAP90, a linear elastic analysis program. The linear degrading analysis was performed using a step process that either eliminated members as they failed, or replaced yielded members with a constant force. Buckled members were assumed to have no post-buckling capacity in the linear elastic analysis, and were assumed to quickly degrade in the inelastic analysis. In the elastic analysis, ultimate load was defined as when the column axial force-bending moment interaction resistance equation reached its limit.

The shapes of the two curves confirm what one would predict from simply examining the geometries of the two bracing schemes. The redundant X-braced bent suffers buckling in the two compression braces almost simultaneously, at which point the tension braces become the sole lateral force resisting system. The stiffness of the structure is also altered at this point. The ultimate strength of the bent reaches about twice the buckling load of the compression diagonals.

The V-braced bent behaved linear elastically until the top compression brace buckled. The upper strut and both columns were immediately overloaded and the entire upper panel formed a mechanism. The ultimate strength is essentially equal to the initial brace buckling load. The lower three panels remained linear elastic and suffered no damage.

It is apparent that the redundant X-bracing scheme is more effective in resisting earthquake forces. The damage is distributed more evenly over the structure, and the progressive failure resulting in reduced structure stiffness is also more desirable. The non-redundant V-bracing scheme, on the other hand, likely suffers from an undesirable "story mechanism" as soon as one of the compression braces buckles, since the adjacent strut will probably not be able to carry the increased demand. Although the inelastic analyses give an indication of the ultimate lateral load, they do not result in a prediction of the ductility ratio, since local buckling and cyclic effects are not included. In order to provide this information, an estimate of the rotation capacity of the various members and details would be needed.

3.6 Summary of Critical Issues and Research Needs on Seismic Evaluation

There are a great number of existing bridges which were built before the current understanding of seismic risk. It has become apparent that some of the assumptions made in the past were unreasonably optimistic. In addition, designing for loads and designing for ductility require different philosophies, so conservative assumptions in strength design are not necessarily compatible with a ductile design approach.

In general, the methods available to calculate the strengths of members are very good. What is lacking is accurate characterization of the cyclic post-yield and/or post-buckling behavior of many structural members. In the absence of reasonable evidence to the contrary, engineers are forced to make the conservative assumption that there is no member ductility, and member failure is assumed to occur when capacity is exceeded. A cumulative damage index approach, such as the evolutionary-degrading hysteretic model developed for tubular steel piers, would be one potential approach to solve this problem, although a substantial research effort would be required to experimentally characterize various other bridge members. The ideal solution from a designers point of view would

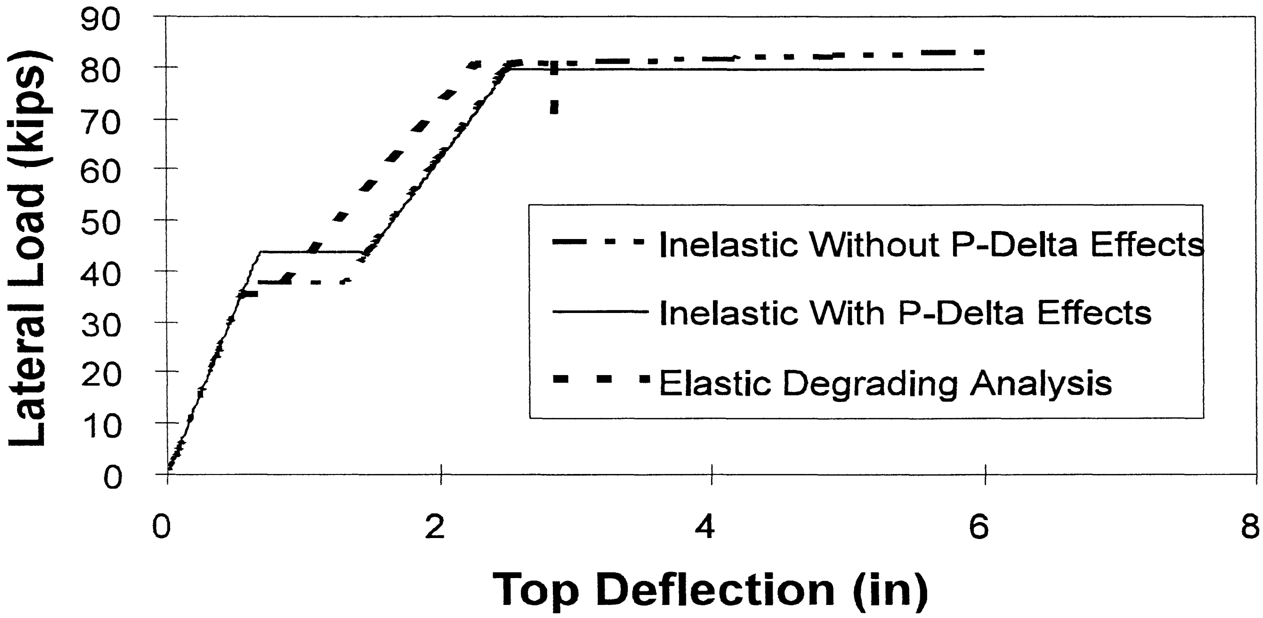


FIGURE 3-20 Lateral load versus deflection from pushover analysis of X-braced frame

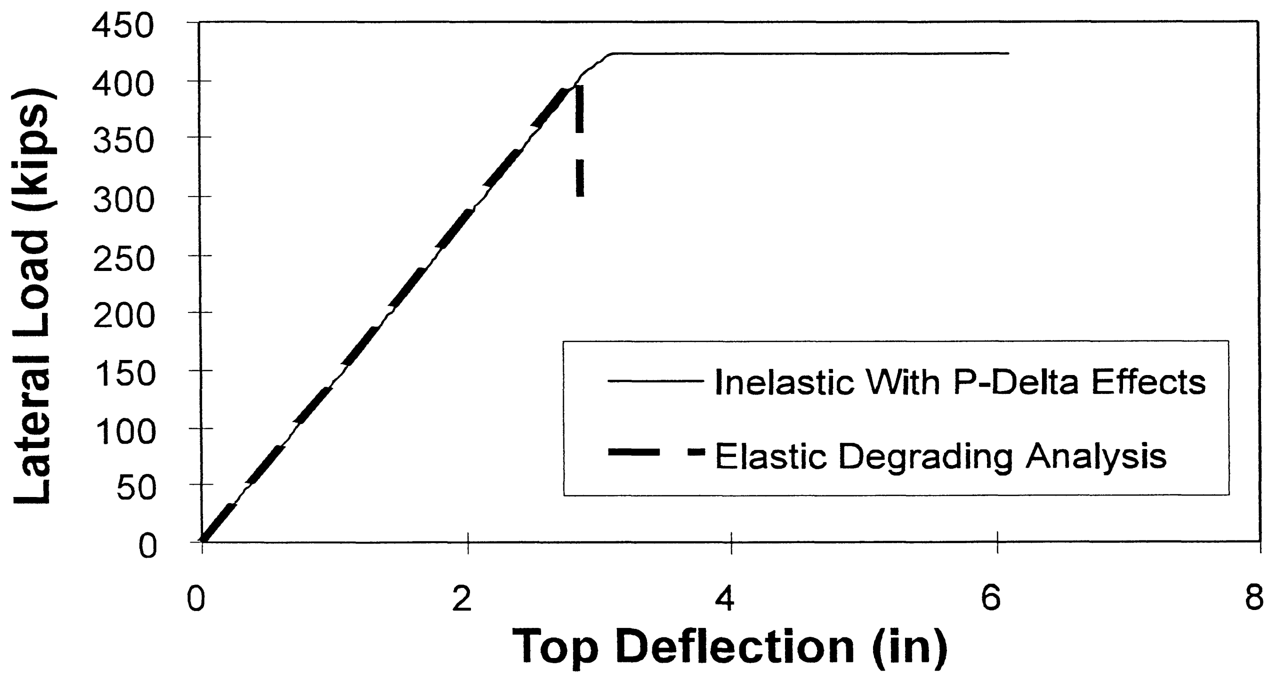


FIGURE 3-21 Lateral load versus deflection for pushover analysis of V-braced frame

be the development of physical theory models, which only require material properties and common geometric or derived engineering properties in order to characterize member behavior. An important step toward developing general analysis models would be to determine width-to-thickness and slenderness rules for seismic cyclic post-yield and post-buckling behavior.

Connections continue to cause problems. While semi-rigid, ductile connections have the potential to provide good seismic resistance, and even act as ductile fuses to protect adjacent brittle members, lack of reliable design and analysis techniques have limited their use in this capacity. In addition, better modelling of the stiffness of connections is needed to accurately analyze structural response. To date, engineers have mostly assumed connections to be fixed or pinned in analyses, and the typical retrofit approach is to strengthen connections or weaken the connected members such that inelastic behavior is forced into the adjacent members. Such is the approach of the post-Northridge moment connections. The work on nonlinear behavior of top-and-seat angles is a good start, but better analytical methods to characterize the moment-rotation behavior and cyclic ductility of typical existing connections in bridges is needed to better model joint behavior.

Great strides have been made in recent years in the seismic evaluation of bridge structures. It is expected that as the database on ground motions continues to expand, more accurate seismic loadings for structures will be developed. Analysis techniques should continue to improve also. Easy to use non-linear analysis techniques need to be developed that can be used in the typical design office. While non-linear computer programs exist, their use is not widespread due to their expense or user-unfriendliness. In addition, even if computer programs were widely available, the dynamic, non-linear, cyclic behavior of steel members and details needs to be better documented since it has been demonstrated that static testing is often not appropriate for characterizing dynamic cyclic behavior. The Caltrans research program should go a long way toward providing the necessary information required for laced compression members, but many other types of members exist.

The "pushover" method has been developed as a relatively simple technique to give an idea of the lateral post-yield and/or post-buckling behavior of bridge structures. This method was used here to examine two different configurations of tower bent bracing. This technique is only as good as the assumed member and connection behavior, however.

SECTION 4
SEISMIC RETROFIT MEASURES FOR STEEL BRIDGES

4.1 Scope

In this section, currently accepted as well as proposed retrofit measures for steel bridges will be described. This includes piers, connections, superstructures, and tower bents. Some recently introduced innovative retrofit techniques are also explored. A summary of critical issues and research needs is also provided.

4.2 Retrofit Measures for Steel Piers

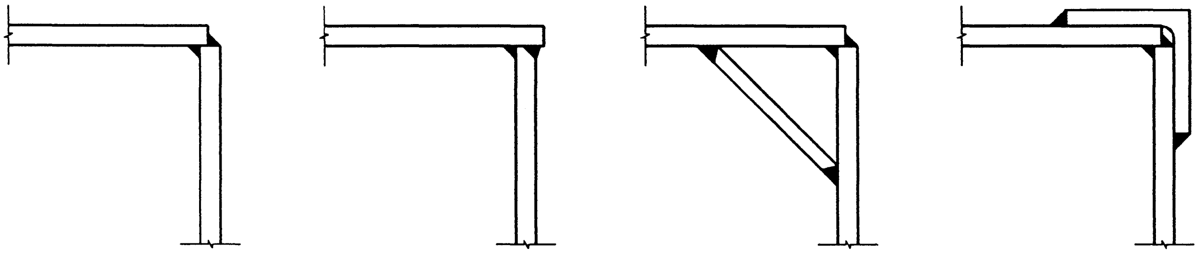
Because of the prevalence of steel bridge piers in Japan, and their demonstrated vulnerability in Kobe, the Japanese have embarked on an intensive research program into the behavior of steel bridge piers with a variety of different details. Some of the results of this research are presented here, although additional comprehensive design and retrofit rules for tubular steel bridge piers should be forthcoming.

Interim recommendations pending the completion of this research program were made. To avoid buckling problems with steel piers in new construction, use of thicker plates (smaller width-to-thickness ratios) and stiffeners that are at least 3 times as rigid as the minimum necessary to prevent elastic buckling were recommended (Watanabe, 1996). To date, width-to-thickness ratios have not been increased, however. In addition, limiting axial compressive stress to 1/10 of yield in order to reduce the P-delta effect was proposed (Watanabe, 1996). It was suggested that existing steel piers be filled with concrete as a retrofit (Watanabe, 1996).

Low cycle fatigue prone details at the corners and bases of portal frames can be avoided, or can be reinforced in new construction, while in existing piers these areas can be reinforced to shift the inelastic behavior elsewhere. It may also be possible to remove material from elsewhere in the structure to "soften" the response at these points of stress concentration and shift the zone of inelastic behavior away from the welded regions.

The buckling problems associated with steel piers in Kobe illustrate that width-to-thickness ratios that are sufficient for statically applied forces may be inadequate when subjected to high cyclic seismic forces. There are many steel pier bents and tower bents in the US that are constructed of rolled and built-up sections utilizing thin plates, channels, angles, or tees, often laced together. These sections may be vulnerable to local buckling prior to, or soon after yielding, when subjected to strong seismic loads. This could result in severe strength degradation, and research should be performed to determine the behavior of typical steel bents under cyclic forces.

Some research performed prior to the Kobe quake seems to have predicted the behavior of steel bridge piers in the quake, and has been used in formulating interim design recommendations in Japan. Work by Usami predicted the failure of piers caused by unzipping of the corner welds (Usami, 1992). Kawashima showed that significant out-of-plane deformation was concentrated in one direction in the first inelastic excursion, after which little additional displacement accumulated, and the column did not experience load reversal (Kawashima, 1989; 1992a). This work also showed that post-



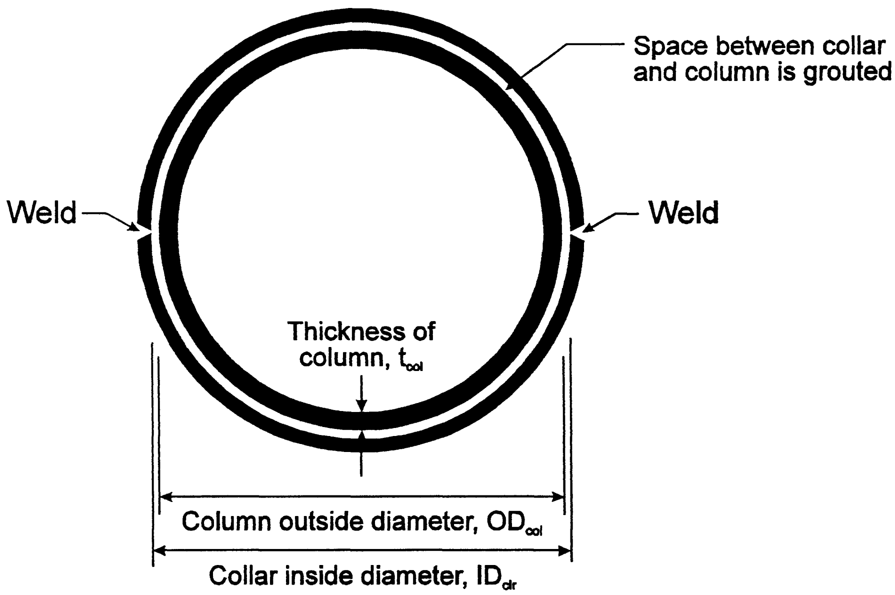
a) Former Detail
b) New Detail
c) Reinforcements for new or existing

FIGURE 4-1 Box column corner details

buckling strength degradation occurred after only a few cycles of loading (Usami, 1992; Kawashima, 1989; 1992a).

Several steel bridge pier repair and/or retrofit techniques have been developed by the Japanese in the wake of the Kobe quake. In order to avoid weld fractures at the corners of rectangular piers, the weld detail has been changed from two fillet welds to an exterior partial penetration weld with an interior fillet weld as illustrated in figure 4-1a and b. Reinforcing techniques include adding a fillet welded diagonal plate in each corner, or welding an angle on the exterior of the column, as shown in figure 4-1c. Partial height circular steel collars are also being used to retrofit circular tubular columns as illustrated in figure 4-2. A space equal to about half the column wall thickness is provided to allow for tolerances and fit-up. This space is then filled with grout.

To increase ductility, rectangular and circular columns are also being retrofitted with internal stiffeners. In rectangular columns existing internal longitudinal stiffeners are being retrofitted by welding eccentric "flanges" onto the outstanding legs, or in other words stiffening the stiffeners.



$ID_{clr} = OD_{col} + t_{col}$

FIGURE 4-2 Circular collar column retrofit

Additional transverse diaphragms are being welded inside rectangular piers as well, and longitudinal stiffeners are being added to the interior of circular piers (Tajima, 1998).

The Japanese are also using concrete infill as a retrofit for tubular steel piers. Of the limited research performed so far, conflicting results have been reported. While it is accepted that concrete infill improves strength, the effect on ductility is not as clear. In one study (Kawashima, 1992b), it was found that concrete filling reduced the ductility of tubular steel pier members. This contrasts with recent work by Ge and Usami, showing that concrete filling improved both ductility and energy absorption capacity, due to the reduction in the strength deterioration of the member (Ge, 1996; Usami, 1992). Concrete-filled circular steel columns have been found to behave better than concrete-filled rectangular steel columns (Kitada, 1992). Principles of reinforced concrete can be applied to concrete-filled steel tubes to calculate ultimate strength, but concrete design equations fail to account for the confining nature of the steel tube, and they also fail to address ductility and energy dissipation.

The research by Ge and Usami shows that providing a steel diaphragm above the concrete fill in the column improves performance (Ge, 1996; Usami, 1992). Performance was also improved by filling the columns to a greater depth. Filling columns at least halfway up is recommended. Based on equal energy concepts, and defining the collapse point as 95% of the maximum load, a force reduction factor (Japanese definition) of 0.3 was found to be reasonable. The US equivalent would be the reciprocal of this, or 3.33, so adopting a value of 3.0 for a US code force reduction factor, R, would be appropriate for concrete filled tubular steel piers.

Another application of concrete infill is to only place the concrete in a relatively limited depth at the base of columns. This concrete can serve a variety of purposes. Several of the steel tower pier retrofits in the California seismic program provide concrete in the base of the piers. In the San Mateo-Hayward bridge, concrete is to be added to the inside of the columns in the bottom portion to aid in transferring load from new shear pins into the column, increase local stability of the column walls below new column reinforcing plates, and provide a gradual transition of strength and stiffness (Prucz, 1996). In the Richmond-San Rafael bridge, concrete will be placed in the bottom portions of the pier legs to prevent local buckling in the plastic hinge region. In order to avoid stiffening the legs, the concrete will be placed in layers separated by a compressible joint material (Vincent, 1996). If a gradual transition of strength and stiffness can be achieved, placing concrete in the base of steel piers can be effective in moving inelastic action away from vulnerable welded base plates, stiffeners, and anchor bolt connection devices.

While open sections cannot be filled with concrete, concrete encasement of these members is an option. Reinforced concrete design principles are applicable in such situations. A study by Caltrans investigating a possible retrofit of the east spans of the San Francisco-Oakland Bay bridge proposed encasing many of the laced steel tower members as shown in figure 4-3, in order to increase their stiffness (Sundstrom, 1996; Maroney, 1996). Another technique, that changes the appearance of a structure even less, is partial concrete encasement.

