8

COMPOSITE STEEL AND CONCRETE

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This chapter illustrates application of the 2000 *NEHRP Recommended Provisions* to the design of composite steel and concrete framed buildings using partially restrained composite connections. This system is referred to as a "Composite Partially Restrained Moment Frame (C-PRMF)" in the *Provisions*. An example of a multistory medical office building in Denver, Colorado, is presented. The *Provisions* set forth a wealth of opportunities for designing composite steel and concrete systems, but this is the only one illustrated in this set of design examples.

The design of partially restrained composite (PRC) connections and their effect on the analysis of frame stiffness are the aspects that differ most significantly from a non-composite design. Some types of PRC connections have been studied in laboratory tests and a design method has been developed for one in particular, which is illustrated in this example. In addition, a method is presented by which a designer using readily available frame analysis programs can account for the effect of the connection stiffness on the overall frame.

The example covers only design for seismic forces in combination with gravity, although a check on drift from wind load is included.

The structure is analyzed using three-dimensional static methods. The RISA 3D analysis program, v.4.5 (Risa Technologies, Foothill Ranch, California) is used in the example.

Although this volume of design examples is based on the 2000 *Provisions*, it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformat of all of the chapters of the 2003 *Provisions*) and substantive technical changes to the 2003 *Provisions* and its primary reference documents. While the general concepts of the changes are described, the design examples and calculations have not been revised to reflect the changes to the 2003 *Provisions*.

Chapter 10 in the 2003 *Provisions* has been expanded to include modifications to the basic reference document, AISC Seismic, Part II. These modifications are generally related to maintaining compatibility between the *Provisions* and the most recent editions of the ACI and AISC reference documents and to incorporate additional updated requirements. Updates to the reference documents, in particular AISC Seismic, have some affect on the calculations illustrated herein.

There are not any general technical changes to other chapters of the 2003 *Provisions* that have a significant effect on the calculations and/or design example in this chapter of the *Guide* with the possible exception of the updated seismic hazard maps.

Where they affect the design examples in this chapter, significant changes to the 2003 *Provisions* and primary reference documents are noted. However, some minor changes to the 2003 *Provisions* and the reference documents may not be noted.

In addition to the 2000 *NEHRP Recommended Provisions* (referred to herein as the *Provisions*), the following documents are referenced:

ACI 318 — American Concrete Institute. 1999. *Building Code Requirements for Structural Concrete*, Standard ACI 318-99. Detroit: ACI.

AISC LRFD — American Institute of Steel Construction. 1999. *Load and Resistance Factor Design Specification for Structural Steel Buildings*. Chicago: AISC.

AISC Manual — American Institute of Steel Construction. 1998. *Manual of Steel Construction, Load and Resistance Factor Design*, Volumes 1 and 2, 2nd Edition. Chicago: AISC.

AISC Seismic — American Institute of Steel Construction. 1997. *Seismic Provisions for Structural Steel Buildings*, including Supplement No. 2 (2000). Chicago:

AISC SDGS-8 — American Institute of Steel Construction. 1996. *Partially Restrained Composite Connections*, Steel Design Guide Series 8. Chicago: AISC.

ASCE TC — American Society of Civil Engineers Task Committee on Design Criteria for Composite Structures in Steel and Concrete. October 1998. "Design Guide for Partially Restrained Composite Connections," *Journal of Structural Engineering* 124(10)..

ASCE 7 — American Society of Civil Engineers. 1998. *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-98. Reston: ASCE.

The short-form designations presented above for each citation are used throughout.

The symbols used in this chapter are from Chapter 2 of the *Provisions*, the above referenced documents, or are as defined in the text. Customary U.S. units are used.

8.1 BUILDING DESCRIPTION

This four-story medical office building has a structural steel framework (see Figures 8-1 through 8-3). The floors and roof are supported by open web steel joists. The floor slab is composite with the floor girders and the spandrel beams and the composite action at the columns is used to create moment resisting connections. Figure 8-4 shows the typical connection. This connection has been studied in several research projects over the past 15 years and is the key to the building's performance under lateral loads. The structure is free of irregularities both in plan and elevation. This is considered a Composite Partially Restrained Moment Frame (C-PRMF) per *Provisions* Table 5.2.2 and in AISC Seismic, and it is an appropriate choice for buildings with low-to-moderate seismic demands, which depend on the building as well as the ground shaking hazard.

