C5. Steel and Cast Iron (Systematic Rehabilitation)

C5.1 Scope

No commentary is provided for this section.

C5.2 Historical Perspective

This section provides a brief review of the history of cast iron and steel components of building structures. The information was provided through discussions with some structural engineers with decades of experience, examination of plans of older buildings constructed in the early part of the 20th century, review of older steel design textbooks, and review of the *Engineering News Record* and ASCE *Transactions* for the period from approximately 1880 through 1930.

History of Steel Materials and Processes. Iron and steel have been used in the construction of buildings for centuries. Cast iron was first developed as early as 200 BC, and it was produced in significant quantities in the United States during the late 18th century and throughout the 19th century. Cast iron has a relatively high carbon content (more than 1.5%) along with silicon and sulphur. As a result, cast iron is hard and brittle, with limited tensile strength. It is difficult to work, so it must normally be used in cast assemblies. Because of its availability and fairly good compressive strength, it was used quite extensively for columns in buildings built in the early to middle 19th century. Engineers preferred not to use cast iron in components that were either part of a lateral load system or developed significant bending or tension, because of brittle and dramatic failures of cast iron components in bridges and other structures. Cast iron continued to be used into the early part of the 20th century, but wrought iron became the more dominant material in the late 19th century, and steel overtook both in the early 1900s.

Wrought iron was first developed through the hand puddled process in 1613. The metal produced by this process was somewhat variable, depending upon the skill of the producer, and only relatively small quantities of metal could be produced. As a result, this early wrought iron could appear in buildings built before approximately 1850, but it is not likely to be a major structural element because of the small volume that could be produced. Mechanical methods for producing larger quantities of wrought iron were developed in the mid-1800s, and wrought iron was used in the structural systems of a substantial number of buildings in the late 1800s and early 1900s. Wrought iron is much more workable than cast iron; it is more ductile and has better tensile capacity. As a result, it was a more versatile construction material than the cast iron that preceded it. However, for columns, cast iron was still viewed as the most economical material until very late in the 1800s.

Steel was largely made possible by the development of the Bessemer process combined with the open hearth furnace. The Bessemer process was patented in 1856, but steel does not appear to have become commonly available until about 1880. This delay was partly due to some legal disputes, as well as fundamental concerns about the properties and quality of the material. In 1880, wrought iron still dominated the structural market, and buildings built in the mid-1890s were still most likely to be built of wrought iron (possibly with cast iron columns) rather than steel, but most engineers of that period believed that low carbon structural steel was the superior material and would dominate future building construction.

In 1894–95, the first specification for structural steel was published (Campbell, 1895). This document did not address building design, but established quality control and standardization requirements for the material. In 1896, the steel manufacturers agreed to establish some standardization in the shapes that they produced, and steel proceeded to totally dominate the structural market during the next 10 years.

A number of tests for steel and structural steel components are reported during the 1890s. Examination of the reported test results suggests that the properties of this early steel were not very different from the A36 steel used in the 1950s and 1960s. The yield stress may have been somewhat lower, and the early standard designation for this mild steel was A9 with a nominal yield stress of 30 ksi. In the late 1890s fire tests were performed on steel members, and engineers became concerned about fire protection. Masonry was used to enclose the steel and provide fire protection in some early buildings, but it appears that concrete encasement became the predominant form of fire protection at about the start of the 20th century. Riveted connections were the primary method for connecting both wrought iron and steel members during this period.

Steel construction proceeded in a fairly continuous manner in the following years, although there was quite a wide variation in the structures and the materials used in the structures because of particular requirements of the designer. Welding techniques were first developed around 1915 and used in a few structures in the 1920s and 1930s, but usage was limited due to poor quality. Mild steel bolts also had limited usage during this period, and A7 steel with a nominal yield stress of 33 ksi arrived on the scene, essentially replacing A9 by 1940. Further standards for steel and steel products were developed, largely due to the efforts of the American Institute of Steel Construction (AISC), established in the 1920s. This second wave of standardization, with the structural designer involved in the process, resulted in greater uniformity in both the steel and structural steel shapes as well as the structural designs themselves.

Some of the early welding techniques employed gas welding, but electric arc welding was also developed in the very early 1900s. During the 1930s the use of flux and shielding of the arc began. Some structural tests on welded components were performed starting in the 1930s, and electric arc welding became common in the 1940s and 1950s. By the mid-1960s, the use of riveted connections was abandoned as high-strength bolts and electric arc welding became the standard connection technique.

Around this time, concrete encasement for fire protection was also disappearing in favor of lighter insulation methods, and A36 steel with a yield stress of 36 ksi became the standard steel. Higher-strength steels were also introduced during this period.

C5.2.1 Chronology of Steel Buildings

C5.2.1.1 Introduction

Due to the brittle nature of iron, it was not possible to produce shapes by hot or cold working. As a result, iron shapes for columns were cast and often patented.

Some typical shapes are shown in Figure C5-1 (Freitag, 1906). Due to lack of good quality control, cast pieces often had inclusions; this greatly reduced the allowable stress for cast iron columns. A good summary of the use of cast iron in the United States was recently published (Paulson, Tide, and Meinheit, 1994).

As noted in the earlier discussion, cast iron was used extensively throughout the 19th century, but its use was primarily for columns, which carried compression with no significant tension or bending. Cast iron performed poorly when it was subjected to these alternate stress states, and wrought iron had filled in as an alternate construction material for these other applications in the second half of the 1800s. Wrought iron and cast iron were largely replaced by steel at the turn of the century.

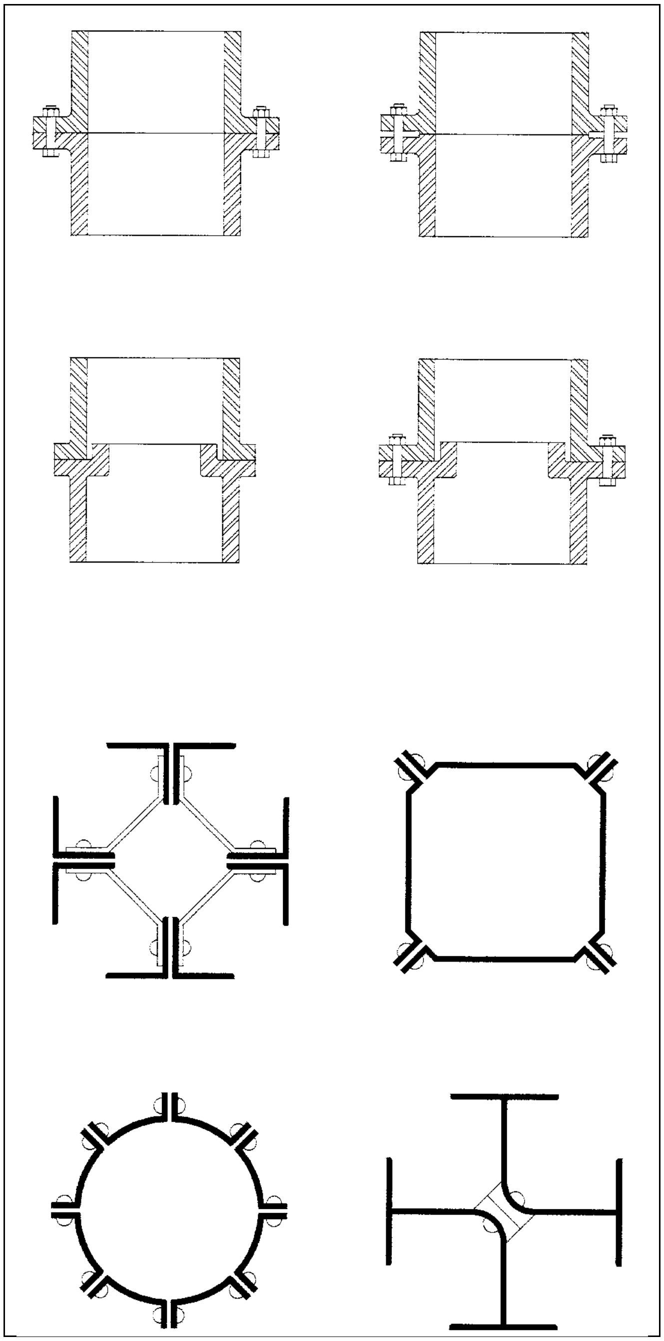


Figure C5-1 Cast and Wrought Iron Column Sections

Wrought iron and steel were more ductile than cast iron and more easily worked, and a wide range of field and shop modifications was possible.

These wrought iron and steel buildings had some common attributes, but in general, the members and connections were unique. Engineers made extensive use of riveted built-up steel and wrought iron members with riveted connections. The members were commonly built up from plates, angles, and channels. These built-up members used tie plates and lacing, and the large number of rivets made them labor-intensive. Connections were formed with haunches, knee braces, and large gusset plates. The first effort to standardize the steel materials and shapes was made in about 1895, but there was relatively little standardization in design. Each engineer would use his own unique member and connection configurations. Further, the design was controlled by local practice and city building codes. As a result, the predicted strength of the member varied widely. An article published in the mid-1890s illustrates this, noting that one column of a given material and geometry could support 100 tons in New York City, 89 tons in Chicago, and only 79 tons in Boston. These local building codes played a role in restricting the use of wrought iron over steel in many cities, and this contributed to the fuzzy transition between the two materials.

The first proposed structural design specification for steel buildings was published by ASCE (Schneider, 1905). This article examined the wide variation in design loads and stress limits, and proposed a standard design procedure, which began to become a reality with the development of the AISC specification and design manual in the 1920s.

While the members and connections were quite variable, there was a lot of similarity in the general structural aspects of these older buildings. First, they usually had massive fire protection. Massive—but lightly reinforced—concrete was used in most buildings constructed after 1900. The concrete was relatively low-strength and often of questionable quality. In addition, these buildings usually had unreinforced masonry for outside walls, and unreinforced clay tile or masonry partitions throughout the interior. These walls and partitions provide the bulk of the strength and stiffness of these older buildings for resisting lateral loads. These buildings were normally designed for wind load but not seismic loading. They were designed as moment frames, with the tacit understanding that infilled walls help to resist lateral loads but do so without any design calculations.

To illustrate further the variability of construction in this era, it should be noted that engineers readily and quickly shifted from one material to another. Concrete encasement was not considered in the evaluation of the strength of steel structures, but it was readily used as a transition between steel and concrete construction. Some engineers shifted from steel to concrete columns, or they connected a reinforced concrete beam to a steel column or beam, and used the encasement for the development of the two different members.

C5.2.1.2 1920 through 1950

In the 1920s, use of the unique, complex built-up members began to be phased out, and standard I and H shapes replaced them as the standard for member design. Partially restrained (PR) connections, such as the riveted T-stub and clip angle connections discussed in Section C5.4.3.3, became the normal connection. Because the clip angle connections were weaker and more flexible, they were used as the beam column connections in shorter buildings or in the top stories of taller buildings. The T-stub connection was stiffer and stronger, and it was used in the lower floors of taller buildings where the connection moments were larger. Stiffened angle or T-stub connections were often used to provide a beam connection to the weak axis of the column.

Lightly reinforced concrete was still used for fire protection. The concrete was sometimes of higher strength, but still often of questionable quality. Unreinforced masonry was still used for outside walls and unreinforced clay tile for masonry partitions throughout the building. Buildings constructed in regions regarded as seismically active were designed for seismic forces, but the design forces were invariably lower than those required today. However, the walls and partitions were not included in the design calculations, and they still provided the bulk of the strength and stiffness of these buildings. Buildings outside of regions of known seismic activity were designed for wind load only.

It should be noted that all buildings constructed during this era used relatively simple design calculations compared to modern buildings. Engineers frequently resorted to observations from past building performance and standard practice; the sophisticated computer calculations used in modern structures were unknown.

Bolts and welding were sometimes used, but rivets were clearly the dominant connection. They were designed as moment frames, but actual structural behavior was strongly influenced by stiff, strong masonry infills and partitions.

C5.2.1.3 1950 through 1970

Significant changes began to appear during this period. The use of rivets was discontinued in favor of high-strength bolts and welding. In the very first structures, bolts were merely used to replace the rivets in connections such as the clip angle and T-stub connection illustrated in Figure C5-2. However, flange plate and end plate connections, such as those discussed in Section C5.4.3.3, were used more frequently. Increased use of and confidence in welding made these connections possible. By using these connections, engineers were often able to develop greater connection strength and stiffness with less labor. Another important change was the replacement of standard concrete fire protection by more modern lightweight materials.

Two more changes are notable. For one, masonry and clay tile walls were less frequently used for cladding and partitions, reducing building weight, although the architectural elements were still significantly heavier and stiffer than those used in steel frames today. However, these panels and finishes were more likely to be attached to the structure rather than being used as an infill to the frame. As a result, buildings built during this era are sometimes less able to utilize this added strength and stiffness than are the older structures. Finally, significant differences began to evolve in the way buildings were designed for regions of high seismic activity, and for other regions. These regional differences were developed because regions with significant seismic design requirements had to deal with larger lateral forces, but also because of the increased emphasis on ductility in seismic design procedures. In less seismically active zones, the weaker, more flexible connections were retained for a longer period of time, while in the seismically active zones the fully restrained FR connection discussed in Section C5.4.2 began to evolve. Also, braced frames and alternate structural systems were used because they could often achieve much greater strength and ductility with less steel and more economical connections.

C5.2.1.4 1970 to the Present

The trends established in the 1960s continued into the following period. First, there was increased emphasis

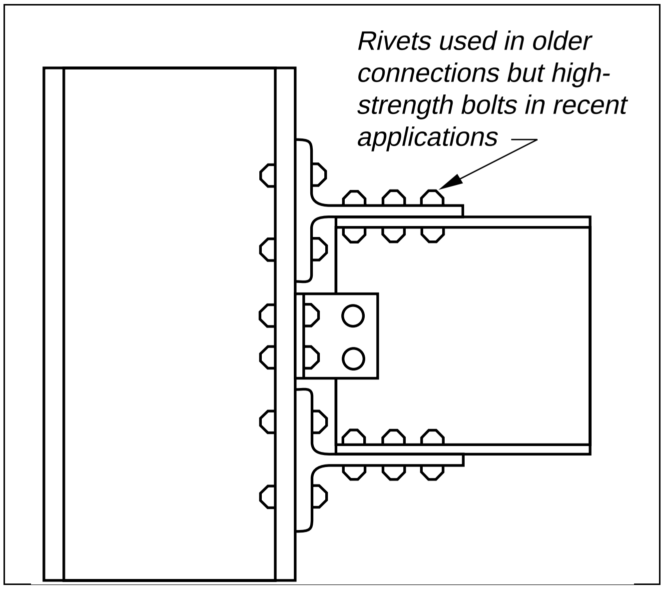


Figure C5-2 Riveted T-Stub Connection

on lightweight fire protection and architectural elements. As a result, the reserve strength and stiffness provided by these elements was reduced.

Second, there was increased emphasis on ductility in seismic design, and extensive rules—intended to assure ductility for moment frames, braced frames, and other structural systems—were established. These rules undoubtedly had some substantial benefit, but compliance was often expensive, and there was a distinct tendency toward using structures with less redundancy, since these less-redundant structures required satisfaction of the ductility criteria at fewer locations. This reduced redundancy also resulted in larger member and connection sizes. This separation of the practice between regions with significant seismic design requirements, and those with little or no seismic design requirements, continued to widen. The less seismically active regions sometimes retained more flexible connections with greater redundancy in the overall structure.

Third, seismic design forces were appearing for the first time in many parts of the United States, and they increased significantly for all parts of the country for some structural systems. Finally, the steel and construction processes themselves were also changing. There was a significant increase in steel produced by reprocessing scrap metal in an electric furnace. As a result, the yield stress of standard steels increased, while the tensile stress remained relatively stable.

Welding evolved from the relatively expensive stick welding shielded arc process to the quicker and more economical flux core, gas shield, and dual shield processes. High-strength bolts were increasingly used as slip-critical friction bolts; however, quality control variations caused by tightening and installation became a major concern. These changes in turn produced changes in the ductility and behavior of many steel structures.

C5.2.2 Causes of Failures in Steel Buildings

Until quite recently, major failures in steel components and buildings were rare. Five steel buildings collapsed or were fatally damaged in Mexico City during the 1985 Michoacan earthquake. This damage was the result of a large torsion irregularity, a resonance condition between the soft soil and the building, and, perhaps, poor fabrication of the built-up square columns. Other typical damage include buckled braces, failure of a few connections, and damage to infills and attached cladding. Loss of entire masonry cladding from entire sides of a building was observed.