Tests of partially encased H-shaped members, cross-sections of which are shown in figure 4-4, have been performed (Takanashi, 1992). Figure 4-4a illustrates what the researchers refer to as the "conventional" encasement reinforcement scheme. The reinforcement of this scheme consists of four longitudinal bars along with bent confining stirrups welded to the web of the steel section. A

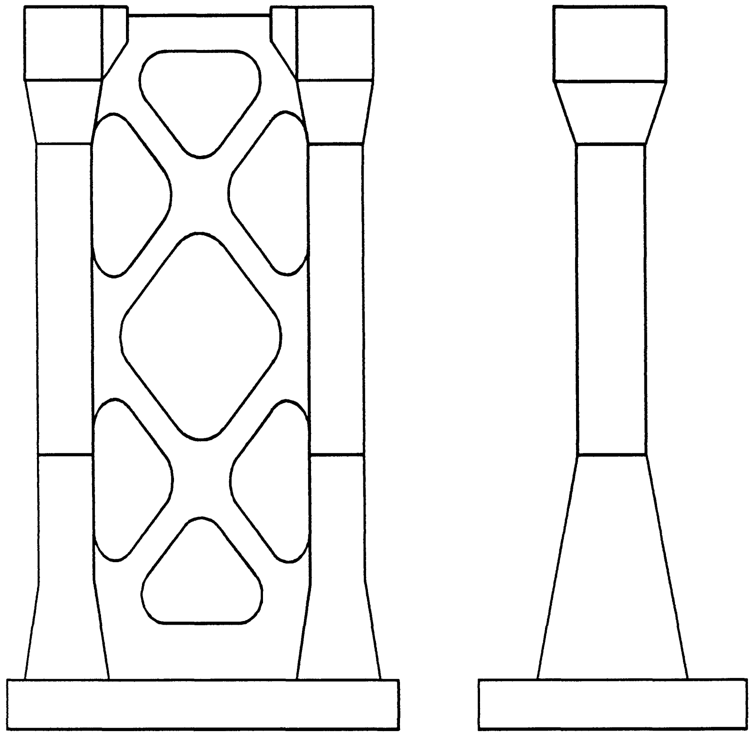


FIGURE 4-3 Appearance of steel pier after concrete encasement (adapted from Maroney, 1996)

reinforcing scheme referred to as "modified" is pictured in figure 4-4b. This scheme includes transverse bars welded between the flanges of the column. These transverse bars, designed to providing additional stability to the flanges, also enhance the composite action of the concrete by acting as shear connectors.

Results reported indicate that both reinforcing arrangements increased the strength of the composite column. The modified reinforcing scheme provided 50-80% better ductility than the conventional scheme, while no detrimental strength or stiffness increase was observed.

4.3 Retrofit Measures for Steel Connections

4.3.1 Welded Connections

Based on recent research, welded connections with low toughness weld metal and without continuity plates are the most vulnerable to seismic loadings (see Section 2.4.1 for proposed Northridge retrofits). Consequently, continuity plates should be required and minimum toughness requirements should be established for weld metal to be used in connections expected to experience high cyclic ductility demands. As mentioned previously, however, full scale dynamic cyclic tests are presently the only way to verify the structural ductility of a welded connection. One alternative to full scale tests is to shift the ductility demand away from welded connections if possible.

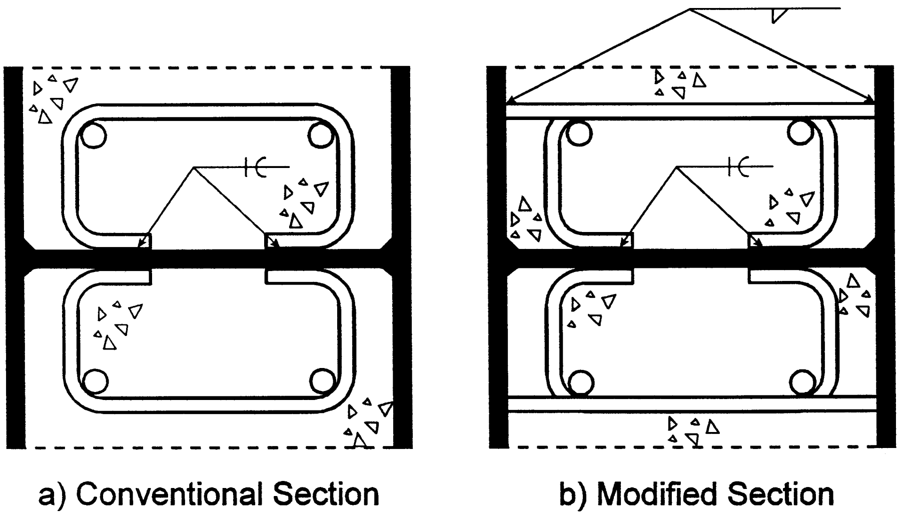


FIGURE 4-4 Cross-sections of partially encased steel columns

4.3.2 Riveted Connections

Many older bridges exist that utilize riveted connections. Testing has demonstrated that rivets display good ductility, with a ductility ratio of about 50, when subjected to tension or tension plus shear. The ductility ratio drops to about 4 in pure shear, however (Astaneh-Asl, 1996b). The strength of the rivets as predicted in current codes was near the yield point of the specimens tested, and the ultimate strength was about 2.5 times the yield strength (Astaneh-Asl, 1996b). The accepted practice to increase the strength of riveted connections is to replace the rivets with high strength bolts. Although bolts and rivets both possess the ability to redistribute loads due to their indeterminate nature, some ductility may be sacrificed when rivets are replaced with high strength bolts, since the ductility ratio of high strength bolts is not as high as that of rivets.

4.3.3 Bolted Connections

High strength bolts are the current fastener of choice for field connections on bridges in many states. The main reasons are that quality control and inspection are not as critical for bolted connections as they are for welded connections, and fatigue is not as big a concern with bolted connections. While not as ductile as rivets, replacing rivets with high strength bolts can increase connection capacity in cases where the rivets are the "weak link". Replacing rivets with similar diameter high strength bolts is a well accepted practice in situations where rivets are removed for any reason. At the same time the rivets are replaced, additional plates or angles can be added to increase connection strength.

For some researchers, there appears to be reasonable concern about the state of slip-critical bolted connections following several cycles of inelastic response. The concern is not over the inelastic action per se, but over the potential loss of clamping force that would occur if the material inside the joint undergoes enough inelastic action for the Poison effect to cause a significant reduction in plate thickness. While not necessarily a strength concern, the loss of clamping force would result in a reduction in the live load fatigue resistance of the connection. Although re-tightening and/or replacing the bolts to restore the clamping force would be one solution, the prospect of performing this task on the number of bolts present on even a medium span structure seems relatively impractical. Several other options may be feasible, however:

- A connection considered a Category C detail for finite life, and a Category D detail for infinite life, that is adequate for the service life of the structure, could probably be regarded as unaffected by the loss of clamping force. This would be especially true if the connection were acceptable for both classes of fatigue as a Category D detail. A post-event screening process based on anticipated remaining life and the age of the structure could be used to identify unacceptable members.
- Pre-event screening using the above criteria could also be used to identify acceptable and unacceptable members that could be marked on the contract drawings for reference after an earthquake.
- Retrofits to move inelastic action away from bolted joints, such as structural fuses, could be used.

4.3.4 Anchor Bolts

The most common method of connecting steel and concrete components is the threaded steel anchor bolt. Anchor bolts adhesively bonded into drilled holes in cured concrete have been shown to perform as well as cast-in-place anchor bolts, making this method suitable for retrofitting anchor bolt connections. Care should be taken to avoid drilling through any existing steel reinforcement in the concrete, however. 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 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 calculated based on a uniform stress is given by:

$$U_p = 4\varphi\sqrt{f_c'} \qquad \textbf{(4-4)}$$

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

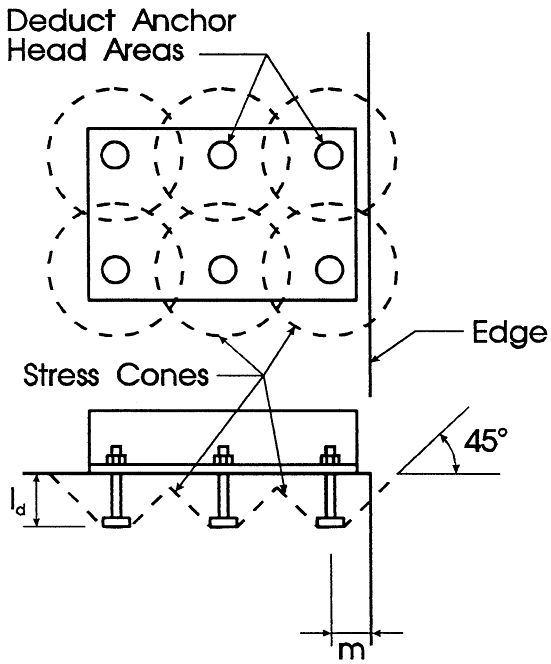


FIGURE 4-5 Failure cone for embedded anchor bolts (adapted from ACI 349, 1978)

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 4-5 (ACI 349, 1978). The inclination angle for calculating projected areas is taken as 45 degrees. The minimum edge distance, m, for anchor bolts under tension only and under tension and shear are given by, respectively:

$$m = D\sqrt{\frac{f_{ut}}{56\sqrt{f_c'}}} \quad \textbf{(4-5)}$$

$$m = D\sqrt{\frac{f_{ut}}{7.5\sqrt{f_c'}}} \quad \textbf{(4-6)}$$

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-LRFD, 1994), would be better for checking the concrete anchorage. The three types of curves are shown in figure 4-6.

When retrofitting bearing seats, reinforcement in the embedment concrete 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).

Anchor bolt connections which are inadequate can be retrofitted in several ways. Adding additional anchor bolts can increase connection capacity, but adequate depth of embedment should be used to avoid brittle concrete fractures. Adding shear keys or shear pins can remove (or supplement) the shear load responsibility of the anchor bolts, such that they are not overloaded in shear. Providing confinement reinforcing for anchor bolts similar to concrete column jacketing could also be effective, if embedment depth is sufficient. Strengthening the concrete using pressure injected polymers or post-tensioning would also be effective.

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 that accomplishes this is to use upset anchor bolts (refer to figure 4-7), 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).

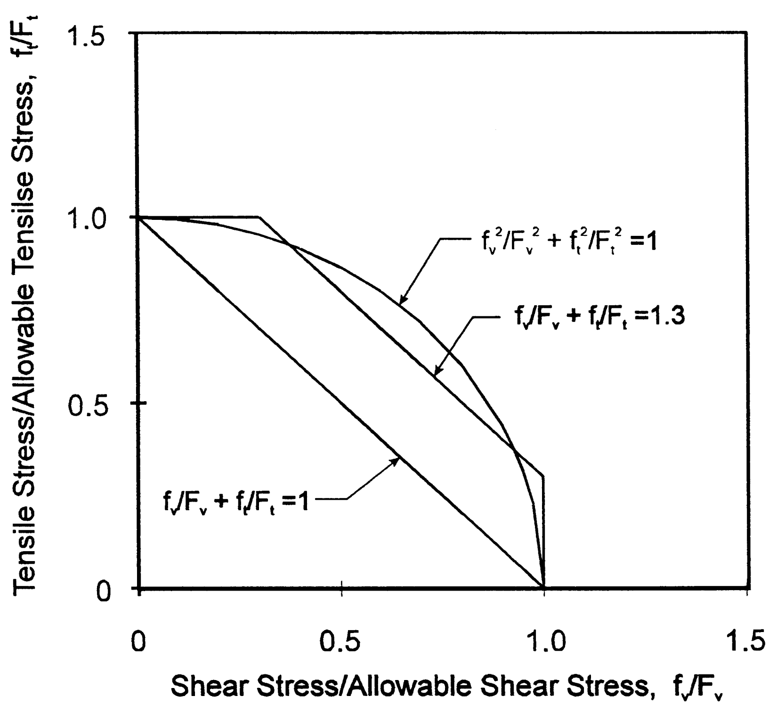


FIGURE 4-6 Tension-shear interaction curves for bolts

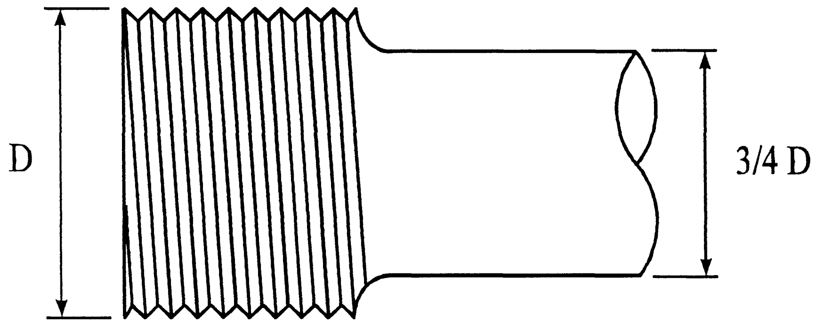


FIGURE 4-7 Bolt with upset threads

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, allowing anchor bolts to deform inelastically and act as a structural "fuse,"can be beneficial, or adequate strength can be supplied for an elastic response 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, 1993; Midorikawa, 1993). Alternatively, an innovative retrofit design for increasing the resistance of the piers of the San Mateo bridge in California by placing additional anchor bolts between existing anchor bolts features sleeves around the anchor bolts as illustrated in figure 4-8, which prevents compressive buckling of the anchor bolts, and permits compressive yielding, resulting in some compressive resistance (Prucz, 1996). The additional length of the retrofit anchor bolts results in a gradual multi-step yielding process, since the existing stiffer anchor bolts will yield first. Another attractive feature of this retrofit is the repairability. The presence of a coupler allows the yielded mild steel portion of the anchor assemblage to be replaced after 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 that 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 has also been suggested that ring springs would be effective in series with bracing or as horizontal thrusters (Erasmus, 1988).

4.4 Retrofit Measures for Bridge Superstructures

4.4.1 Truss Bridges

Caltrans has embarked on a seismic retrofit program to upgrade all of the major toll bridges in the San Francisco bay area. The spans of these bridges include short and long trusses, both simply

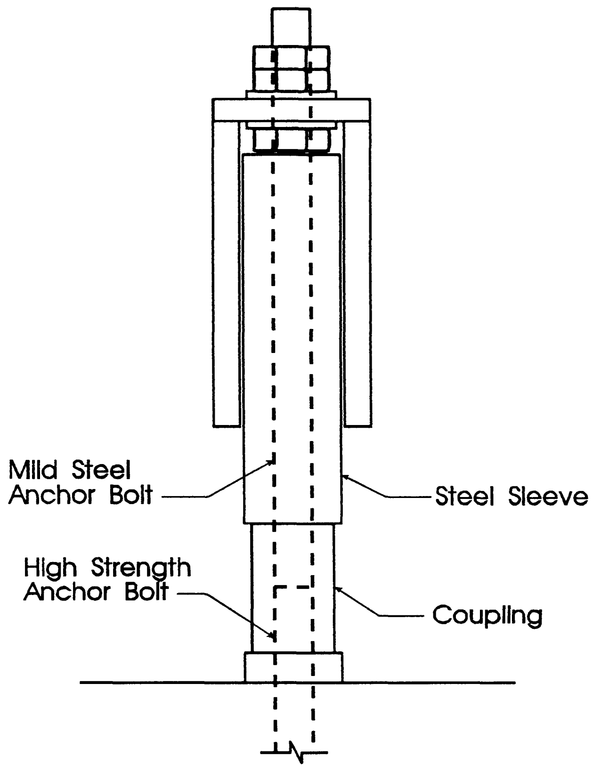


FIGURE 4-8 Anchor bolt with sleeve to permit compression yielding (adapted from Prucz, 1996)

supported and continuous, as part of their crossings. Designs for the retrofits have been presented and some of the identified vulnerabilities and renovations of the truss portions will be summarized here. While the choice of retrofit strategy for each bridge was greatly affected by the expected intensity of the potential earthquake and the assigned performance level designated from Table 1-1, it was recognized that simple member strengthening alone would be uneconomical and/or impractical in most cases due to the large numbers of vulnerable members. For this reason, many schemes included the use of base isolation and/or structural damping to reduce the response of the structure and minimize the number of member retrofits required. Reducing the response of the superstructure has the added advantage of reducing demands on the substructure. It should be recognized that bridges in areas of lower seismicity are not as vulnerable due to the lower inertial forces developed, and the required retrofits may not be as extensive, but similar techniques should be effective.

Initial linear-elastic analyses of the east (truss) spans of the San Francisco-Oakland Bay Bridge (SFOBB) indicated that demand-to-capacity ratios of most structural elements were greater than 1 (Maroney, 1996). This demonstrated that if simple member strengthening were used, an extremely extensive program would be required to provide an elastic response, which was deemed unrealistic. In addition, the laced built-up sections were identified as being vulnerable to severe strength degradation in post-buckling (see Section 3.3.2 on laced members). Connections of tension members were also calculated to be understrength relative to adjacent members, and brittle fractures were a concern. Caltrans engineers speculated that these connections had the potential to fail in a similar manner to the building moment connections in the Northridge earthquake. While this conclusion was initially controversial, it has been substantiated by researchers working on the toll bridge project (Maroney, 1996).

After examining several retrofit strategies for the east spans of the existing SFOBB, it was decided that to best avoid disruption of traffic, respect the lack of inelastic capacity in the laced members and connections, and to minimize post earthquake repairs, an isolation strategy would be most effective (Maroney, 1996). The friction pendulum bearing system by EPS was selected to replace the traditional bearings that already exist because of the design characteristics, low profile, and the abundance of test data verifying performance.

Seismic studies of the east spans of the SFOBB indicate the one of the main vulnerabilities of double deck steel bridges such as this is the differential sway movement between the upper and lower decks. Research on half-scale models of the frames as well as tests on some of their components showed that although significant damage was likely, the existing sway frames of the SFOBB were very ductile under cyclic loading (Astaneh-Asl, 1996b). The research also showed that to limit the amount of inelasticity and damage, and increase the performance level from minimal to limited in Table 1-1, the laced truss verticals could be retrofitted with perforated plates (Astaneh-Asl, 1996b).

The two Carquinez Strait cantilever truss bridges fall into the minimum performance level category in Table 1-1. This essentially means that the bridge should not collapse under the maximum credible earthquake at the site. For this reason the principle of increasing ductility of the structures with little or no increase in strength was followed in the proposed retrofit design (Ballard, 1996). The structure carrying westbound traffic was constructed in 1927 and is composed of laced members of various configurations. The structure carrying eastbound traffic was constructed in 1958 and has a similar geometry, but uses perforated plates instead of lacing on the members. In order to utilize the full capacity of each member, it was proposed that the connections on both structures be strengthened such that gross section yielding governed over net section fracture, and all primary compression members be stiffened such that global buckling controlled over local buckling. Several other retrofit schemes were investigated, including allowing rocking of the piers, using isolation bearings, and using viscous dampers between the superstructure and the towers. While these strategies all reduced the response of the superstructure somewhat, none provided enough of a member retrofit cost reduction to justify their use.

The proposed retrofit of the 1927 Carquinez Strait bridge consists mostly of strengthening connections and adding stiffeners to prevent local buckling of members (Jones, 1996). Connection retrofit consisted generally of replacing rivets with high strength bolts and stiffening the edges of gusset plates. Eyebars expected to experience compression forces are to be clamped together, in order to shift the buckling mode from out-of-plane to in-plane, which will eliminate damage to the connection areas. Utilizing a planned deck replacement by detailing the new deck to act as a horizontal diaphragm and strut will eliminate the need for a significant retrofit of the bottom chords and lateral bracing system.

The proposed retrofit of the 1958 Carquinez Strait bridge consists of bolting stiffeners to vulnerable members to reduce b/t ratios to more stringent criteria than required for reaching yield before buckling (Hinman, 1996). For some bottom chords in the suspended truss spans strengthening is required in addition to stiffening. Very high compression and tension demands were calculated for the X-bracing between bottom chords of the trusses. Calculations also showed that in many cases the tension leg of the X-bracing was adequate when considered as a tension only system. In these cases, the members will be stiffened to maximize compression ductility in order to survive cyclic load reversals.

The Richmond-San Rafael bridge includes two double deck cantilever trusses spanning the shipping channels. Constant depth double deck trusses connect the cantilever trusses to each other as well as the approach structures. The performance criteria of the Richmond-San Rafael bridge, like the Carquinez Strait bridges, is minimum performance, i.e., no collapse, under the safety evaluation earthquake. The retrofit philosophy was to allow inelastic behavior in pre-selected ductile elements, in order to reduce demands on the remainder of the structure (Vincent, 1996). Additional methods to be applied to limit bridge response include eccentrically braced steel frame towers, special moment resisting towers, lead-core rubber isolation bearings, controlled tower rocking, and hydraulic dampers.