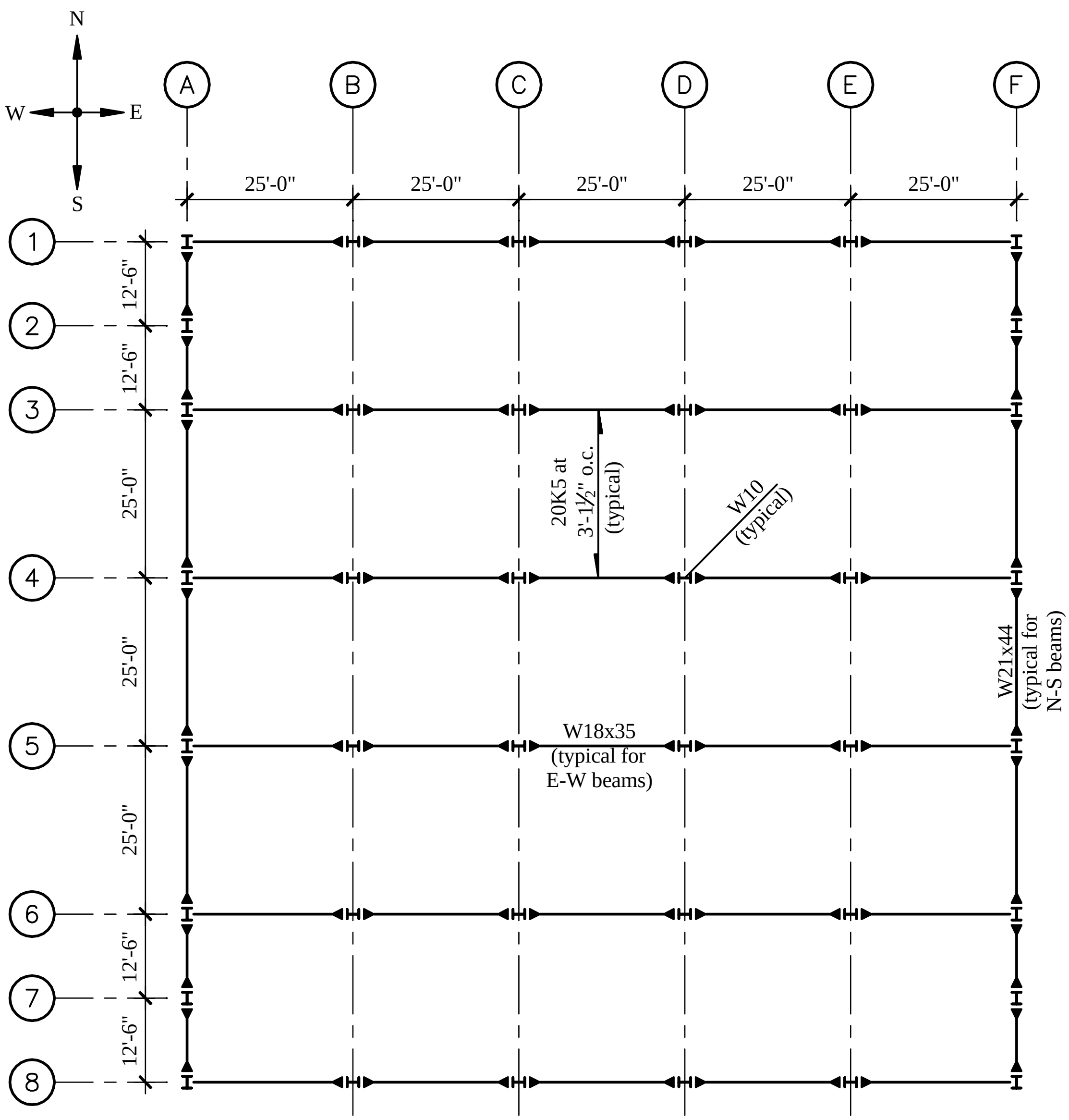

Figure 8-1 Typical floor plan (1.0 ft = 0.3048 m).