Prior to the 1994 Northridge, California earthquake, the steel moment frame was considered to be the ideal structural element to resist earthquakes because of its excellent ductility. However, during this earthquake over two hundred buildings experienced fractured beam-column or column-baseplate connections. The reasons for this poor performance are complex, and still under investigation. One significant factor was lack of quality control of the entire welding process, in combination with the use of weld filler that has almost no notch toughness. Other factors contributed to this poor behavior, such as the thickness of the column and beam flanges, the stiffness and strength of the panel zones, triaxial stress effects, high confinement of the joints, and poor welding procedures, for example, high heat input, rapid cooldown, and conditions allowing hydrogen embrittlement. A discussion of the different types of fractures and ways of preventing or repairing them is given in FEMA 267 (SAC, 1995). The increased beam depths used in current designs also played an important role (Roeder and Foutch, 1996), along with poor quality in construction.

C5.3 Material Properties and Condition Assessment

C5.3.1 General

No commentary is provided for this section.

C5.3.2 Properties of In-Place Materials and Components

C5.3.2.1 Material Properties

No commentary is provided for this section.

C5.3.2.2 Component Properties

Identification of critical load-bearing members, transfer mechanisms, and connections must be established on the basis of a review of available data. It is often possible to classify structural member types—whether rolled or built-up—and material grade and general properties, by examining the original building drawings and construction documents. Local verification of matching members and materials to the construction documents is necessary in order to examine any gross changes that may have occurred since construction began. If these drawings and documents are not available, the subject building's components must be determined (e.g., size, condition), and the material type(s) identified.

C5.3.2.3 Test Methods to Quantify Properties

A variety of building material data is needed for conducting a thorough seismic analysis and rehabilitation design. For metallic structures, which are often enclosed or encased in the architectural fabric, these needs range from verification of physical presence to specific knowledge of material properties, member behavior, connection details and type, and condition. Many buildings have been structurally altered during their service life existence, without corresponding drawing updates or other notification. Verification of gravity and lateral-load-resisting members and their connection configuration is essential.

After member and connection presence and types are confirmed, mechanical properties must be quantified. The amount of effort needed to establish properties varies considerably, depending on the availability of building drawings and data. Several common steps may be taken to gain confidence regarding the materials used and their properties. These steps, in preferred order, include:

- Retrieval of building drawings, specifications, improvement records, and similar information
- Definition of the age of the building (e.g., when the building materials were procured and erected)
- Comparison of age and drawing information to reference standards
- Field material identification with in-place nondestructive testing
- Acquisition of representative material samples from existing members and performance of laboratory mechanical tests (e.g., tensile, offset yield, impact, chemical)
- Performance of in-place metallurgical tests to determine the relative state of the crystalline structure and presence of structural damage

Finally, the physical condition of the structural system must be examined to determine whether defects are present that would prevent any member from performing its function. For accessible members and connections, visual inspection should be performed for condition assessment. Other methods for quantifying the physical condition of a structure are specified in the *Guidelines*, Section 5.3.2.

A wide range of evaluation methods and tools exists for verifying the existence, and determining the mechanical properties and physical condition, of a metallic building element. Also, many reference standards for material behavior are given in the following reference standards for metallic structures:

1. American Institute of Bolt, Nut and Rivet Manufacturers (defunct)

   Tentative Specifications for Cold Riveted Construction

2. American Institute for Hollow Structural Sections (formerly Welded Steel Tube Institute)

   Structural Steel Tubing

3. American Institute for Steel Construction (AISC)

   Manual of Steel Construction

   Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings

   AISC Iron and Steel Beams, 1873 to 1952

4. American Iron and Steel Institute (AISI)

   Specification for the Design of Cold-Formed Steel Structural Members

   Specification for the Design of Cold-Formed Stainless Steel Structural Members

   Sectional Properties of Corrugated Steel Sheets

   AISI Standard Steels

   Fastening of Lightweight Steel Framing

   Load and Resistance Factor Design Specification for Cold-Formed Steel Structural Members

5. American Society for Metals (ASM)

   "Properties and Selection: Irons, Steels and High-Performance Alloys," *ASM Handbook, Volume 1*

   "Nondestructive Testing and Quality Control," *Metals Handbook, 9th Edition, Volume 8*, 1992

   "Failure Analysis and Prevention," *Metals Handbook, 10th Edition, Volume 10*, 1989

   "Corrosion," *Metals Handbook, Ninth Edition, Volume 13*, 1987

   "Nondestructive Testing and Quality Control," *Metals Handbook, Volume 17*, 1989

   "Metallography and Microstructures," *ASM Metals Handbook, Volume 9*, 1985

6. American Society of Mechanical Engineers (ASME)

   Bibliography on Riveted Joints

7. American Society of Civil Engineers (ASCE)

"Specification for the Design of Cold-Formed Stainless Steel Structural Members," *ANSI/ASCE 8-90*

Bibliography on Bolted and Riveted Joints (Manual 48)

"Guideline for Structural Condition Assessment of Existing Buildings," *ASCE Standard 11-90*, 1991

8. American Society for Testing and Materials (ASTM)

Annual Book of Standards (material specifications for base metals and all forms of connector material)

"Standard Practice for Measuring Thickness by Manual Ultrasonic Pulse-Echo Contact Method," *ASTM E797-87*, 1987

"Metals—Mechanical Testing; Elevated and Low-Temperature Tests; Metallography," *Annual Book of Standards, Volume 03.01*, 1993

(Particular emphasis on Designations A370, E8 [tensile], E9 [compression], E10/18 [hardness], E110 [portable hardness], E290 [ductility], and E399 [fracture toughness])

9. American Welding Society

Structural Welding Code—Steel, AWS D1.1

Code for Arc and Gas Welding in Building Construction

Filler Metal Specifications

10. Industrial Fasteners Institute (IFI)

Fastener Standards

11. International Standards Organization

Steel Construction—Materials and Design

12. Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation

Specifications for Assembly of Structural Joints Using High-Strength Bolts

Specification for Structural Joints Using ASTM A325 or A490 Bolts (Allowable Stress Design and Load and Resistance Factor Design)

13. Steel Deck Institute (SDI)

SDI Design Manual for Composite Decks, Form Decks and Roof Decks

14. Steel Joist Institute (SJI)

Standard Specifications, Load Tables and Weight Tables for Steel Joists and Joist Girders

50 Year Steel Joist Digest

15. United States Department of Commerce, National Institute of Science and Technology (formerly National Bureau of Standards)

Simplified Practice Recommendation R-216-46 (discontinued)

16. Welded Steel Tube Institute (now American Institute for Hollow Structural Sections)

Welded Carbon Steel Mechanical Tubing

Dimensions and Properties of Cold Formed Welded Structural Steel Tubing

C5.3.2.4 Minimum Number of Tests

The material testing requirements described in the *Guidelines* should be considered as a minimum. Where construction documents and drawings are not available, the design professional must insist that some inspection and material testing be done if the evaluation and rehabilitation is to proceed. This must be done even if removal and replacement of architectural features results in some inconvenience to the occupants.

ASTM Designation A370 contains standard test methods for determining tensile, bend, impact, and hardness properties of steel and iron elements. Testing of in situ materials may be done on smaller specimens than those described in A370, but the dimensions must be scaled down proportionately. Included in this specification, ASTM Designations E9 and E11 provide procedures for computing compressive strength and Young's, tangent, and chord moduli.

C5.3.2.5 Default Properties

For older buildings where steel components are encased in concrete, or for buildings with great historical importance, it may be prohibitively expensive to do all of the testing required by one of the nonlinear procedures. A lesser amount of testing may be done if it is supplemented with additional analysis. The upper and lower bounds on component force demands must be estimated. The first analysis should be done using the minimum strength values determined through testing, supplemented by default values. A second analysis must be done where lower bound material strengths are used for columns and connections and upper bound material strengths are used for braces and beams. The upper bound strengths should be 30 to 50% greater than the default values given in Tables 5-1 and 5-2.

C5.3.3 Condition Assessment

C5.3.3.1 General

Establishing the physical presence of metallic structural members in a building may be as simple as direct visual inspection and measurement, or as complex as using gamma radiography (through the architectural fabric) or boroscopic review through drilled access holes—methods that may be necessary if access is not permitted. The survey should include both base element and connector materials and details. For elements encased in concrete or fireproofing, this verification may be done by removing such encasements at critical locations.

It is well recognized that metallic components degrade if exposed to an aggressive environment. Corrosion is especially degrading in terms of lost material, reduction of properties, and propensity for creating locally embrittled areas. Assessment of in-place physical condition may be accomplished through visual inspection, nondestructive testing (NDT), and sampling and destructive testing techniques. Quantification of condition may consist of taking ultrasonic material thicknesses for comparison to original/nominal thickness, comparing existing material response to sound and vibration to that of new (calibrated) material, or using recently developed tomographic methods. Depending on the physical conditions of the element/connections, the number of tests necessary to gain confidence will vary substantially. Recommended guidelines for visual condition assessment are contained in ASCE Standard 11-90 for both base metals and connectors. Of particular interest during the survey are any existing conditions not reflected in the design documents (e.g., different end connectors), presence of any degradation, integrity of any surface coatings, and signs of any past movement.

Visual inspection of weldments should be made in accordance with American Welding Society D1.1, "Structural Welding Code—Steel." Structural bolts should be verified to be in proper configuration and tightened as required in AISC's *Steel Construction Manual*. Rivets should also be verified to be in proper configuration and in full contact, with "hammer sounding" conducted on several random rivets to ensure that they are functional.

Other nondestructive testing methods that may be used include liquid penetrant and magnetic particle testing (weld soundness), acoustic emission (system and element behavior), radiography (connector condition), and ultrasonics (numerous uses). Nondestructive testing should be used when visual inspection identifies ongoing degradation, or when a particular element or connection is critical to seismic resistance and requires further verification. Information on these methods and descriptions of their application are contained in a number of references.

It is recommended that all critical building elements be visually inspected, if possible, based on access and available time.

C5.3.4 Knowledge (κ) factor

No commentary is provided for this section.

C5.4 Steel Moment Frames

C5.4.1 General

Steel moment frames are categorized by the connection type. The connections vary widely between modern welded connections with high-strength bolts, and older riveted connections with gusset plates, angles, and T-sections connecting standard rolled shapes and complex built-up members. Modern connections with welded flanges and bolted webs deform and rotate very little, and are regarded as fully restrained (FR) connections.

Partially restrained (PR) connections develop significant rotation and deformation within the connection. Many riveted and bolted connections qualify as PR connections, but the connection strengths and stiffnesses vary widely. Figure C5-3 shows the relative deformability and stiffness of different connections.

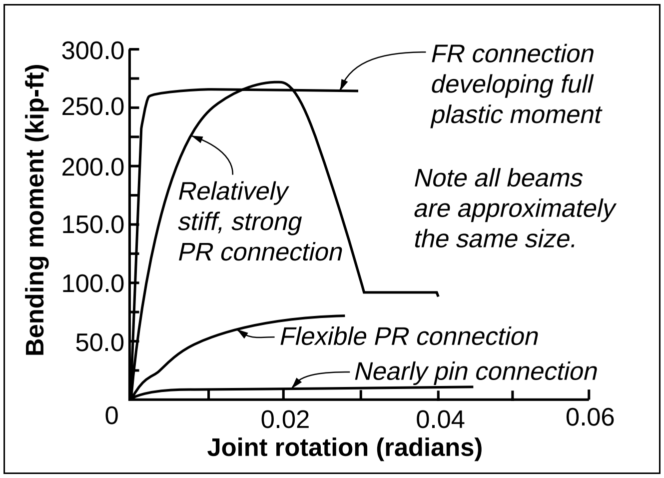


Figure C5-3 M-θ Relationships for FR and PR Connections

C5.4.2 Fully Restrained Moment Frames

C5.4.2.1 General

Fully restrained (FR) moment frames have nearly rigid connections. The connections must be at least as strong as the member, and the deformation of the connections can contribute no more than 5% of the story drift. Special Moment Frames are typically designed for small seismic forces, because they dissipate large quantities of energy through flexural yield of beams and columns or shear yield of the panel zone. As a result, local flange and web buckling and lateral torsional buckling of beams and columns of Special Moment Frames must be controlled in the hinging regions, even for end rotations as large as four to six times the rotation at yield. Ordinary Moment Frames must also meet limited ductility requirements, but the plastic end rotation requirements are smaller, and the slenderness limits for the web, flange, and lateral torsional buckling are less severe. The terms Ordinary and Special Moment Frames are not used in the *Guidelines*, but the limits used in the *Guidelines* are based on limits associated with these two moment frames in other documents, such as AISC (1994a).

FR moment frame members that are encased in concrete for fire protection are unlikely to experience the deformation associated with local buckling that is encountered with bare steel frames. This prevents the deterioration associated with local buckling, and allows the steel to develop its full ductility and yield capacity without the many local stability concerns outlined in AISC (1994a). As a result, these encased frames are assumed to satisfy the requirements of Special Moment Frames.

Special Moment Frames historically had a very good reputation for ductility and seismic performance, but because a significant number of these frames experienced cracking in the 1994 Northridge earthquake, special provisions are included in this document.

C5.4.2.2 Stiffness for Analysis

The stiffness and the resulting deflections and dynamic period of FR moment frames are determined by the usual structural analysis procedures. The contributions of elastic deformation of the connections to frame deflection are not addressed, because these contributed frame deflections are relatively small compared to deflections caused by member deformations. Elastic stiffness is dependent upon the geometric properties of the members; for modern steel frames with lightweight fire protection, these are the properties of the bare steel section. For older steel frames that are encased in concrete for fire protection, composite member properties should be used for elastic analysis if the concrete is in contact with the steel. This increased stiffness may be very significant, and can lead to larger seismic forces.

During inelastic analysis, changes in incremental stiffness occur due to yielding, and the inelastic stiffness is therefore interrelated with the strength. FR moment frames yield in the beams, columns, and panel zones during inelastic deformation. Stiffness must be reduced at these locations when yielding occurs. Computer models such as those developed for PR connections and described in Section C5.4.3.2 are sometimes used to approximate panel zone yield deformation. While the stiffness is reduced for yielded members and panel zones, the elastic stiffness is still used for all other members and connections.

The yield deflections and strength rules included in Section 5.4.2.2 are based on typical plastic design models such as those used in the AISC LRFD

Specification (AISC, 1994b). The yield deflections for beams and columns are based on conservative approximations. The true frame deflection at initiation of significant yielding may be slightly larger than predicted, and as a result, the true ductility demand should be somewhat smaller than predicted by these guidelines. This conservative procedure is based on the assumption of cantilevered members with inflection points at mid-height of the column and mid-span of the beam. The method further assumes that the rotation all occurs in the most flexible element. The members are assumed to remain elastic until the full plastic moment is developed. The plastic moment capacity for members under combined loading is adjusted for the axial load by linear interpolation.

C5.4.2.3 Strength and Deformation Acceptance Criteria

The significant deformation given in Table 5-4 is plastic end rotation. This was chosen to be consistent with the concrete chapter, and because some popular computer programs give plastic end rotation as standard output. The majority of test results give chord rotation, which is depicted in Figure 5-2, as the deformation response. There is little actual difference between the two for large deformations. The chord rotation may be estimated as the plastic end rotation plus the yield rotation.

The strength of individual members and components is defined by plastic analysis techniques, except that linear interpolation is sometimes used for transitions between one established condition and another.

Composite action due to concrete encasement is not considered in the resistance, because the bond stress or shear transfer mechanism is important to member behavior, and the condition of this interface is uncertain in existing structures. Further, the additional strength contributed by composite action of FR moment frames often is relatively small. While the strength provided by encasement is not factored in, the stiffness provided to the steel by the concrete is considered.

A. Linear Static and Dynamic Procedures

There is no strict story drift limit for steel frames. For the Immediate Occupancy Performance Level, a drift level less than 0.01 is desirable. This limit is selected because steel frames normally experience their first significant yielding at an inter-story drift ratio of between 0.005 and 0.010. Steel is a ductile material and no significant damage is expected at the 0.01 drift level. Practical drift limits for Life Safety and Collapse Prevention performance might be 0.02 and 0.04, respectively.

Significant inelastic deformation is permitted in ductile elements for the Life Safety and Collapse Prevention Performance Levels. Collapse Prevention *m* values by definition represent maximum permissible post-yield deformation for components based on the Collapse Prevention limit state. They are to be specified for each type of component, recognizing the types of forces (axial, shear, flexure) and considering the mode of failure. Table 5-3 indicates the components to be covered. When using the linear procedures, *m* factors reduce the seismic design forces because of inelastic behavior and component ductility. Good inelastic performance indicates good energy dissipation and the ability of the component to hold together through significant inelastic deformations. For Life Safety, *m* values are invariably smaller than *m* values for Collapse Prevention because the Life Safety limit state can tolerate less damage to the structure.

