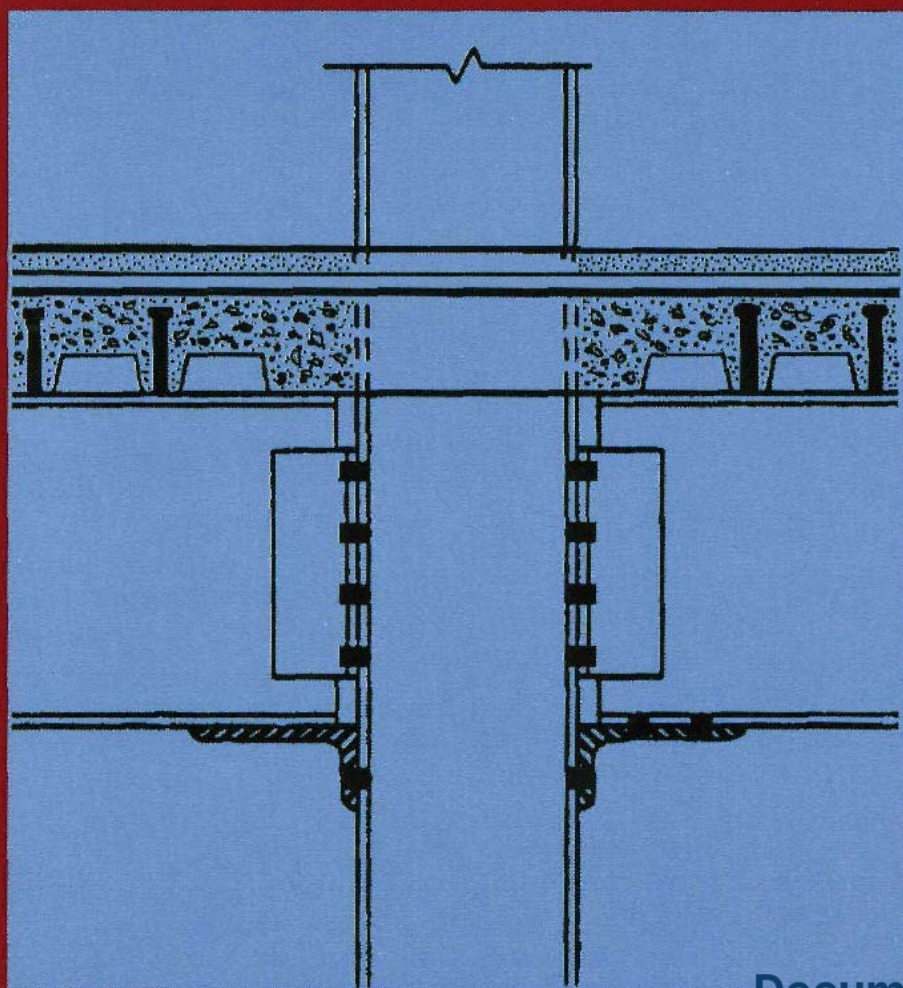




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Steel Design Guide Series

Partially Restrained Composite Connections



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Steel Design Guide Series

Partially Restrained Composite Connections

A Design Guide

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Preface

This booklet was prepared under the direction of the Committee on Research of the American Institute of Steel Construction, Inc. as part of a series of publications on special topics related to fabricated structural steel. Its purpose is to serve as a supplemental reference to the AISC *Manual of Steel Construction* to assist practicing engineers engaged in building design.

This document is intended to provide guidelines for the design of braced and unbraced frames with partially restrained composite connections (PR-CCs). The design procedures and examples in this guide represent a refinement of the work presented by Ammerman and Leon^{7,8} and is thoroughly documented in more recent work by the authors.^{12,21} The design of structures utilizing PR-CCs for gravity and wind loads falls under the provisions of Section A2.2 of the LRFD Specification for Structural Design of Buildings. Design for seismic loads is allowed under Section 7.4.1 of the latest version of the NEHRP provisions.

The guide is divided into four parts. The first part is an introduction dealing with topics pertinent to partially restrained (PR) analysis and design, and discusses some of the important design choices utilized in the design procedures and examples. The second part contains detailed, concise design procedures for both braced and unbraced frames with partially restrained composite connections. The third part consists of a detailed design example for a four-story building. The design is for an unbraced frame in one principal direction and for a braced frame in the other. The fourth part contains design aids in the form of Tables and Appendices.

It is important that the reader recognize that the guide is intended to be a self-contained document and thus is longer than comparable documents dealing with similar topics. The reader is advised, on a first reading, to read Parts I and III carefully, consulting Part IV as necessary. Once the reader is familiar with the topic, he/she will only need to consult Parts II and IV in doing routine design work.

The design guidelines suggested by the authors that are outside the scope of the AISC Specification or Code do not represent an official position of the Institute and are not intended to exclude other design methods and procedures. It is recognized that the design of structures is within the scope of expertise of a competent licensed structural engineer, architect, or other licensed professional for the application of principles to a particular structure.

Acknowledgments

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Part I BACKGROUND

1. INTRODUCTION

Partially restrained connections, referred to as PR connections in the LRFD provisions¹ and Type 3 connections in the ASD provisions,² have been permitted by the AISC Specifications since 1949. With some notable exceptions, however, this type of connection has not received widespread application in practice due both to (a) the perceived complexity of analysis required, and (b) the lack of reliable information on the moment-rotation characteristics of the connections as required by design specifications. The notable exceptions involve specific types of connections that have been demonstrated, through experience in the field and extensive analytical work,^{3,4} to provide equivalent response under design conditions to that of rigid connections. The Type 2 or "wind" connections allowed under the ASD provisions are a good example of this approach. In these cases the specification essentially prequalifies a simple connection under gravity loads as a rigid connection under lateral loads. In reality, of course, these connections are neither fully rigid (FR) nor simple but partially restrained (PR). The code uses this artifice to simplify the analysis and design, but requires a guaranteed rotational and strength capacity from these connections.

After 10 years of research and development a new type of semi-rigid connection, labelled the Partially Restrained Composite Connection or PR-CC,* can be added to this list.⁵⁻¹² The word "composite" is used to indicate that this connection engages the reinforcing steel in the concrete slab to form the top portion of the moment resisting mechanism under both live loads and additional dead loads applied after the end of construction (Figure 1). The bottom portion is typically provided by a steel seat angle with web angles providing the shear resistance. This connection may be used to economize beam sizes for gravity loading or to resist lateral loads in unbraced frames. The design of this type of system is based not only on the work of the senior author at the University of Minnesota,^{5-12,21} but also on that of many researchers throughout the U.S. and Europe.^{11,13-19} The extensive experimental work required in the development of these connections is discussed elsewhere^{5,6,9} and will not be repeated here.

Part I of this design guide is organized as follows. First, some discussion of partially restrained connection behavior

will be given to put PR-CC design in its proper context. Second, the advantages and limitations of PR-CCs are discussed in the context of simplified or code-oriented design. Third, the assumptions and theory applied in their design are described. Fourth, detail recommendations for the connections under both gravity and lateral loads are given. In Part II a step-by-step procedure is presented in outline form followed by corresponding detailed calculations for an example problem in Part III. The 1993 *Load and Resistance Factor Design* (LRFD) Specification¹ is used in the design and ASCE 7-93²⁰ is used for load determination. Tables and design aids are included in Part IV to facilitate the design.