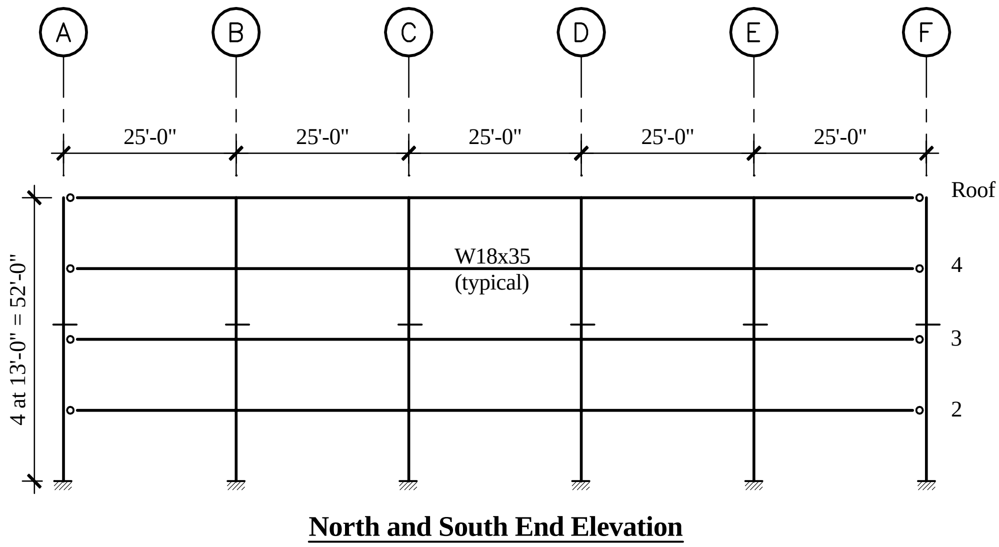

Figure 8-2 Building end elevation (1.0 ft = 0.3048 m).

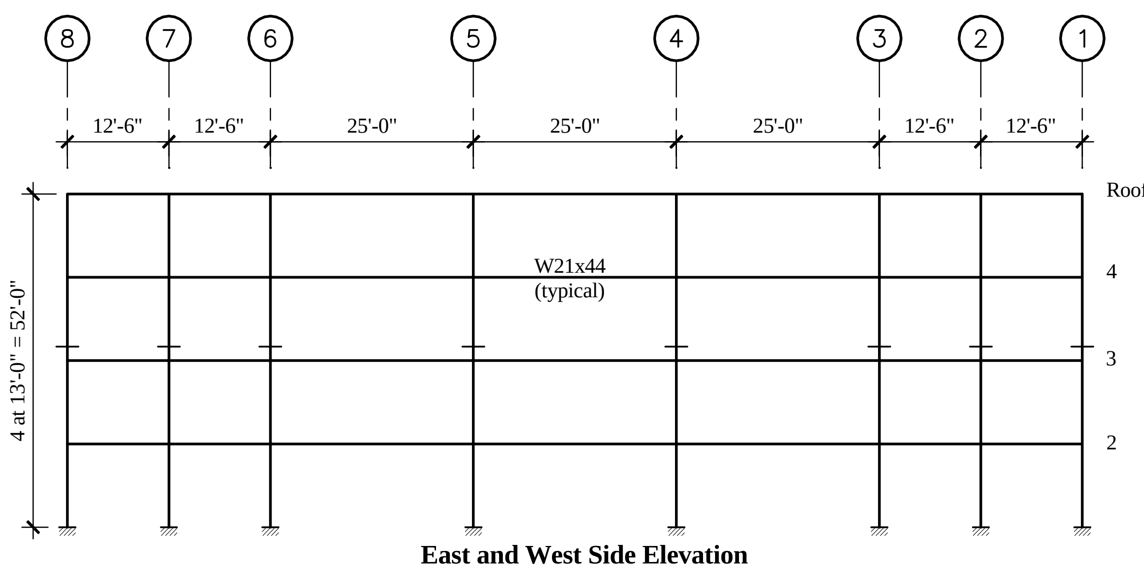

Figure 8-3 Building side elevation (1.0 ft = 0.3048 m).