Historically, Special Moment Frames have been regarded as very ductile structural systems that can tolerate plastic deformations on the order of four times the yield deformation with little or no deterioration in strength or ductility. Larger inelastic deformations are possible if some deterioration is tolerated. Ordinary Moment Frames are somewhat less ductile. The Collapse Prevention *m* values given for beams and columns in moment frames in Table 5-3 are based upon member behavior. The more restrictive limits on frame properties with larger *m* values are based upon AISC (1994a) limits for Special Moment Frame behavior. The least restrictive limits on frame properties with smaller *m* values are based upon Ordinary Moment Frame behavior. Interpolation is allowed between these extreme limits; however, it must be emphasized that these are member ductility limits, and separate limits are applied to the connections of FR steel moment frames.

A number of FR steel moment frames experienced cracking in the joints and connections during the Northridge earthquake. As a result, the *m* values for FR moment frame connections are evaluated separately in Table 5-3. This evaluation was achieved by examining the results of more than 120 experiments on FR moment connections under inelastic cyclic loading, all performed in the United States in the past 30 years (Roeder and Foutch, 1996). This evaluation clearly showed that the flexural ductility achieved with FR moment frame connections is dramatically reduced with deeper beams. The empirically determined equation,

$$m = 7.5 - 0.125\ d_b \qquad \text{(C5-1)}$$

is based on a least squares fit to experimental results. This equation has been slightly reduced for safety for use with the *Guidelines*. The term, d_b, is the beam depth. This same experimental data showed that flexural ductility is significantly reduced in beams with panel zone yielding. This occurs because of the severe local deformation occurring near the welded connection with panel zone yield deformation. The ductility achieved with the panel zone itself may be very large, but there is significantly larger strain hardening with shear yield of the panel zone than with flexural yielding. As a result, the bending moments in the welded connection grow significantly larger during panel zone yielding, and the second set of connection limits is provided.

B. Nonlinear Static Procedure

The NSP uses a nonlinear pushover analysis to evaluate inelastic behavior. The deformations permitted in each element utilize a logic that is very close to that employed in the evaluation of *m* values. Table 5-4 defines the deformation limits for FR moment frames.

C. Nonlinear Dynamic Procedure

The deformation limits provided in Table 5-4 also apply to the deformations achieved in the NDP.

C5.4.2.4 Rehabilitation Measures for FR Moment Frames

A. Component Strength Enhancement Techniques

- **Columns**
  - Shear capacity—Add steel plates parallel to web (doubler or at flanges) or encase in concrete.
  - Moment capacity—Add steel plates to flanges or parallel to web, or encase in concrete.
  - Axial—Add steel plates or encase in concrete.
  - Combined—See above.
  - Stability—Provide steel plates, stiffeners, bracing members, or concrete encasement.
  - Strong column-weak beam—Strengthen column using techniques noted above.
  - Concrete encasement—Remove or modify in cases where concrete causes potential undesirable failure mode.
- **Beams**
  - Shear—Add steel plates parallel to web (doubler or at flanges) or encase in concrete. These are probably only needed over a certain length adjacent to connections.
  - Moment—Add steel plates to both flanges, bottom flange only (if composite action is reliable), or beam encasement, or augment composite slab participation. Effects on strong column-weak beam conditions should be considered. Again, these are probably only needed over a certain length adjacent to connections.
  - Stability—Provide lateral bracing for unsupported flange(s) (usually only the bottom flange, since the top flange is braced by the concrete diaphragm) with perpendicular elements or stiffeners. Both strength and stiffness need to be considered.
  - Concrete encasement—Remove or modify encasement or composite action where they create potential undesirable failure modes.
- **Connections**
  - Beam flange to column—The choice depends on the type of connection. For fully welded connections, modify in accordance with FEMA 267 (SAC, 1995). For flange plates, add plates, and/or welding.
  - Beam to column web—Add welding; replace rivets with high-strength bolts.
  - Concrete encasement—Remove or modify encasement or composite action where it creates potential undesirable failure modes.
  - Column base fixity—Add anchor bolts; add welding; add stiffening plates to column and base plate.

- **Joints**

  – Panel zone shear strength—Add doubler plates with various details.

  – Column flange stiffness—Add continuity plates or stiffen flanges with additional plates.

  – Column web crippling—Add continuity plates and/or doubler plates, or concrete encasement.

  – Column web tearing—Add continuity plates and/or doubler plates, or concrete encasement.

B. Rehabilitation Measures for Deformation Deficiencies

Almost all member-strengthening techniques will also enhance member stiffness. The amount of stiffening can vary substantially depending on the technique. Only minor stiffening will result from additional welding, replacement of rivets, or addition of continuity plates; moderate stiffening from addition of steel plates, or augmentation of composite action; and the most substantial stiffening from concrete encasement. Effects on frame strength and failure modes must be considered.

C. Connection Between New and Existing Components—Compatibility Requirements

- **Within Component**

  When choosing rehabilitation measures, the following compatibility requirements apply to connections between new and existing components.

  – Built-up steel sections—Consider the load transfer mechanism between pieces of built-up section (stitch or lacing plates) by welding, bolting, or riveting as it affects strength and stiffness, both elastic and cyclic.

  – Composite beam elements—Consider the interaction of steel beam and concrete slab, the load transfer mechanism (shear connectors or puddle welds), and the effects of both on element strength and stiffness, both elastic and cyclic.

  – Concrete encasement—Consider the interaction of concrete and steel, the load transfer mechanism (friction or shear connectors), and the effects of both on element strength and stiffness, both elastic and cyclic.

- **Within Frame**

  – Connection stiffness and strength—Connection size (especially older systems) may alter frame response, increasing stiffness by reducing clear member lengths. Weak connections limit the load to frame elements.

  – Joint stiffness and strength—Weak joints limit the load to frame elements, but may cause local stress concentrations (column flange kinking).

- **Between Frame and Other Vertical Lateral-Force-Resisting Elements**

  – Stiffness compatibility—Consider the frame/wall effect in tall structures (reverse shears in walls or braced frames at upper stories).

  – Collector/drag elements—The method of distribution of loads to elements should be considered.

- **Interaction with Diaphragm Stiffness**

  – Load distribution—Consider whether rigid versus flexible diaphragms.

  – Load transfer mechanism—Consider mechanisms such as collectors/drags, shear connectors, puddle welds, friction, and bearing, and their effects on strength and stiffness.

  – Diaphragm yielding mechanism—Consider limit load to frames, and the effect on local drifts.

D. Connections in FR Frames

Connections in FR frames must be at least as strong as the weaker member being connected. Rigid connections are commonly used in modern seismic design, and the procedures for dealing with them are documented in other references. Full-pen beam-to-column connections performed poorly during the 1994 Northridge earthquake. Enhancement techniques are given in FEMA 267 (SAC, 1995).

C5.4.3 Partially Restrained Moment Frames

C5.4.3.1 General

Partially restrained (PR) moment frames are those steel moment frames in which the strength and stiffness of the frame is dominated or strongly influenced by the strength and stiffness of the connection. Because of this, the connection strength, M_{CE}, and the rotational spring stiffness, K_θ, are important considerations. In FR moment frames, the analysis of the frame is performed with the assumption that the originally undeformed angle between connected members is retained during seismic deformation. This assumption is not valid with PR connections. Typical moment-rotation relationships for FR and PR connections are depicted in Figure C5-3. Finite element analyses that include the rotational springs as well as the stiffness of the beams and columns must be performed as depicted in Figure C5-4, where K_s is the spring stiffness.

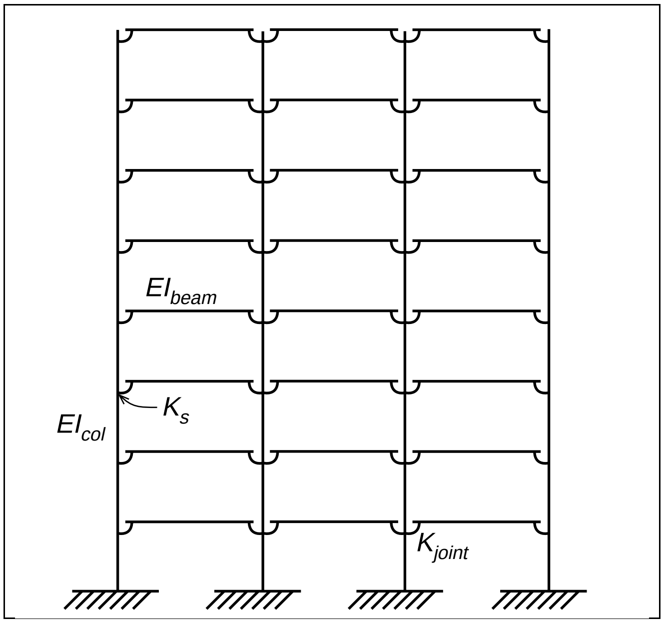


Figure C5-4 Model of PR Frame

While the strength and stiffness of PR connections are limited, many PR connections can sustain very large deformations without failure of the connection or structure. Experimental research has shown that the joint rotation of the connection is an important limiting factor for Life Safety and Collapse Prevention. Therefore, the joint rotation, θ, of each joint due to the application of the unreduced seismic loading must be determined as part of the nonlinear structural analysis. This maximum rotation is then compared to the rotation limits in Table 5-6 of the *Guidelines*. Typical hysteresis behavior of PR connections is shown in Figure C5-5.

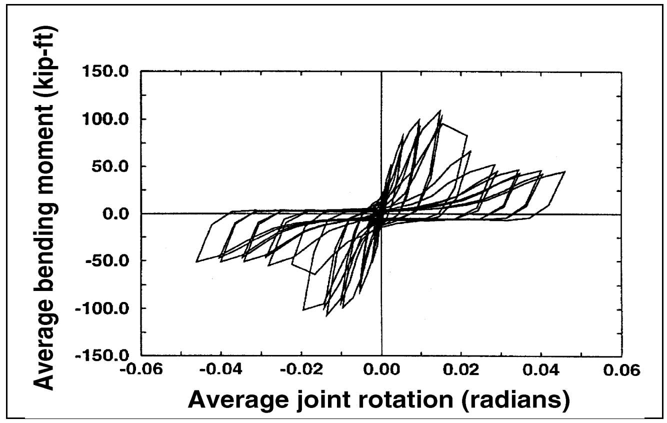


Figure C5-5 Hysteresis of PR Connection

C5.4.3.2 Stiffness for Analysis

The rotational spring stiffness, K_θ, is an important part of the structural analysis of frames with PR connections. However, experimental research has shown that the connection stiffness varies widely based on parameters such as connector size and type, thickness of steel elements, and depth of beam. Composite action due to concrete encasement also significantly increases the stiffness of some connections. The tangent modulus stiffness and the secant modulus stiffness also decrease with increasing joint rotation. Empirical models have been developed for a range of connection types, but these models are inexact and do not cover the full range of connections provided. The simplified models used in this document are based on the experimental observations that connections that are stronger are usually also stiffer. All PR connections experience significant yield and reduction of stiffness at joint rotations on the order of 0.005 radians. A realistic estimate of connection strength is essential to the seismic evaluation and rehabilitation of these structures, so the approximate connection stiffness in Equation C5-2 is employed. That is,

$$K_\theta = \frac{M_{CE}}{0.005} \qquad \text{(C5-2)}$$

Section 5.4.3 provides guidance in evaluating the connection strength, M_{CE}, used to approximate the stiffness. The rotational spring stiffness provided by Equation C5-2 is invariably an intermediate stiffness. It is smaller than the maximum stiffness at zero load, and much larger than the tangent stiffness at failure. This stiffness is needed to establish the initial dynamic period and the seismic forces of the structure.

Composite action due to encasement for fire protection dramatically increases both the strength and stiffness of some PR connections. The engineer has the option of including this additional resistance in the calculation of M_{CE}, but this calculation is more difficult and requires additional effort. In the absence of this added effort, the simplified resistance calculations provided in this document are believed to be conservative. Therefore, the engineer has the conservative option of neglecting this extra resistance in making the design calculations. It is essential, however, that the engineer not neglect the added stiffness, since this would result in a potentially nonconservative underestimate of the seismic forces. Therefore,

$$K_\theta = \frac{M_{CE}}{0.003} \qquad \text{(C5-3)}$$

is proposed for the special case where the connection is encased and develops composite action. The composite action is neglected in the connection strength calculation.

The rotational spring stiffness is important, but relative frame stiffness determines whether the frame has PR or FR connections. It is preferred that a computer model with frame elements and rotational spring elements, as illustrated in Figure C5-4, be used in determining the frame stiffness. However, many engineers and structural analysis computer programs are not able to easily accommodate the rotational spring. Therefore, a simplified analysis method is proposed as an alternative to a full PR frame analysis. This alternative method allows an analysis with rigid connections, but the beam stiffness, EI_b, is reduced to $EI_b\,adj$—adjusted to account for the rotational spring stiffness of the joint. This adjusted stiffness may be substituted in an ordinary rigid-connection frame analysis.

The fundamental assumptions of the adjusted model are based on the simple single-story moment frame subassemblage illustrated in Figure C5-6. This frame has rigid connections with a bending stiffness EI for the beams and columns; an average beam span length, l_b; and an average story height, h. The centerline member lengths are used, and panel zone rigidity is neglected.

The elastic story drift-deflection, u, can be estimated by the equation

$$u = \frac{Ph^3}{12EI_c} + \frac{Phl_b^2}{12EI_b} \qquad \text{(C5-4)}$$

where

h = Story height, in.
l_b = Beam length, in.
I_b = Moment of inertia of beam, in.^{4}
I_c = Moment of inertia of column, in.^{4}

It can be seen that the deflection is made up of two parts: bending of columns and bending of beams. If the loads and beam and column stiffness are unchanged, the moment and beam curvature are unchanged, and the story drift deflection for a frame with flexible connections becomes

$$u = \frac{Ph^3}{12EI_c} + \frac{Phl_b^2}{12EI_b} + \frac{Ph^2}{2K_S} \qquad \text{(C5-5)}$$

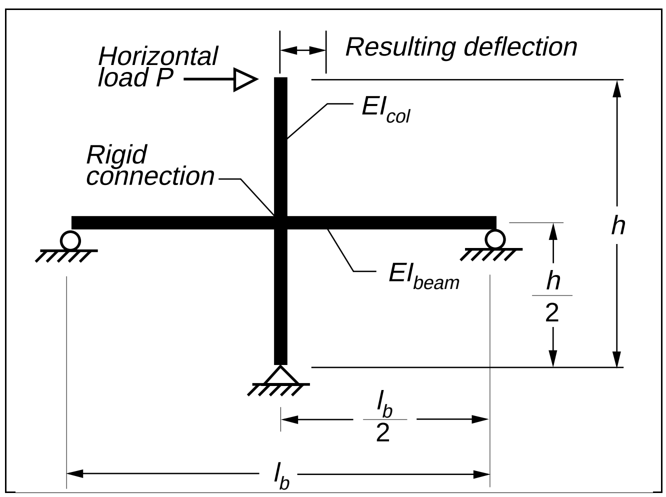


Figure C5-6 Frame Subassemblage

As indicated, a third term is added to this frame deflection based on the rotational spring stiffness of the connection. The simplified model allows the engineer to use Equation C5-4 to achieve the same deflection as achieved with Equation C5-5, that is,

$$u = \frac{Ph^3}{12EI_c} + \frac{Phl_b^2}{12EI_b adj} \tag{C5-6}$$

where

$$EI_b adj = \frac{1}{\frac{6h}{l_b^2 K_\theta} + \frac{1}{EI_b}} \tag{C5-7}$$

Only the bending stiffness of the beam is adjusted. This is an important distinction, because it is essential that the story drift and frame stiffness be estimated while the joint rotation is conservatively and at least approximately retained. The rotation of the column at the joint is the same for the deflections achieved with Equations C5-5 and C5-6. However, in Equation C5-5, the column rotation is achieved by the sum of a joint rotation, θ, and a beam end rotation. That is, the true joint rotation is somewhat smaller than the column rotation. Therefore, the rotation of the column at the joint is used conservatively as the joint rotation, θ, with this simplified analysis procedure.

While the spring stiffness of the connections must be considered in elastic analysis of PR frames, the elastic properties of the members are the same as those used in FR steel frames. Composite properties of the member should be used for encased members with the concrete encasement in contact with the steel. The stiffness of masonry infill walls, and other structural and nonstructural elements, should also be included as in the FR frame analysis.

Figure C5-5 shows a typical moment rotation hysteresis curve for a PR connection. The slope of this curve is the spring stiffness. For inelastic analysis, the computer models must recognize that the rotational spring stiffness of the connection changes dramatically with the deformation. These models are necessarily quite complicated, and relatively few computer models are available at this time. In nonlinear procedures, the variable rotational spring stiffness should be included in the computer model as illustrated in Figure C5-4. With this procedure, the rotational connections between the beams and columns is replaced by rotational springs with variable (nonlinear) spring stiffness. Direct transfer of shear and axial forces is permitted by the connection. A step-by-step nonlinear procedure can be performed by incremental changes in the rotational spring stiffness. The discussion provided in Section C5.4.3.3 on individual connection types provides insight into the variation of stiffness for different PR connections.