Approximately 25% of the primary truss members of the Richmond-San Rafael bridge will be strengthened by replacing existing lacing bars, adding cover plates, and replacing rivets with high strength bolts. In addition, new sway bracing will be installed on the cantilever truss spans at locations where it was originally omitted, and existing portal and sway bracing will be strengthened to control transverse drift demands on the truss posts caused by racking of the upper and lower decks. New longitudinal mid-height bracing members will be added to the cantilever trusses to reduce the unbraced length of the truss posts and prevent instability under lateral racking. Vertical post-floorbeam connections will be strengthened in order to force flexural yielding into the posts under lateral forces. Additional bracing in the plane of the trusses will be added to prevent lateral torsional buckling of the posts and ensure stabilty of the plastic hinge region.

The Benicia-Martinez bridge includes a 4884 foot constant depth deck truss, consisting of both continuous and simply supported sections, including drop-in spans. This bridge has been designated a lifeline route, resulting in a full performance level criteria from table 1-1, meaning that an essentially elastic response is required for all elements. The piers and the bearings were identified as the most vulnerable elements of the bridge, so isolation bearings have been proposed to limit the response of the superstructure (Imbsen, 1996). This not only reduces forces in the superstructure, but also limits the forces transferred down to the piers.

Most of the existing main truss members of the Benicia-Martinez bridge were found to have sufficient capacity to resist seismic loads. The non-redundant hangers for the drop-in spans were found to be vulnerable. New vertical and diagonal members were proposed for the trusses, to torsionally stiffen the drop-in spans, reducing demands on the hangers. New transverse shear keys are also proposed to limit relative motion between the drop-in and fixed spans. The top lateral bracing is the load path for carrying inertial forces of the deck to the sway frames at the piers. Deficient lateral braces will be strengthened by adding cover plates and/or replacing members. Strengthening of connections and members will be performed to obtain the required elastic response.

In statically determinate truss bridges, the failure of one main truss element can result in collapse of the structure. For this reason, it should be ensured that main truss elements do not fail during the maximum seismic event. From the previous proposed seismic retrofit schemes, some general conclusions regarding trusses can be drawn. Connections continue to be one of the most vulnerable areas, especially for main tension members. The approach on the Carquinez Strait bridges was to strengthen connection capacities to 30% greater than the tensile yield of the member. In compression members, local buckling is also a big concern, due to low cycle fatigue problems associated with it. Bolted stiffening elements are one typical retrofit to reduce local buckling problems. Overall buckling is not as big a concern unless it is in a main compression member, in which case buckling must be prevented. In X-braced members, for instance, it may be sufficient to ensure compressive cyclic ductility through stiffening, and allow the bracing to act in tension only. Strengthening can be done by bolting on additional material, but this also places additional demands on the connections, and is more expensive than stiffening. Finally, when retrofitting, it may be easier and less expensive to provide isolation elsewhere in the structure, such as the bearings or the piers, reducing the response of the truss superstructure, such that substantially less individual member retrofit is required.

4.4.2 Laced members

Once laced members have been identified as vulnerable, there are several retrofit schemes available to improve cyclic performance including:

- replacing lacing with perforated cover plates
- bolting a solid web in the interior of the member
- replacing the entire member with a new box beam

These methods are very effective, and are recommended when only a limited number of members require retrofit. When faced with an entire bridge full of potentially overloaded laced members, however, retrofitting every single one may be impractical, especially if work must be performed under traffic. Design proposals for many of the California steel toll bridges slated for retrofit resolved this problem by using some means of isolation to reduce demand below capacity for most of the laced members, then retrofitting the remaining members that were still overloaded.

4.4.3 Slab-on-Girder Bridges

As mentioned in section 3.4.3, the bearing diaphragms or cross-braces in slab-on-girder bridges have been identified as potential locations for ductile "fuses". While the existing bearing diaphragms are potentially adequate to behave as these "fuses," it is unlikely that they possess the strength and/or ductility to perform satisfactorily. Diaphragm systems found to be inadequate for seismic demands must be either modified or replaced to improve behavior. Some research has been performed and is continuing on several ductile end diaphragm schemes. Diaphragms utilizing a shear panel system (SPS), an eccentric braced frame (EBF), and a TADAS (triangular plates added damping and stiffness) system (illustrated in figure 4-9) have been investigated (Zahrai, 1998).

Some conclusions based on this research include:

- The ductile end diaphragm approach works best on bridges having relatively low lateral stiffness. For bridges with girders having high lateral stiffness, other approaches (for example base isolation) may be more effective. For this reason it is recommended that bearing stiffeners be trimmed to provide only the minimum width necessary to satisfy their strength and stability requirements.
- Possibly due in some part to the preceding point, longer bridges with properly designed ductile end diaphragms absorb more earthquake energy and more effective retrofits are obtained for bridges with a smaller number of girders.

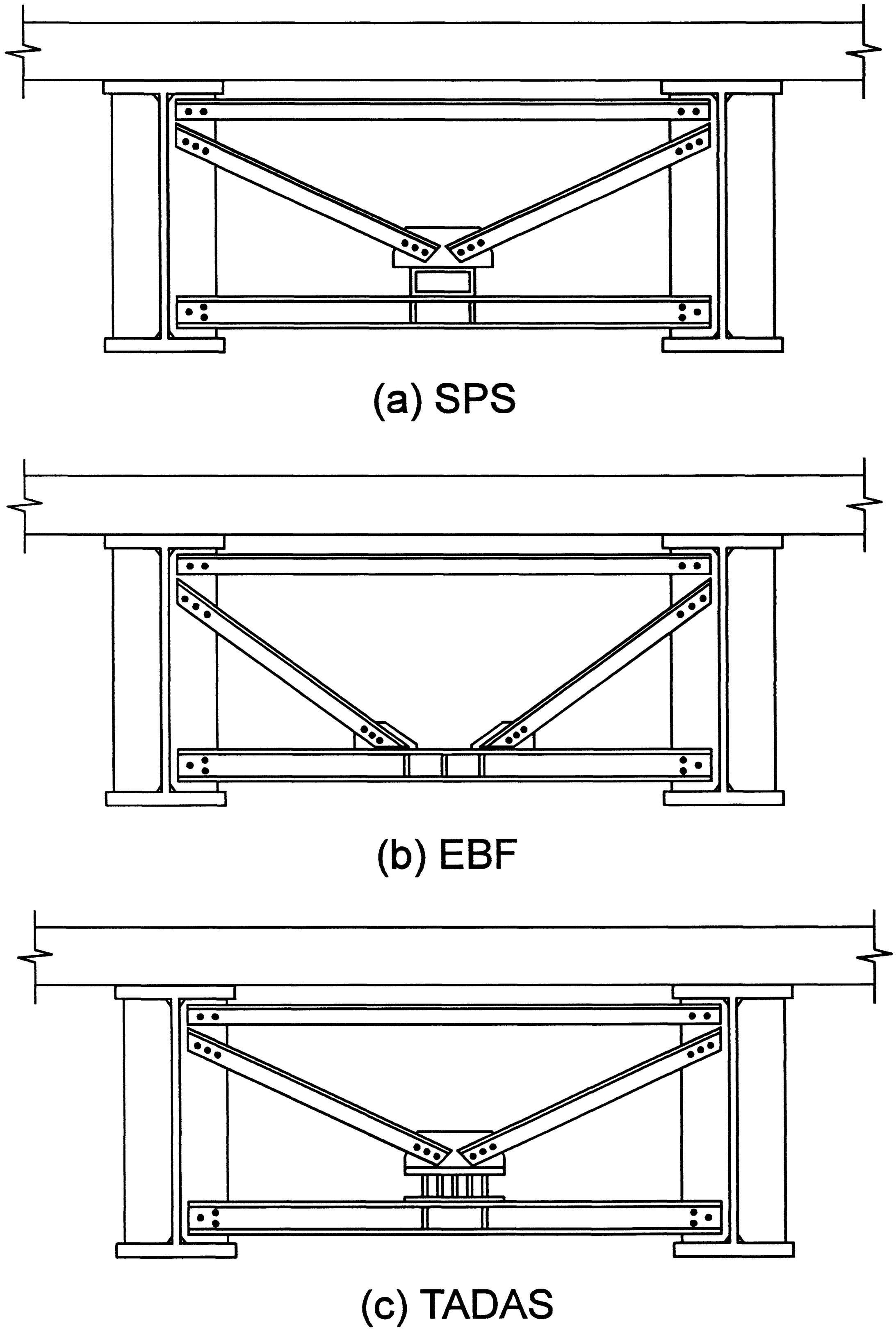


FIGURE 4-9 Several ductile end diaphragm schemes

! For short to medium span bridges, seismic loads are such that all lateral inertial forces can be carried by one ductile end diaphragm, and diaphragms can be omitted between all other girders. Inserting end diaphragms between all girders makes it difficult to find an appropriate structural shape among the available sections and can lead to unnecessarily overstrength diaphragms.

! Welding of ductile end diaphragm connections is recommended to avoid the potential slippage associated with bolted connections which leads to pinched hysteretic behavior.

A simple and reliable calculation method, using a trilinear load-displacement relationship and a strength-versus-ductility relationship based on equal energy concepts, suitable for hand calculations, has also been developed and is presented here (Zahrai, 1998).

First determine the basic design parameters:
M = mass of the entire superstructure of the bridge
A = seismic acceleration coefficient (code specified or site specific)
n_g = number of girders
n_d = number of ductile end diaphragms at each support
L = girder spacing
R = force reduction factor - derived using principles of equal energy

Since the steel bridges of interest usually have a low period of vibration in the transverse direction, and the lateral stiffness of the girders is small compared to that of the braces and the energy dissipating device (resulting in a small slope on the second part of the trilinear curve), the bilinear relationship between R and μ (ductility) is usually adequate provided an equivalent yield strength of V_{inel} is used.

$$R = \sqrt{2\mu - 1} \qquad \textbf{(4-7)}$$

Calculate the generalized mass and generalized stiffness of the superstructure:

$$m^* = \int_0^L \frac{M}{L} (u(x))^2 \, dx \qquad \textbf{(4-8)}$$

$$K^* = \int_0^L EI_D \left(\frac{d^2u(x)}{dx^2} \right)^2 dx \qquad \textbf{(4-9)}$$

where:
u(x) = assumed deflected shape - usually the first mode of vibration
E = modulus of elasticity
I_D = moment of inertia of the entire bridge about an axis perpendicular to the deck

The stiffness of the substructure K_{subs}, should also be determined at this point, although the stiffness of abutments can generally be assumed to be large enough that substructure stiffness is neglected.

Calculate the elastic seismic base shear resistance, V_e, for one end of the span, that would be needed if a fully elastic response was desired. Typically this is done using a code formula such as:

$$V_e = C_s \left(\frac{8Mg}{\pi^2} \right) / 2 = C_s \frac{4Mg}{\pi^2} \approx 0.4 C_s Mg \qquad \textbf{(4-10)}$$

where:
g = acceleration due to gravity
C_s = seismic response coefficient, typically equal to:

$$C_s = \frac{1.2\ A\ S}{T^{2/3}} \leq 2.5A \qquad \textbf{(4-11)}$$

where:
S = site coefficient
T = lateral period of vibration

For bridges with a short period of vibration such as the steel bridges of interest here, the upper C_s limit of 2.5A is usually suitable for preliminary design. In addition, for bridges with laterally stiff decks, V_e should include half of the bridge mass, i.e. $0.5C_sMg$.

Alternatively, V_e can be calculated using a pseudo acceleration, P_{sa}, based on a smooth design spectra representative of the average seismic demand for a set of earthquake records.

Calculate $V_{inel} = V_e / R$, where V_{inel} is the inelastic lateral load resistance of the entire ductile diaphragm panel. At this point it should be verified that V_{inel} is less than the lateral resistance of the substructure, including a comfortable safety factor (say 2.0). This will ensure that the principal target objective of protecting the substructure is achieved.

Determine the design lateral load to be resisted by the energy dissipation device at the target ductility level, given by:

$$V_d = \frac{V_{inel} - n_g V_g}{n_d} \qquad \textbf{(4-12)}$$

where:
V_g = lateral resistance of one stiffened girder

For trilinear hysteretic systems, an iterative approach must be employed to determine V_d, since V_g cannot be calculated accurately without knowing the transverse displacement. For the first iteration V_g can be taken as the yield value. It is apparent from this equation that girders with a high lateral stiffness have the potential to dominate the transverse response. It is also difficult to accurately calculate the girder lateral stiffnesses due to uncertainties in their fixity conditions. In many cases it may be reasonable to assume full fixity at the composite deck and a pinned condition at the bearing. It should be noted that the most conservative assumption for design in this case is not a pinned-pinned but a fixed -fixed vertically stiffened girder. For these reasons it is recommended that the lateral stiffness of the girders be reduced by trimming the vertical stiffeners to the minimum necessary for strength and stability.

Once the design force of the energy dissipation element has been determined, the remaining structural members and connections of the ductile diaphragm should be designed to resist a force of $1.5V_d$. This 50% overstrength factor recognizes the uncertainties in the design of the ductile element, and ensures that these members will not fail prematurely.

At this point the energy dissipating device is designed. For the devices illustrated here, additional information is available in the literature. Other energy dissipators not listed here could also be effective.

For a TADAS device, the two key parameters, the plastic flexural strength, V_T, and the stiffness, K_T, can be determined from:

$$V_T = \frac{N_T\, b_T\, t_T^2\, F_y}{4h_T} \qquad \textbf{(4-13)}$$

$$K_T = \frac{N_T\, E\, b_T\, t_T^3}{6h_T^3} \qquad \textbf{(4-14)}$$

where:
N_T = number of triangular plates
b_T = steel plate base widths
t_T = steel plate thickness
h_T = steel plate height
F_y = steel yield strength
E = modulus of elasticity of steel

In this case, $V_T = V_d$. It is also recommended that the height of girder to steel plate height ratio, H/h_T, fall between 10 and 12 in order to meet distortion demands. h_T/b_T ratios between 1 and 1.5 have been determined to be better energy dissipators. A large number of thinner plates is preferable to mitigate potential problems due to plastic interaction of flexure and shear forces. While t_T is constrained by the available plate thicknesses, it is possible to design many different yet effective TADAS devices which satisfy the above constraints.

Alternatively, to design an eccentrically braced diaphragm as the energy dissipation device, the shear force V_l in the link would be:

$$V_l = \frac{H}{L_s} V_d \qquad \textbf{(4-15)}$$

where:
H = height of girder
L_s = girder spacing

The plastic shear capacity of a wide flange beam is given by (AISC-LRFD, 1994):

$$V_{l,p} = 0.6 F_y\, t_w\, d_l \quad \textbf{(4-16)}$$

where:
F_y = steel yield strength
t_w = web thickness
d_l = web (link) depth

For best performance, the section should be chosen such that $V_{l,p}$ is as close as possible to V_l. The moment simultaneously applied to the link must be less than the reduced moment capacity, M_p^*, of the link yielding in shear calculated as:

$$M_p^* = t_f\, b_f\, F_y \left(d_l - t_f\right) \quad \textbf{(4-17)}$$

where:
t_f = flange thickness
b_f = flange width

In order to ensure the desired shear link behavior and avoid flexural link behavior, the link length, e, should be limited to:

$$e < e_{max} = 1.6 \frac{M_p^*}{V_{l,p}} \quad \textbf{(4-18)}$$

To obtain the desired balance, link length or link beam size can be adjusted. A link length of 1/8 to 1/12 of the girder spacing, L_s, is recommended. Deeper link beams are also preferred as their larger flexural stiffness increases the overall stiffness of the device, ensuring that the device yields prior to the stiffened girders.

The design of an SPS device would be similar to the design of the EBF, with the exception that V_l = V_d, and the panel height should be limited to half of the link length calculated by equation 4-18, since the link is only in single curvature. A link height of 1/8 to 1/10 of the girder depth, H, is recommended.

The stiffness of the ductile diaphragm, K_{DD}, should now be calculated using an appropriate method.

For TADAS devices:

$$K_{DD} = K_{TADAS} = \frac{E}{\dfrac{l_b}{2 A_b \cos^2\theta_b} + \dfrac{L_s}{4 A_{bb}} + \dfrac{6 h_T^3}{N_T b_T t_T^3} + \dfrac{L_s (h_T + d_{bb}/2)^2}{12 I_{bb}} + \dfrac{H \tan^2\theta_b}{2 A_g}} \quad \textbf{(4-19)}$$

where:
l_b = brace length
A_b = brace area
θ_b = angle of brace with horizontal

d_{bb} = depth of bottom beam
A_{bb} = area of bottom beam
I_{bb} = moment of inertia of bottom beam
H = height of stiffened girder
A_g = area of stiffened girder

For EBF diaphragm:

$$K_{DD} = K_{EBF} = \frac{E}{\dfrac{l_b}{2A_b\cos^2\theta_b} + \dfrac{a}{2A_l} + \dfrac{e^2H^2}{12L_sI_l} + \dfrac{1.3eH^2}{aL_sA_{s,l}} + \dfrac{H\tan^2\theta_b}{2A_g}} \quad \textbf{(4-20)}$$

where:
e = length of link
A_l = cross-sectional area of link
$A_{s,l}$ = shear area of link
I_l = moment of inertia of link

For shear panel system:

$$K_{DD} = K_{SPS} = \frac{E}{\dfrac{l_b}{2A_b\cos^2\theta_b} + \dfrac{L_s}{4A_{bb}} + \left(\dfrac{h_l^3}{3I_l} + \dfrac{2.6h_l}{A_{s,l}}\right) + \dfrac{L_s(h_l + d_{bb}/2)^2}{12I_{bb}} + \dfrac{H\tan^2\theta_b}{2A_g}} \quad \textbf{(4-21)}$$

where:
h_l = height (length) of the link
I_l = moment of inertia of the link

Recognizing that the stiffnesses of the various elements are linked together as springs in series, the equivalent stiffness of the system, K_e, can be written as:

$$K_e = \frac{1}{\dfrac{1}{K^*} + \dfrac{1}{K_{ends}} + \dfrac{1}{K_{subs}}} \quad \textbf{(4-22)}$$

where:
K^* = generalized stiffness of the bridge
K_{subs} = stiffness of substructure
In many cases where the substructure is stiff (such as abutments) this term may become negligible
K_{ends} = sum of the stiffnesses of the ductile diaphragms and the girders given as:

$$K_{ends} = \sum K_{DD} + \sum K_g \quad \textbf{(4-23)}$$

where:
K_{DD} = stiffness of the ductile diaphragms from above
K_g = stiffness of the stiffened girders (highly dependent on chosen end conditions)

Once the effective stiffness is found, the lateral period of the bridge is calculated as:

$$T = 2\pi\sqrt{\frac{m^*}{K_e}} \tag{4-24}$$

At this point, equation 4-11 can be used to check whether the value of C_s assumed initially was appropriate. If not, repeat the procedure until C_s converges.

Once C_s has converged, calculate the effective deflection, δ_e, as:

$$\delta_e = \frac{V_e}{K_{ends}} \tag{4-25}$$

and use this value to determine the lateral force per stiffened girder, V_g, from:

$$V_g = K_g \delta_e \le V_{y,g} \tag{4-26}$$

where:
$V_{y,g}$ = lateral yield force of the stiffened girder

The actual force reduction factor, based on the true trilinear behavior of the system, can now be calculated from:

$$R = \frac{V_e}{V_{inel}} = \frac{V_e}{n_d V_d + n_g V_g} \tag{4-27}$$

If the value of R differs significantly from the original target value, go back and modify the design as required.