The building is located in a relatively low hazard region (Denver, Colorado), but some internal storage loading and Site Class E are used in this example to provide somewhat higher seismic design forces for purposes of illustration, and to push the example into Seismic Design Category C.

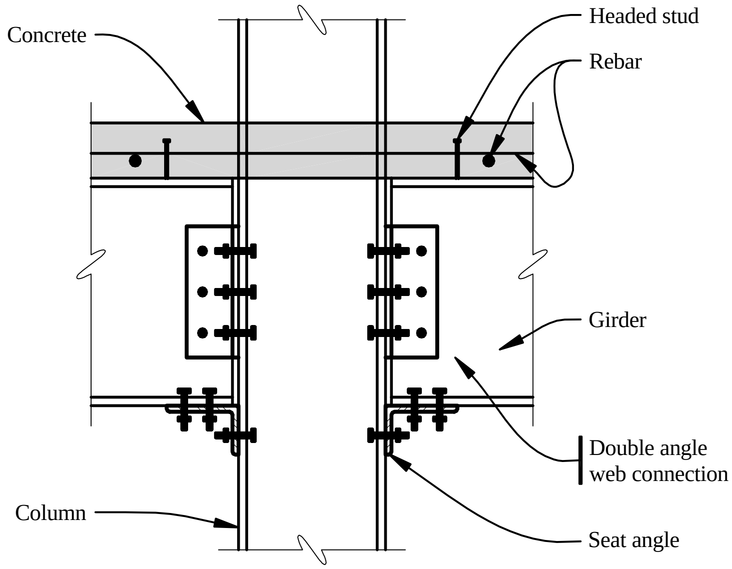

Figure 8-4 Typical composite connection.

There are no foundations designed in this example. For this location and system, the typical foundation would be a drilled pier and voided grade beam system, which would provide flexural restraint for the strong axis of the columns at their base (very similar to the foundation for a conventional steel moment frame). The main purpose here is to illustrate the procedures for the partially restrained composite connections. The floor slabs serve as horizontal diaphragms distributing the seismic forces, and by inspection they are stiff enough to be considered as rigid.

The typical bay spacing is 25 feet. Architectural considerations allowed an extra column at the end bay of each side in the north-south direction, which is useful in what is the naturally weaker direction. The exterior frames in the north-south direction have moment-resisting connections at all columns. The frames in each bay in the east-west direction have moment-resisting connections at all except the end columns. Composite connections to the weak axis of the column are feasible, but they are not required for this design. This arrangement is illustrated in the figures.

Material properties in this example are as follows:

1. Structural steel beams and columns (ASTM A992): F_y = 50 ksi
2. Structural steel connection angles and plates (ASTM A36): F_y = 36 ksi
3. Concrete slab (4.5 inches thick on form deck, normal weight): f_c' = 3000 psi
4. Steel reinforcing bars (ASTM A615): F_y = 60 ksi

The floor live load is 50 psf, except in 3 internal bays on each floor where medical records storage imposes 200 psf, and the roof snow load is taken as 30 psf. Wind loads per ASCE 7 are also checked, and the stiffness for serviceability in wind is a factor in the design. Dead loads are relatively high for a steel building due to the 4.5" normal weight concrete slab used to control footfall vibration response of the open web joist system and the precast concrete panels on the exterior walls.

This example covers the following aspects of seismic design that are influenced by partially restrained composite frame systems:

1. Load combinations for composite design
2. Assessing the flexibility of the connections
3. Incorporating the connection flexibility into the analytical model of the building

4. Design of the connections

8.2 SUMMARY OF DESIGN PROCEDURE FOR COMPOSITE PARTIALLY RESTRAINED MOMENT FRAME SYSTEM

For buildings with low to moderate seismic demands, the partially restrained composite frame system affords an opportunity to create a seismic-force-resisting system in which many of the members are the same size as would already be provided for gravity loads. A reasonable preliminary design procedure to develop member sizes for a first analysis is as follows:

1. Proportion composite beams with heavy noncomposite loads based upon the demand for the unshored construction load condition. For this example, this resulted in W18x35 beams to support the open web steel joists.