C5.4.3.3 Strength and Deformation Acceptance Criteria

The strength and deformation of PR frames are dominated by the connections. Member properties are identical to those used for members in FR frames, and are defined by plastic analysis techniques similar to those used by AISC (1994a). While composite action due to concrete encasement is seldom used in estimating the resistance of members in FR or PR frames, the engineer is encouraged to utilize both the stiffness and resistance provided by composite action for PR connections. This increased stiffness and resistance is particularly great for any of the weaker, more flexible connections.

The *m* factors used for the linear procedures and the deformation limits employed for nonlinear procedures are very sensitive to connection failure mode and the connection type. As a result, more detailed discussion of individual PR connection types is provided in this *Commentary*. The *m* factors and deformation limits are summarized in Tables 5-5 and 5-6. It should be emphasized that the limits for PR connections in these tables often require adjustment for deeper beams.

Flange Plate Connections. Flange plate connections that are welded to the column and bolted to the beam, as shown in Figure C5-7, are relatively stiff and strong PR connections. In fact, the flange plates could be designed for strength and stiffness such that the behavior could be classified as fully restrained (SAC, 1995). These connections exhibit fairly good hysteretic behavior with moderate pinching. Flange plate connections may also be welded to both the beam and the column as shown in Figure C5-8. Both types may be close to the stiffness limit required to qualify as an FR connection. They are relatively modern connections that are seldom encased in concrete for fire protection. Therefore, composite action due to encasement is not a major concern.

It is important that the failure modes considered in the analysis include plastic bending capacity of the beam, plastic capacity of the net section (including consideration of the critical row of bolts or the narrow point of a welded plate), resistance of the connectors (welds and bolts) themselves, local buckling of the flange plate, and weld strength between the flange plate and the column flange.

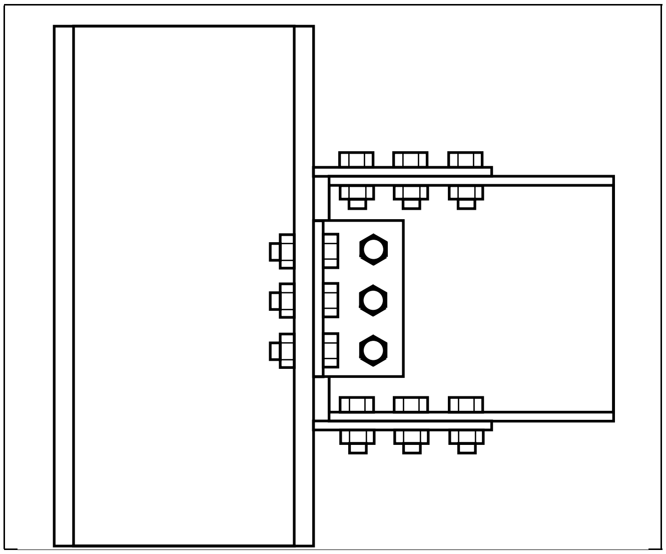


Figure C5-7 Bolted Flange Plate Connection

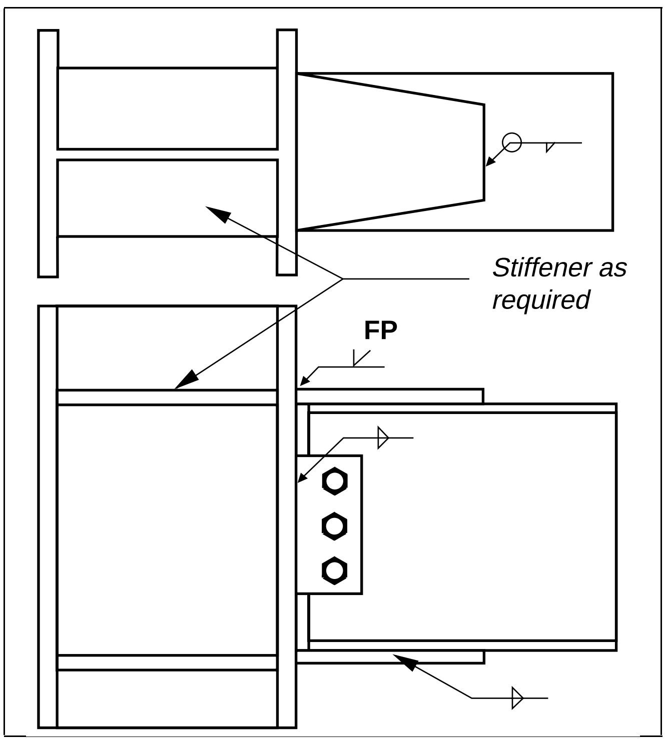


Figure C5-8 Welded Flange Plate Connection

The ductility appears to be greatest when the net section of the flange plate controls the resistance of the connection, and the ductility is lowest when weld resistance controls the strength of the connection. The moment capacity of the connection should be taken as the smallest moment produced by these different failure modes. The relative ductility of these different failure modes is considered in the definitions of *m* values and connection rotation limits in Tables 5-5 and 5-6. For more details on individual test results, see references by Popov and Pinkney (1969) and Harriott and Astaneh-Asl (1990).

End Plate Connections. End plate connections such as shown in Figure C5-9 are also stiff and strong PR connections, sometimes qualifying as an FR connection for stiffness analysis. Their use became more common around 1960, since they typically require high-strength bolts. This type of connection is most ductile if flexural yielding of the beam or the end plate occurs. It fails abruptly at small deformations if tensile failure of either the high-strength bolts or the weld occurs. These differences in relative ductility are reflected in the *m* values and deformation limits provided in Tables 5-5 and 5-6. It is important that the failure modes considered in the analysis include the plastic capacity of the beam, the local bending plastic capacity of the plate, the local bending plastic capacity of the column flange, the capacity of the fillet or penetration welds between the end of the beam and the end plate, and the tensile capacity of the bolts, including prying action.

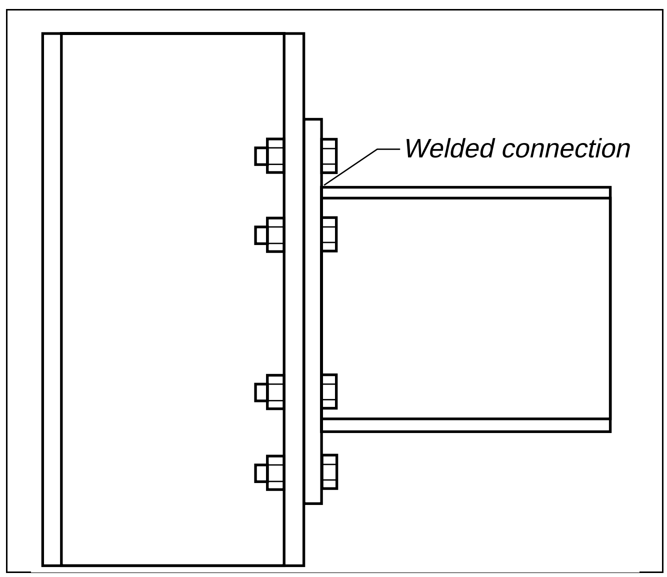


Figure C5-9 End Plate Connection

The moment capacity of the connection should be taken as the smallest moment produced by these different failure modes. However, it should be recognized that there is considerable uncertainty in the various calculations, and so the *m* values for thin plate failure modes (i.e., local bending of end plate) should be used only if the capacity achieved with all other failure modes exceeds the plastic bending of the end plate by 25%. The *m* value for thick plate or stiffened plate failure modes should be used only if the capacity achieved with all other failure modes exceeds the plastic bending capacity of the beam by 25%. Otherwise, the lower value should be employed. If these overstrength requirements are met, the AISC strength calculations appear to be appropriate for seismic evaluation.

Empirical models for connection nonlinear monotonic moment rotation behavior have been developed. The formula by Frye and Morris (1975) for end plates without column stiffeners is

$$\theta = 1.83 \times 10^{-3} \times (KM) + 1.04 \times 10^{-4} \times (KM)^3 + 6.38 \times 10^{-6} \times (KM)^5 \qquad \text{(C5-8)}$$

where

$$K = d^{-2.4} \times t^{-0.4} \times f^{1.1} \qquad \text{(C5-9)}$$

M = Applied moment

d = Distance between center of top and bottom bolt line

t = End plate thickness

f = Bolt diameter

θ = Rotation of end of beam relative to column

The formula by Frye and Morris (1975) for end plates with column stiffeners is

$$\theta = 1.79 \times 10^{-3} \times (KM) + 1.76 \times 10^{-4} \times (KM)^3 + 2.04 \times 10^{-4} \times (KM)^5 \qquad \text{(C5-10)}$$

where

$$K = d^{-2.4} \times t^{-0.6}$$

More details on individual test results and failure modes for end plate connections are given in Tsai and Popov (1990), Johnstone and Walpole (1981), Whittaker and Walpole (1982), Murray and Kukreti (1988), and Sherbourne (1961).

T-Stub Connections. T-stub connections have been used for at least 70 years; Figure C5-10 illustrates a typical connection. Riveted details such as those illustrated in the figure were used for the first half of this period; high-strength bolts have been used in more recent practice. During the early part of this period, these connections were encased in massive, lightly reinforced concrete for fire protection. T-stub connections are of intermediate strength and stiffness, but approach FR behavior if carefully designed. The connection will seldom develop the full plastic capacity of the beam, but it will develop a significant portion of this beam-bending capacity. As a result, composite action due to the concrete encasement will often significantly increase the strength and rotational spring stiffness of the connection.

A number of failure modes are possible with these connections. The *m* values and deformation limits are very sensitive to the failure mode. Greater ductility and larger inelastic deformations can be achieved in connections with flexural yielding in the flanges of the T-sections. The smallest ductility and inelastic deformation can be achieved on connections where the inelastic deformation is controlled by the tensile connectors between the T-section and the column flange. The limits established in Tables 5-5 and 5-6 reflect these differences in behavior. The guidelines provided in Section 5.4.3.3 provide approximate estimates of the resistance and failure mode of T-stub connections. Accurate calculation of the connection failure modes and resistance is difficult because of the interaction between flexure in the flanges and tension in the connectors through prying action in the connection. As a result, the equations in the *Guidelines* are very approximate and quite conservative in their estimates of the resistance.

More detailed procedures have been developed for estimation of the connection resistance and failure mode. These procedures are considerably more accurate, but they require more effort and calculation. They also permit consideration of composite action due to concrete encasement. One such procedure for riveted T-stub connections is outlined below in this *Commentary.*

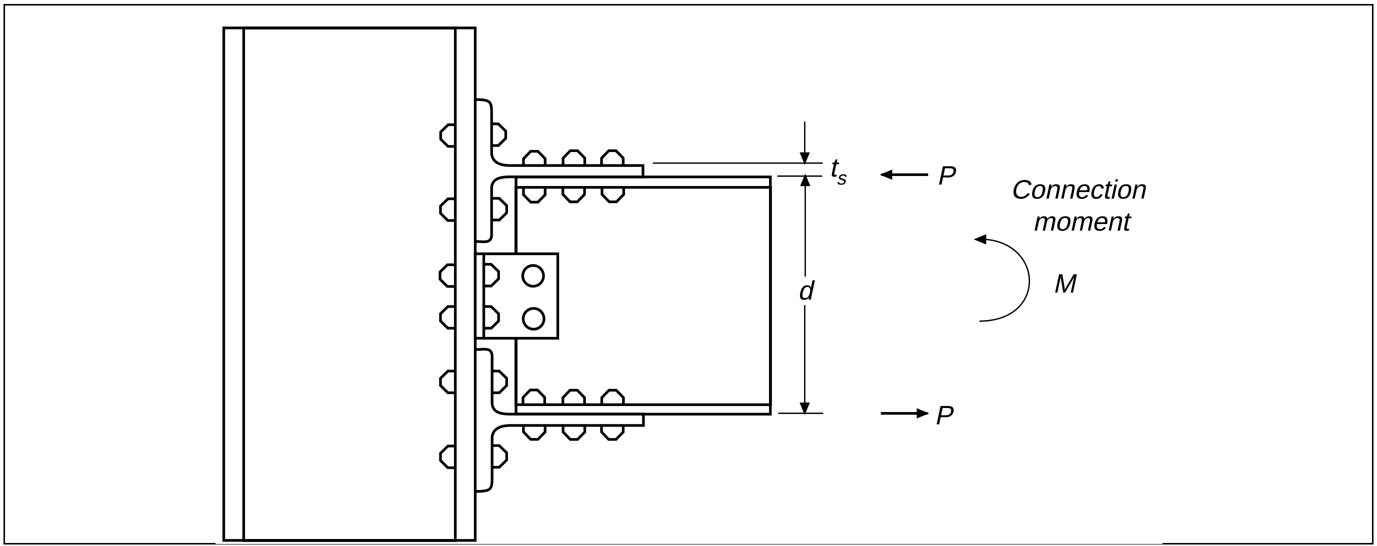


Figure C5-10 T-Stub Connection

For riveted bare steel connections, Figure C5-10 illustrates the general configuration of the connection. The connection moment can be approximated with the flange forces, P, as shown in the figure. The maximum flange force can be determined by examining a number of different failure modes and determining which mode leads to the smallest flange force. The flange force can then be directly translated into a moment capacity, M_{CE}, of a bare steel connection, or it can be combined with other calculations to predict M_{CE} for an encased connection.

T-Stub Connections: Plastic Moment Capacity of the Beam. The ultimate capacity of the connection is limited by the expected plastic capacity of the beam, so that $M_{CE} < Z F_{ye}$, where Z is the plastic section modulus of the steel and F_{ye} is the expected yield stress of the beam.

T-Stub Connections: Shearing of Rivets Between the Beam Flange and the T-Section. The expected force, P_{CE}, must be transferred from the beam flange to the stem of the T-section. The shear strength of the connectors provides another limit on the moment capacity, so that

$$P_{CE} \le A_c F_{ve} N_{Stem} \tag{C5-11}$$

and

$$M_{CE} = P_{CE} d_b \tag{C5-12}$$

where

d_b	=	Beam depth
A_c	=	Gross cross-sectional area of a single connector
F_{ve}	=	Expected shear strength of the connector
N_{Stem}	=	Number of connector shear planes

T-Stub Connections: Tension in the Stem of the T-Section. The ultimate tensile capacity of the stem (or web) of the T-section may also control the resistance of the connection, and it should be checked by the normal AISC tension member criteria; that is,

$$P_{CE} \le F_{ye} A_g \tag{C5-13}$$

$$P_{CE} \le F_{te} A_e \tag{C5-14}$$

and

$$M_{CE} \le P_{CE}(d + t_s) \tag{C5-15}$$

where

F_{ye} = Expected yield of steel in T-section stem

F_{te} = Expected tensile strength of steel in T-section stem

A_e = Net effective area of stem

A_g = Gross area of stem

t_s = Thickness of stem

T-Stub Connections: Local Plastic Bending of Flange of T-Section. Flexure of the flange of the T-section must also be considered. Prying forces are necessary to develop these flexural moments, and the prying forces increase the tensile forces in the connectors. Prying action plays a different role in older steel connections than it does in connections with modern high-strength bolts. Mild steel rivets yield and elongate more in tension than do high-strength bolts. This tensile yielding limits the prying action, so that a balance between flexure and tensile yield may occur. Flexure of the flange has the equilibrium conditions described in Figure C5-11. The local flange moments are limited by the plastic bending capacity of the flange, and this limits the force, P_{CE}. Thus, the ultimate capacity of the T-stub connection is approximated by

$$P_{CE} \le \frac{wt_s^2 F_{ye}}{d'} \qquad \text{(C5-16)}$$

and

$$M_{CE} < P_{CE}(d + t_s) \qquad \text{(C5-17)}$$

where d' is as shown in Figure C5-11 and t_s is the thickness of the stem.