2. CHARACTERIZATION OF CONNECTION BEHAVIOR

The behavior of structural connections can be visualized for design purposes with the aid of moment-rotation $M-\theta$ curves (Figure 2). These curves are generally taken directly from individual tests or derived by best-fit techniques from the results of multiple tests.^{22,23} All design specifications require that the structural engineer have a reliable $M-\theta$ curve for the PR connections to be used in design since such curves syn-

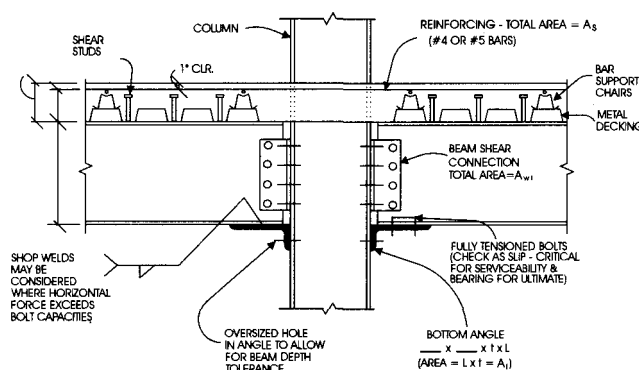


Fig. 1. Partially restrained composite connection (PR-CC).

* The label PR-CC is meant to encompass the connections previously labelled semi-rigid composite connections (SRCC) by the senior author.

the size the connection's main characteristics: stiffness, strength, and ductility.⁶ The application of PR-CCs to design implies that reliable M - θ relationships have been developed and are simple enough to use in design. The M - θ equations developed for SRCCs will be discussed in detail in Section 4.

In Figure 2(a), the stiffness of the connection corresponds to the slope of the M - θ curve. For most connections, such as PR-CCs, the slope changes continuously as the moment increases. The real stiffness of the connection at any stage of the M - θ curve corresponds to the tangent stiffness ($K_{tan} = \Delta M / \Delta \theta$). However, for design purposes it is customary to assume a linear approximation for the service range ($\theta < \theta_{ser}$), generally in the form of a secant stiffness ($K_{conn} = M_{ser} / \theta_{ser}$). This stiffness is generally less than the initial stiffness of the connections (K_i), and corresponds closely to the unloading stiffness ($K_{unloading}$).

Based on the initial (K_i) or service stiffness (K_{conn}), connec-

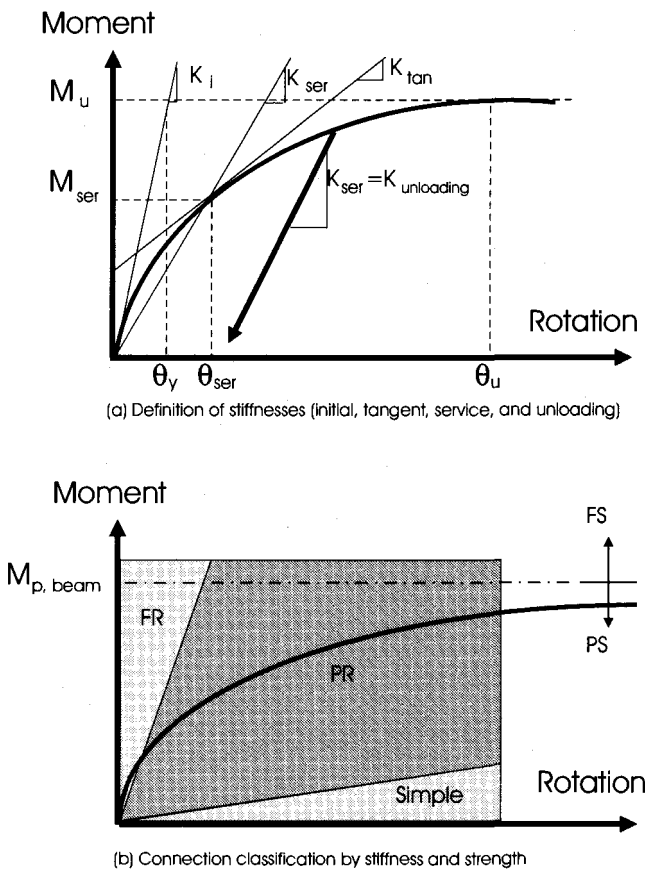


Fig. 2. Characterization of connection behavior.

tions can be classified as fully restrained (FR), partially restrained (PR) or simple depending on the degree of restraint provided (Figure 2(b)). The current approach in design is to assume that for members framing into relatively rigid supports, if the connection stiffness is about 25 times that of the girder (i.e., $(K_{conn} L_g / EI_g) > 25$), the connection can be considered rigid. Conversely, if the connection provides a stiffness less than 0.5 times that of the girder, then it should be considered simple.* The classification by stiffness is valid only for the service load range and for connections which do not exhibit significant non-linear behavior at M_{ser} .

Insofar as strength is concerned, joints can be classified either as full strength (FS) when they are capable of transferring the full moment capacity of the steel beam framing into them or as partial strength (PS) when they are not (Figure 2(b)). The schematic moment-rotation curve for a PR-CC shown in Figure 2(b) does not reach the full M_p capacity, and thus is a partial strength connection. Partial strength is desirable in seismic design because it permits a calculation of the maximum forces that a structural element will be required to withstand under the uncertain ground motions that serve as an input. If the designer knows what is the maximum moment that a connection can transmit, he/she can insure that other key elements, columns for example, remain elastic and suffer no damage even when the seismic input far exceeds the code prescribed forces. This design philosophy, known as capacity design,²⁴ is employed in this design guide. Capacity design requires that any hinging region be carefully detailed to dissipate energy and that all other elements in the structure remain basically elastic when the maximum plastic capacity of these regions is reached. Following this design philosophy, the detailing of the PR-CCs is driven by the need to provide a stable, ductile yielding mechanism such as tension yielding of the angle legs rather than a sudden, brittle failure such as bolt shearing.

Ductility is required in structural design so that some moment redistribution can occur before the connection fails. In applications for unbraced frames, and particularly if seismic loads are important, large ductilities are required. Ductilities can be defined in relative terms (θ_u / θ_y , or ultimate rotation capacity divided by a nominal yield one, see Figure 2(a)) or in absolute terms ($\theta_u > 0.05$ radians, for example). The required ductilities are a function of the structural system being used and whether large cyclic loads need to be considered in the design. In general cyclic ductilities greater than 6 (relative ductility) or 0.035 radians (absolute ductility) are desirable for frames with PR-CCs designed in areas of low to moderate seismic risk. Demands in unbraced frames for areas where wind governs the design or for braced frames are lower.

* The values of 25 and 0.5 selected here were chosen arbitrarily; ranges from 18 to 25 for the FR limit and 0.2 to 2 for the simple limit are found in the literature. The selection of specific values is beyond the scope of this guide. These values are cited only for illustrative purposes.

The PR-CCs described in this guide meet the criteria for areas of low to moderate seismic risk and can be used for the other design conditions described above.

It is important to recognize at the outset that for design purposes an exact, non-linear moment-rotation curve such as those shown in Figure 2 may not be necessary. In fact, only two important points need to be known for design. The first corresponds to the serviceability level where the stiffness, K_{conn} , must be known for deflection and drift calculations. The second point is the ultimate strength (M_{ult}) and rotation (θ_{ult}) achievable by the connection to insure that adequate plastic redistribution of stresses can occur.