2. Proportion other composite beams, such as the spandrel beams in this example, based upon judgment. For this example, the first trial was made using the same W18x35 beam.

3. Select a connection such that the negative moment strength is about 75 percent of the plastic moment capacity of the bare steel beam.

4. Proportion columns based upon a simple portal analogy for either stiffness or strength. If stiffness is selected, keep the column's contribution to story drift to no more than one-third of the target. If strength is selected, an approximate effective column length factor of $K = 1.5$ is suggested for preliminary design. Also check that the moment capacity of the column (after adjusting for axial loads) is at least as large as that for the beam.

Those final design checks that are peculiar to the system are explained in detail as the example is described. The key difference is that the flexibility of the connection must be taken into account in the analysis. There are multiple ways to accomplish this. Some analytical software allows the explicit inclusion of linear, or even nonlinear, springs at each end of the beams. Even for software that does not, a dummy member can be inserted at each end of each beam that mimics the connection behavior. For this example another method is illustrated, which is consistent with the overall requirements of the *Provisions* for linear analysis. The member properties of the composite beam are altered to become an equivalent prismatic beam that gives approximately the same flexural stiffness in the sway mode to the entire frame as the actual composite beams combined with the actual connections. Prudence in the use of this simplification does suggest checking the behavior of the connections under gravity loads to assure that significant yielding is confined to the seismic event.

Once an analytic model is constructed, the member and connection properties are adjusted to satisfy the overall drift limits and the individual strength limits. This is much like seismic design for any other frame system. Column stability does need to account for the flexibility of the connection, but the AISC LRFD and the *Provisions* approaches considering second order moments from the translation of gravity loads are essentially the same. The further checks on details, such as the strong column rule, are also generally familiar. Given the nature of the connection, it is also a good idea to examine behavior at service loads, but there are not truly standard criteria for this.

8.3 DESIGN REQUIREMENTS

8.3.1 *Provisions* Parameters

The basic parameters affecting the design and detailing of the buildings are shown in Table 8.1 below.

Table 8-1 Design Parameters

Parameter	Value
S_s (Map 1)	0.20
S_1 (Map 2)	0.06
Site Class	E
F_a	2.5
F_v	3.5
$S_{MS} = F_aS_s$	0.50
$S_{M1} = F_vS_1$	0.21
S_{DS} = 2/3S_{MS}	0.33
S_{D1} = 2/3S_{M1}	0.14
Seismic Design Category	C
Frame Type per *Provisions* Table 5.2.2	Composite Partially Restrained Moment Frame
R	6
Ω_0	3
C_d	5.5

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

The frames are designed in accordance with AISC Seismic, Part II, Sec. 8 (*Provisions* Table 5.2.2). AISC SDGS-8 and ASCE TC describe this particular system in detail. Given the need to determine the flexibility of the connections, it would be difficult to design such structures without reference to at least one of these two documents.

8.3.2 Structural Design Considerations Per the *Provisions*

The building is regular both in plan and elevation. *Provisions* Table 5.2.5.1 indicates that use of the Equivalent Lateral Force procedure in accordance with *Provisions* Sec. 5.4 is permitted.

Nonstructural elements (*Provisions* Chapter 14) are not considered in this example.

Diaphragms must be designed for the required forces (*Provisions* Sec. 5.2.6.2.6), however this is not unique to this system and therefore is not explained in this example.

The story drift limit (*Provisions* Table 5.2.8) is 0.025 times the story height. Although the C_d factor is large, 5.5, the seismic forces are low enough that conventional stiffness rules for wind design actually control the stiffness.

Orthogonal effects need not be considered for Seismic Design Category C, provided the structure does not have a plan structural irregularity (*Provisions* Sec. 5.2.5.2.2).

8.3.3 Building Weight and Base Shear Summary

The unit weights are as follows:

Non-composite dead load:

4.5 in. slab on 0.6 in. form deck, plus sag	58 psf
Joist and beam framing	6 psf
Columns	2 psf
	66 psf

Composite dead load:

Fire insulation	4 psf
Mechanical and electrical	6 psf
Ceiling	2 psf
Partitions	20 psf
	32 psf

Exterior wall:

Precast concrete panels:	0.80 klf

Records storage on 3 bays per floor 120 psf

(50 percent is used for seismic weight; minimum per the *Provisions* is 25 percent)

The building weight, W, is found to be 8,080 kips. The treatment of the dead loads for analysis is described in more detail subsequently.