Equations C5-16 and C5-17 limit the capacity of the connection based on flexure in the connecting elements. However, this flexure requires a prying force, as can be seen in Figure C5-11. The prying force introduces an additional tension in the tensile connectors, and a coupled mode of failure may occur. As a result, the capacity may be reduced to

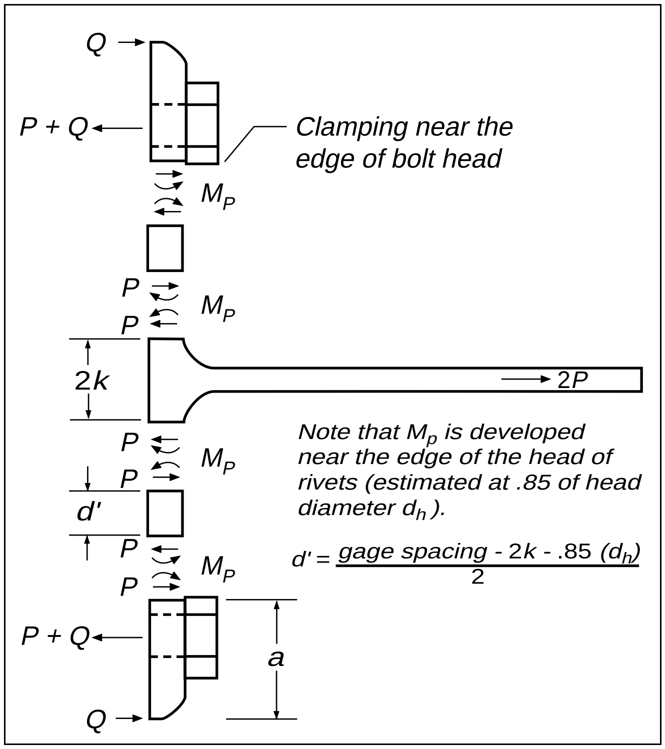


Figure C5-11 Prying Action in T-Stub Connection

$$P_{CE} \le \frac{\dfrac{0.5wt_s^2 F_{ye}}{a} + (F_{ye} A_c N_{VL})}{1 + \dfrac{d'}{a}} \qquad \text{(C5-18)}$$

$$M_{CE} \le (d + t_s/2)P_{CE} \qquad \text{(C5-19)}$$

N_{VL} is the number of tensile connectors between the flange of the T-section and the column flange.

T-Stub Connections: Tension of Rivets Between T-Section and Column. The tensile capacity of the connectors between the vertical leg of the angle or T-section and the column face may also control the resistance of the connection.

The equations

$$P_{CE} = F_{ye} A_c N_{VL} \qquad \text{(C5-20)}$$

and

$$M_{CE} \le (d + t_s/2)P_{CE} \qquad \text{(C5-21)}$$

can be used for the T-stub connection.

N_{VL} = Number of connectors acting in tension

A_c = Net area of each connector

t_s = Thickness of the T-stub stem

d = Vertical distance to the center of the connectors

F_{ye} = Expected yield stress of the connectors

These equations limit the moment capacity of the connection based on the tensile capacity of the connector. If the above equations produce the smallest moment capacity of the connection, the connection capacity may be further reduced by

$$P_{CE} \leq \frac{0.5 w t_f^2 F_{ye}}{d'} \tag{C5-22}$$

and

$$M_{CE} \leq (d + t_s/2) P_{CE} \tag{C5-23}$$

where

w = Length of T-stub, in.

t_f = Thickness of T-stub flange, in.

for a T-stub connection if Equation C5-22 or C5-23 produces a smaller moment capacity than Equation C5-20 or C5-21, respectively.

Web connectors and composite action due to encasement for fire protection may contribute to the resistance of these connections. The later commentary on clip angle connection design methods will describe methods for incorporating these added factors. However, it should be noted that the additional capacity provided by the web connection and composite action due to concrete encasement is likely to be relatively small for T-stub connections, because the flange connection is relatively strong.

The resistance predicted by the previous procedure will usually be larger than that predicted by Equations 5-23 and 5-24, and the stiffness can be estimated by combining this resistance with Equations C5-2 and C5-3. The stiffness of bare steel connections can also be estimated by application of a secant modulus to empirical equations such as

$$\theta = 2.1 \times 10^{-4} \times (KM) + 6.2 \times 10^{-6} \times (KM)^3 - 7.6 \times 10^{-9} \times (KM)^5 \tag{C5-24}$$

where

$$K = d^{-1.5} \times t^{-0.5} \times f^{1.1} \times L^{-0.7} \tag{C5-25}$$

M = Connection moment, kip-in.

d = Depth of beam, in.

t = Thickness of clip angle plus column flange, in.

f = Bolt diameter, in.

L = Length of T-stub section, in.

θ = Rotation of end of beam relative to column, rad

More information on individual test results and failure modes for T-stub connections is given by Roeder, Leon, and Preece (1994), Hechtman and Johnston (1947), Rathbun (1936), and Batho and Lash (1936).

Clip Angle Connections. Clip angle connections, as illustrated in Figure C5-12, have a similar history to that of T-stub connections. Rivets were used until about 1960, and high-strength bolts have been used more recently. For many years, the connections were encased in massive, lightly reinforced concrete for fire protection. Clip angle connections are among the weaker and more flexible PR connections. The connection will usually develop only a small portion of the plastic capacity of the beam. As a result, composite action due to the concrete encasement will at most invariably provide a significant increase to the strength and rotational spring stiffness of the connection. A number of failure modes are possible with clip angle connections. The *m* values and deformation limits provided in Tables 5-5 and 5-6 are based on the failure mode. Greater ductility and larger inelastic deformations can be achieved in connections with flexural yielding in the outstanding leg (OSL) of the clip angle. The smallest ductility and inelastic deformation occurs when the resistance is controlled by the tensile connectors between the OSL and the column flange. The limits established in Tables 5-5 and 5-6 reflect these priorities. The prediction of the failure mode of these connections is very important. Equations 5-17 through 5-22 of Section 5.4.3.3 of the

Guidelines provide approximate equations for estimating the resistance and failure mode. Accurate calculation of the connection failure modes and resistance is difficult because of the interaction between flexure in the flanges and tension in the connectors through prying action in the connection. As a result, the equations in the *Guidelines* are very approximate and conservative.

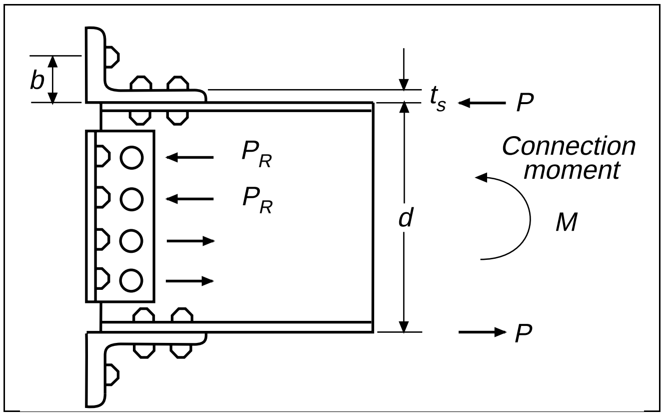


Figure C5-12 Clip Angle Connection

More detailed procedures have been developed for estimation of the connection resistance and failure mode. These procedures are more accurate, but they require more effort and calculation. They also permit consideration of composite action due to concrete encasement. One such procedure for riveted clip angle connections is outlined in this *Commentary*, as follows.

For riveted bare steel clip angle connections, Figure C5-12 illustrates the general configuration of the connection. The connection moment can be approximated with the flange force, P. The expected flange force, P_{CE}, can be determined by finding the smallest force provided by different failure modes. The flange force can then be directly translated into a moment capacity, M_{CE}, of a bare steel connection, or it can be combined with other calculations to predict M_{CE} for an encased connection.

Clip Angle Connections: Shearing of Rivets Between the Beam Flange and the Clip Angle. The force, P, must be transferred from the beam flange to the OSL of the clip angle. The shear strength of the connectors provide one limit on the moment capacity, so that

$$P_{CE} \le A_b F_{Ve} N_{OSL} \qquad \text{(C5-26)}$$

and

$$M_{CE} = P_{CE} d_b \qquad \text{(C5-27)}$$

where

d_b = Beam depth
A_b = Cross-sectional area of single connector
F_{Ve} = Expected shear strength of connector
N_{OSL} = Number of connector shear planes in OSL of angle

Clip Angle Connections: Tension of Outstanding Leg (OSL) of Clip Angle. The ultimate tensile capacity of the OSL may also control the resistance of the connection, and it should be checked by the normal AISC tension member criteria; that is,

$$P_{CE} \le F_{ye} A_g \qquad \text{(C5-28)}$$

$$P_{CE} \le F_{te} A_e \qquad \text{(C5-29)}$$

and

$$M_{CE} \le P_{CE}(d_b + t_s) \qquad \text{(C5-30)}$$

Clip Angle Connections: Local Plastic Bending of Flange of Clip Angle. Flexure of the vertical leg of the angle must also be considered. Prying forces are necessary to develop these flexural moments, and the prying forces increase the tensile forces in the connectors. However, prying action plays a different role in older riveted connections than it does in connections with modern high-strength bolts. Mild steel rivets yield and elongate more than high-strength bolts and this limits the prying action. In a clip angle connection, the flexure of the vertical flange has the equilibrium conditions described in Figure C5-13. The moments *M2* and *M4* limit the force, P, that can be transferred by the vertical leg. They are also limited by the plastic bending capacity of the leg. Thus,

$$P_{CE} \le \frac{0.5 w t_s^2 F_{ye}}{d' - \frac{t_s}{2}} \qquad \text{(C5-31)}$$

and

$$M_{CE} < P_{CE}(d + d') - \frac{0.25 w t_s^2 F_{ye}}{d' - \frac{t_s}{2}} \qquad \text{(C5-32)}$$

where d' is as defined in the figure and w is the length of the angle.

Clip Angle Connections: Prying Forces and Tension of Rivets Between Clip Angle and Column. Flexure requires a prying force, as can be seen in Figure C5-14. The prying force introduces an additional tension in the tensile connectors, and a coupled mode of failure may occur. As a result, the capacity of the connection produced by Equation C5-32 may be reduced by

$$P_{CE} \leq \frac{\frac{0.25 w t_s^2 F_{ye}}{a} + (F_{ye} A_c N_{VL})}{1 + \frac{d' - \frac{t_s}{3}}{a}} \qquad \text{(C5-33)}$$

$$M_{CE} \leq P_{CE}(d + d') - (F_{ye} A_c N_{VL} - P_{CE}) a \qquad \text{(C5-34)}$$

N_{VL} is the number of tensile connectors between the angle and the column flange. The prying force may be relieved, however, by tensile yielding of the connector. Under these conditions, the tensile capacity of the connectors between the vertical leg of the angle and the column face may directly control the resistance of the connection; that is,

$$P_{CE} = F_{ye} A_c N_{VL} \qquad \text{(C5-35)}$$

and

$$M_{CE} \leq (d + b) P_{CE} \qquad \text{(C5-36)}$$

where b is the vertical distance to the center of the connectors as shown in Figure C5-14, and F_{ye} is the expected yield strength of the connectors.

Web connectors and composite action due to encasement for fire protection may contribute to the resistance of these PR connections. These contributions may be particularly significant for the clip angle connections, because the clip angle flange connection is weaker than most other PR connections. The procedures for calculating these additional contributions are similar for all types of PR connections, and a brief description of procedures for completing this calculation follows.

Contribution of Web Connection to Moment Capacity. The smallest moment capacity, M_{CE}, and its associated flange force, P_{CE}, obtained in previous calculations, determine the mode of failure and moment capacity of the bare steel flange connection. The web connection also contributes to the moment capacity as illustrated in Figure C5-12. The web connectors develop forces that combine to form couples as illustrated in the figure. The calculations required to determine the forces developed in the web are similar to those used in determining the moment capacity provided by the flange connection. The addition of the web connector moment generally improves the estimate of the ultimate capacity of the connection, since past research has indicated that consideration of only the moment capacity contributed by the flanges will underestimate the true resistance. The underestimate is particularly significant for weaker and more flexible PR connections such as clip angle connections. However, a larger rotation is required to develop this additional moment in the web connection than is required to develop the moment capacity of the flanges. Thus, some connections with limited rotational ability—such as those with tensile failure of the column flange connectors—will not be able to develop this additional moment capacity. The additional moment capacity due to the web connection can be added to the contribution of the flange connection.

Contribution of Composite Action to the Moment Capacity. For encased connections, composite action develops additional moment resistance that can be considered. The critical mode of failure for the flange connections of the bare steel is again determined by the procedures described earlier for determination of the moment capacity due to the flange bare steel connection. For this mode of failure, the critical tensile flange force, P_{CE}, and the centroid of the location of this tensile force remain unchanged after the connection is encased. This tensile force is then balanced by the compressive force of the concrete using the normal ACI Ultimate Strength Design rectangular stress block, as illustrated in Figure C5-14. The location of the neutral axis and the ultimate capacity are readily determined by equilibrium calculations. These calculations again

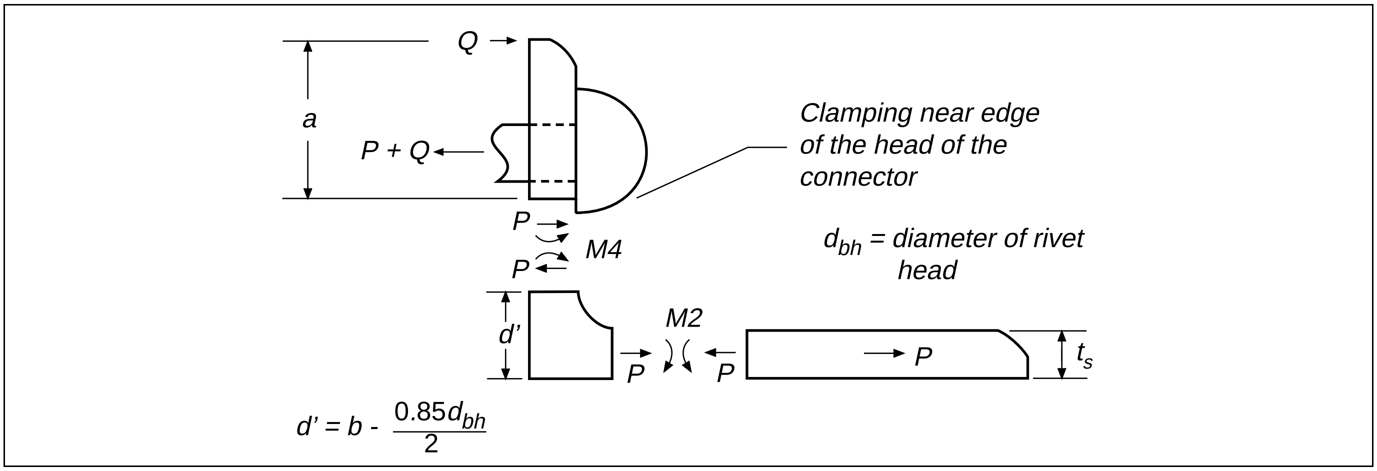


Figure C5-13 **Forces in Clip Angle**

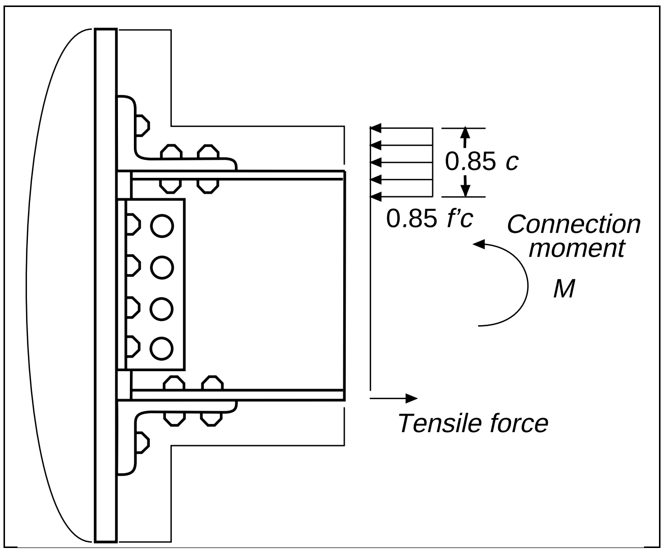


Figure C5-14 **Moment Resistance by Clip Angle Connection**

neglect the capacity of the web connectors, and past research has shown this to be a lower bound of the connection resistance.

The web connectors should also be considered, as illustrated in Figure C5-15. The web connectors are primarily in tension when the connection is encased, as illustrated in the figure. Flange connectors for the compression flange may be included if they are located well above the neutral axis. The tensile capacity of the web connectors is included only if they are located well below the neutral axis. A larger rotation is required to activate the web connectors in composite action than is required to activate the moment resistance of the flange connectors. Connections that developed a large rotation, such as those with flexural yielding of the clip angle, easily develop the moment resistance predicted by the model with composite actions. Some connections with smaller rotational capacity, such as those with tensile yield of connectors, do not develop the full composite moment resistance, including the web connection. The calculated moment capacity with web connectors and composite action may be larger than the experimental values in a few cases. However, this prediction is consistently closer to the true moment capacity of the connection. The moment capacity calculated by this procedure is all-inclusive, and it should not be added to the bare steel contributions.