3. ADVANTAGES AND LIMITATIONS

There are several practical advantages to PR-CCs. By using reinforcing in the slab the need for a top angle or top plate is eliminated. This provides a more economical solution for several reasons:

- (a) The top force and moment arm are increased resulting in either (1) a reduction of the forces in the connection for a given design moment, or (2) an increase in the connection moment capacity. The difference in strength can be substantial because the ultimate capacity of a seat angle in tension is only about one-third of its capacity in compression (area of its leg times its yield stress). Thus an A 36 ½-in. top angle 8-in. wide (total force = $8 \times 0.5 \times 36 \times 0.33 = 48$ kips) can be replaced with four #4 Grade 60 reinforcing bars (total force = $0.2 \times 4 \times 60 = 48$ kips). The capacity of the connection can then be controlled by the amount of steel in the slab. In addition, in a floor system with shallow beams (say W14s or W16s) the increase in moment arm (Y_3) can add 20 to 25 percent additional capacity.
- (b) In gravity design PR connections result in an efficient increase of the end moments. For a composite section, the strength in positive bending is typically on the order of 1.8 times that of the steel beam alone (M_p). Under a uniformly distributed load, if simple connections are used, the structural efficiency of the system is low because the large capacity of the system is required only at the centerline; most of the section strength is wasted. Similarly, if rigid connections are used the efficiency of the composite system is considerably reduced because the end moments ($wL^2/12$) are large where the section strength is small (M_p), and the midspan moments are small ($wL^2/24$) are small where the section strength is large ($1.8M_p$). Only the use of semi-rigid connections and composite action allows the designer to "balance" the connection such that the demand (external moment) is balanced by the supply (section capacity).

- (c) The use of PR-CCs reduces the required beam size and/or reduces deflection and vibration problems because of the composite action provided by the slab. The use of reinforcing bars, as opposed to the common steel mesh used for temperature and shrinkage crack control, is necessary to achieve these benefits. The use of distributed steel reinforcing bars around the columns considerably reduces crack widths over beam and column lines.
- (d) From the construction standpoint the need to cut and resupport the steel decking around the support is eliminated. The placement of some additional reinforcing bars in the slab should not represent significant additional costs.

Connection research on PR frames until recently considered only bending about the strong axis of wide flange columns. In this guide some preliminary recommendations for extending their use to the weak axis of columns in braced frames are given. When used on the weak axis the web angles are typically not used and the connection strength is reduced slightly. In general a stiffened seat is used to help carry the shear force in this situation.

Because of its increased flexibility relative to rigid (Type 1 or FR) connections, the system is most applicable in structures that are ten stories or less, and it should be limited to use with lateral wind forces or seismic loading with ground accelerations less than or equal to 0.2g only, pending further research.

It should also be clear that PR-CCs cannot, in general, be used as substitutes for rigid connections on a one-to-one basis. This implies that more connections will have to participate in resisting the lateral loads in a SRCC frame. The key to the economy of the system is that it allows the designer to turn simple connections into semi-rigid ones by adding only slab steel. The latter is inexpensive and is already being used by many designers to control cracking over column lines. Thus the additional costs for material and labor will be small. The gains in structural efficiency and redundancy will far outweigh the additional construction costs. The recent experience with the Northridge earthquake clearly points out the need for redundancy and ductility in steel lateral load resisting systems. PR-CCs clearly provide a superior level of performance in this respect and can be adopted as a secondary lateral-load resisting system in areas of high seismic risk and as the primary system in areas of low to moderate seismic risk.

4. CONNECTION $M-\theta$ CURVES

The most accurate way of modelling the $M-\theta$ behavior of a semi-rigid connection such as that shown in Figure 2 is through either a continuous exponential or a piecewise linear

function. In advanced computer programs, spring elements with similar $M-\theta$ characteristics can be input at the ends of the beams to simulate the behavior of the connections. Frames can then be analyzed under a variety of load combinations and the second order effects included directly through the use of a geometric stiffness matrix.

The design procedure proposed here simplifies the analysis to a two-level approach:

- a first order elastic analysis with linear springs at service to check beam deflections and frame drift. These results will be extended to the case of factored loads in order to check the beam-column strength equations.
- a simplified second-order, rigid-plastic analysis with a weak beam-strong column mechanism will be used to check ultimate strength and stability of the frame.

The first level is very similar to what would be used today for a rigid frame design. Many commercially available computer programs incorporate linear springs and thus this type of analysis is well within reach of the average practitioner.

The second level is used here as opposed to the conventional B1 and B2 approach for frame stability because the development of that technique for PR frames, and for frames using PR-CCs in particular, is still underway.²⁵ Several other alternatives, including (a) a rigorous analysis that models both the non-linearities in the connections and the $P-\Delta$ effects directly, or (b) an analysis with linear springs, using a secant stiffness to θ_u , are possible. The second-order plastic analysis described here is useful for preliminary design. The final design should be checked using advanced analysis tools if the geometry of the frame is not regular with respect to vertical

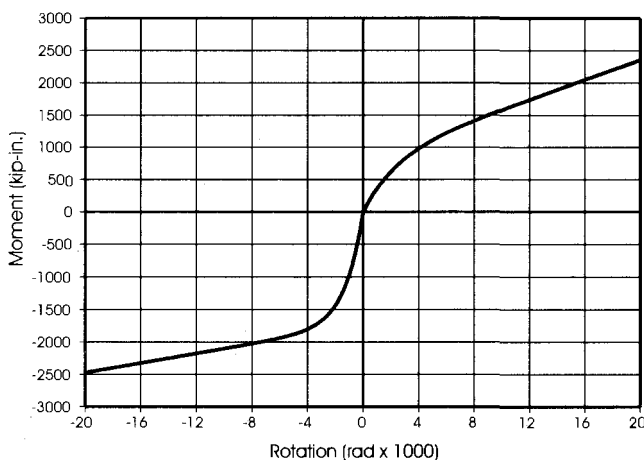


Fig. 3. Complete $M-\theta$ curve for a typical PR-CC.

and horizontal stiffness distribution. The simplifications required to carry out this two-level approach will be discussed in Section 5.

As noted earlier, specifications require that the engineer have a good idea of the strength and stiffness characteristics of these connections before he/she utilizes them in design. For PR-CCs, the work of Leon et al.^{5,26,27} has led to the following expression for the $M-\theta$ curve under negative bending (slab steel in tension):

$$M_n^* = C1(1 - e^{-C2\theta}) + C3\theta$$

where

$$C1 = 0.18(4 \times A_s F_{yb} + 0.857 A_f F_y)(d + Y3)$$

$$C2 = 0.775$$

$$C3 = 0.007(A_f + A_{w1})F_y(d + Y3)$$

θ = girder end rotation, radians

d = girder depth, in.

$Y3$ = distance from the top flange of the girder to the centroid of the reinforcement, in.

A_s = steel reinforcing area, in.²

A_f = area of bottom angle, in.²

A_{w1} = gross area of double web angles for shear calculations, in.²

F_{yb} = yield stress of reinforcing, ksi

F_y = yield stress of seat and web angles, ksi

Since the connection behavior is not symmetrical with respect to either strength or stiffness, a similar expression is needed for positive bending (bottom angle in tension):

$$M_n^* = C1(1 - e^{-C2\theta}) + (C3 + C4)\theta \quad (2)$$

where

$$C1 = 0.2400 \times [(0.48 \times A_{w1} + A_f) \times (d + Y3) \times F_y]$$

$$C2 = 0.02W \times (d + Y3/2)$$

$$C3 = 0.0100 \times (A_{w1} + A_f) \times (d + Y3) \times F_y$$

$$C4 = 0.0065 \times A_{w1} \times (d + Y3) \times F_y$$

These curves were derived from tests and FE parametric studies.^{5-6,26-27} The complete curve given by Equations 1 and 2 for a typical PR-CC is shown in Figure 3. This corresponds to a connection of a W18x35 A36 beam with 8 #4 Grade 60 bars in the slab. The bottom angle area is 2.38 in.² and the area of the web angles is 4.25 in.² The effective depth is 17.7 inches assuming $Y3$ equal to 4 inches.