The Seismic Response Coefficient, C_s, is equal to 0.021:

$$C_s = \frac{S_{D1}}{T\dfrac{R}{I}} = \frac{0.14}{1.12\left(\dfrac{6}{1}\right)} = 0.021$$

The methods used to determine W and C_s are similar to those used elsewhere in this volume of design examples. The building is somewhat heavy and flexible. The computed periods of vibration in the first modes are 2.12 and 2.01 seconds in the north-south and east-west directions, respectively. These are much higher than the customary 0.1 second per story rule of thumb, but low-rise frames with small seismic force demands typically do have periods substantially in excess of the rule of thumb. The approximate period per the *Provisions* is 0.66 seconds, and the upper bound for this level of ground motion is 1.12 seconds.

The total seismic force or base shear is then calculated as follows:

$V = C_sW = (0.021)(8,080) = 170$ kips (*Provisions* Eq. 5.3.2)

The distribution of the base shear to each floor (again, by methods similar to those used elsewhere in this volume of design examples) is found to be:

Roof	(Level 4):	70 kips
Story 4	(Level 3):	57 kips
Story 3	(Level 2):	34 kips
Story 2	(Level 1):	8 kips
Story 1	(Level 0):	0 kips
	Σ:	169 kips (difference is rounding; total is 170)

Without illustrating the techniques, the gross service level wind force following ASCE 7 is 123 kips. When including the directionality effect and the strength load factor, the design wind force is somewhat less than the design seismic base shear. The wind force is not distributed in the same fashion as the seismic force, thus the story shears and the overturning moments for wind are considerably less than for seismic.

8.4 DETAILS OF THE PRC CONNECTION AND SYSTEM

8.4.1 Connection M-θ Relationships

The composite connections must resist both a negative moment and a positive moment. The negative moment connection has the slab rebar in tension and the leg of the seat angle in compression. The positive moment connection has the slab concrete in compression (at least the "a" dimension down from the top of the slab) and the seat angle in tension (which results in flexing of the seat angle vertical leg). At larger rotations the web angles contribute a tension force that increases the resistance for both negative and positive bending.

Each of these conditions has a moment-rotation relationship available in AISC SDGS-8 and ASCE TC. (Unfortunately there are typographical errors in ASCE TC: A "+" should be replaced by "=" and the symbol for the area of the seat angle is used where the symbol should be that for the area of the web angle.) An M-θ curve can be developed from these equations:

Negative moment connection:

$$M_n^- = C_1(1 - e^{-C_2\theta}) + C_3\theta \qquad \text{(AISC SDGS-8, Eq. 1)}$$

where:

$C_1 = 0.18(4 \times A_s F_{yrb} + 0.857 A_L F_y)(d + Y_3)$
$C_2 = 0.775$
$C_3 = 0.007(A_L + A_{wL})F_y\,(d + Y_3)$
θ = girder end rotation, milliradians (radians/1000)
d = girder depth, in.
Y_3 = distance from top flange of the girder to the centroid of the reinforcement, in.
A_s = steel reinforcing area, in.^{2}
A_L = area of seat angle leg, in.^{2}
A_{wL} = gross area of double web angles for shear calculations, in.^{2} (For use in these equations A_{wL} is limited to 150 percent of A_L).
F_{yrb} = yield stress of reinforcing, ksi
F_y = yield stress of seat and web angles, ksi

Positive moment connection:

$$M_n^+ = C_1(1 - e^{-C_2\theta}) + (C_3 + C_4)\theta \qquad \text{(AISC SDGS-8, Eq. 2)}$$

where:

$C_1 = 0.2400[(0.48A_{wL}) + A_L](d + Y_3)F_y$
$C_2 = 0.0210(d + Y_3/2)$
$C_3 = 0.0100(A_{wL} + A_L)(d + Y_3)F_y$
$C_4 = 0.0065\,A_{wL}\,(d + Y_3)F_y$

From these equations, curves for M-θ can be developed for a particular connection. Figures 8-5 and 8-6 are M-θ curves for the connections associated with the W18x35 girder and the W21x44 spandrel beam respectively, which are used in this example. The selection of the reinforcing steel, connection angles, and bolts are described in the subsequent section, as are the bilinear approximations shown in the figures. Among the important features of the connections demonstrated by these curves are:

1. The substantial ductility in both negative and positive bending,
2. The differing stiffnesses for negative and positive bending, and
3. The substantial post-yield stiffness for both negative and positive bending.