The resistance predicted by the previous procedure will usually be larger than that predicted by Equations 5-17 and 5-22, and the stiffness can be estimated by combining this resistance with Equations 5-14 and 5-15. The stiffness of bare steel connections can also be estimated by application of a secant modulus to empirical equations such as

$$\theta = 0.2232 \times 10^{-4} \times (KM) + 0.1851 \times 10^{-7} \times (KM)^3 - 0.3289 \times 10^{-11} \times (KM)^5 \quad \text{(C5-37)}$$

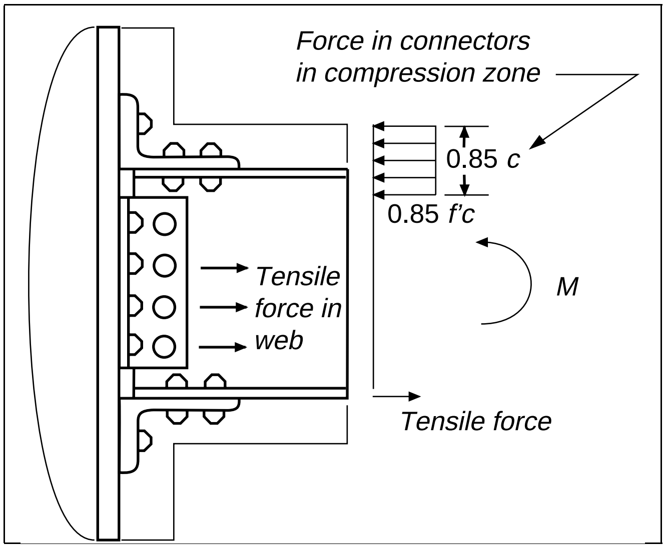


Figure C5-15 Effects of Web Rivets and Slab

where

$$K = d_b^{-1.2870} \times t^{-1.1281} \times t_a^{-0.6941} \times L^{-0.6941} \times \left(g - \frac{f}{2}\right)1.3499 \quad \text{(C5-38)}$$

g = Gage in flange angle

t = Thickness of clip angle

t_a = Thickness of web angles

f = Bolt diameter

L = Length of clip angles

M = Connection moment

θ = Rotation of end of beam relative to column

More information on individual test results and failure modes for T-stub connections may be found in Roeder et al. (1994), Azizinamini and Readziminski (1989), Hechtman and Johnston (1947), Rathbun (1936), Batho and Lash (1936), and Batho (1938).

C5.4.3.4 Rehabilitation Measures for PR Moment Frames

As stated in the *Guidelines*, many of the rehabilitation measures given for FR frames also apply to PR frames (see Section 5.4.2.4).

Older PR moment frames may be too flexible even if the beams and columns are encased in concrete. If this is the case, additional stiffness may be achieved by several means. Steel braces may be added in either a concentric or eccentric manner. Reinforced concrete or masonry infills may be added to some of the bays of the frames. Methods for designing and/or evaluating the effects of infills are given in the *Guidelines* Chapters 6 and 7. New steel frames may be attached to the outside of the building, but connections and load paths must be checked carefully.

C5.5 Steel Braced Frames

C5.5.1 General

Braced frames do not appear to be too common in seismic areas before the 1950s and 1960s, even though their use has been dated back to the 1920s in nonseismic areas. In the earlier applications, the bracings appear to have played a secondary role in lateral-load-carrying function, with the primary frames being moment frames and masonry infilled frames. They have generally taken the form of light vertical trusses, which were often knee-braced types. These older vertical trusses were connected by rivets and generally encased in fireproofing concrete, and they generally did not develop the capacity of the members. Use of bracings has, however, been common in one- and two-story structures, especially industrial types. Tension-only diagonals have often been used in one- and two-story applications.

More complete braced-frame systems started evolving after the 1950s, especially in low- to nonseismic areas. Braced frames were still generally combined with moment frames as dual frames in seismic areas.

Diagonal members and their connections form the basic components. The brace member may consist of single or double angles, channels or T-sections, circular or rectangular tubes with or without concrete filler, or tension rods or angles. The connection of the brace to the frame is generally by gusset plates with rivets, bolts, or welding.

C5.5.2 Concentric Braced Frames (CBFs)

C5.5.2.1 General

Concentric braced frames (CBFs) are very efficient structural systems in steel for resisting lateral forces due to wind or earthquakes because they provide complete truss action. That is the main reason for their popularity.

However, this framing system has not been considered as ductile in past or current design practice for earthquake resistance. The nonductile behavior of these structures mainly results from early cracking and fracture of bracing members or connections during large cyclic deformations in the post-buckling range. The reason lies in the code philosophy. Instead of requiring the bracing members and their connections to withstand cyclic post-buckling deformations without premature failures (i.e., for adequate ductility), the codes generally specify increased lateral design forces. It has recently been recognized that CBFs designed according to the past or current code procedures may not survive a major earthquake without serious consequences.

During a severe earthquake, bracing members in CBFs experience large deformations in cyclic tension in the post-buckling range, which cause reversed cyclic rotations to occur at plastic hinges in much the same way as they do in beams and columns of moment frames. In fact, braces in a typical CBF should be expected to yield and buckle at rather moderate story drifts of about 0.3% to 0.5%. In a severe earthquake the braces could undergo post-buckling axial deformations up to 10 to 20 times their tension yield deformation. In order to survive such large cyclic deformations without premature failure, the bracing members and their connections must be properly detailed. This often has not been the case in past design practice.

Early brace failures were observed in testing of the United States-Japan full-size, six-story structure with hollow tubular bracing in an inverted V pattern (Foutch, Goel, and Roeder, 1987). Two recently completed analytical studies (Tang and Goel, 1987; Hassan and Goel, 1991) investigated the seismic behavior due to severe ground motions of a number of concentric-braced structures designed according to different design philosophies. Included in the studies were CBFs with and without backup moment frames. It was found that structures designed strictly in accordance with the 1988 UBC procedure showed early brace fractures leading to large story drifts of up to 6% to 7% or more, which results in excessive ductility demands on beams and columns.

In the post-buckling range of a bracing member, local buckling of compression elements limits the plastic moment capacity and, consequently, the compression load capacity of the member. More importantly, however, the extent and severity of local buckling has a major influence on fracture life (ductility) because of high concentration of reversed cyclic strains at those locations. Therefore, in order to prevent early fracture of bracing members, their width-thickness ratios (compactness) must be kept within much smaller limits than those used in current practice. For rectangular tubular sections, a limit of $95/\sqrt{F_y}$ has been suggested (Tang and Goel, 1987), which is half of that specified in AISC (1994a). This is reasonable because plastic design is based on ductility under monotonic loading, whereas seismic design counts on the ability of structural elements to withstand large cyclic inelastic deformations in the event of a severe earthquake.

If the ductility of bracing members is ensured by using compact sections, as suggested above, and other frame members are properly designed by considering the strength of the braces, there is no need to use increased seismic design forces for a CBF. Thus, a number of structures were designed by using compact rectangular tubular bracing members $(b/t < 95/\sqrt{F_y})$ and $R_w = 12$ (same as specified by the 1988 UBC for SMRF). Also, the "penalty factor" of 1.5 (1988 UBC) was deleted in calculating the forces in chevron braces. Dual systems, as well as those without backup Special Moment Frames, were designed by this approach, and their responses to several severe ground motion records (peak accelerations of about 0.5g) were studied. No brace fractures occurred in these frames and their responses were much better than those of the code-designed structures. The story drifts were generally under 3%. The hysteretic loops of shear force in the first story of a ductile braced structure with backup SMRF are shown in Figure C5-16.

As mentioned earlier, local buckling has been found to be the most dominant factor influencing the ductility and energy dissipation capacity of bracing members. For rectangular tube sections, which are very popular for braces, an alternative to using smaller width-thickness ratios is to use plain concrete infill. Concrete infilling has been found to reduce the effective width-thickness ratio by as much as 50%, thus increasing the fracture life by up to 300% (Lee and Goel, 1987). The width-thickness ratio of angle sections should be kept under $52/\sqrt{F_y}$. Double angles used in toe-to-toe shape perform much better than the conventional back-to-back configuration (Aslani and Goel, 1989). For built-up sections, such as double angles or double channels, a stitch spacing such that L/r of the individual elements

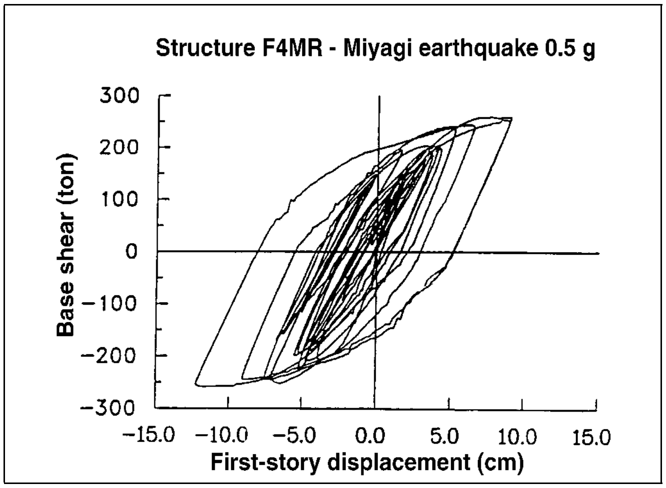


Figure C5-16 Response of Braced Story with Moment Frame Backup

does not exceed 0.4 times the KL/r of the overall member was recommended (Xu and Goel, 1990). For single gusset plate connections in members buckling out of plane, the gusset plates should have a clear length of about two times their thickness in order to allow for restraint-free plastic rotations during cyclic post-buckling of the member (Astaneh-Asl et al., 1986). Some of these recommendations, such as using concrete infill in tubular members and increasing the number of stitches in built-up members, can be used in seismic upgrading of existing structures.

As a result of the research findings discussed above, provisions were introduced for Special (ductile) Concentric Braced Frames (SCBF) in the 1994 *Uniform Building Code* and the 1994 NEHRP *Recommended Provisions for Seismic Regulations for New Buildings* (BSSC, 1995). The older provisions for CBFs were retained as applicable to Ordinary Concentric Braced Frames (OCBF). In both provisions the R_w or R factors were adjusted to reflect the additional requirements to ensure ductile behavior of bracing members.

C5.5.2.2 Stiffness for Analysis

The purpose of a Linear Static or Dynamic Procedure is to evaluate the acceptability of components, elements, and connections in a rather simplistic manner. Unlike other framing systems, seismic behavior and performance of a CBF are very much governed by those of the bracing members and their connections. Use of a linear procedure for evaluation purposes is usually based on the premise that the component is capable of reaching maximum displacements under expected reversed cyclic deformations without any major drop in actual strength. Since this is usually not the case for a CBF, the factor C_3 is introduced in Section 3.3.1. Also, the m values as given in Table 5-7 have been derived by taking the pertinent factors into consideration. Professional judgment should be applied as appropriate. Use of Nonlinear Static or Dynamic Procedures is highly recommended for more precise evaluation.

The major components of a CBF are beams, columns, and braces. Because of the truss action, a CBF is considerably stiffer than a moment-resisting frame of equal strength, prior to buckling or yielding of bracing members at moderate story drift levels. Under increasing story drifts, the buckling of compression braces is followed by yielding of tension braces, after which the truss action partially breaks down, but the columns develop very substantial additional shear strengths through flexure. The strength and stiffness contribution of columns comes not only from those in the braced bays, but also from all other columns that are designed to support gravity loads only. This is because the columns in steel frames are generally made continuous even when the beam-to-column connections are not moment-resisting. Thus, CBF structures can possess very substantial overstrength after buckling of the compression braces. For nonlinear procedures, all columns may be included in the model with proper regard to their continuity and base connection details.

The force-deformation behavior of a brace is governed by the tension yield force, $P_y = AF_y$, the compression buckling load, and the post-buckling residual compression force, which are functions of the yield stress and the slenderness ratio of the brace. The residual force is also influenced by compactness, cross-section shape, and other details of the member. A typical force versus axial deformation response of a steel brace is shown in Figure C5-17. For this brace the residual force was about 20% of the buckling load, a percentage that is about the same for many brace configurations. Tests on a variety of bracing members have been carried out at the University of Michigan (Gugerli and Goel, 1982; Aslani and Goel, 1989). Other test results for brace components are available from the following sources: Lee and Goel, 1987; Xu and Goel, 1990; Fukuta et al., 1989; Goel and El–Tayem, 1986; Fitzgerald et al., 1989; Astaneh-Asl et al., 1986. Results of testing and/or analysis of braced frame elements have been reported by the following: Khatib et al., 1988; Ricles and Popov, 1987; Khatib et al., 1987; Bertero et

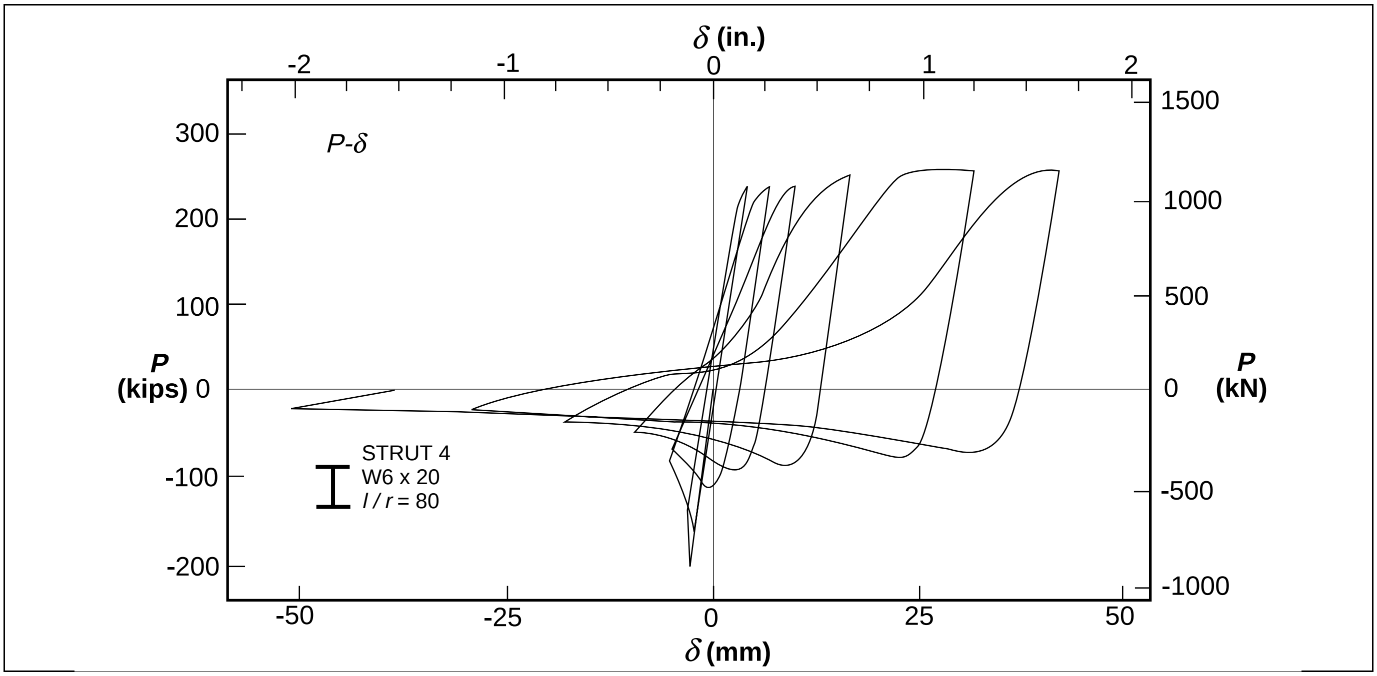


Figure C5-17 Typical Load versus Axial Deformation Behavior for a Brace

al., 1989; Wijanto et al., 1992; Uang and Bertero, 1986; Takanashi and Ohi, 1984; Midorikawa et al., 1989; Whittaker et al., 1989; Goel, 1986; Redwood, et al., 1991; Wijanto et al., 1992; Foutch et al., 1987; Roeder, 1989; Fukuta et al., 1984; Bertero et al., 1989; and Yang, 1984.

The hysteretic behavior of a brace may be modeled fairly accurately by using phenomenological models (Jain and Goel, 1978) or physical theory models (Ikeda and Mahin, 1984). The axial force versus axial deformation behavior of the Jain-Goel model is shown in Figure C5-18. A brace model similar to this should be used for Nonlinear Static or Dynamic Procedures (Rai, Goel, and Firmansjah, 1995). For a more simplified NSP, the axial force-deformation behavior of a brace in compression could be modeled as an elasto-plastic element with the yield force equal to the residual force. The residual force can be determined from Table 5-8 and Figure 5-1. However, an elastic analysis would also need to be done to determine the maximum axial force delivered to the column, the beam, and the beam-column connections.