The design should be repeated until acceptable convergence is obtained. It is reported that only two or three iterations were found to be required in most cases. From the final solution and actual R value, the actual displacement ductility demand, μ, should be calculated (using equation 4-7 or other appropriate equation). The maximum lateral drift of the bridge at the diaphragm location, δ_{max}, is:

$$\delta_{max} = \mu\,\delta_y \qquad \text{where} \qquad \delta_y = \delta_{y,d} = \frac{V_d}{K_{DD}} \tag{4-28}$$

Since the maximum ductility capacity of shear links is commonly expressed in terms of the maximum link deformation angle, γ_{max}, obtained by dividing the maximum relative displacements of the link ends by the link length, the maximum drift of the SPS and the EBF diaphragms are respectively limited to:

$$\delta_{max} < e\gamma_{max} \tag{4-29}$$

$$\delta_{max} < \frac{eH}{L_s}\gamma_{max} \tag{4-30}$$

where the generally accepted γ_{max} limit is 0.009. A more accurate equation for SPS diaphragms taking into account the bottom beam rotation can be used:

$$\delta_{max} < e\left(\gamma_{max} + \frac{V_d L_s \left(h_1 + d_{bb}/2\right)}{12 E I_{bb}}\right) \tag{4-31}$$

If these limits are exceeded, the shear panel design should be modified until the limits are satisfied. A maximum drift limit of 2% is also recommended at least until experimental evidence justifies use of a higher value.

Since the specially detailed devices are designed to dissipate large amounts of seismic energy in a stable manner, concentrating all the cyclic inelastic deformations into the devices is the objective. Ideally, the braced diaphragm assembly should be on the order of 5-10 times stiffer than the stiffened girder webs to accomplish this.

Finally, lateral bracing should be supplied for the ends of the ductile energy dissipating elements. The lateral supports and their connections should be designed for a load of 6% of the nominal strength of the beam flange, or:

$$F_{brace} = 0.06 F_y t_f b_f \tag{4-32}$$

In addition, the laterally unsupported length, L_u, of the beams of the TADAS, EBF, and SPS systems should not exceed:

$$L_u = 200 \frac{b_f}{\sqrt{F_y}} \tag{4-33}$$

The ductile diaphragm technique should be very effective in resisting motions transverse to the bridge structure. Other methods must be employed to resist longitudinal motions, however.

One of the other well documented vulnerabilities of slab-on-girder bridges is insufficient girder seat length, which can result in a dropped span. Increasing the bearing seat length or providing catch systems, tying girders together across piers, or providing cable or bar restrainers, have all been used effectively as retrofits to reduce the possibility of a dropped span. Making girders continuous across piers can also result in an increased live load capacity of the bridge. Design methods for these are available elsewhere.

4.5 Retrofit Measures for Steel Towers and Tower Bents

In the Caltrans seismic retrofit program to upgrade all of the major toll bridges in the San Francisco bay area, various steel towers have been assessed, and retrofits proposed. Designs for the retrofits have been presented and some of the identified vulnerabilities and renovations of the various steel towers will be summarized here.

Most of the towers of the east spans of the San Francisco-Oakland Bay Bridge consist of built-up steel members with laced X-braces. In the proposed retrofit, the towers constructed with vulnerable built-up members and connections will have all their members encased in concrete. At locations without traditional bearings, on the cantilever truss section, it is proposed that isolation be provided by "softening" the response of the towers at these locations. This will be done by removing the cross-bracing between the legs of the towers and encasing the legs in concrete. This retrofit is intended to increase flexibility in the lateral direction while increasing stiffness in the longitudinal direction (Sundstrom, 1996). The concrete encasement solution eliminates local buckling, allows for strengthening where necessary, and eliminates the problem of lead paint removal by encasing it (Maroney, 1996).

The steel box towers of the west (suspension) spans of the San Francisco-Oakland Bay Bridge were also found to be inadequate, especially in the longitudinal direction. To achieve an elastic response, doubler plates would need to be bolted on to decrease bending and axial stresses in the plates of the towers. In addition, many of the latticed diagonal bracing members would have to be reinforced with plates, and many connections would need to have their rivets replaced with high strength bolts. For one tower, an estimated 270 tons of additional steel and replacement of 40,000 rivets with bolts would be required (Reno, 1996).

A more reasonable alternative involves installation of stiffeners to improve the post yield performance of the steel plates in the towers. Laced cross-bracing members that are expected to yield will be retrofitted in order to obtain compact sections. High strength bolts will be used to replace rivets in connections that need strengthening. New anchor bolts will also be installed, allowing additional elongation and rocking of the tower base, and internal shear keys will be added as well. Viscous dampers are being contemplated to try to eliminate the slamming of the stiffening trusses into the towers (Reno, 1996).

On the San Mateo-Hayward bridge, the multi-cell steel box towers will have steel doubler plates bolted on the outside in order to increase ductility. The single cell rectangular steel spandrel beams between the tower columns will be internally stiffened with bolted transverse stiffeners in order to prevent shear buckling. As previously discussed in Section 4.3.4, new anchor bolt assemblages will provide ductile anchorages at the base of the towers. Steel pins are proposed to transfer shear from the base of the steel pier to the concrete. To help transfer the shear to the pins, concrete is to be added inside the steel boxes at the base. This concrete is also expected to increase local stability of the column walls, and provide a gradual transition of strength and stiffness at the column base (Prucz, 1996).

The towers of the Golden Gate suspension bridge are also constructed of multi-cell steel boxes. The proposed retrofit of this bridge includes dampers between the stiffening trusses and the towers to eliminate pounding between these two elements (Ingham, 1996). This reduces the demand on the towers somewhat, but the cellular box walls are still susceptible to local buckling in compression. Width-to-thickness ratios are only sufficient to allow the tower to reach yield, with buckling occurring soon after. To reduce the vulnerability of the tower walls, I-shaped stiffeners will be placed along the vertical centerlines of the plates at the base (between diaphragms) in the extreme compression zones. The stiffeners are expected to improve the displacement ductilities of the towers from the existing 1 to a retrofitted value of 4 (Ingham, 1996). The tower retrofit also will include stiffening of some of the portal and diagonal struts that connect the towers, as well as strengthening the strut connections by replacing the rivets with high strength bolts.

Lateral resistance for the 1927 Carquinez Strait bridge is provided by chevron braced steel towers. The bracing consists of riveted built-up latticed members. Virtually every connection required strengthening in order to achieve the goal of 130% of the nominal yield strength of the connected member (Jones, 1996). This was to be accomplished by replacing rivets with high strength bolts and thickening gusset plates where necessary. Back-up plates were proposed for many chevrons at bracing connections to eliminate the possibility of net section fracture. The towers were analyzed using the actual width-to-thickness ratios of the bracing members, since in most cases the width-to-thickness ratios fell between the AISC limits (AISC, 1994) for non-compact sections (able to reach yield before local buckling) and compact sections (able to fully plastify before local buckling). These analyses indicated that in most cases braces were adequate to meet the ductility demand, even though they did not always meet AISC width-to-thickness requirements for plastic design. AASHTO width-to-thickness ratios are comparable to AISC values (AASHTO, 1994).

Longitudinal resistance for the 1927 Carquinez Strait bridge is provided by A-frame towers. Pushover analyses of these towers indicated that they were inadequate for the 30 in. longitudinal ductility demand. It was deemed impractical to increase the member ductility sufficiently to meet this demand, so an alternative solution was proposed. A structural fuse allowing limited rocking of the tower on shock absorbing elastomeric bearing pads has been proposed to limit forces on the tower (Jones, 1996). Shear keys and restraining devices are also part of the design.

The 1958 Carquinez Strait bridge employs similar geometry to the 1927 bridge, but it was one of the first bridges to use high strength (T1) steel, welded built up members, and high strength bolted connections (Hinman, 1996). Proposed retrofits to the towers of the 1958 bridge generally followed the same philosophy as the 1927 bridge. Member connections are all to be strengthened to be stronger than the member. Inadequate bracing members are to be stiffened to provide better compression ductility. Two towers with A-frames are to allow limited rocking in the longitudinal direction, similar to the ones on the 1927 bridge. Allowing this rocking in the two A-frame piers results in additional load being carried by the braced tower at the center of the bridge. Although permitting rocking of this tower was considered also, it was decided to maintain the longitudinal resistance at this tower. This will require increasing the cross-section of the columns of the tower by 20%, and strengthening the tie-downs of the tower by using transfer beams and post-tensioned strand anchors.

The Richmond-San Rafael bridge includes two double deck cantilever trusses spanning the shipping channels with constant depth double deck trusses connecting the cantilever trusses to each other as well as the approach structures. The retrofit philosophy was to allow inelastic behavior in pre-selected ductile elements, in order to reduce demands on the remainder of the structure (Vincent, 1996). The proposed steel tower retrofits for this bridge incorporate several earthquake resistance concepts from the steel building industry.

The existing built-up latticed member tower bracing will be removed and in the taller towers replaced with dual eccentrically braced frames (EBFs). Dual frames are being used in order that the active links can be braced out-of-plane. These new EBFs will be mostly independent of the existing gravity load system, carrying no gravity load, and only providing bracing to the existing columns. The EBFs will be used to provide resistance in both the lateral and longitudinal directions at four-legged towers. A unique hinged yoke will be utilized to transfer longitudinal forces only from the truss chords, without subjecting the chord to bending and torsion.

Lateral resistance at the shorter two-legged towers will be provided using moment resisting frames (MRFs). The MRFs will incorporate scalloped "dogbone" connections such as those pictured in figure 2-3. The scallops are intended to move the plastic hinge location approximately 1'-3" away from the column face, and to avoid the problems associated with welded moment connections in the Northridge earthquake.

The anchorages of the pier box columns on the Richmond-San Rafael bridge will be strengthened, and cover plates added at transition regions and splices to avoid the potential of yielding at these locations. Concrete will be placed in the bottom of the columns in layers separated by a compressible joint material to prevent local buckling in the plastic hinge region.

After the failure of one of the spans of the Oakland Bay Bridge during the 1989 Loma Prieta earthquake (Housner, 1990), a study was performed to determine the most economical retrofit method to assure satisfactory performance during subsequent earthquakes. Due to the importance of the structure, a strategy to provide an "almost elastic" response was sought. To provide this response with rigid piers, considerable strengthening of the substructure would be required in addition to extensive strengthening of the superstructure (Rosenbaum, 1992; Astaneh-Asl, 1993). Providing a semi-rigid pier connection, and permitting some limited rocking of the pier, was found to increase the period of the bridge. Although in the case of the Bay Bridge the increased period was subsequently found not to be an effective approach, depending on the frequency of the earthquake input, other bridges might benefit from this approach.

4.6 Innovative Retrofit Measures

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 dissipation. Devices utilizing these characteristics can function as structural "fuses," providing sacrificial elements that 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 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 4-10, 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, that use these three techniques of seismic protection.

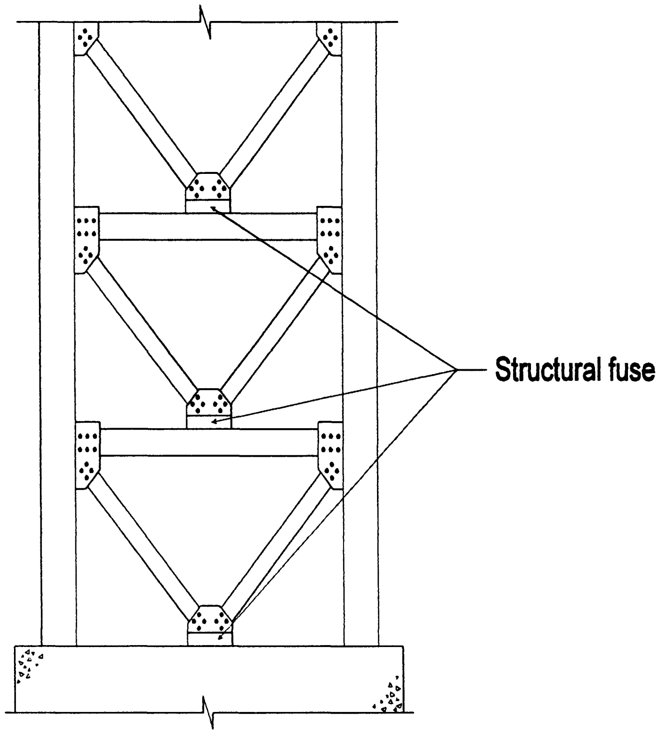


FIGURE 4-10 Concentric bracing with structural fuse at connection

4.6.1 Slotted Friction Connections

The slotted bolted connection (SBC) shows promise in providing a device that 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.

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 4-11 (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, which provides a consistent, repeatable energy dissipation (Grigorian, 1994). 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 exposed members should be confirmed before using these in bridges.

4.6.2 Disposable Knee Bracing

Another lateral bracing scheme that has received little attention to date is the Disposable Knee-Brace (DKB) (Aristizabal-Ochoa, 1986) or Knee-Brace-Frame (KBF) (Balendra, 1993). The only difference between the two systems is that the braces in the DKB are designed for tension only. Like

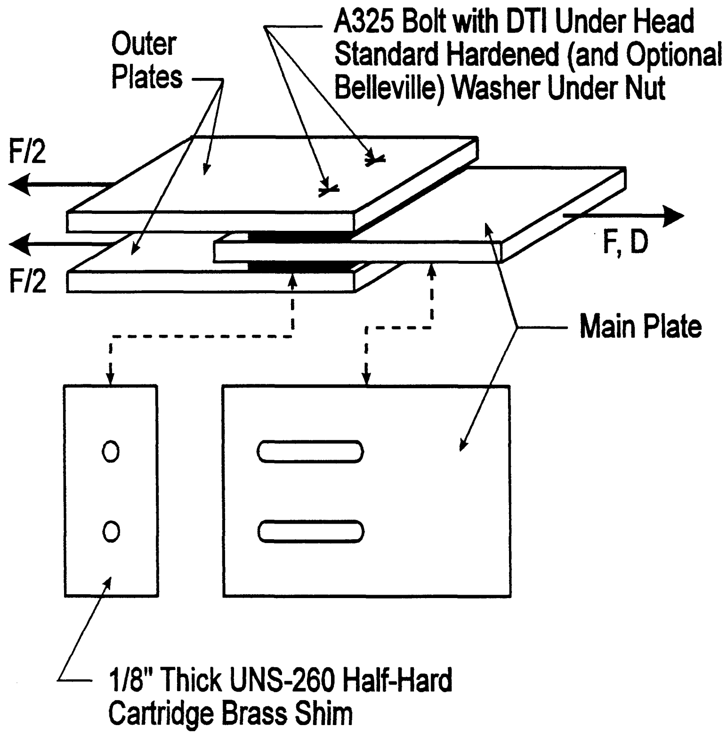


FIGURE 4-11 Slotted bolted connection (adapted from Grigorian, 1994)

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 4-12. 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.

The main advantage the KBF has over other seismically resistant frame configurations is the 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, although 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. The diagonal braces are not part of the gravity load bearing frame, so 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 any damaged brace be entirely replaced.

The main disadvantage of the KBF is that bracing members would have several extra connections apiece, which is a very labor intensive task in both fabrication and erection.

(a)

(b)

(c)

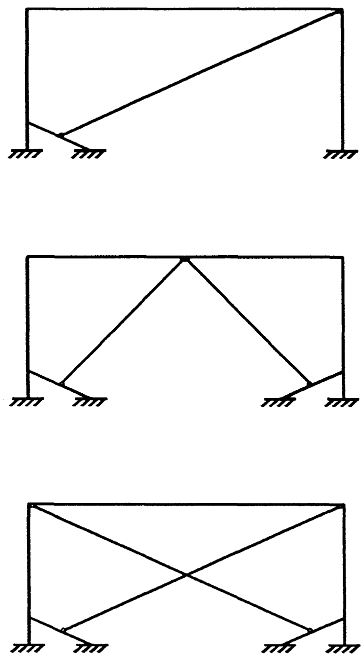


FIGURE 4-12 Disposable knee-brace (DKB) or knee-brace-frame (KBF)

4.6.3 Steel Plate Added Damping and Stiffness (ADAS) Devices

Steel plate added damping and stiffness (ADAS) devices consist of a series of vertical steel plates, as illustrated in figure 4-13, and fall into the material yield device category. Devices such as this are attractive due to their simplicity of construction, consistent response at any temperature, ordinary maintenance requirements, and ease of replacement following an earthquake. Research has shown that ADAS devices are very reliable energy dissipators that exhibit stable hysteresis for displacement amplitudes as large as 14 times the yield displacement, Δ_y, of the device (Xia, 1992). Tests have shown that in the displacement range of 6 times Δ_y or less, the ADAS device hysteretic behavior is dependent only on the yield force, P_y, and the yield displacement, Δ_y, and an extremely large number of yielding reversals (over 100 cycles in the tests) can be sustained (Xia, 1992). This means that a bilinear force-displacement model is appropriate for simulating ADAS behavior.

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 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 important design parameters (Xia, 1992).

The selection of the ADAS device yield force should consider both strength and energy demands based on the expected seismic activity at the site. The yield force of the device should provide sufficient energy dissipation while remaining within the design ductility ratio. Guidelines for the

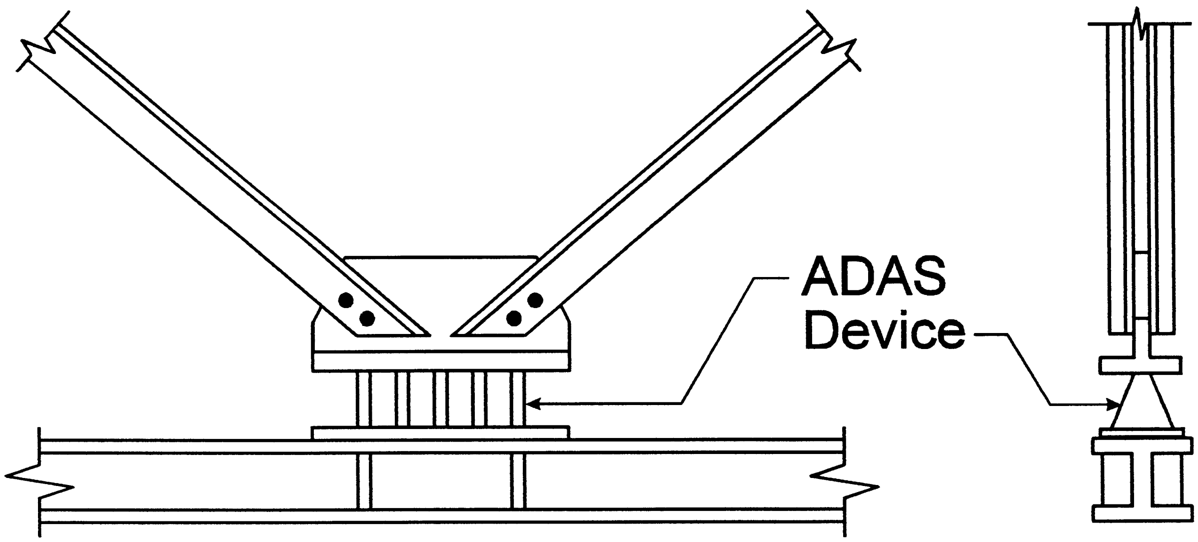


FIGURE 4-13 Added Damping and Stiffness (ADAS) device

design of ADAS devices for buildings are listed in the following paragraph, and similar considerations should apply to applications in bridge structures.

The selection of the yield displacement of the device is very important in limiting the ductility ratio. The recommended device yield displacement is 0.0014 to 0.0020 of the story height. For bridges the story height would correspond to the distance between horizontal struts. The effect of brace stiffness to device stiffness ratio, B/D, is small, and a value of 2 is recommended for bracing member design. The optimal stiffness ratio of the horizontal ADAS stiffness to the structural story stiffness, SR, was also found to be about 2. The effect of strain hardening of the ADAS device should be included when designing the strength of the structural members supporting the device. A procedure for designing triangular plate ADAS systems, or TADAS systems, is presented in Section 4.4.3.

4.6.4 Shear Panel Systems (SPS)

Shear panel systems work on the same principle as active shear links in EBFs or a yielding panel zone of a beam-column connection. Essentially, instead of using a portion of one of the structural members to provide a structural fuse through hysteretic shear yielding, the shear yielding apparatus is isolated as a separate device, and then located where the structural fuse is desired. Again, one suitable location would be at the locations of lateral bracing member connections. There are several advantages of isolating the shear panel system including:

- Increases design flexibility of the hysteretic shear yielding device and the other structural members by uncoupling them - the device and the other members need no longer be the same section as required in other shear yielding systems.
- Minimizes the replacement of members in retrofit situations - for example the entire lateral bracing system need not be replaced.
- Makes repair easier following an earthquake - only the SPS device need be replaced, not the entire horizontal strut.