Fortunately, experience has shown that PR-CCs in unbraced frames seldom unload into positive moment even under the full factored loads. Thus use of Equation 1 is justified for the service load level and up to the factored loads. Equation 1, however, is still cumbersome for use in design. Given the detailing requirements for capacity design described in Section 7, it is more practical to develop design tables for specific connections. Such tables are shown as Tables 1 and 2, which contain all the necessary design infor-

mation for a series of "prequalified connections."* In this guide all the connections designed are "prequalified connections" which have been checked for a large number of failure mechanisms and loading conditions.

Table 1 shows some of the key values to be used in design: the ultimate strength of the connection (ϕM_n) and the stiffness for checking drift ($K-lat$). Table 1 is divided into two parts, showing values for both angles with 36 ksi and 50 ksi nominal yields. In these tables $Y3$ is the distance from the top flange of the beam to the centroid of the slab steel. The derivation of the values in Tables 1 and 2 are discussed in the next section, while the detailing is discussed in Section 7.

5. ANALYSIS

Once the $M-\theta$ characteristics are known the next problem is how to analyze frames containing such connections. In this section the analysis and design assumptions used in the design examples (Part III) will be discussed.

5.1 Service Load Range

There are several ways to evaluate the performance of beams with PR connections under gravity and lateral loads. They range from using modified slope-deflection or moment distribution equations to using elements with non-linear springs in a computer program that incorporates $P-\Delta$ effects directly. The following observations are pertinent:

- The latest versions of the better commercial structural analysis packages (stiffness-based methods) allow designers to specify linear springs at the ends of beam elements. Design procedures should strive to use these elements since the availability of multi-linear or fully non-linear (exponential) spring elements in these software packages is not foreseen in the near future.
- While the behavior of the connections is non-linear, the use of a secant stiffness up to about 2.5 milliradians of rotation does not introduce significant error in the force or displacement calculations. Thus the use of linear spring is justified for design of PR-CCs provided the designer keeps in mind that this approach will probably overestimate the forces at the connections but underestimate the deflections.
- Modified slope-deflection, moment distribution, and similar classical approaches, while of great value for those familiar with their implementation, are tedious and prone to errors.¹⁷
- For those interested in gaining a better insight into connection behavior, a beam-line analysis, described in

detail below, is the preferred method. Note that use of the beam line technique is not advocated for design; it is merely a great educational tool and it is used here in that vein.

In both (a) and (c) above the only unknown is the stiffness to be assigned to the connections. From a simple rigid-plastic analysis where (a) all rotations are lumped at the PR joints and column bases, and (b) a strong column-weak beam mechanism is assumed, it can be shown that the rotation is proportional to the allowable drift. For an allowable drift of $H/400$, the corresponding rotation is 0.0025 radian or 2.5 milliradians. Since the deformations of the beams and columns are not included in this calculation, this value overestimates the rotations of the connections. This simplified analysis does not include any $P-\Delta$ effects which are expected to be negligible at this level even for PR frames. From experience with PR-CCs, it appears that to check service drifts a secant stiffness measured at a rotation of 2 milliradians is sufficiently conservative to avoid too many redesign iterations. The values of the stiffness for drift calculations for the "prequalified connections" are shown in Table 1 as $K-lat$. Note that the secant stiffness used is different from the tangent stiffness that would be obtained by differentiating Equation 1 directly and substituting a value of $\theta = 0.002$ radians.

Following a similar line of reasoning, one could derive conservative values for deflections under gravity loads. Assuming an allowable vertical deflection of $L/360$, a value of $\theta_{ser} = 0.0025$ seems reasonable. Solving Equation 1 for the moment (M) at the service rotation leads to a similar stiffness for gravity loads ($K-grav = M/0.0025$). These moments, M , are tabulated in Table 2, Part IV, for the "prequalified connections". Table 2 is given for different values of $Y3$ and is divided into connections for braced and unbraced frames because the detailing requirements differ as will be described later. The reader is cautioned not to confuse $K-lat$, the connection stiffness for lateral drift, with $K-grav$, the connection stiffness for live load deflections. While the difference in the rotations at which they are calibrated is small, this effect has been integrated directly into the design procedure.

5.2 Beam Line Analysis for Gravity Loading at Service

The connection must be designed to resist the support moments resulting from gravity loads after the slab has cured and the member is acting as a composite beam. The magnitude of negative gravity moment will always be less than that assuming a fully rigid connection and is dependent on the stiffness of the connection. This can be determined by a beam-line analysis. The three key elements for the beam-line analysis are the moment-rotation relationship of the connection, the

* The tables are included at end of this guide (Part IV) and are kept separate from the text to facilitate their use in later designs.

simply supported end rotation of the beam, and the fixed end moment assuming a fully rigid connection of the beam. Note that the beam line as defined herein is only applicable in the elastic range.

The moment-rotation relationship for one of the typical connections in Table 1 (W18×35 with 8 #4 bars, $Y_3 = 5$ in., $F_y = 36$ ksi) is shown as a solid line in Figure 4. To simplify the beam line analysis the moment-rotation relationship will be reduced to a linear spring. The linear spring is represented in Figure 4 by the dashed line. The corresponding stiffness is given by $K_{grav} = M1/\theta1 = 147/0.0025 = 58,800$ kip-ft/radian. The values of $M1$, again, are tabulated in Table 2.

Two values are needed to define the beam line: the fixed end moment, M_F , and simply supported end rotation, θ_{ss} . These values can be determined by conventional beam analysis methods such as slope deflection, virtual work, or moment area, or can be found in reference tables for most loading patterns. These values have been tabulated for the most common loading patterns in Table 3, Part IV. The fixed end moment depends on whether the connection at the other end is PR or pinned. If the far end restraint is PR then the

fixed-fixed end moment (M_{ff}) is used and if it is pinned the fixed-pinned end moment (M_{fp}) is used. With the above key elements established, two lines can be drawn, and the intersection of those lines will provide the actual moment and rotation under gravity loading as shown in Figure 4. This intersection point can be solved directly by an equation which results from the solution of simultaneous equations for the two lines in the beam line analysis. The equation of the connection line is:

$$M = K\theta \quad (3)$$

The equation for the beam line is:

$$M = M_F - \left(\frac{M_F}{\theta_{ss}} \right) \theta \quad (4)$$

The value of θ at the intersection of these lines is given by:

$$\theta = \frac{M_F}{K_{grav} + \frac{M_F}{\theta_{ss}}} \quad (5)$$

The exact solution, the intersection of the solid line and the beam line, can be obtained by setting Equations 1 and 4 equal to one another and solving for θ . This is tedious and generally yields a value very close to that from the linear approximation. Therefore the use of the exact solution is not warranted for preliminary design purposes.

5.3 Connection Ultimate Strength (Gravity Loads)

The ultimate capacity of the connection is based on work by Kulkarni.²⁶ A resistance factor (ϕ) of 0.85 is recommended and is the same value used for composite beam design in Chapter I of the LRFD Specification. $M2$ in Figure 4 and Table 2 is the moment which corresponds to a rotation, $\theta2$, of 20 milliradians. Most of the connections tested have reached and exceeded this value. Considerable connection yielding and deformation is present at this stage. This moment is included in Figure 4 and the design tables for two reasons. First, it illustrates the ductility of the connections. Second, if the user has software which allows a bi-linear spring to be input for connections, $M1$ and $M2$ are useful values which allow a bi-linear curve to approximate the actual curve.