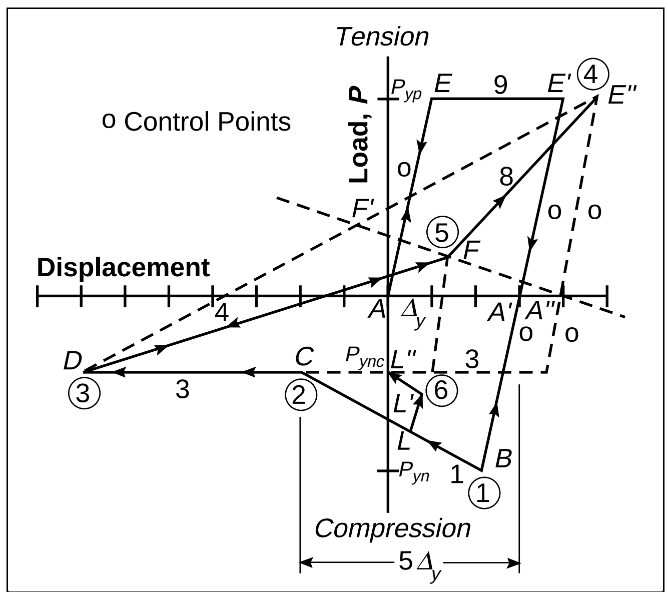


Figure C5-18 Axial Hysteresis Model—Load Starting in Tension

C5.5.2.3 Strength and Deformation Acceptance Criteria

The effective length factor is very important for calculating the expected strength of the brace. For diagonal, V, or inverted V braces attached to the column and beam with gusset plates through welded connections, the clear length of the brace should be used with a *k* of 0.8 for in-plane buckling and 1.0 for out-of-plane buckling. For bolted connections, a *k* value of 0.9 should be used.

C5.5.2.4 Rehabilitation Measures for Concentric Braced Frames

A. Component Strength Enhancement

Columns. The provisions for rehabilitating columns in moment frames are applicable to CBFs.

Beams. Provisions are the same as for moment frames:

Braces. Rehabilitation measures for braces include the following:

- Shear—Add steel plates parallel to the shear force, or encase in concrete.
- Moment—Add steel plates or encase in concrete.
- Axial —Add steel plates to increase section strength and/or reduce member slenderness; encase in concrete; provide secondary bracing members to reduce unbraced length; or replace with a section with greater capacity.
- Combined stresses—Use measures similar to those for axial braces.
- Stability—Stiffen element or connections by additional steel plates; provide secondary bracing elements; encase in concrete; or replace with a section with greater capacity.
- Concrete encasement—Remove or modify in cases where concrete causes undesirable failure mode.
- Element section properties
  - High b/t ratios—Infill with concrete, or replace with different section.
  - Spacing or capacity of stitch plates—Strengthen existing stitch connections, or provide stitch plates. If stitch plates are already in place, provide additional stitch plates.

Connections. Rehabilitation measures for connections include the following.

- Brace connections—Add welds or bolts; replace rivets with high-strength bolts; add plates to strengthen the connection.
- Concrete encasement—Remove or modify in cases where concrete causes an undesirable failure mode.
- Column base strength—Use same measures as for moment frames.

System Enhancements. The following system enhancements should be considered:

- “K” bracing—Remove bracing or strengthen column such that strength and stiffness are sufficient to transfer maximum bracing forces.
- Knee bracing—Use the same measures as for “K” bracing.
- Chevron bracing—Strengthen beam as required to develop maximum unbalanced bracing loads.
- Tension-only systems—Replace bracing with elements capable of resisting compression loads, or add stiffening elements.

B. Rehabilitation Measures for Deformation Deficiencies

The following rehabilitation measures for adding stiffness to the building should be considered.

- Add steel plates.
- Encase in concrete.
- Replace existing braces.
- Add concrete or masonry infills.
- Add reinforced concrete shear walls.

C5.5.3 Eccentric Braced Frames (EBF)

C5.5.3.1 General

The eccentrically braced frame represents a hybrid framing system that is both stiff and ductile. The presence of the link beam, created by offsetting the point of action of the braces that frame into a beam, is primarily responsible for both the high stiffness of the frame and the good ductility characteristics.

The link beam is called short if $e < 1.6M_p/V_n$, and long if $e > 2.6M_p/V_n$, where e is the length of the link, M_p is the nominal plastic moment capacity of the section, and V_n is the nominal plastic shear capacity of the section. Links in the intermediate range of lengths are subject to interaction between moment and shear. A short link is stiffer than a long link, but it is also prone to greater ductility demands. Frame stiffness decreases rather rapidly with link length. The length of a link is generally chosen to maximize frame stiffness within the limits of available link ductility.

C5.5.3.2 Stiffness for Analysis

Elastic shearing deformations are important to the stiffness of the link element, which is typically modeled as a beam. The stiffness associated with flexural deformation is given by

$$K_b = \frac{12EI}{e^3} \qquad \text{(C5-39)}$$

where E is Young's modulus, I is the second moment of the cross-sectional area, and e is the length of the link.

Similarly, the stiffness associated with shear deformation is given by

$$K_s = \frac{GA_w}{e} \qquad \text{(C5-40)}$$

where G is the shear modulus and $A_w = t_w(d_b - 2t_f)$ is the area of the web. The ratio of bending to shear stiffness, $\beta = K_b/K_s$, characterizes the importance of shearing deformation to the stiffness. The stiffness of the link can be expressed in terms of β and the combined stiffness K given by

$$K = \frac{K_b K_s}{K_b + K_s} = \frac{K_b}{1 + \beta} \qquad \text{(C5-41)}$$

The stiffness coefficients associated with unit rotation of one end, and unit translation of one end, of a link are given in Figure C5-19. It should be noted that for long beams, $\beta \rightarrow 0$ and the stiffness coefficients are the customary values used in ordinary structural analysis. When analyzing an EBF with a structural analysis program, the effects of shearing deformations must be accounted for by the program.

For a short link, energy associated with overloading is dissipated primarily through inelastic shearing of the link web. For a long link, the overload energy is dissipated primarily through plastic hinging at the ends of the link. The shear yielding energy dissipation mechanism is more efficient than the flexural plastic hinging mechanism.

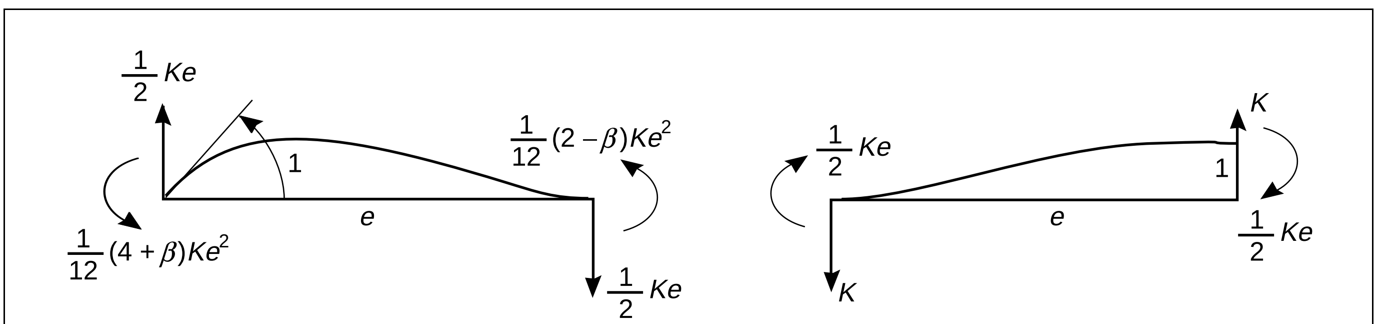


Figure C5-19 Stiffness Coefficients for a Link of Length e

The plastic capacity of a link is governed by shear-moment interaction. For design purposes, the shear-

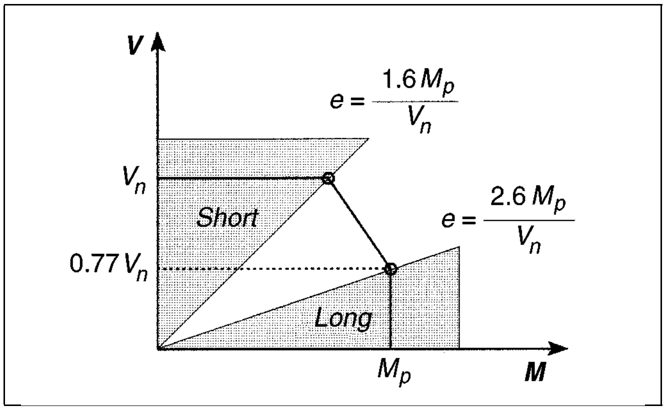


Figure C5-20 Shear-Moment Interaction

moment interaction diagram is idealized as shown in Figure C5-20. The nominal moment capacity of a beam is given by

$$M_p = F_y Z$$

where F_y is the uniaxial yield strength of the material and Z is the plastic section modulus. The nominal shear yield strength of a beam is given by

$$V_n = 0.6 F_y A_w$$

where $0.6F_y$ is the shear yield strength and $A_w = T_w = t_w(d_b - 2t_f)$ is the area of the web. These values provide the bounds on moment and shear that a link can sustain, as illustrated in the shear-moment interaction diagram of Figure C5-20. Moment M, shear V, and link length e are related through static equilibrium. The radial lines that emanate from the origin of the moment-shear interaction plot represent equilibrium lines for constant values of e.

The values $1.6M_p/V_n$ and $2.6M_p/V_n$ that define the bounds of short and long links in Figure C5-20 are based upon empirical observations. These different regions of link behavior are important to the following issues: (1) placement and detailing of web and flange stiffeners in the link region, (2) the strength of the link element, and (3) the ductility that the link element can supply. For short links, web buckling is the primary concern, while for long links local flange buckling is important. The requirements for placement and detailing of stiffeners can be found in Section 10.3 of AISC (1994a).

For a short link, the web yields while the flanges remain elastic. Therefore, the plastic capacity of a short link does not depend upon the moment carried by the link, and hence the shear capacity is $Q_{CE} = V_n$. A long link yields through the formation of a plastic hinge. The influence of the shear stresses on the yielding is so small that they do not affect the strength of the link. As the link yields, the forces tend to redistribute so that the full plastic moment develops on both ends of the link. Static equilibrium insists that $V = 2M_p/e$. Thus, the shear capacity can be equivalently expressed as $Q_{CE} = 2M_p/e$. The smallest link length that can be considered a long link is $e = 2.6M_p/V_n$. The shear capacity for a link of this length is therefore $Q_{CE} = 0.77V_n$. The capacity of a link of intermediate length is given by linear interpolation between the limiting values of short and long links; that is,

$$Q_{CE} = \left[1.37 - 0.23\frac{eV_{CE}}{M_{CE}}\right]V_{CE} \qquad \text{(C5-42)}$$

for $1.6 < EV_n/M_p < 2.6$.

The deformation of a link beam is characterized in terms of the angle between the axis of the link and the axis of the beam adjacent to the link, as shown in Figure C5-21. The link deformation angle at first yield can be computed as the shear force divided by the stiffness

$$\gamma_y = \frac{Q_{CE}}{Ke}$$

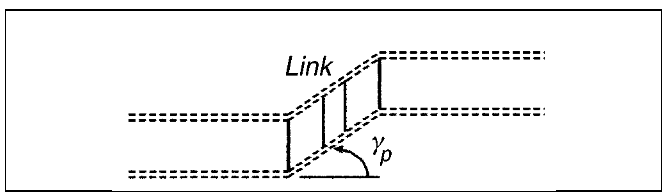


Figure C5-21 Link Rotation Angle

C5.5.3.3 Strength and Deformation Acceptance Criteria

The deformation capacity, γ_p, of a link beam depends upon the length of the link as well as the web and flange stiffening details. An idealization of link behavior is shown in Figure C5-22. The limit state for γ_p is web or flange buckling, as significant deterioration of link behavior begins after buckling. For adequately stiffened short links, the rotation capacity is approximately $\gamma_p = 0.12 \text{ rad}$.

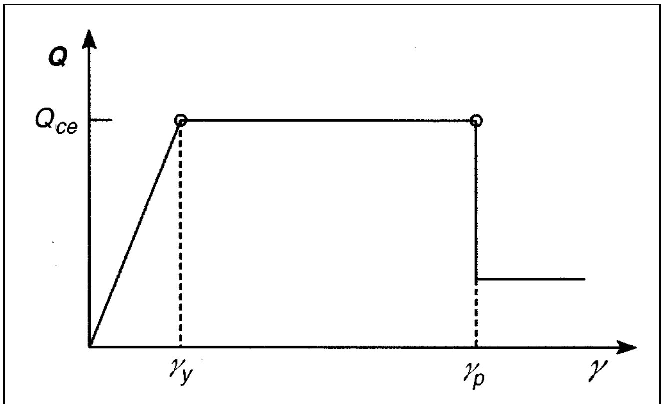


Figure C5-22 *Deformation Capacity Definitions for a Link*

Among reports giving experimental results are Ricles and Popov, 1987 and 1989; Hjelmstadt and Popov, 1983; Yang, 1982; Malley and Popov, 1983; Nishiyama et al., 1989; Whittaker et al., 1987 and 1989; Popov and Ricles, 1988; Foutch, 1989; and Foutch et al., 1987.

C5.5.3.4 Rehabilitation Measures for Eccentric Braced Frames

No commentary is provided for this section.

C5.6 Steel Plate Walls

No commentary is provided for this section.

C5.7 Steel Frames with Infills

The stiffness and resistance provided by concrete and/or masonry infills may be much larger than the stiffness of the steel frame acting alone with or without composite action. However, gaps or incomplete contact between the steel frame and the infill may negate some or all of this stiffness. These gaps may be between the wall and columns of the frame or between the wall and the top beam enclosing the frame. Different strength and stiffness conditions must be expected with different discontinuity types and locations. Therefore, the presence of any gaps or discontinuities between the infill walls and the frame must be determined and considered in the design and rehabilitation process. The resistance provided by infill walls may also be included if proper evaluation of the connection and interaction between the wall and the frame is made and if the strength, ductility, and properties of the wall are properly included.

Frames Attached to Masonry Walls. Attached walls are by definition somewhat separate from the steel frame. The stiffness and resistance provided by the walls may be large. However, the gaps or incomplete contact known to exist between the steel frame and the wall negate some or all of this strength and stiffness. As a result, the stiffness provided by attached masonry walls is excluded from the design and rehabilitation process unless integral action between the steel frame and the wall is verified. If complete or partial interaction between the wall and frame is verified, the stiffness is increased accordingly. The seismic performance of unconfined masonry walls is far inferior to that of confined masonry walls; therefore, the resistance of the attached wall can be used only if strong evidence as to its strength, ductility, and interaction with the steel frame is provided.

C5.8 Diaphragms

C5.8.1 Bare Metal Deck Diaphragms

C5.8.1.1 General

Diaphragms for bare steel decks are typically composed of corrugated sheet steel of 22 gage to 14 gage. The depths of corrugated sheet steel ribs vary from 1-1/2 to 3 inches in most cases, and attachment of the diaphragm to the steel frame occurs through puddle welds to the deck, typically at a spacing of one to two feet on center. This type of diaphragm is typically used only for roof construction. For large roof structures, supplementary diagonal bracing may be present for additional support.

The distribution of forces for existing diaphragms for bare steel decks is generally based on the flexible diaphragm assumption. Flexibility factors for various available types of diaphragms are available from manufacturers' catalogs. For systems where values are not available, it is best to interpolate with similar systems that do have values.

For bare metal decks, interaction between new and existing elements of the diaphragms (stiffness compatibility) must be considered as well as interaction with existing frames. Load transfer mechanisms between new and existing diaphragm elements and existing frames may need to be considered in flexibility of the diaphragm. (Analyses need to verify that diaphragm strength is not exceeded, so that elastic assumptions are still relatively valid.)

C5.8.1.2 Stiffness for Analysis

Inelastic properties of diaphragms are generally not included in inelastic seismic analyses. This is because diaphragm strength is generally quite high compared to demands, especially when concrete topping is present.

More flexible diaphragms, such as bare metal deck, could be subject to inelastic action. Procedures for developing models for inelastic response of wood diaphragms in URM buildings could be used as the basis for an inelastic model of a bare metal deck diaphragm condition. If the weak link of the diaphragm is connector failure, then the element nonlinearity obviously cannot be incorporated into the model.

C5.8.1.3 Strength and Deformation Acceptance Criteria

Among the deficiencies most commonly found in bare metal deck diaphragms are:

- Inadequate connection between metal deck and chord or collector components
- Inadequate strength of chord or collector components
- Inadequate attachment of deck to supporting members
- Inadequate strength and/or stiffness of metal deck

C5.8.1.4 Rehabilitation Measures

Typical methods for correcting deficiencies in bare metal decks include:

- Adding shear connectors for chord or collector forces
- Strengthening existing chords or collectors by the addition of new steel plates to existing frame components
- Adding puddle welds or other shear connectors at panel perimeters
- Adding diagonal steel bracing to supplement diaphragm strength
- Replacing nonstructural fill with structural concrete
- Adding connections between deck and supporting members

New bare metal deck diaphragms should be designed and constructed in accordance with the recommendations of the Steel Deck Institute (SDI), given in the SDI Diaphragm Design Manual.