4.6.5 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. The main advantage of dampers is that they can be overlaid on existing structural systems so retrofit work is minimized. It has been shown (Tsai, 1993) that viscoelastic dampers can significantly reduce the response of a bridge to earthquake loading.

4.6.6 Fiber-Reinforced Composites

The use of fiber-reinforced composites for structural applications continues to increase. Their light weight and high strength make them attractive materials for seismic applications. Bonded composite plates have been shown to be an effective method of increasing the static flexural strength of reinforced concrete members. Research has shown that fiber wrapping of reinforced concrete columns improves strength and ductility, and this procedure is being used as a retrofit technique in California. There appears to be no reason that fiber composites could not be used as a retrofit for steel structures as well. Although additional research would have to be performed to verify behavior, bonded fiber composites could potentially be used to strengthen and/or locally stiffen steel members for improved cyclic ductility.

4.7 Summary of Critical Issues and Research Needs for Seismic Retrofitting of Steel Bridges

Steel structures are relatively easy to retrofit compared to concrete structures. Once the vulnerable elements have been identified, additional material can be added by welding or bolting or taken away by cutting to alter the strength and/or stiffness of elements.

In the retrofit of steel piers, towers,or bents, when only additional ductility is desired, stiffeners can be added to postpone local buckling. If additional strength is desired as well, coverplates or collars can be added. Corners of rectangular tubular members can be strengthened to eliminate failures at these locations. As an alternative, members can be filled or encased in concrete to increase the strength and stiffness, but care must be taken to make sure that this is not at the expense of ductility.

Connections continue to be one of the most vulnerable areas, especially for main tension members. For welded connections, strengthening the connection or weakening the connected member such that inelastic behavior occurs in the adjacent member and not the welded connection is the present philosophy. The current approach for bolted and riveted structures is to strengthen connection capacities such that gross section yield governs over net section fracture. In riveted structures this often entails replacing the rivets with high strength bolts. For anchor bolts, similar approaches apply, although in some cases allowing yielding of the anchor bolts may be desirable.

In superstructures, compression member local buckling is a big concern, due to low cycle fatigue problems associated with it. Bolted stiffening elements are one typical retrofit to reduce local buckling problems. Overall buckling is not as big a concern unless it is in a main compression member, in which case buckling must be prevented. In X-braced members, for instance, it may be sufficient to ensure cyclic ductility through stiffening, and allow the bracing to act in tension only. Strengthening can be done by bolting on additional material, but this also places additional demands on the connections, and is more expensive than stiffening.

Accepted retrofits for vulnerable laced members include replacing the lacing with perforated plates, adding an interior web to the member, or replacing the entire member with a new box beam.

Slab-on-girder superstructures can be retrofitted to survive transverse forces by either strengthening the bearing diaphragms, or, to limit forces on the substructure, by providing a ductile end diaphragm system. Longitudinal forces need to be resisted in other ways, however. Providing continuity across multiple spans of a bridge greatly assists in resisting seismic forces as well, although longitudinal forces should not be concentrated at one support.

Finally, when retrofitting, it may be easier and less expensive to provide some means of isolation, damping, and/or energy dissipation at selected locations within the structure, or elsewhere in the structure, such as the bearings or the piers, reducing the response of the superstructure, such that substantially less individual member retrofit is required. These devices fall generally into one of three categories, friction devices, viscous or viscoelastic devices, or material yield devices. Often this approach is referred to as providing structural "fuses," since potentially vulnerable members are protected by force limiting devices located elsewhere. Examples of such devices are eccentrically braced frames, shear panel systems, TADAS systems, slotted friction connections, knee-brace frames, and visco-elastic dampers. Other devices may be available as well, and may be more appropriate for a particular situation.

SECTION 5
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS FOR FUTURE RESEARCH

5.1 Summary and Conclusions

Steel bridges performed extremely well in the recent Northridge earthquake. No main load carrying steel bridge members suffered severe damage; most damage was confined to steel diaphragms or cross-frames, and steel-concrete interfaces. The unexpected poor performance of bolted web-welded flange moment connections in steel buildings was the bad news that came out of the Northridge earthquake. Reinforcing the connection using a bolted stiffened connection or reducing the beam area away from the connection (i.e., dogbone) in order to move the plastic hinge away from the column face were among several proposed solutions. Although the use of welded connections such as these is rare in bridges due to fatigue concerns, these concepts can be used effectively to relocate plastic hinges away from connections in bridges as well.

In the recent Kobe earthquake, steel bridges performed well from the standpoint of resistance to collapse. While damage was far greater than expected, much of the damage to steel superstructures was the result of failures of other components such as bearings, anchor bolts, restrainers, and substructures. Steel tubular pier columns experienced a variety of unanticipated problems including local and global buckling, and two rectangular steel columns collapsed completely due to corner weld failures. The extent of damage and failures of steel components was far less than in concrete components, however.

Tubular steel pier performance in Kobe led to Japanese recommendations that stiffeners be sized to three times the minimum to prevent elastic buckling and axial compressive stress be limited to 1/10 of yield to reduce P-delta effects and limit inelastic shortening. Adding transverse diaphragms and/or longitudinal stiffeners has been used to retrofit steel piers to increase ductility, and understrength corners of rectangular piers have been reinforced with welded plates or angles. Partially filling piers with concrete has also been used as a retrofit, and while it is accepted that increased strength and stiffness result, the effect on ductility is still debated. Encasing steel members in concrete or steel jackets are alternative repair or retrofit techniques.

Most of the damaged steel structural members in both the Northridge and Kobe earthquakes have little directly in common with typical steel bridge structures in the United States. Bolted web-welded flange beam-column connections and tubular steel piers are not typically employed in steel bridges constructed in the U.S. This is not to say that because they do not contain these features, U.S. steel bridges are seismically safe and we need not worry. On the contrary, these problems have shown that simply using steel does not guarantee dynamic cyclic ductility in a bridge, and steel bridge engineers should take several lessons from the Northridge and Kobe earthquakes, including:

- Greater attention should be paid to weldments, especially welded connections, to ensure that the required ductility is present. This may include using higher toughness weld metal, using more stringent welding procedures, or using a connection which shifts high strain demands away from welds, such as one of the concepts that came out of research on the Northridge failures.
- Connections continue to be vulnerable, and potentially vulnerable connections may benefit from some of the Northridge retrofit concepts.

- Designs based on seismic cyclic ductility should not extrapolate on historical research, or assume that static cyclic ductility capacity indicates dynamic cyclic ductility capacity.
- Local buckling should be postponed by using generous width-to-thickness ratios or stiffeners to avoid large cyclic strain reversals leading to low cycle fatigue failures and/or fast strength degradation for members expected to buckle globally.

Some of the damage to structures in recent earthquakes can also be attributed to recent advances in analysis techniques and methods, especially computer analyses. While these tools improve the calculation of the distribution of loads to members, allowing a more efficient design of the structure, a reduction in redundancy and overall factor of safety also occurs. This can result in the seismic resistance demands being concentrated in fewer elements, with a failure of one of these elements being more catastrophic. Analysis programs also make it easy to neglect the resistance of members that are assumed to be pinned. Even if the resistance contribution of a member is ignored, it must be able to undergo compatible displacements with the remainder of the structure without suffering failure.

Connections continue to be vulnerable to seismic loadings. Riveted connections on older bridges can be retrofitted by replacing the rivets with high strength bolts and adding additional connection plates if necessary, although some ductility may be sacrificed. Many older bridges also employ laced or latticed members which can be vulnerable to post-buckling strength degradation. Current Caltrans research aims to better characterize latticed member behavior, including axial force-moment interaction. Removal and replacement of lacing bars with perforated plates is the currently accepted method of retrofit.

Use of high strength bolts for connecting members is the current method of choice for most steel bridges. The indeterminate, redundant nature of bolted connections is a benefit in earthquake resistance, although the potential for partially-restrained behavior makes analysis and consequently retrofit design more difficult.

Fracture of anchor bolts through the threads can be avoided by using upset threads or providing a sleeve around the anchor bolts. Shear keys can be used to provide supplemental shear resistance in addition to the anchor bolts.

Global and local stability of steel compression members remains a concern. Global buckling is especially a problem in non-redundant structures when fast strength degradation occurs. Both better cyclic post-buckling behavior and plastic hinge rotation capacity are obtained when local buckling is postponed in steel members. Local buckling contributes greatly to low cycle fatigue problems. Research into the use of lower width-to-thickness requirements in areas expected to experience inelastic behavior is warranted.

Damage to bearing cross-frames which apparently protected the substructure of several concrete slab-on-girder bridges in the Northridge earthquake led to a proposal that bearing cross-frames would be an ideal location for a ductile "fuse", and research has verified the “ductile end diaphragm” concept. Preliminary results indicate that for short and medium span bridges, retrofitting with only one ductile end diaphragm at each support is often sufficient.

Extensive research into the three building frame lateral resisting systems, moment resisting frames (MRFs), concentrically braced frames (CBFs), and eccentrically braced frames (EBFs), indicates that properly proportioned and detailed, any of the three systems provides adequate seismic ductility. There appears to be no reason why they could not be equally effective for bridges. Some challenges, such as how to brace the active link, will have to be met for EBFs, but these can be overcome.

As an alternative to ensuring that the structural members themselves possess sufficient strength or ductility, "devices" that provide a combination of isolation, damping, and/or energy dissipation can be incorporated into the structure. Properly designed and installed in series or parallel with existing structural members, these devices can serve as structural "fuses," controlling the structural response, limiting forces, and protecting potentially vulnerable members. Several of these devices, which fall generally into one of three categories, friction devices, viscous or viscoelastic devices, or material yield devices, have been illustrated here, but others (some proprietary) exist. Devices such as these are very attractive in retrofit applications, as they may be able to reduce the response such that little or no member strengthening or stiffening is required.

Unlike concrete, where plastic hinging is limited to tops and bottoms of pier columns, the geometry of steel bridge structures is such that multiple locations and methods of supplying the isolation, damping and/or energy dissipation required to resist the maximum earthquake loading are available. The designer of a retrofit should take full advantage of this flexibility in choosing the optimal resistance strategy.

The overall Project 106 objectives are to study various aspects of seismic vulnerability of existing highway construction. Task 106-E-5.5 is one of the few tasks that investigates the seismic vulnerability of existing steel bridge structures. Due to historically good performance, and general confidence in the ductility of steel, existing steel bridges have previously been assumed to be seismically safe, and research has concentrated on concrete. This faith was shown to be overly optimistic by the performance of steel structures in recent earthquakes. Despite the poor behavior of certain steel details, however, the high strength-to-weight ratio and inherent ductility make steel an excellent material for seismic resistance. This task is important in order that steel bridges do not continue to be overlooked when assessing the seismic vulnerability of highway structures. It will ensure a more efficient allocation of limited resources when upgrading seismically deficient structures.

5.2 Recommendations for Future Research

Retrofitting existing bridges to current seismic standards is an expensive proposition. Getting the maximum seismic upgrade for minimal cost and effort is paramount. This is especially true in the eastern and central portions of the U.S., where seismic risk is generally perceived as extremely low. Research dollars should concentrate on retrofit techniques that require a minimum of field work, a minimal disruption in traffic, and a reasonable assurance that the structural integrity of the bridge will at least survive the maximum credible earthquake at the site.

Since the “do nothing” approach is always the most attractive, research should concentrate on improved connection and member characterization techniques, as well as readily available and easy to use analysis techniques to rate the existing bridge structures more accurately for current seismic resistance.

For improving the seismic resistance of bridges, research should continue into the development of structural "fuses." These devices are attractive because they usually do not require substantial field work and existing member retrofits, they are generally unobtrusive and do not alter the historical "look" of bridges, and, since they are generally part of the lateral resistance system, they can be easily removed and replaced, not necessarily immediately, after an earthquake.