The connection ultimate strength is defined in both the positive and negative directions. The negative bending ultimate strength (M_n^-), when the bottom angle is in compression, is:

$$M_n^- = 0.245(4A_s F_{yrb} + A_w F_y)(d + Y_3) \quad (6)$$

The positive bending ultimate strength (M_n^+) is:

$$M_n^+ = 0.25(1.25A_{wl} + 1.35A_l)(F_y) \left(D + \frac{Y_3}{2} \right) \quad (7)$$

The area of the angle, A_l , is equal to the width of the horizontal

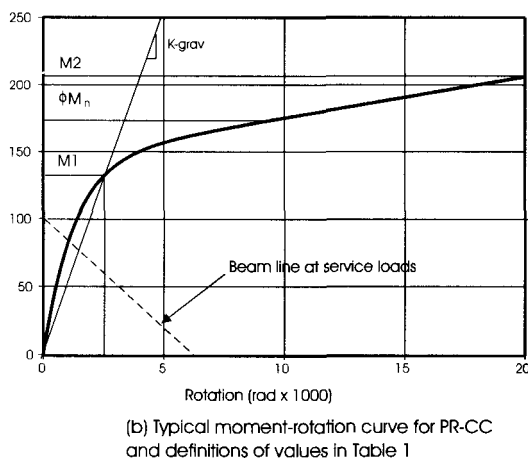
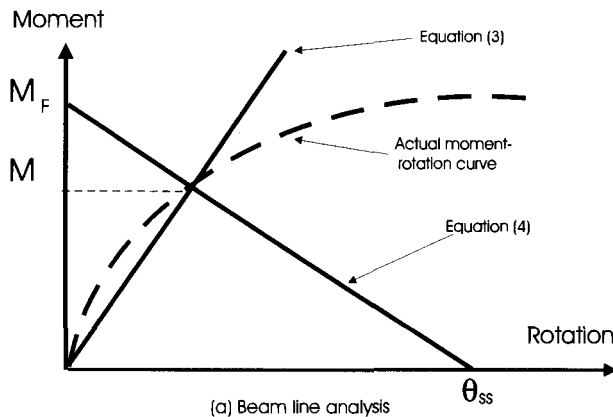


Fig. 4. Beam line analysis.

leg times the thickness of the angle leg. The area A_{wl} is equal to the gross area of the web angles in shear, and A_s is the total area of steel reinforcing provided in the concrete slab over a width not to exceed seven times the steel column width. The values from Equation 6 evaluated at 10 milliradians and including a ϕ factor of 0.85 are tabulated for the different connections in Table 1 as ϕM_n . These values have been arbitrarily selected as the design strength for these connections.

The connection can also be used in braced frames without web angles. This would be a simple modification from the current seated beam design useful in designing connections to the weak axis of a column. The bottom angle required for the seated beam would generally be adequate to supply the bottom part of the force couple and a small amount of reinforcement in the slab would provide the top force. The seat angle would need to be thickened or stiffened as needed to take care of the shear force. The ultimate capacity of this connection is:

$$M_n = A_s F_{yb} (d + Y3) \quad (8)$$

Tables 1 and 2, Part IV, provide key information regarding the moment-rotational relationship, ultimate moment capacity, and connection stiffness for a series of typical connection types using steel reinforcement ranging from an area of 1.2 in.² to 3.1 in.² and beam depths from 12 to 24 inches. The connections selected meet the criteria of explained above plus the detailing requirements discussed in the next section. The force given in the tables is for the design of bolts or welds between the beam bottom flange and angle.

5.4 Frame and Beam Ultimate Strength

Ultimate strength checks will be made for both individual beams and the frame as a whole using plastic analysis.²⁸⁻³⁰ The applicable load combinations for ultimate beam capacity from ASCE 7-93 are:

$$1.4D \quad (9)$$

$$1.2D + 1.6L + 0.5(L_r, S, R) \quad (10)$$

$$1.2D + 1.6(L_r, S, R) + (0.5L, 0.8W) \quad (11)$$

For frame ultimate capacity they are:

$$1.2D + 1.3W + 0.5L + 0.5(L_r, S, R) \quad (12)$$

$$1.2D + 1.0E + 0.5L + 0.2S \quad (13)$$

$$0.9D \pm (1.3W, 1.0E) \quad (14)$$

$$E = \pm 1.0Q_E \pm 0.5A_1 D \quad (15)$$

5.4.1 Beam Ultimate Capacity

The load combination used to calculate the beam load factor is the most critical of combinations given by Equations 9-11.

Commonly the most critical load combination is given by Equation 10. The load factors for beam mechanisms of four different common load cases and for three different connection relationship are shown in Table 4. The general form for these load factors is:

$$\lambda_b = \frac{d}{(P_u \text{ or } w_u L)L} [(aM_{p,c1}) + (bM_{p,c2}) + (cM_{p,b})] \quad (16)$$

where

- λ_b = is the load factor,
- $a, b, c,$ and d = the coefficients given in Table 4, Part IV,
- $M_{p,c1}$ and $M_{p,c2}$ = are the negative bending ultimate design capacities (ϕM_n) of connections 1 and 2, and
- $M_{p,b}$ = is the ultimate moment capacity of the composite beam in positive bending.

5.4.2 Frame Ultimate Capacity

An approximate second order rigid plastic analysis is carried out to determine the overall adequacy of the frame. The controlling combination is generally given by Equations 12 or 13. The collapse mechanism governing this type of design is a weak girder-strong column one (Figure 5).

In plastic analysis two possibilities, proportional and non-proportional loading, arise. Proportional loading, in which both the lateral and gravity loads are increased simultaneously, is commonly used. This design procedure, however, is calibrated to non-proportional loading. In this case the gravity loads are held constant and the lateral loads are increased. Thus, if Equation 12 or 13 is used, the gravity loads (D, L, L_r , and/or S) are kept constant while the lateral loads (W or E) are increased from zero to failure. The multiplier on the lateral loads at failure is the ultimate load factor for the frame, λ_f .

To obtain λ_f the second order effects must be considered.

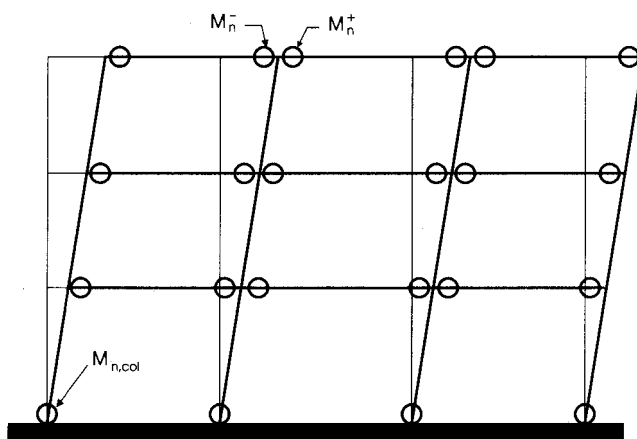


Fig. 5. Plastic collapse mechanism.

Here an approximate method, called the mechanism curve method²⁸ is used. Before calculating λ_p the first order load factor must be calculated. The first order rigid plastic load factor, λ_p , is calculated as:

$$\lambda_p = \frac{\Sigma \phi M_n}{\Sigma V_i h_i} \quad (17)$$

where ϕM_n is the moment capacity of a hinge or connection, V_i is the factored lateral force at story i , and h_i is the height from the base to story i . In this equation the numerator represents the internal resisting forces provided by all hinging regions, while the denominator represents the external loads. Thus any value of λ_p greater than one represents a safe condition. The summation of the connection design strengths are over all the connections, while the summation of $V_i h_i$ is from 1 to S , the number of stories.

The calculation of the internal resisting moments requires computing the resistance provided by all elements hinging: the column bases, the external and the internal moments. Symbolically:

$$\Sigma \phi M_n = \Sigma \phi M_{n,col}(R) + \quad (18)$$

$$\Sigma \{ (N-1)(\phi M_n^+ + \phi M_n^-)_{Inte} \} + \Sigma \{ (\phi M_n^+ + \phi M_n^-)_{Exte} \}$$

where

$\Sigma \phi M_{n,col}(R)$ = the summation of the reduced design plastic capacity of the columns at the base of the structure,

N = the number of bays, and

"Inte" and "Exte" refer to the interior and exterior frame connections.