It should be recognized that these curves, and the equations from which they were plotted, do not reproduce the line from a single test. They are averages fit to real test data by numerical methods. They smear out the slip of bolts into bearing. (There are several articles in the *AISC Engineering Journal* that describe actual test results. They are in Vol. 24, No.2; Vol. 24, No.4; Vol. 27, No.1; Vol. 27, No. 2; and Vol 31, No. 2. The typical tests clearly demonstrate the ability of the connection to meet the rotation capabilities of AISC Seismic, Section 8.4 - inelastic rotation of 0.015 radians and total rotation capacity of 0.030 radians.)

[Based on the modifications to AISC Seismic, Part II, Sec. 8.4 in 2003 *Provisions* Sec. 10.5.16, the required rotation capabilities are inelastic rotation of 0.025 radians and total rotation of 0.040 radians.]

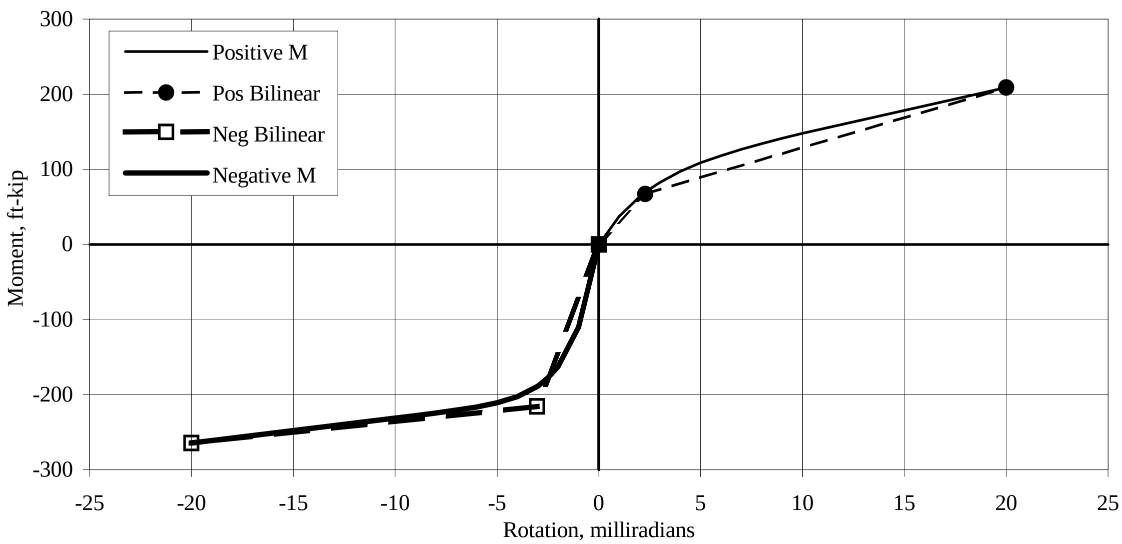

Figure 8-5 M-θ Curve for W18x35 connection with 6-#5 (1.0 ft-kip = 1.36 kN-m)