SECTION 6
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APPENDIX A

Standard Cross-Frames and Diaphragms of Various States

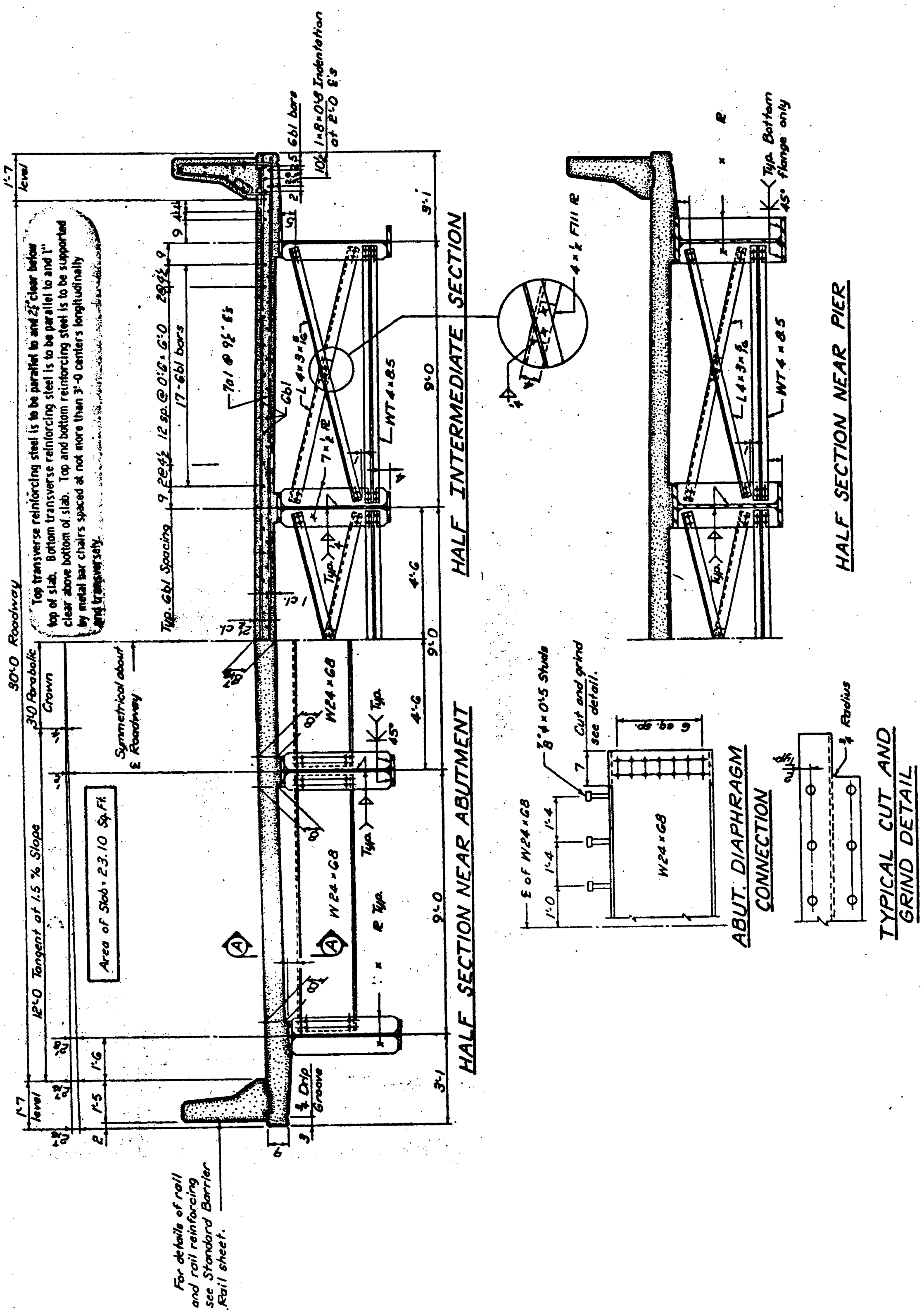


Figure A1-1 Iowa Welded Girder Cross-Section - 30' Roadway, 1978

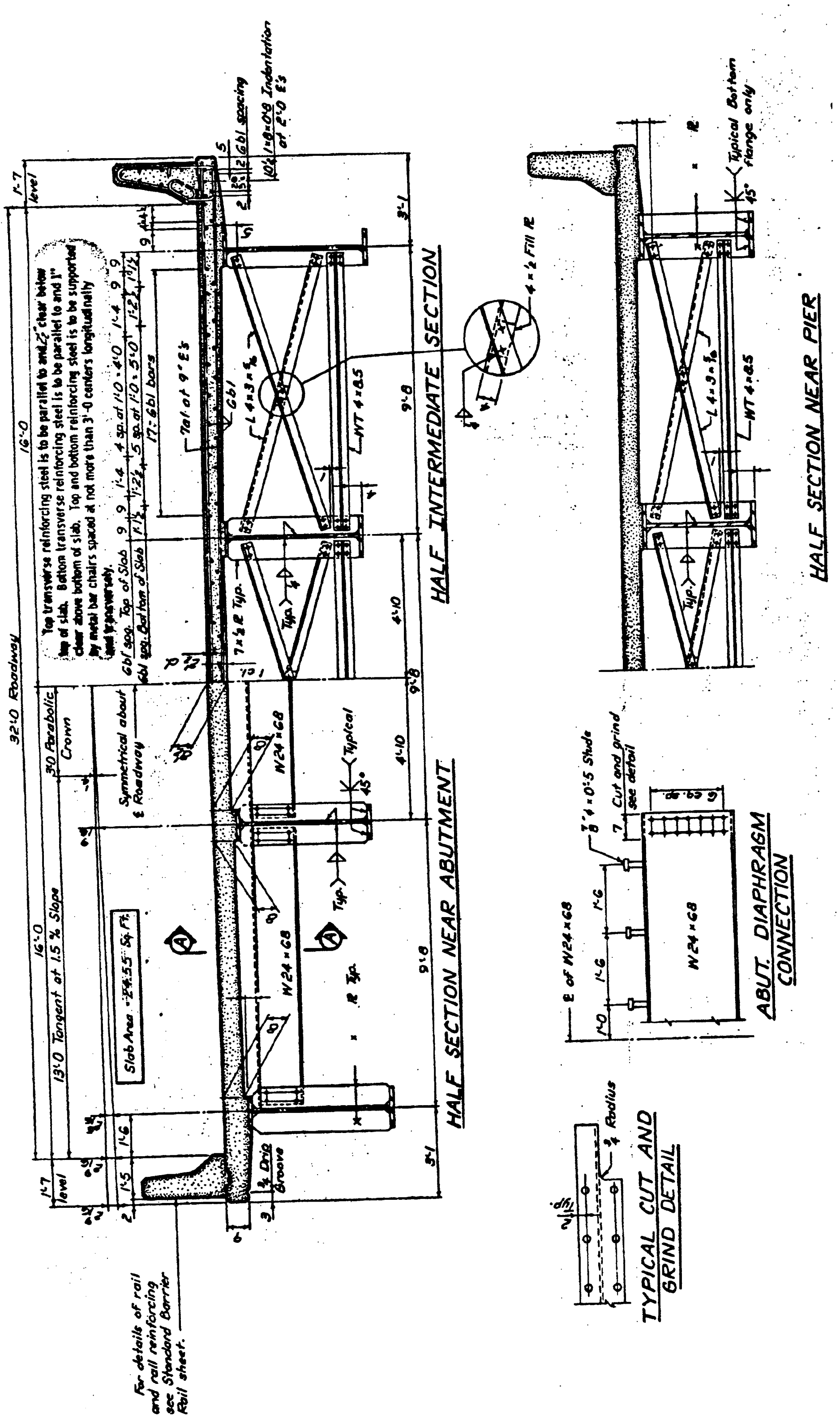


Figure A1-2 Iowa Welded Girder Cross-Section - 32' Roadway, 1978

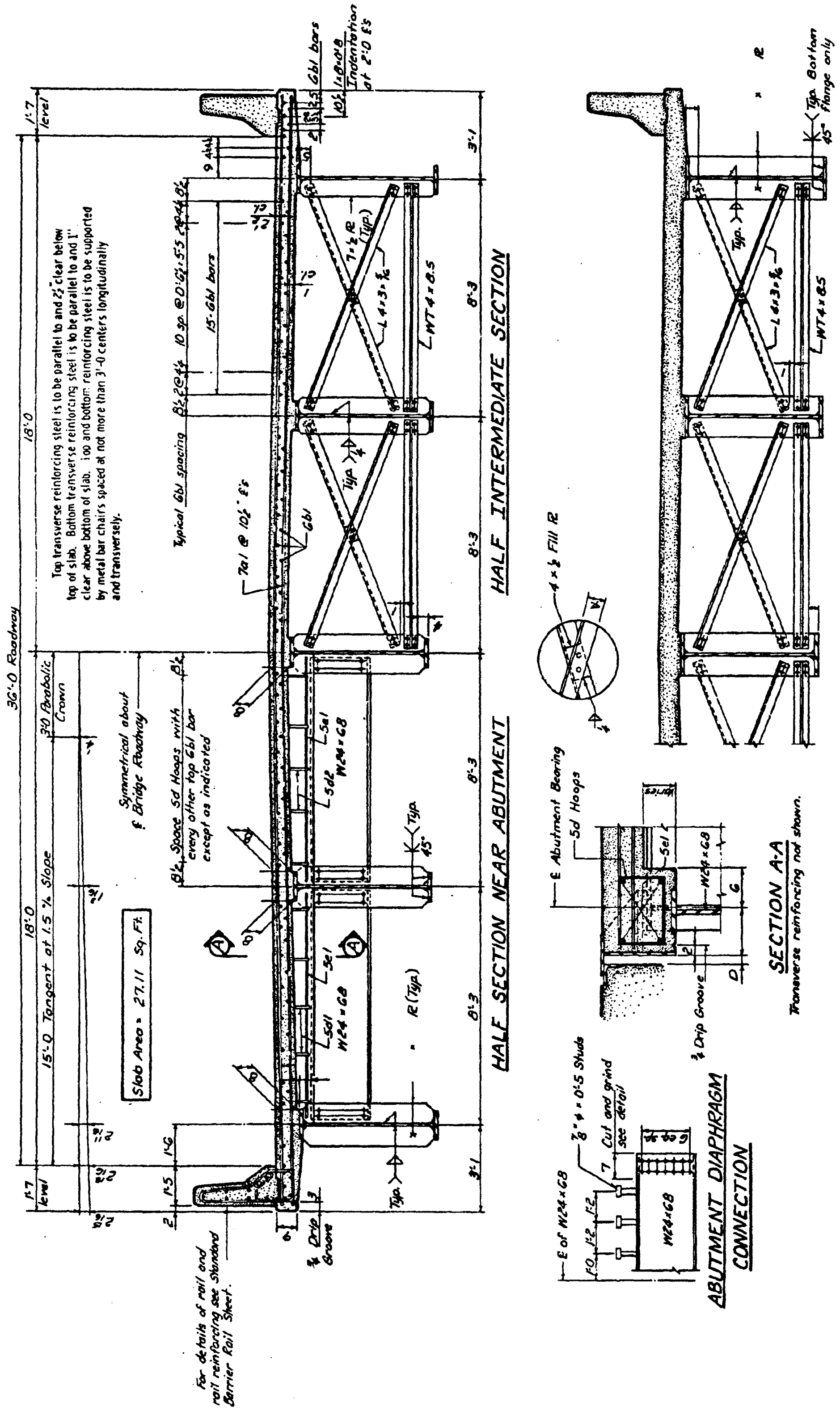


Figure A1-3 Iowa Welded Girder Cross-Section - 36' Roadway, 1978

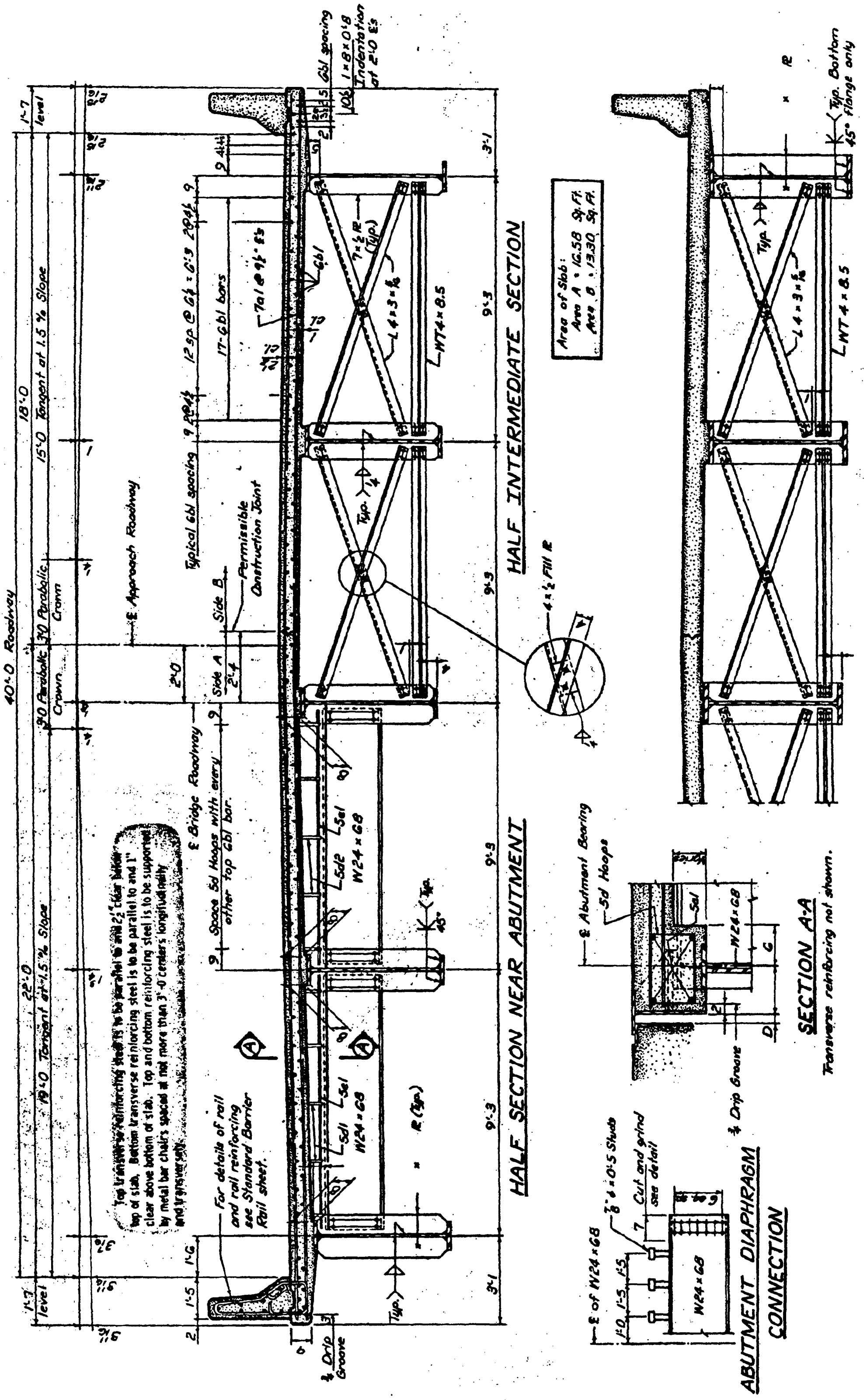


Figure A1-4 Iowa Welded Girder Cross-Section - 40' Roadway, 1978

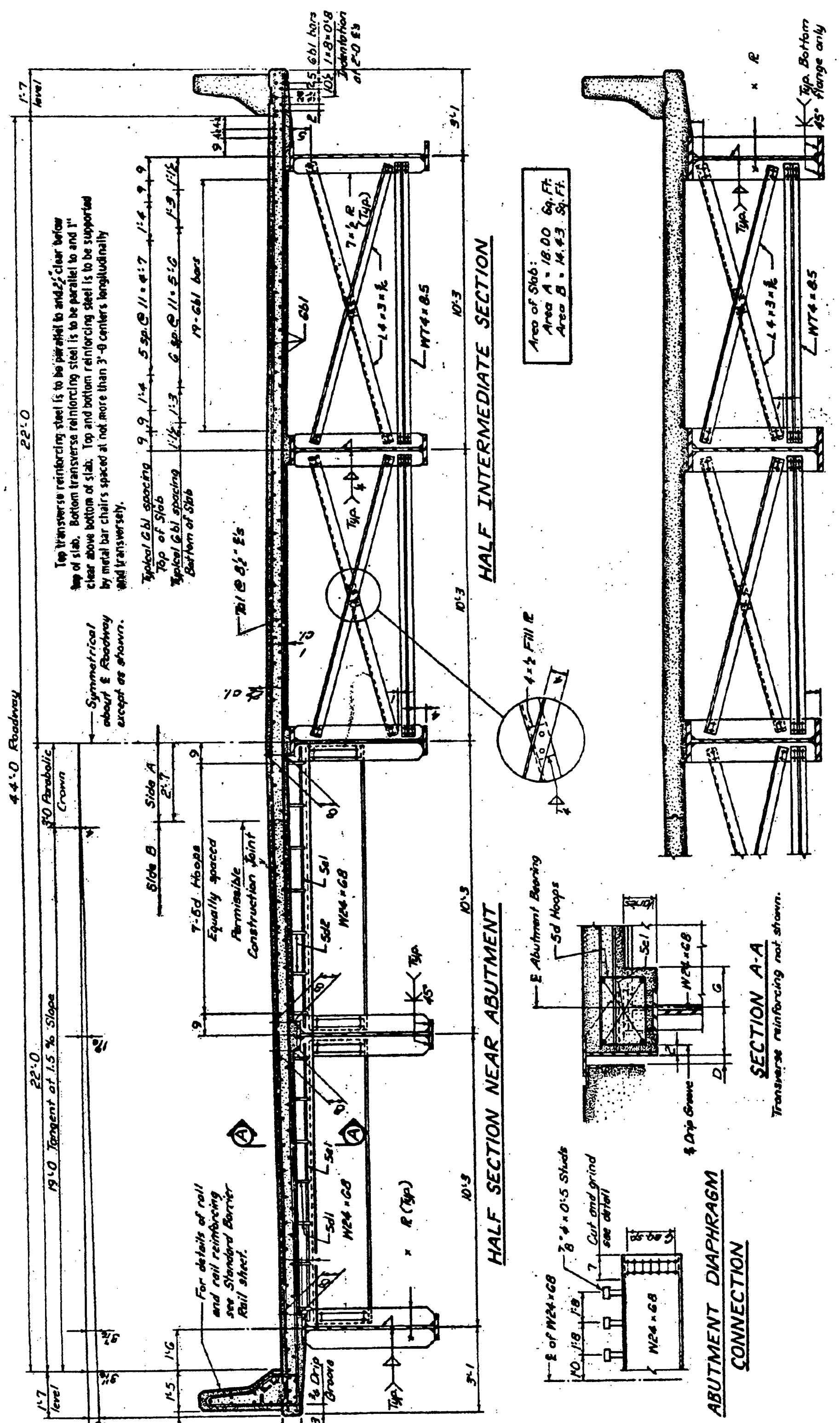


Figure A1-5 Iowa Welded Girder Cross-Section - 44' Roadway, 1978

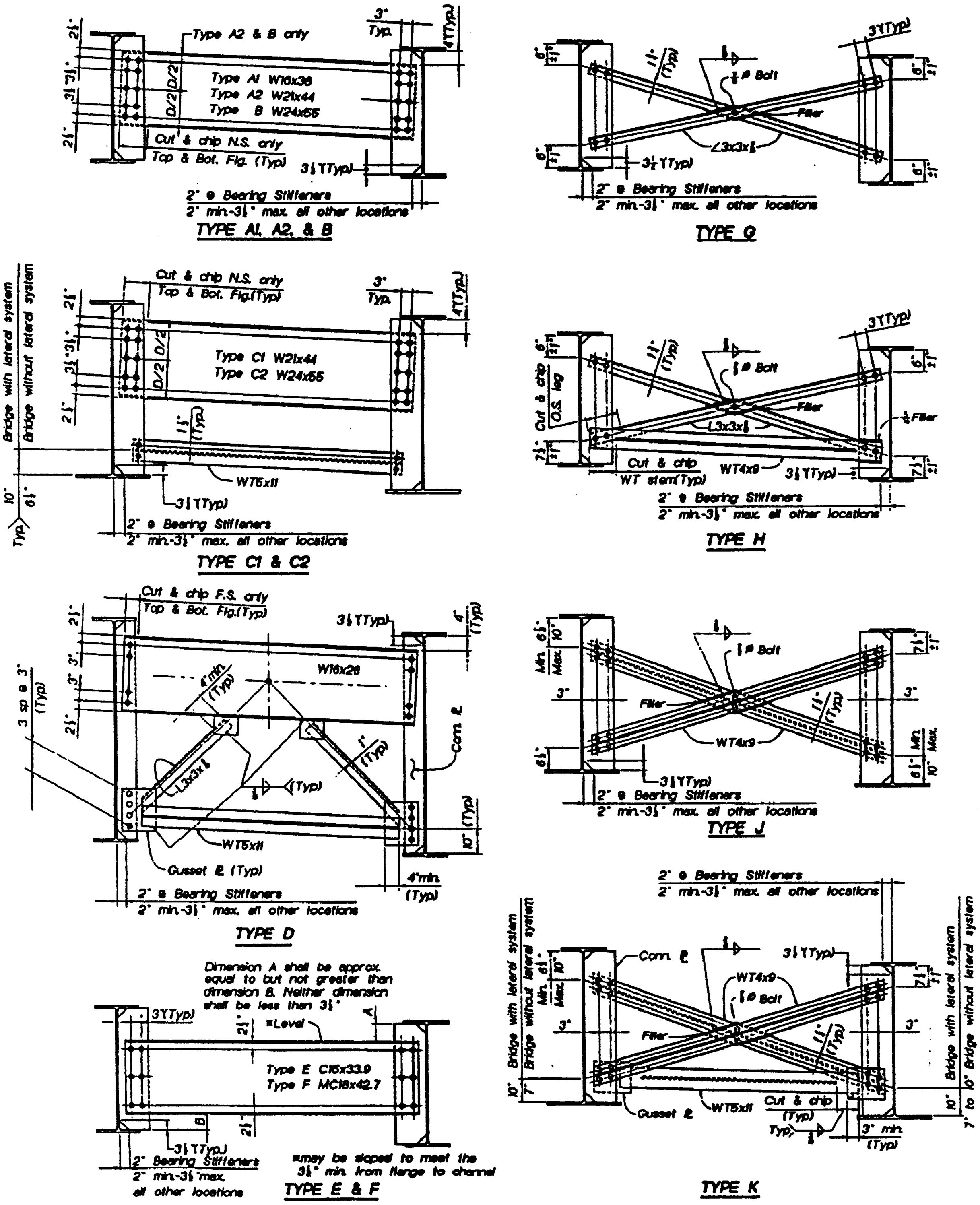
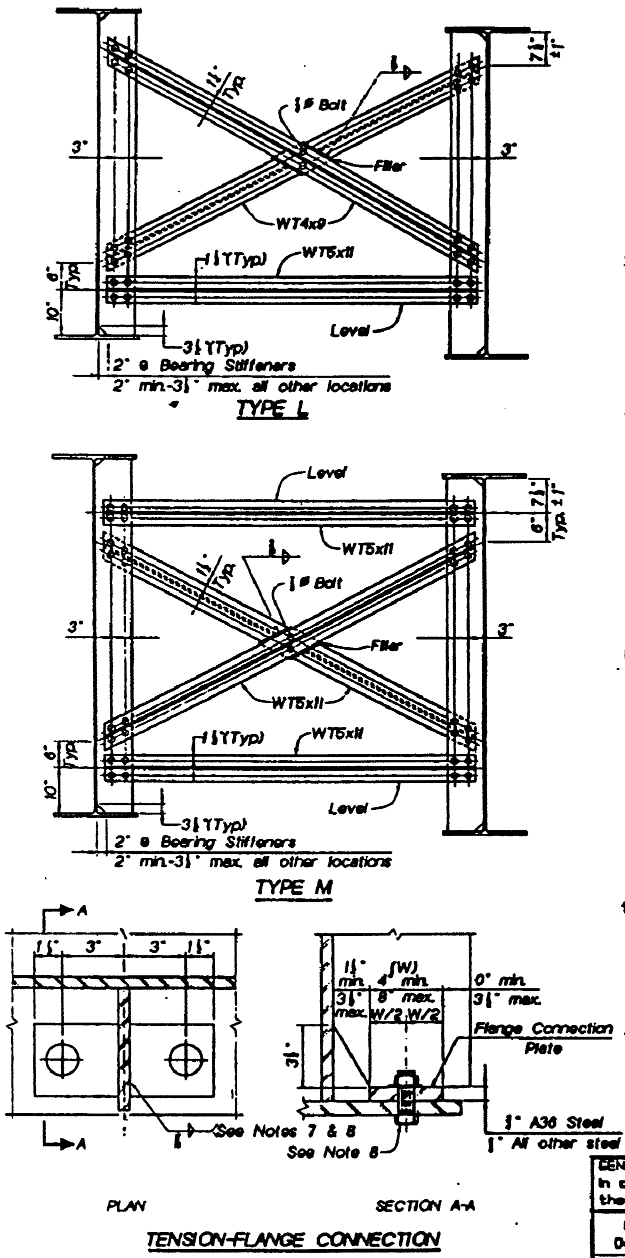


Figure A1-6 Maine Standard Details - Diaphragms and Cross-Frames, 1993



FABRICATION NOTES

1. All bolts shall be 7/8" dia. H.S. Bolts. Hole sizes for bolts shall conform to Section 504.23 of the Standard Specifications, and edge-distances shall be 1½" min. unless otherwise shown.
2. Connection plates and gusset plates shall have a minimum thickness of 3/8" and shall have sufficient width to provide erection clearance. Connection plates shall have a minimum width of 7". For all stiffeners and bent connection plates, the plate thickness will be given on the design drawings.
3. Depending on the skew angle, stiffeners and connection plates shall be welded to the web plates with either fillet welds or a single bevel groove weld. Fillet welds shall be the minimum size specified by the AWS Structural Welding Code D1.5, table 2.2 unless otherwise shown on the design drawings. Fit-up shall meet the requirements of AWS D1.5, Art. 3.3, Assembly.
4. All stiffeners and connection plates shall extend to both the top and bottom flanges and shall be welded to the flanges with a fillet weld on both sides of the plate, except as indicated by note 5 and/or 8. Fillet weld size shall be as specified under note 3.
5. Connection plates and stiffeners used as connection plates shall be connected to flanges in tension and stress reversal areas with the "Tension-Flange Connection" detail. All other stiffeners shall fit within 1/16" (tight fit) at flanges in tension and stress reversal areas and shall not be welded.
6. Bearing stiffeners shall be machined to have full bearing, and shall have a fillet weld on each side.
7. All fillet welds which connect stiffeners or connection plates to either a flange or web plate, shall be started and stopped approximately ½ inch from the ends or edges of the plate.
8. Bolt tension-flange connection plate to flange before welding stiffener or diaphragm connection plate to it.
9. All dimensions shown as "__±1" are variable in order to allow a series of crossframes to have the same slopes and/or dimensions.
10. For unpainted applications all steel for diaphragms and crossframes shall be A.S.T.M.-A588. For bridges specified to be painted the steel fore diaphragms and connection plates shall be A.S.T.M.-A36, except other steel classifications may be used subject to the approval of the Engineer.
11. Use only those items called for on the design drawings.

GENERAL NOTE

In case of conflict between these Standard Details and the Design Drawings, the requirements of the Design Drawings shall be followed.