In this equation the summation of positive (ϕM_n^+) and negative moment capacities (ϕM_n^-) assumes that the connections on either side of each joint have reached their ultimate design capacity. If the exterior connections are simple, then the last term above is zero. To account for the presence of axial load on the plastic capacity of the base columns the following approach is used. If $P_u < 0.15P_y$ then $\phi M_{n,col}(R) = \phi M_{n,col}$, or else:

$$\phi M_{n,col}(R) = 1.18 \left(1 - \frac{P_u}{P_y} \right) \phi M_{n,col} \quad (18a)$$

where

P_u = the factored load on the column for the lateral load combination, and

P_y = is the axial yield capacity of the column. Now the approximate ultimate load factor including second order effects may be calculated by:

$$\lambda_p = \lambda_p - S_p \lambda_p^2 \left(\frac{\Sigma P \delta}{\Sigma M_n} \right) \quad (19)$$

where

- P = is the story axial load for the frame under analysis,
- δ = is the interstory drift at 1.0E (or 1.0W),
- ΣM_n = is the *nominal* summation of design moment values, and
- S_p = is the sway parameter calibrated for these frames (see table below and Table 5).

Values of S_p for Different Frame Geometries			
No. of Stories	Story Height (ft)		
	12	14	16
4	4.85	4.40	3.10
6	3.70	2.95	2.55
8	2.45	1.95	1.35

The S_p values above may be interpolated. Note that these values have been calibrated to frames designed with PR-CCs by the present procedure. These values are currently under further evaluation and should not be used with any other frame and connection types.

6. DESIGN CONSIDERATIONS

This section explains a number of the design choices made by the authors in selecting, checking and detailing the connections. The topics are separate and are arranged in the order they appear in the design procedure.

6.1 Deflections for Beams with PR Connections

The effect of having semi-rigid connections must be included in service deflection checks. The following equation gives the deflection (δ_{SR}) of a symmetrically loaded beam with equal or unequal connection stiffnesses

$$\delta_{sr} = \delta_{FF} + \frac{C_\theta \theta_{sym} L}{4} \quad (20)$$

where

δ_{SR} = is the deflection of the beam with semi-rigid connections,

δ_{FF} = is the deflection of the beam with fixed-fixed connections,

C_θ = is a deflection coefficient, and

θ_{sym} = is the service load rotation corresponding to a beam with both connections equal to the stiffest connection present.

When the beam has equal connection stiffnesses C_θ equals one. When the connection stiffnesses are different C_θ may be found in Table 6. The values in Table 6 depend on the ratio of

the less stiff to more stiff connection and on the ratio of the semi-rigid to the fixed-fixed end moment for the stiffer connection. If K_a is the stiffness of the stiffer connection, the ratio of semi-rigid to fixed-fixed end moment can be expressed as:

$$\frac{M_{SR}}{M_{FF}} = \frac{1}{1 + \alpha}, \alpha = \frac{2EI}{K_a L} \quad (21)$$

where

M_{FF} and M_{SR} = the fixed-fixed and semi-rigid end moments, respectively.

For design purposes it is beneficial to assume a service rotation for preliminary deflection requirements and then check that deflection after connections have been chosen by either beam line analysis or from:

$$\theta = \frac{M_{FF}}{K_a + \frac{2EI}{L}} \quad (22)$$

Using a 2.5 milliradian service rotation, the connection will add an additional $L/1600$ to the deflection when the connection stiffnesses are equal. If $L/360$ is the service limit, this approach now requires that the service load deflection based on a fixed-fixed beam approach be kept below $L/465$.

When the beam has one semi-rigid connection and one pinned connection the following equation provides a conservative deflection for any connection stiffness:

$$\delta_{SR} = \delta_{FP} + \frac{\theta L}{5.4} \quad (23)$$

where

δ_{FP} = the beam deflection with one end fixed and the other end pinned and

θ = the actual rotation of the semi-rigid connection.

The rotation θ may be found by a beam line analysis using the fixed-pinned end moment, M_{FP} .

6.2 Lateral Drift

When used in unbraced frames, the flexibility of the connections will cause the lateral deflections of the frame to increase over that which would occur if the connection was fully rigid. To illustrate this effect, the contributions of the columns (Δ_c), beams (Δ_b), and connections (Δ_{conn}) to the total drift (Δ_{total}) can be separated as illustrated in Figure 6.

For preliminary design, the engineer can either estimate the size of the columns based on experience or use a trial-and-error approach combined with a computer program. A hand method to estimate the column sizes, based on the approach given in Figure 6, is included in Appendix A.

In general the design of frames with PR-CCs does not require that the column sizes be increased significantly over those used for an equivalent rigid frame. This is because the

design of frames with PR-CCs takes advantage of the additional stiffness in the beams provided by the composite action (see next section). Thus the additional flexibility due to the PR connections is balanced by a larger beam stiffness and the column sizes need to be increased generally by only one or two sections.

The flexibility of the column base plate connections should be incorporated into these calculations. Drifts in the first floor will probably control the design of many low-rise PR frames. As for unbraced FR frames, the assumption of full fixity at the base should not be made unless careful analysis and detailing of the column base plate justify it.

6.3 Beam Stiffness

In modelling PR-CC frame behavior, the effective moment of inertia of the beams (I_{eq}) should take into account the non-prismatic nature of the beam, i.e. the variation in moment of inertia for a composite beam with SRCC between areas of positive and negative bending. The moment of inertia in positive areas (I_B) can be determined in the traditional way for composite beams and it is recommended that the lower

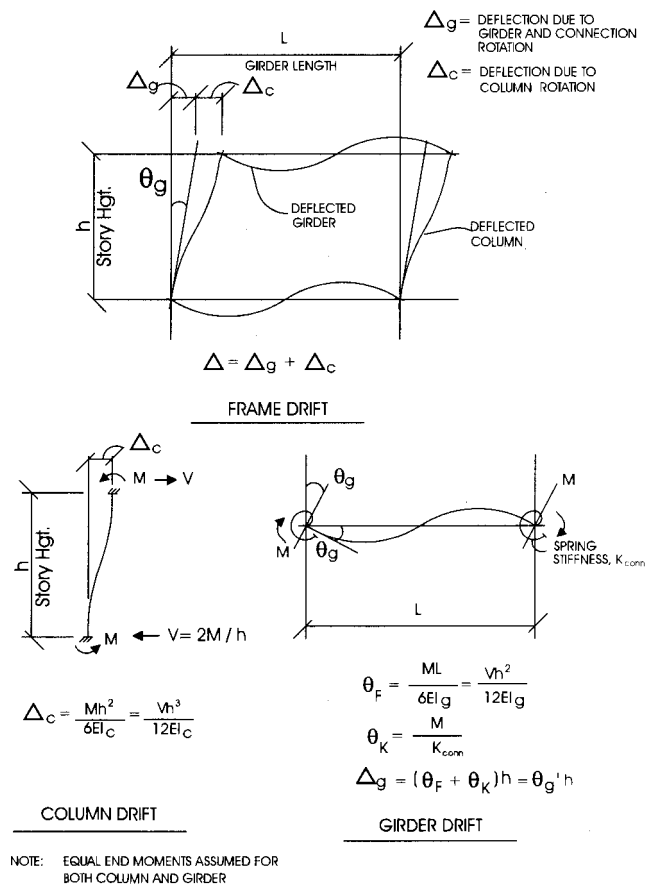


Fig. 6. Components of PR frame drift.

bound tables in the LRFD Manual be used for its determination. The moment of inertia in the negative areas (I_n) is a function of the steel beam and the reinforcing in the slab. This can be determined using the parallel axis theory. Table 7 provides values for several combinations of reinforcing and beam sizes for a Y3 (distance from the top flange to centroid of the reinforcing) equal to 3, 4, 5, and 6 inches.