C5.8.2 Metal Deck Diaphragms with Structural Concrete Topping

C5.8.2.1 General

No commentary is provided for this section.

C5.8.2.2 Stiffness for Analysis

No commentary is provided for this section.

C5.8.2.3 Strength and Deformation Acceptance Criteria

Deficiencies that have been identified for metal deck diaphragms with structural concrete topping include:

- Inadequate connection between metal deck and chord or collector components (puddle welds and/or shear studs)
- Inadequate strength of chord or collector components
- Inadequate attachment of deck and concrete to supporting members
- Inadequate strength and/or stiffness of metal deck and composite concrete fill

C5.8.2.4 Rehabilitation Measures

Typical methods for correcting deficiencies include:

- Adding shear connectors for chord or collector forces

- Strengthening existing chords or collectors by the addition of new steel plates to existing frame components; also, attaching new plates directly to the slab with attachments such as embedded bolts, or epoxy
- Adding diagonal steel bracing to supplement diaphragm strength

New metal deck diaphragms with structural concrete topping should be designed and constructed in accordance with SDI recommendations or manufacturers' catalogs. Also, diaphragm shear capacity can be calculated considering the strength of concrete above the deck ribs in accordance with UBC or ICBO reports.

C5.8.3 Metal Deck Diaphragms with Nonstructural Concrete Topping

C5.8.3.1 General

No commentary is provided for this section.

C5.8.3.2 Stiffness for Analysis

No commentary is provided for this section.

C5.8.3.3 Strength and Deformation Acceptance Criteria

Deficiencies that have been identified for metal deck diaphragms with nonstructural concrete topping include

- Inadequate connection between metal deck and chord or collector components
- Inadequate strength of chord or collector components
- Inadequate attachment of deck to supporting members
- Inadequate strength and/or stiffness of metal deck and nonstructural concrete fill

C5.8.3.4 Rehabilitation Measures

Typical methods for correcting deficiencies in metal decks with nonstructural topping include

- Adding shear connectors for chord or collector forces
- Strengthening existing chords or collectors by the addition of new steel plates to existing frame elements, or attaching new plates directly to the slab with embedded bolts or epoxy
- Add puddle welds at panel perimeters of bare deck diaphragms
- Adding diagonal steel bracing to supplement diaphragm strength
- Replacing nonstructural fill with structural concrete

New metal deck diaphragms with structural concrete topping should be designed and constructed in accordance with SDI recommendations or manufacturers' catalogs. Also, diaphragm shear capacity can be calculated considering the strength of concrete above the deck ribs in accordance with UBC or ICBO reports.

C5.8.4 Horizontal Steel Bracing (Steel Truss Diaphragms)

C5.8.4.1 General

Horizontal steel trusses are generally used in combination with bare metal deck roofs or conditions where diaphragm stiffness is inadequate to transfer shear forces. It is more common for long spans or in situations with a longer overall width of diaphragm. Other examples are special roof structures of exposition halls, auditoriums, and others. The addition of horizontal steel trusses is one enhancement technique for weaker diaphragms.

The size and mechanical properties of the tension rods, compression struts, and connection detailing are all important to the yield capacity of the horizontal truss. Standard truss analysis techniques can be used to determine the yield capacity of the horizontal truss. Special attention is required at connections between different members of the horizontal truss. Connections that will develop the yield capacity of the truss members and reduce the potential for brittle failure are desired.

Stiffness can vary with different systems, but is most often fairly flexible with a fairly long period of vibration. Classical deflection analysis procedures can be used to determine the stiffness of the horizontal truss. Span-to-depth ratios of the truss system can have a significant effect on the stiffness of the horizontal truss. Lower span-to-depth ratios will result in increased stiffness of the horizontal truss. For equivalent lateral-force methods, factoring of the lateral force will be required to predict the actual deflection of the truss system.

More flexible, lower-strength horizontal truss systems may perform well for upgrades to the Life Safety Performance Level. Upgrades to the Damage Control Performance Range or the Immediate Occupancy Performance Level will require proportional increases in yield capacity and stiffness to control lateral displacements. Displacements must be compatible with the type of construction supported by the horizontal truss system.

Chord and collector elements for the above-listed diaphragms are generally considered to be composed of the steel frame elements attached to the diaphragm. For diaphragms with structural concrete, special slab reinforcement may be used in combination with the frame elements to make up the chords and/or collectors. The load transfer to the frame elements, which act as chords or collectors in modern frames, is generally through shear connectors. In older construction, the load transfer is made through bond when the frame is encased for fire protection.

C5.8.4.2 Stiffness for Analysis

Inelastic behavior may not be generally permitted in a steel truss diaphragm. Deformation limits to be established are to be more consistent with that of a diaphragm.

Classical truss analysis methods can be used to determine which members or connections of the existing horizontal truss require enhancement. Analysis of existing connections, and enhancement of connections with insufficient yield capacity, should be performed in a manner that will encourage yielding in the truss members rather than brittle failure in the truss connections.

C5.8.4.3 Strength and Deformation Acceptance Criteria

No commentary is provided for this section.

C5.8.4.4 Rehabilitation Measures

Deficiencies that may occur in existing horizontal steel bracing include the following:

- Various components of the bracing may not have strength to transfer all of the required forces.
- Various components of the bracing may not have sufficient ductility.
- Bracing connections may not be able to develop the strength of the members, or an expected maximum load.
- Bracing may not have sufficient stiffness to limit deformations below acceptable levels.

Typical methods for correcting deficiencies include the following:

- Diagonal components can be added to form a horizontal truss; this may be a method of strengthening a weak existing steel-framed floor diaphragm.
- Existing chord components may be strengthened by the addition of shear connectors to enhance composite action.
- Existing steel truss components may be strengthened by methods similar to those noted for braced steel frame members.
- Truss connections may be strengthened by the addition of welds, new or enhanced plates, and bolts.
- Where possible, structural concrete fill may be added to act in combination with steel truss diaphragms. Gravity load effects of the added weight of the concrete fill must be considered in such a solution.

Design of completely new horizontal steel bracing elements should generally follow the procedures required for new braced frame elements.

C5.8.5 Archaic Diaphragms

C5.8.5.1 General

No commentary is provided for this section.

C5.8.5.2 Stiffness for Analysis

No commentary is provided for this section.

C5.8.5.3 Strength and Deformation Acceptance Criteria

No commentary is provided for this section.

C5.8.5.4 Rehabilitation Measures

Deficiencies that may occur in existing archaic diaphragms include the following:

- The lack of steel reinforcing severely limits the ability of the element to resist diagonal tension forces without significant cracking.
- Diagonal tension could jeopardize the compression forces in the brick arches, creating a situation that could lead to loss of support.
- Connections between the brick work and steel may not be able to transfer the required diaphragm forces.
- The diaphragm may not have sufficient stiffness to limit deformations below acceptable levels.

Typical methods for correcting deficiencies include the following.

- Diagonal elements can be added to form a horizontal truss.
- Existing steel members may be strengthened by the addition of shear connectors to enhance composite action.
- Weak concrete fill may be removed and replaced by a structural reinforced concrete topping slab. Gravity load effects of the added weight of the concrete fill must be considered in such a solution.

C5.8.6 Chord and Collector Elements

C5.8.6.1 General

No commentary is provided for this section.

C5.8.6.2 Stiffness for Analysis

No commentary is provided for this section.

C5.8.6.3 Strength and Deformation Acceptance Criteria

No commentary is provided for this section.

C5.8.6.4 Rehabilitation Measures

Deficiencies that have been identified for chords and collectors include:

- Inadequate connection between diaphragm and chords or collectors
- Inadequate strength of chord or collector
- Inadequate detailing for strength at openings or re-entrant corners

Typical methods for correcting deficiencies include the following:

- The connection between diaphragms and chords and collectors can be improved.
- Chords or collectors can be strengthened with steel plates. New plates can be attached directly to the slab with embedded bolts or epoxy. Also, reinforcing bars can be added to the slab.
- A structural slab can be added to improve compressive capacity of existing chords and collectors.
- Chord members can be added.

New chord and collector components should be designed in accordance with the requirements of the AISC Manual or ACI Building Code.

C5.9 Steel Pile Foundations

C5.9.1 General

No commentary is provided for this section.

C5.9.2 Stiffness for Analysis

Two analytical models are commonly used to analyze pile foundations: the equivalent soil spring model and the equivalent cantilever model. These are shown schematically in Figure C5-23.

The equivalent soil spring model is often used for the design of pile foundations for bridges. The properties of the soil spring are dependent on the soil properties at the site. Both linear and nonlinear models are available. A complete description of the model and a computer program for its implementation are given in FHWA (1987).

Before the development of the equivalent soil spring model, the primary model used to obtain the stiffness and maximum moments for piles was the equivalent cantilever method, represented in Figure C5-24. The pile is considered to be a cantilever column. The stiffness of the pile is assumed to be the same as for a free-standing cantilever column with a length of L_s. The maximum moment in the pile is assumed to be the same as for a free-standing cantilever column with a length of L_M. The lengths L_S and L_M depend on EI of the pile and a soil constant as given in Figure C5-24. Additional information on pile capacity may be found in Davisson (1970) and in most foundation engineering textbooks.

C5.9.3 Strength and Deformation Acceptance Criteria

In most situations the calculation of the pile strength is straightforward, since buckling is not a consideration unless the pile extends above the ground surface or through a liquefiable soil. A pile that extends above the ground surface may be analyzed as a free-standing column with length $L_C = (L_F + L_S)$ and $K = 1.0$ where L_C is the equivalent column length, L_F is the length above ground, and L_S is as given in Figure C5-24. For piles that pass through a liquefiable soil, guidance should be sought from a geotechnical engineer.

C5.9.4 Rehabilitation Measures for Steel Pile Foundations

No commentary is provided for this section.

C5.10 Definitions

No commentary is provided for this section.

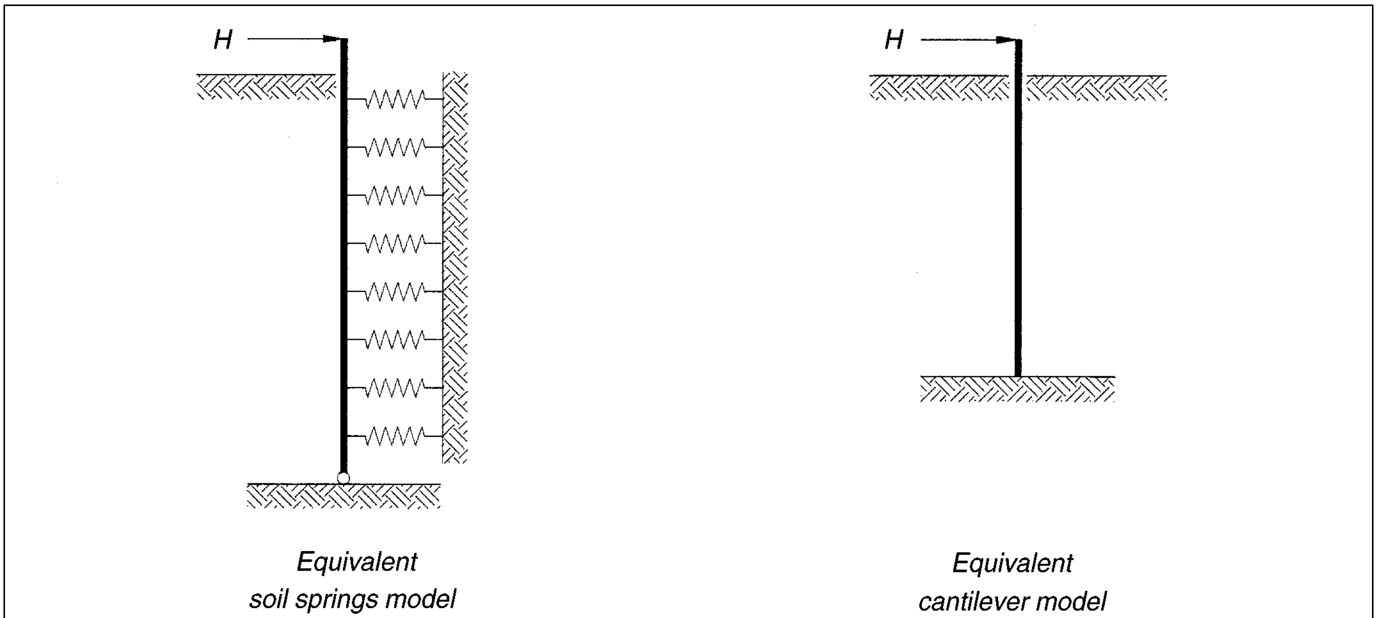
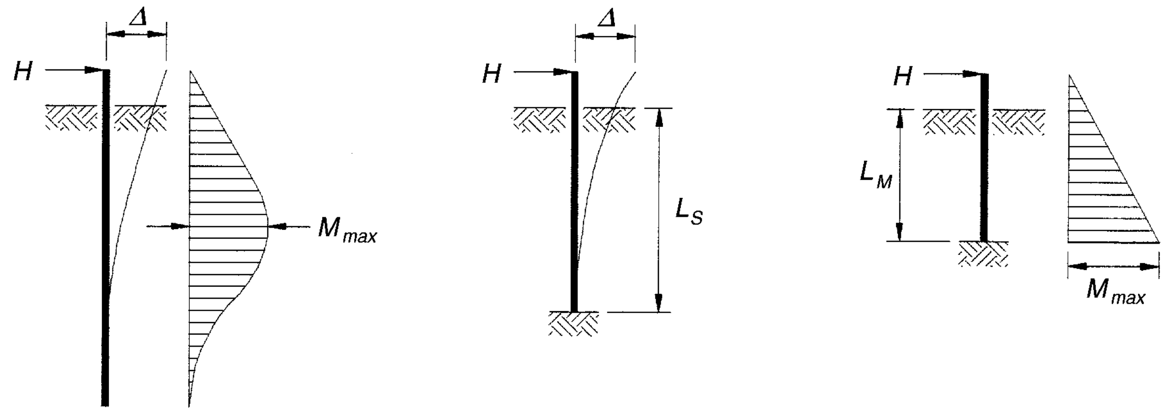


Figure C5-23 Models for Pile Analysis

C5.11 Symbols



	L_S	L_M
Cohesive soil constant K_h	$1.4\sqrt[4]{\frac{EI}{K_h}}$	$0.44\sqrt[4]{\frac{EI}{K_h}}$
Cohesionless soil constant n_h	$1.8\sqrt[5]{\frac{EI}{n_h}}$	$0.78\sqrt[5]{\frac{EI}{n_h}}$

Figure C5-24 Equivalent Cantilever Model for Piles

This list may not contain symbols defined at their first use if not used thereafter.

A_c Gross cross-sectional area of connector, in.^{2}

A_e Net effective area of stem, in.^{2}

A_g Gross area of T-stub stem, in.^{2}

A_w Area of web of link beam, in.^{2}

E Modulus of elasticity, 29,000 ksi

F_{ve} Expected shear strength of connector, ksi

F_y Yield strength, ksi

F_{ye} Expected yield strength, ksi

G Shear modulus, ksi

I_b Moment of inertia of beam, in.^{4}

$I_b adj$ Adjusted moment of inertia of beam, in.^{4}

I_c Moment of inertia of column, in.^{4}

K Stiffness of a link beam, kip/in.

K Coefficient for Equations C5-9, C5-25, and C5-38

K_b Flexural stiffness of link beam, kip-in./rad

K_θ Rotational stiffness of a partially-restrained connection, kip-in./rad

M_{CE} Expected flexural strength of a member or joint, kip-in.

M_{CE} Expected flexural strength, kip-in.

N_{OSL} Number of connectors in outstanding leg of clip angle, dimensionless

N_{stem} Number of connectors in stem of T-stub connection, dimensionless

N_{VL} Number of tensile connectors in T-stub connection, dimensionless

P Force, kips

P_{CE} Expected strength, kips

Q_{CE} Effective expected shear strength of link beam, kips

Z Plastic section modulus, in.^{3}

d Dimension of end plate connection, in.

d_b Beam depth, in.

f Bolt diameter, in.

h Story height, in.

k_s Rotational stiffness of connection, kip-in./rad

k_s Shear stiffness of link beam, kip/in.

l_b Length of beam, in.

m Modification factor used in the acceptance criteria of deformation-controlled components or elements, indicating the available ductility of a component action.

t Plate thickness, in.

t_f Flange thickness, in.

t_s Stem thickness of T-stub, in.

t_w Thickness of web of link beam, in.

u Deflection, in.

w Width of T-stub, in.

Δ Generalized deformation, dimensionless

γ_p Deformation capacity of link beam, radians

γ_y Yield deformation of link beam, radians

θ Rotation, radians

C5.12 References

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