REVISIONS Description	APPROVED MeDOT	FHWA	STATE OF MAINE DEPARTMENT OF TRANSPORTATION
Original Plan	JUNE 1993		STANDARD DETAILS BD 112 - 93 DIAPHRAGMS & CROSSFRAMES
			SHEET OF AUGUSTA, MAINE JUNE 1993

Figure A1-7 Maine Standard Details - Diaphragms and Cross-Frames, 1993

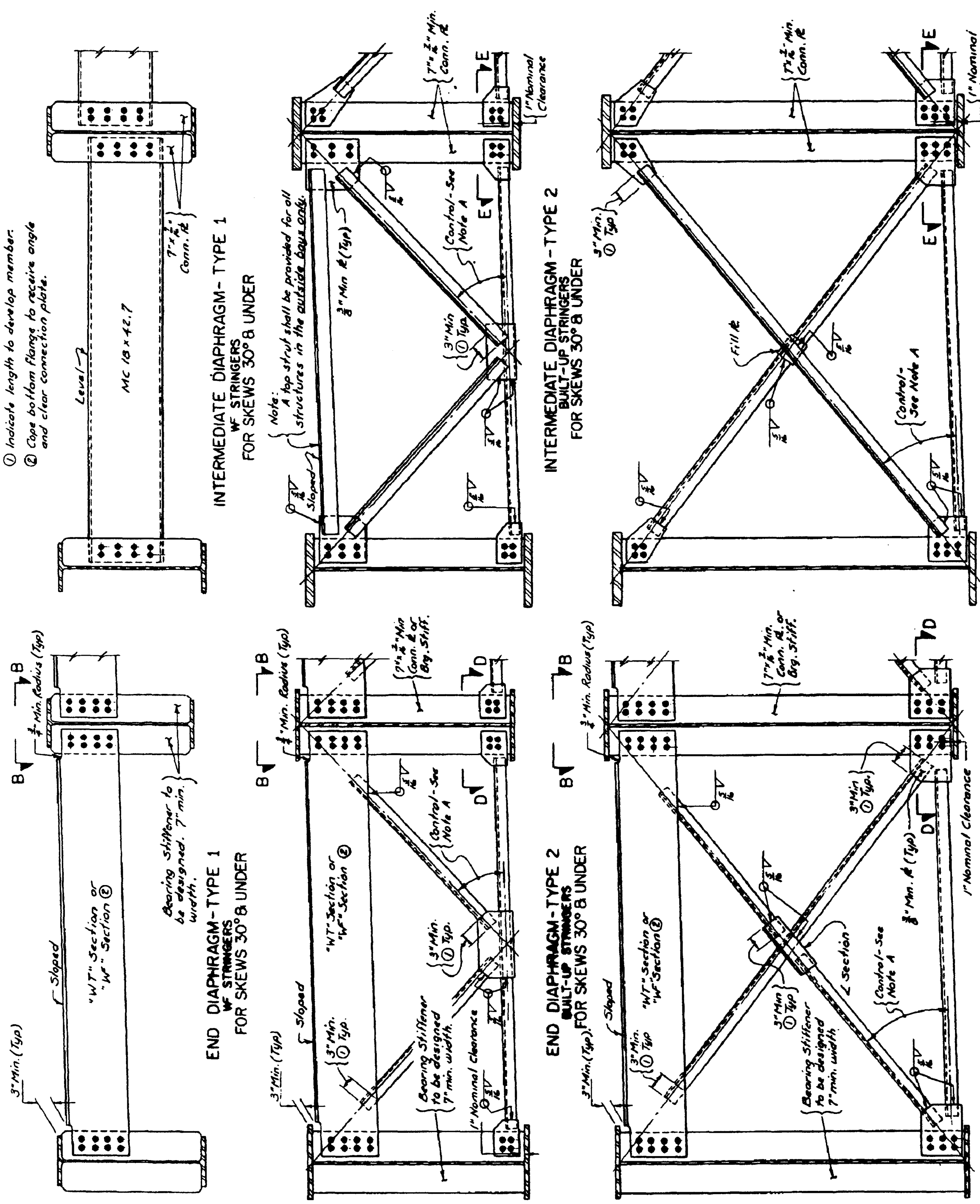


Figure A1-8 New York Diaphragm Details - Bridges with Straight Stringers, 1979

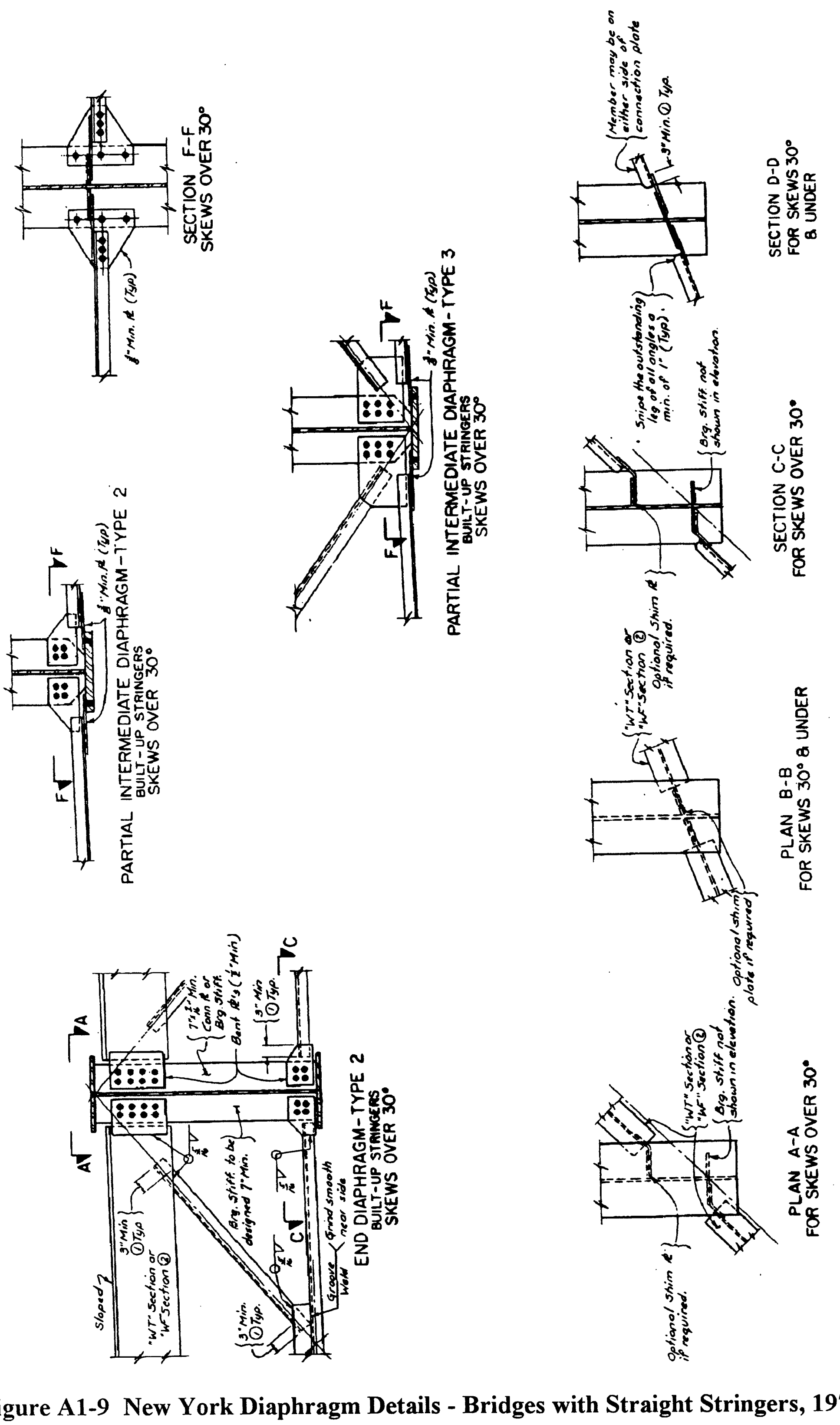


Figure A1-9 New York Diaphragm Details - Bridges with Straight Stringers, 1979

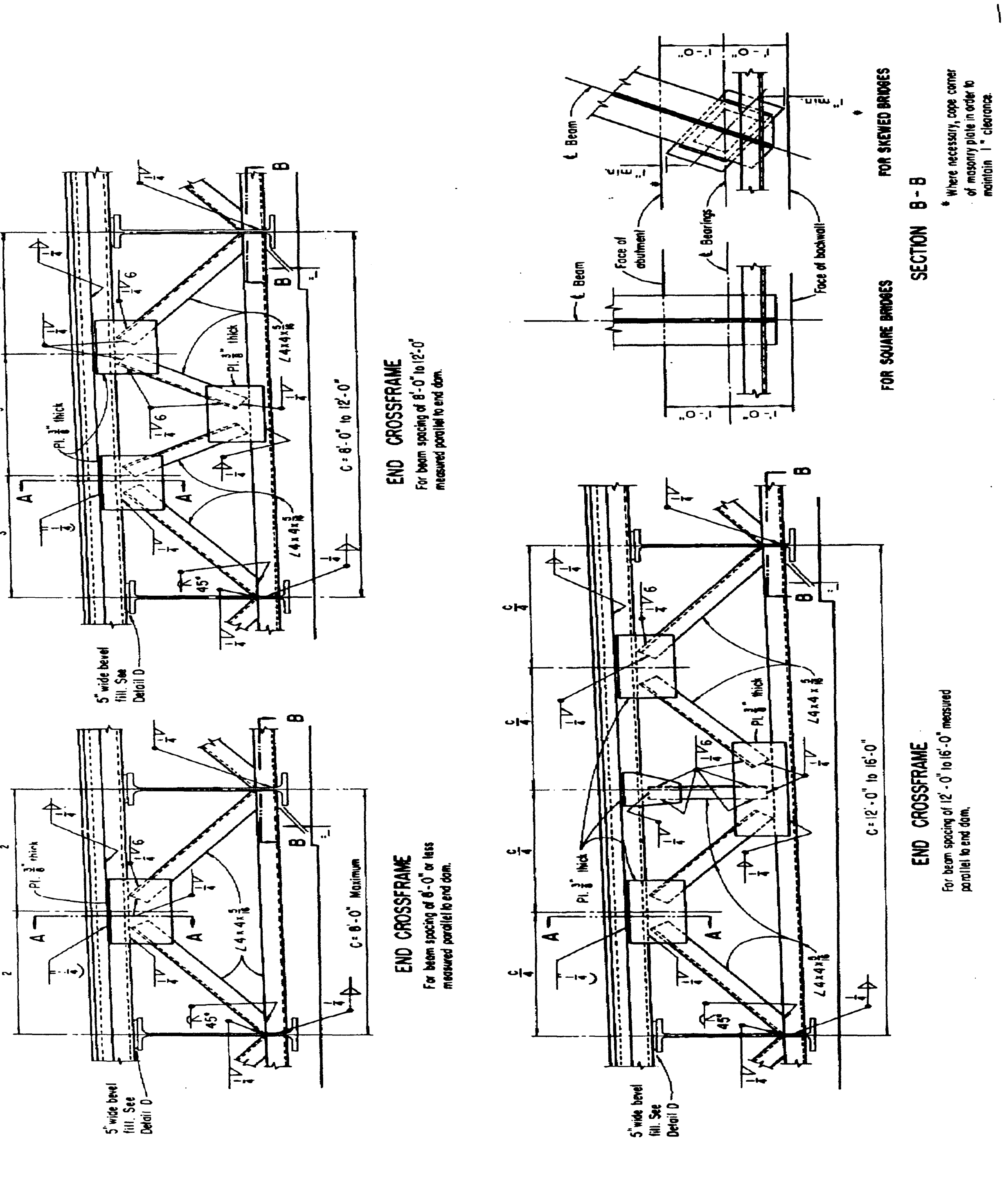


Figure A1-10 Ohio Standard Superstructure Details for Steel Beam and Girder Bridges, 1969

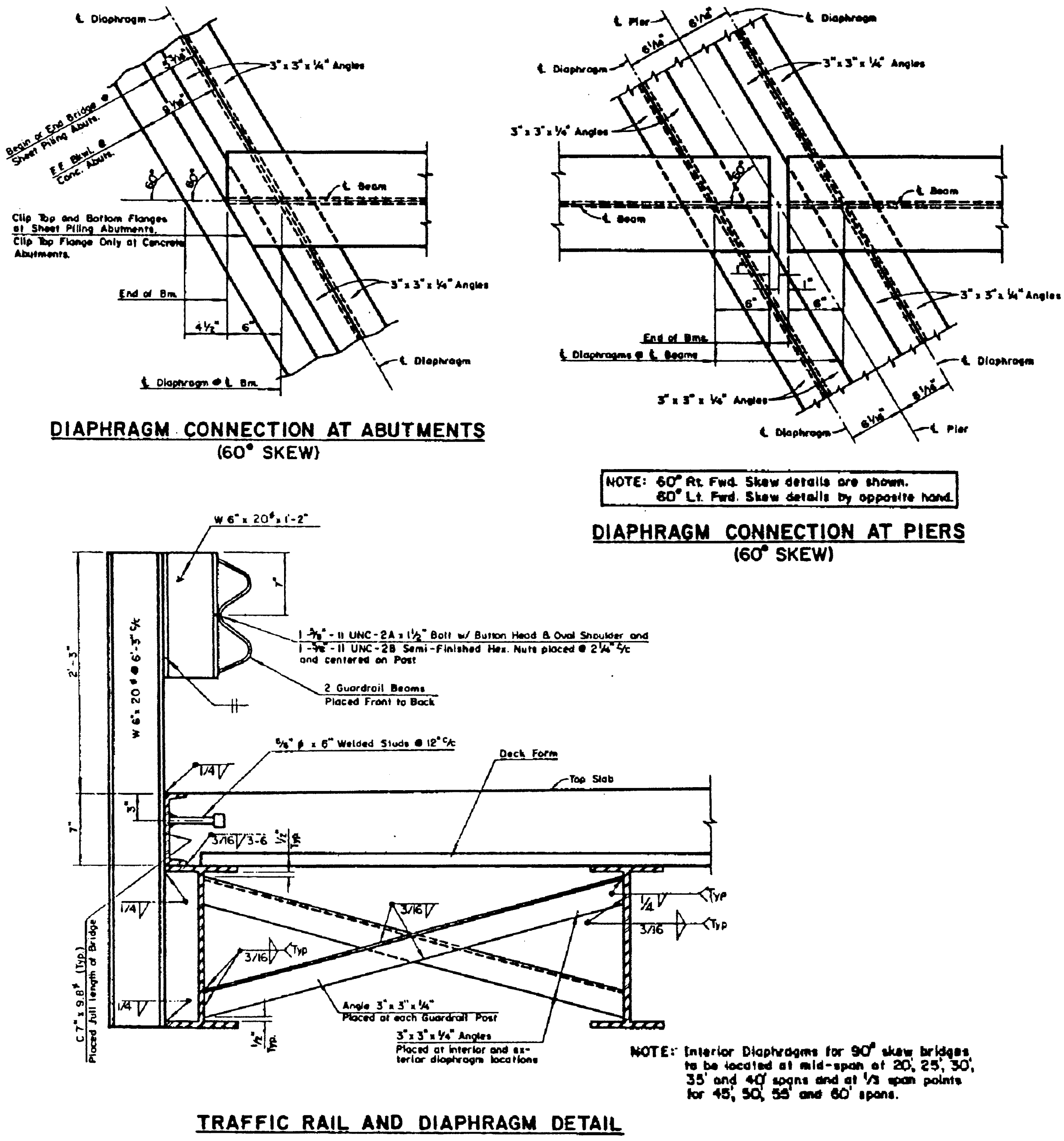


Figure A1-11 Oklahoma I-beam Bridge, Non-composite, 1981

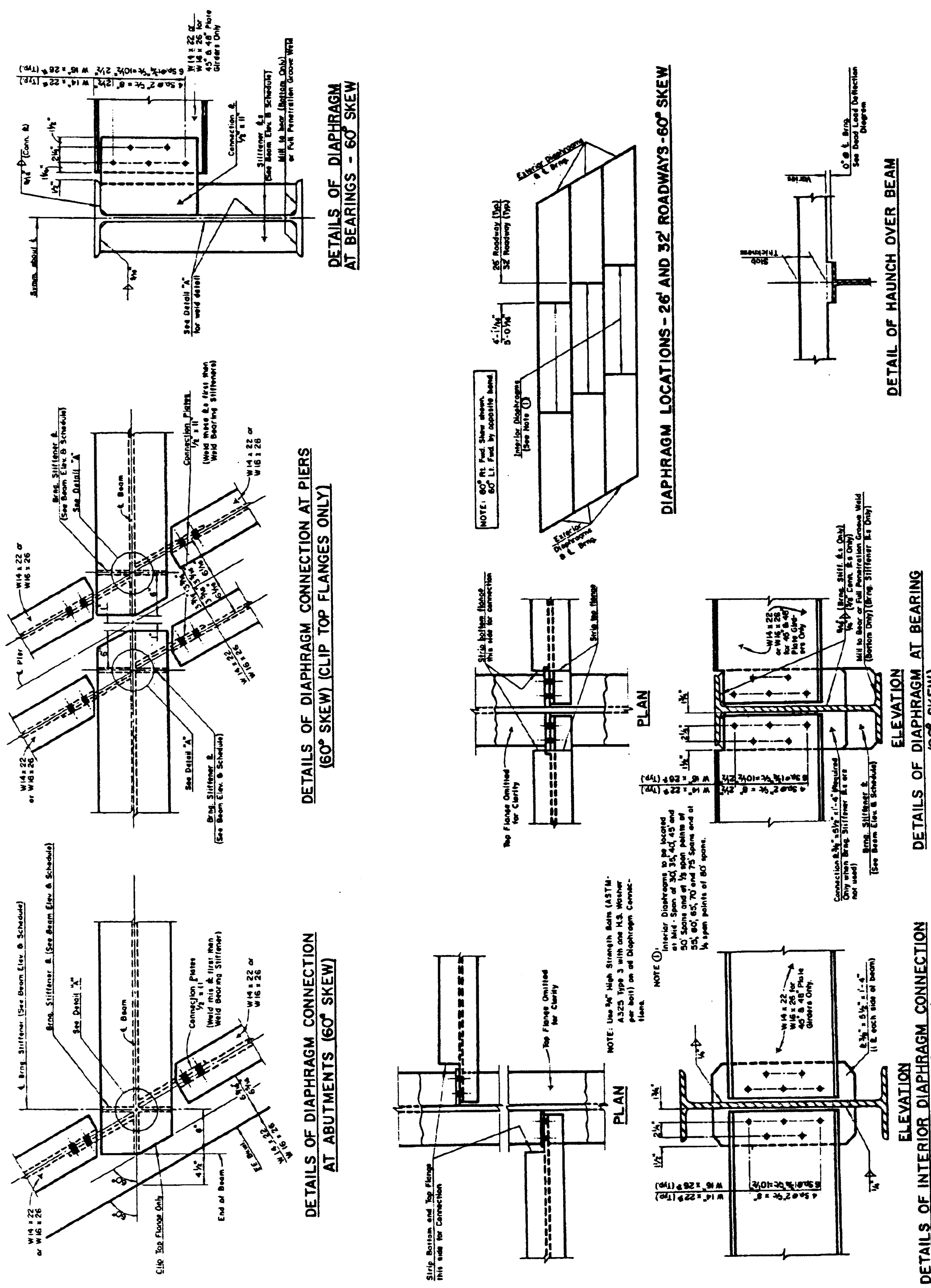


Figure A1-12 Oklahoma Miscellaneous Details for Composite I-beams and Plate Girders, 1981