If the positive moment of inertia is denoted as I_{LB} and the negative moment of inertia is denoted as I_n , then I_{eq} is the "prorated" average of the two. For beams with SRCC connections at both ends it is recommended that the following value be used:

$$I_{eq} = 0.6I_{LB} + 0.4I_n \quad (24)$$

When one end has a SRCC and on end pinned:

$$I_{eq} = 0.75I_{LB} + 0.25I_n \quad (25)$$

6.4 PR Connection Effect on Column End Restraint

PR connections reduce the amount of end restraint provided by the beams to the columns when compared to FR connections. This must be considered when carrying out stability checks. The effective moment of inertia of a beam including the effect of the PR connections to be used in calculating G factors is:^{25,31}

$$I_{eff} = I_{eq} \left(\frac{1}{1 + C\alpha} \right) \quad (26)$$

$$\alpha = \frac{2EI_{eq}}{LK_{tan}} \quad (27)$$

where

L and I_{eq} = are the beam length and equivalent moment of inertia,

K_{tan} = is the connection tangent stiffness, and $C = 1$ for braced frames and $C = 3$ for unbraced ones.

The main problem in utilizing this formula is that K_{tan} at the factored load where stability is being checked must be known for each connection. Several simplifications to this approach have been proposed:

- (a) For a frame subjected to lateral loads the connections on one side of the column will continue to rotate in the same direction as the rotations imposed by the gravity loads, while the connection on the other side will rotate in the opposite direction.^{25,31} For the connection that continues to load, the stiffness of the connection will decrease and in the limit (i.e. at very large rotations) this stiffness will be zero. The connections on the other side of the column will unload along a path with a stiffness close to the service level stiffness. In calculating G one can then assume that for one side of the connection the effective beam stiffness (I_{eff}) in Equation 26 can be calculated by setting $K_{tan} = 0$ while for the

other side $K_{tan} = K_{tar}$. This results in only one side of the connection, the unloading side, contributing to G. This procedure is overconservative.

- (b) A similar reasoning for braced frames implies that both connections are loading and that therefore their restraint to the column is negligible. For this case $K=1$.
- (c) For unbraced frames, a better, less conservative estimate can be made by assuming that the loading connection has not reached its ultimate capacity. In this case the stiffness of the loading side can be approximated as the slope of a line connecting the service ($M1, \theta1$) and ultimate ($M2, \theta2$) points. The stiffness for the unloading side should still be taken as K_{ser} .
- (d) Recently it has been suggested that the use of a secant stiffness to the ultimate point ($M2, \theta2$) should also provide a reasonable lower bound to the frame stability. In this case both connections are assumed to have the same stiffness.
- (e) If an advanced analysis is carried out, then the K-factors can be calculated in the usual manner by using an equivalent stiffness as given by:

$$I_{eff} = I_{eq} \left(\frac{1}{2 \left(1 + \frac{3}{\alpha/2} \right) - \frac{M_f}{M_n}} \right) \quad (28)$$

where

α is calculated from Equation 27 using the tangent stiffness, and

M_f and M_n are the changes in moment during the last step in the loading at the far and near end of the element, respectively.

For the design example, the stability was checked following the procedure described in (a). A more thorough treatment of this topic, including an example utilizing the same frame as in this design guide, can be found in.³¹ In Chapter 3 of this reference, in addition, there is extensive treatment of the extension of the story-based stability procedures to PR frames.

6.5 Bottom Angle Connection

For unbraced frames the bottom angle thickness should be increased so that approximately the same stiffness is provided in the positive direction as the negative direction. To accomplish this the yield force in the bottom angle, $A_t \times F_y$, should be at least 1.2 times the force in the reinforcement, $A_s \times F_{yrb}$, assuming the angle width remains constant. For braced frames the bottom angle is sized for a force equal to $A_s F_{yrb}$.

As shown in Figure 1, the bottom angle is usually connected to the bottom flange of the beam by ASTM A325 or A490 bolts. A 6-in. long angle leg can normally accept 4 bolts (2 rows of 2), but in some cases a 7- or 8-in. leg may be necessary. Bolt bearing and shear must be checked at ultimate

loading assuming some bolt slippage occurs. For service loading, however, it is important that the bolts not slip to ensure that the spring stiffness response is maintained. For this reason, an additional check should be made for service gravity and wind loading against the slip-critical shear values for the bolts, and the bolts should always be fully tensioned. Welding the angle to the bottom flange can also be considered for large forces; in this case the serviceability check need not be performed. Welding of the angle to the column is discouraged since the ductility of the system depends on the ability of the angle to deform plastically as a two member frame.

For each set of reinforcement a set of bottom angles and bolts have been chosen that have passed all the required connection checks by LRFD. These angles and bolts are

shown in Table 8. The force in the bottom angle that was designed for was based on the ultimate capacity design approach. Two of the same type of bolts as for the horizontal leg were used in the vertical leg of the angle for connections to resist tension in unbraced frames. Prying action of the angle was considered. If any other angle and bolt set is used all connection checks must be carried out.

7. DETAILING

For SRCCs, the authors and their co-workers have developed the following recommendations (Figures 7 and 8):

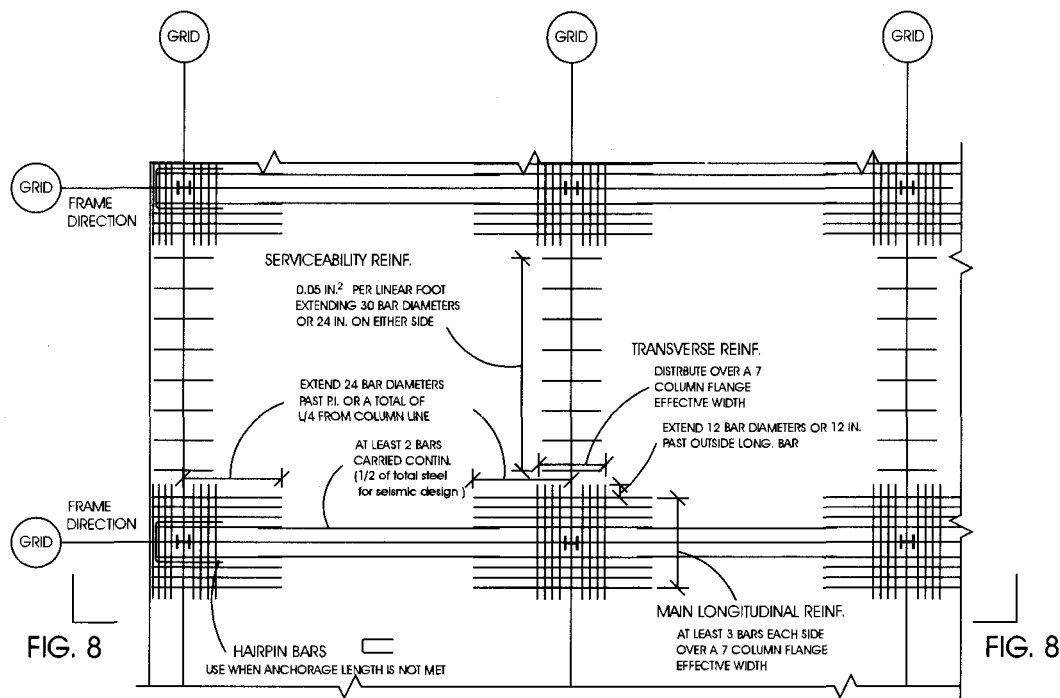


Fig. 7. Detailing requirements (plan view).

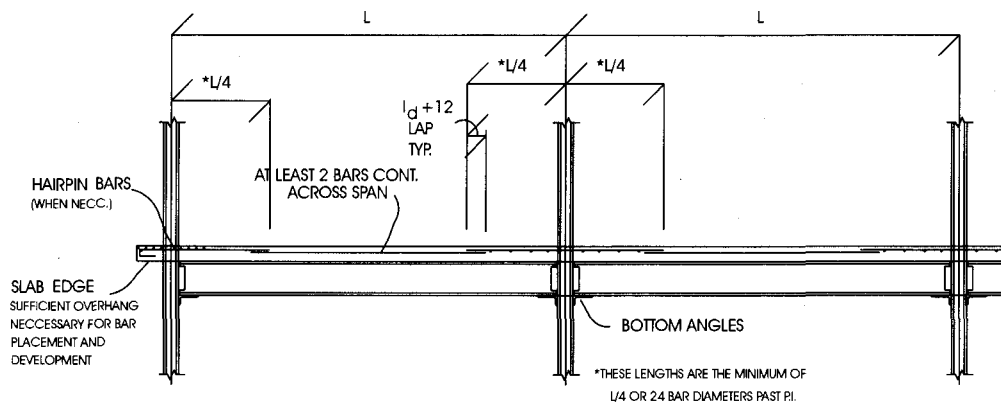


Fig. 8. Detailing requirements (elevation).

- (1) For designs where seismic forces control and a weak beam-strong column mechanism is desirable:

$$\Sigma M_{p, \text{columns}} / \Sigma M_{p, \text{conn}} > 1.2 \quad (29)$$

In this equation the moment capacities of the columns should account for the decrease due to axial loads (Equation 18), while the moment capacity of the connections should be increased by 1.25 to account for the overstrength of the slab steel. The usual ϕ factors should be included in this calculation, and thus the ratio of nominal capacities should be greater than 1.6.

- (2) The longitudinal slab steel should be kept within a column strip less than or equal to seven column flange widths. Tests have shown that the steel must be close to the column to be activated at low drifts. Since the intent is to obtain a connection that is stiff at service loads, the placement of the slab steel is a key detailing issue.
- (3) The slab steel should extend at least l_d plus 12 inches past the point of inflection or $L/4$, whichever is longest. At least two bars should be run continuously for unbraced frames governed by wind. At least two bars for the case where wind governs or one half of the steel for the case where seismic governs, should be run continuously for unbraced frames since the point of inflection can change drastically under seismic loading.
- (4) The bar size should be kept small (between #4 and #6), and at least three bars on either side of the column should be used.
- (5) Transverse steel must be provided at each column line, and must extend at least 12 inches into the slab strip. To reduce serviceability problems a minimum of 0.05 in.² of steel per lineal foot must be provided over the girders, with this reinforcement extending at least 24 inches or 30 bar diameters, whichever is greater, on either side of the girder. Reinforcing transverse to the direction of the moment connection serves a structural purpose and deserves attention. Moments imposed by lateral loads cause a transfer of forces from the reinforcing to the column by means of shear in the slab and bearing at the columns. The transverse reinforcing, therefore, acts as concrete shear reinforcing for this mechanism and it is recommended that the area of the transverse reinforcing be made approximately equal to the main reinforcing.
- (6) The development of the equations for $M-\theta$ curves for PR-CCs assumed that friction bolts (i.e., slip-critical) are used in the seat angle. The intent is not to prevent slip at service loads, but to minimize it.
- (7) Full shear connection in the form of headed shear studs should be provided. Partial shear connection can be used for non-seismic cases, but the designer is cautioned that there is no experimental evidence to justify any design guidelines in this area.

- (8) Other failure modes such as local buckling of the beam flange or web in negative moment regions, yielding of the column panel zone, bolt bearing stresses, and spacing requirements should be checked as per current specifications.

Because the reinforcing in the slab is an integral part of the connection, the quantity, spacing, and location of the reinforcing should be monitored very closely during construction.

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Part II

DESIGN PROCEDURES

1. INTRODUCTION

Two practical design procedures for designing PR-CCs are presented in this section. The first procedure is for PR-CC use in braced frames. In this case the connections provide continuity for composite beams or girders carrying gravity loads. The beam size or the amount of composite action required may be reduced because of the use of PR-CCs. Partial composite action is permitted in these members since they are not part of the lateral load resisting system. The second procedure presented is for PR-CC use in unbraced frames. This design is centered around providing enough connection stiffness to meet interstory drift criteria, as the frame's stiffness and not strength typically controls the design. For the main girders in the lateral load resisting system only use of full interaction is permitted.

Both procedures are based on a two-level approach; elastic analysis for service loads and plastic analysis for ultimate strength. This approach was chosen because of the nature of the moment-rotation relationship of PR-CCs. Under service loads the connections are approximated as linear elastic springs. At ultimate loads, plastic analysis is used because of its simplicity. Consequently, painstaking techniques to determine exactly where the connection is on the nonlinear moment-rotation are not necessary for ultimate strength checks. Beams are analyzed by plastic analysis as described in Part I. For unbraced frames, the capacity of the frame under nonproportional loading is determined by second-order plastic analysis as outlined in Part I.

The procedures are given in step-by-step outline form. For completeness all of the important steps are given. The design of a frame with PR-CC's only entails a departure from conventional design in the selection of the amount of end restraint and moment desired (Step 2 in the design of braced frames and Step 5 in the design for unbraced frames.) Both procedures are geared towards design using the AISC LRFD Manual and many references will be made to design provisions found in this manual. In addition, the Tables found in Part IV of this document will be referenced.

A few notes on the notation that is used throughout the procedures must be made. The dead load on the members is divided into the portion that is applied before composite action, DL_b , which includes weight of the slab, steel framing and decking, and the dead load after composite action, DL_a , which includes superimposed dead loads such as ceilings, mechanical systems, and partitions. The factored simply supported moment is denoted as M_u . The amount of composite

action in the beams is designated by the plastic neutral axis (PNA), as defined by AISC LRFD. Thus a PNA equal to the top of the top flange (TFL) is considered full composite action, and a PNA equal to position 7, as defined by AISC LRFD, is considered to be the minimum composite action (25 percent composite by LRFD).

2. DESIGN PROCEDURE FOR BRACED FRAMES

2.1 Introduction

Partially restrained composite connections may be utilized in braced frames for beams framing into columns to reduce the beam size or amount of composite action required. In addition many of the filler beams can also be designed following this procedure. In many instances beams usually considered simply supported may be designed with PR-CCs with very few modifications in order to improve their deflection and vibration characteristics. The following paragraphs include a brief overview of this design procedure which is given in a step-by-step form in Section 2.2.

In the first step the minimum beam size is determined based on construction loading conditions, assuming unshored construction. In the second step the capacity of the bare beam chosen for construction conditions is compared with the requirements of ultimate strength and service deflections for a composite section based on the same beam. It is the aim of this procedure to utilize the beneficial effects of PR-CCs so that the "construction beam" may be adequate for ultimate strength and serviceability. Therefore, the second step is used to determine if (a) it is possible to use PR-CCs with the "construction beam", (b) the beam needs to be increased in size, or (c) the superimposed loads are so small that the "construction beam" is adequate at low composite action and semi-rigid connections are not required.

After the need for PR-CCs has been determined, the magnitude of end restraint required for strength and stiffness is determined in Step 3, and the connection is chosen. In Step 4 the connection details are established, including the seat angle, web angle, and connection reinforcement.

The ultimate strength of the connections is checked in Step 5 by plastic analysis. Finally, the connections are checked for compatibility at service loads. This is done to verify that the connections' rotations are less than that assumed for deflection checks.