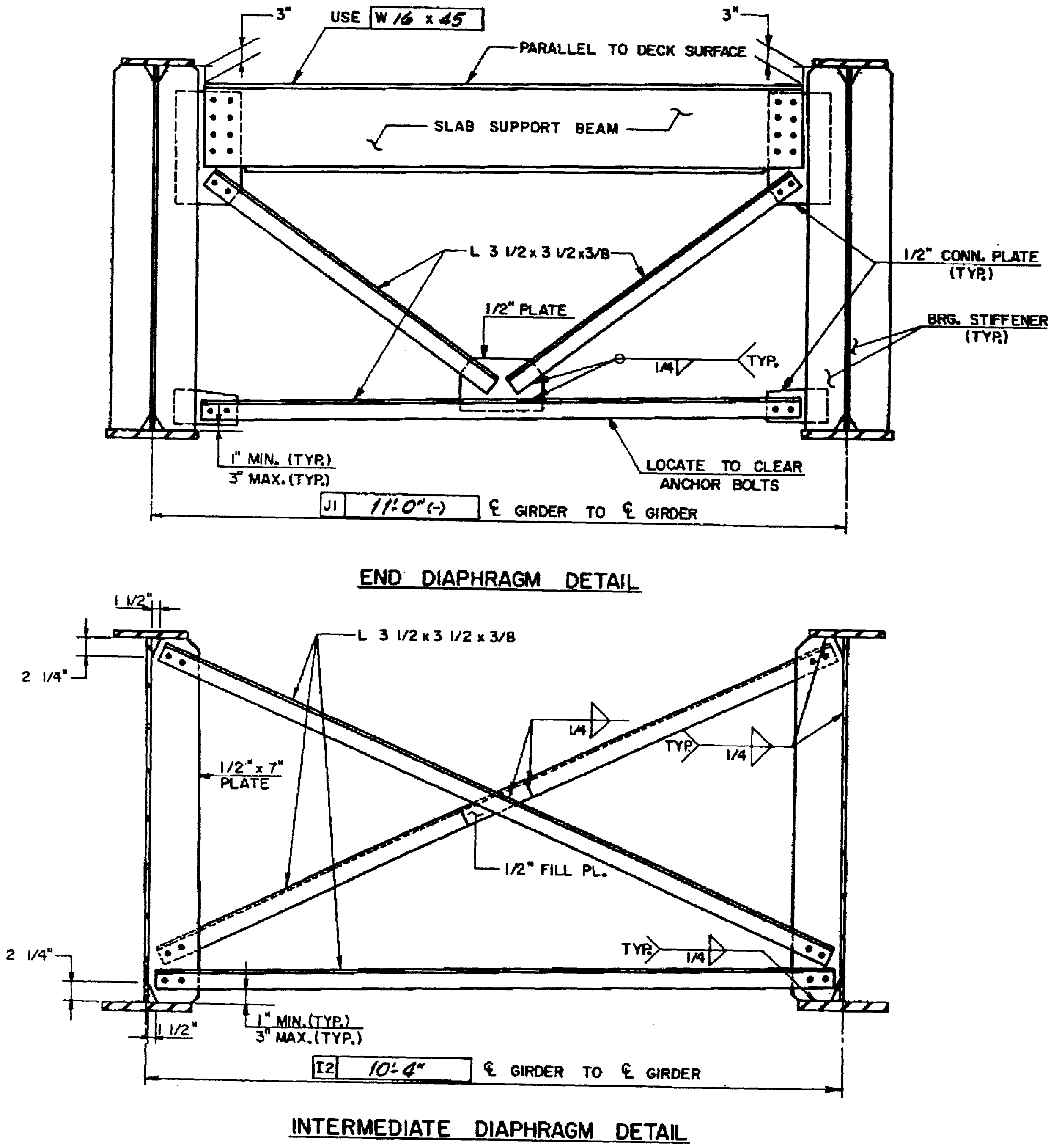


Figure A1-13 Pennsylvania Simple Span Composite Steel Multi-Girder Bridge Diaphragm Details, 1983

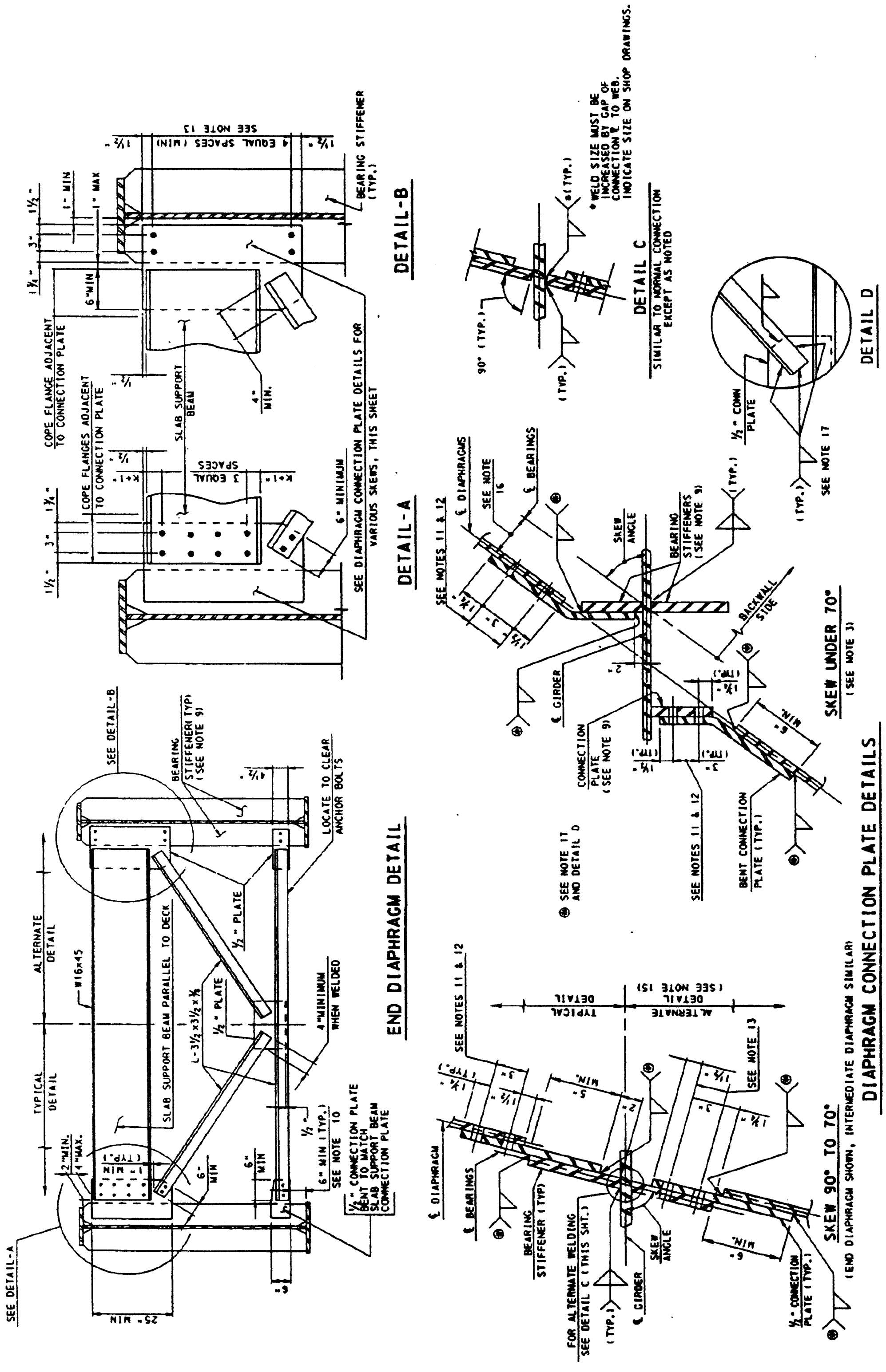


Figure A1-14 Pennsylvania Standard Steel Diaphragms for Steel Beam/Girder Structures (Straight Girders Only), 1994

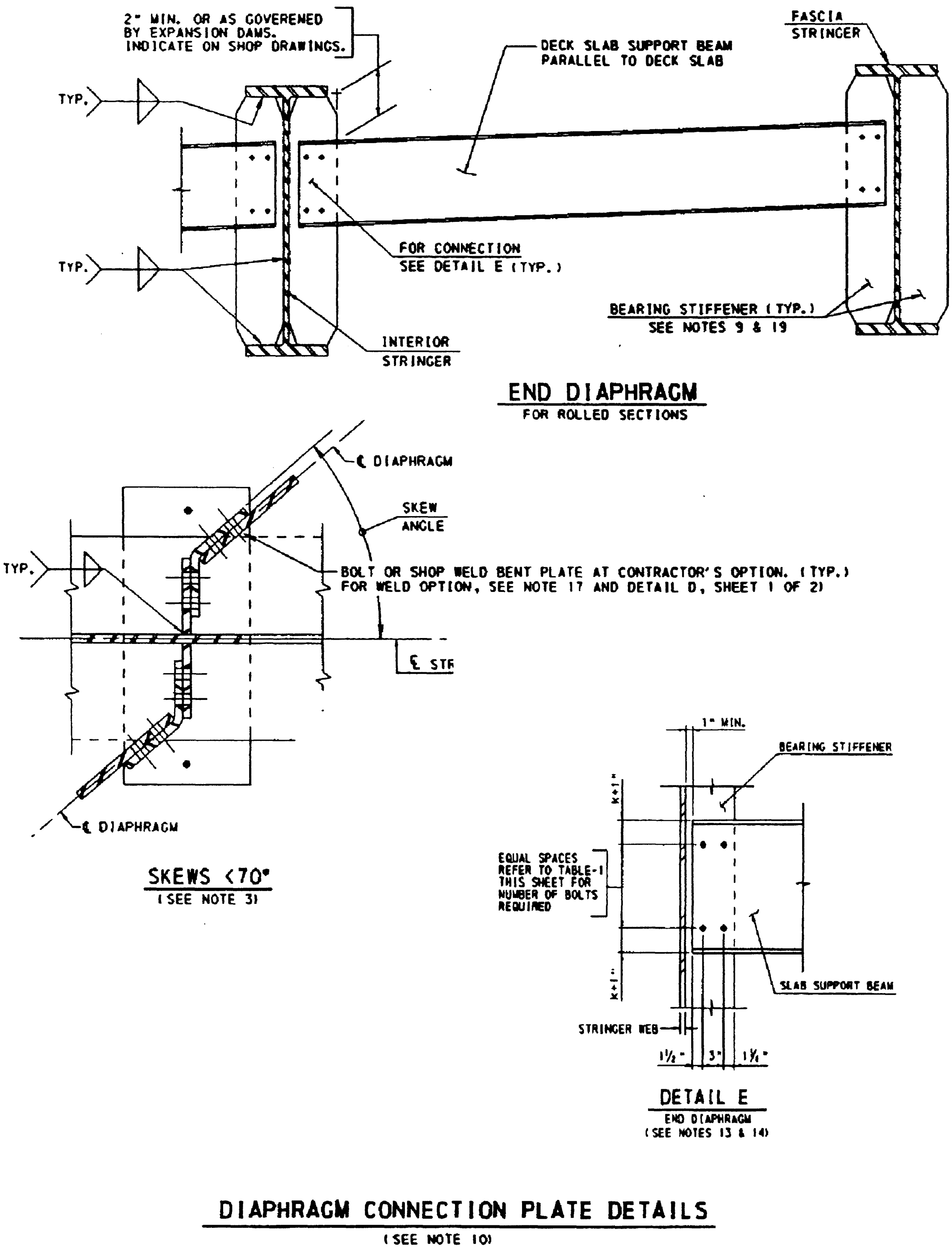
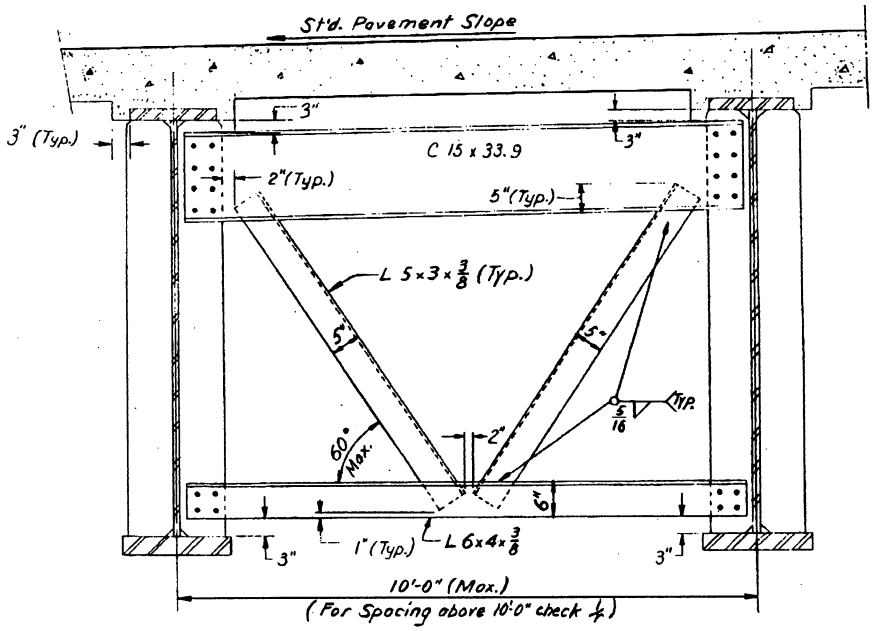


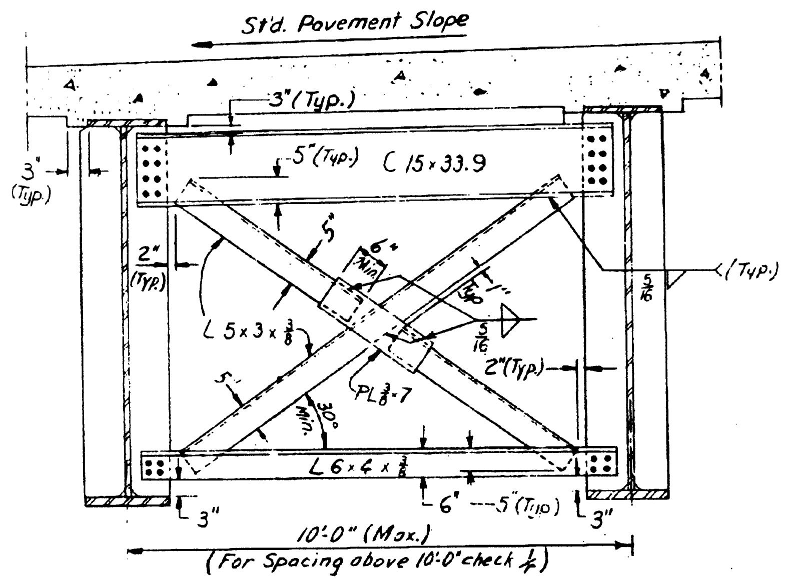
Figure A1-15 Pennsyvania Standard Steel Diaphragms for Steel Beam/Girder Structures (Straight Girders Only), 1994



TYPICAL END DIAPHRAGM
(WITH V DIAGONALS UP TO 60° ANGLE)
STRAIGHT CROSSING

82Jul27 - HBN V-2-07.20-3

Figure A1-16 Virginia, 1982



TYPICAL END DIAPHRAGM
(WITH X DIAGONALS, WHEN V DIAGONALS REACHED 60° ANGLE)
STRAIGHT CROSSING

82Jul27 - HBN V-2-07.20-4

Figure A1-17 Virginia, 1982

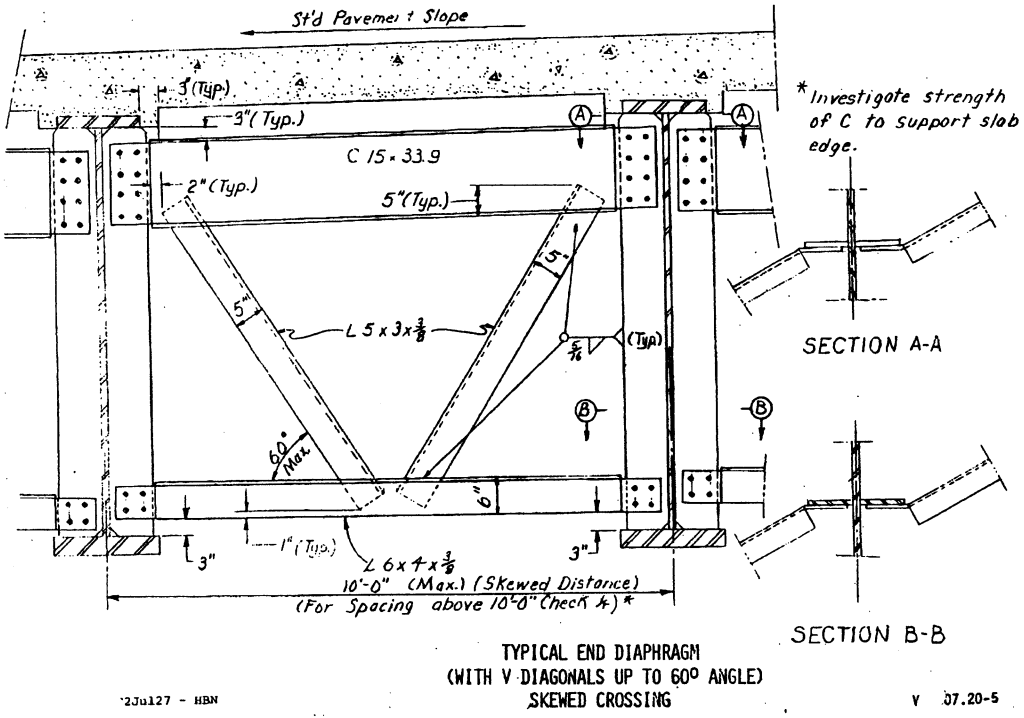


Figure A1-18 Virginia, 1982

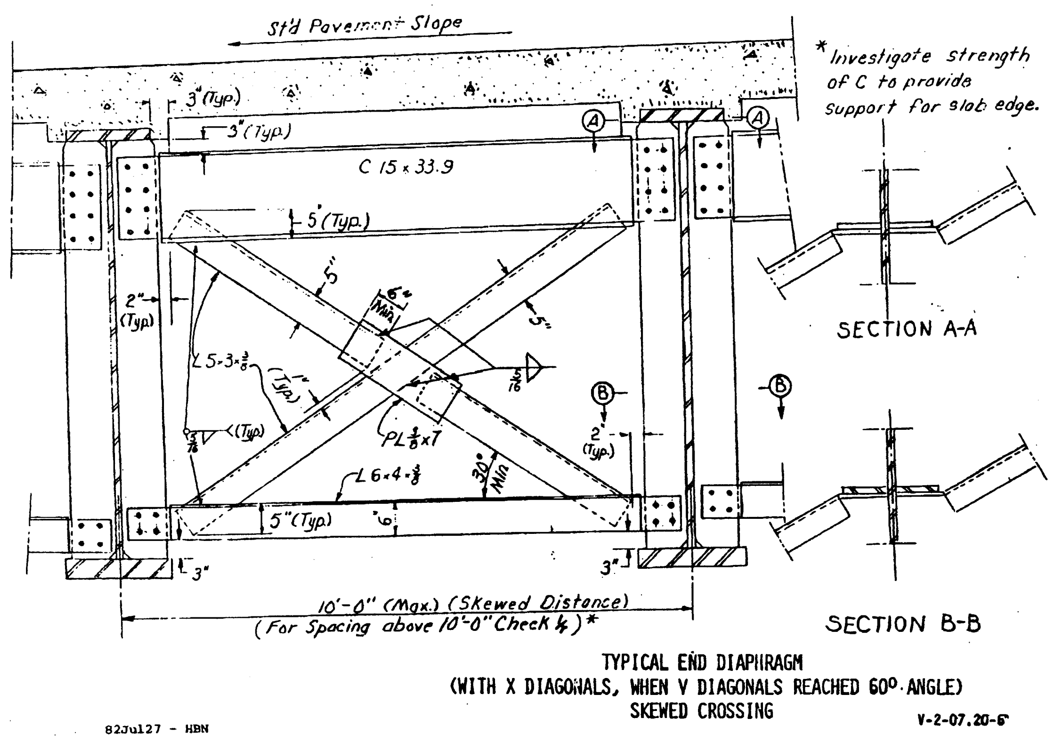


Figure A1-19 Virginia, 1982

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