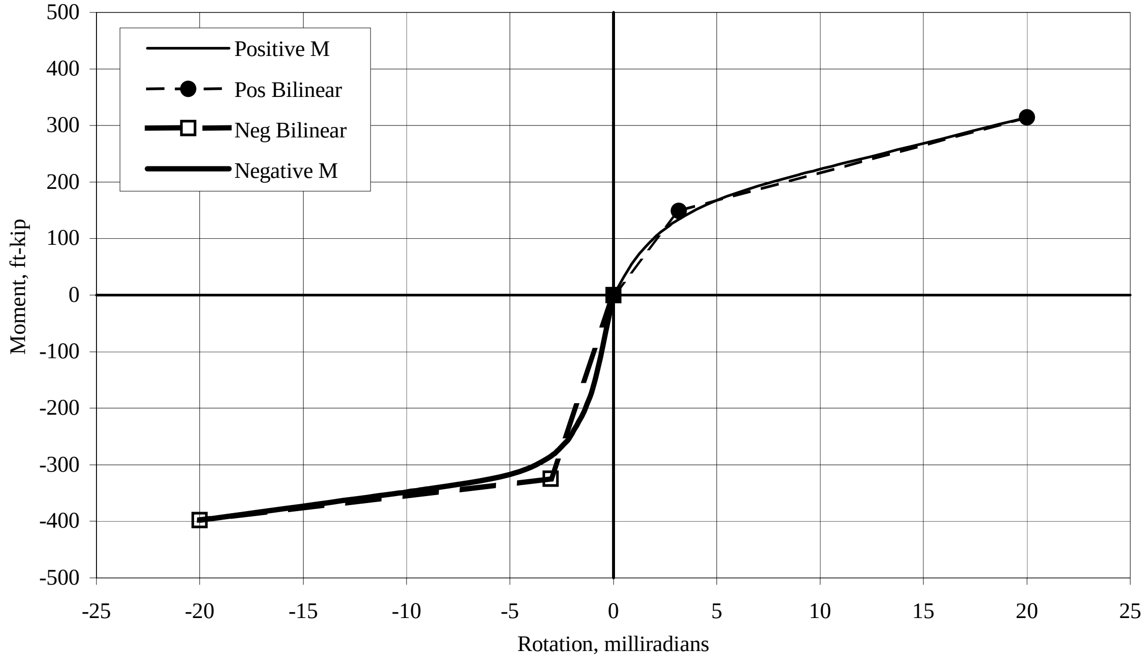

Figure 8-6 M-θ Curve for W21x44 connection with 8-#5 (1.0 ft-kip = 1.36 kN-m).

8.4.2 Connection Design and Connection Stiffness Analysis

Table 8-2 is taken from a spreadsheet used to compute various elements of the connections for this design example. It shows the typical W18x35 girder and the W21x44 spandrel beam with the connections used in the final analysis, as well as a W18x35 spandrel beam for the short exterior spans, where a W21x44 was used in the end. Each major step in the table is described in a line-by-line description following the table. [Based on the modifications to AISC Seismic, Part II, Sec. in 2003 *Provisions* Sec. 10.5.16, the nominal strength of the connection must be exceed R_yM_p for the bare steel beam, where R_y is the ratio of expected yield strength to nominal yield strength per AISC Seismic, Part I, Table I-6-1.]

Table 8-2 Partially Restrained Composite Connection Design

Line		Girder	Spandrels	
	Basic Data			
2	Beam size	W18x35	W21x44	W18x35
3	Span, ft	25	25	12.5
4	Area of beam, in.^{2}	10.3	13	10.3
5	I, of beam alone, in.^{4}	510	843	510
6	Z, plastic modulus of beam, in.^{3}	66.5	95.4	66.5
7	Beam depth, in.	17.7	20.7	17.7
8	Slab thickness, in.	7.0	7.0	7.0
9	Y_3 to rebar, in.	5.5	5.5	5.5
10	Column	W10x77	W10x88	W10x77
11	Flange width, in.	10.2	10.3	10.2
12	Flange thickness, in.	0.87	0.99	0.87
13	Flange fillet, k_1, in.	0.88	0.94	0.88
	Basic Negative Moment Capacity			
15	Reinforcing bars	6-#5	8-#5	6-#5
16	A_s, rebar area, in.^{2}	1.86	2.48	1.86
17	T_r, rebar tension, kips	111.6	148.8	111.6
18	M_n^- , nominal negative moment, ft-kips	215.8	324.9	215.8
19	% M_p (M_n^-/beam M_p)	78%	82%	78%
20	Check: > 50%? (75% per ASCE TC)	OK	OK	OK
	Seat Demands for Negative Moment			
22	Seat angle	L7x4x$^1/_2$x8	L7x4x$^5/_8$x8.5	L7x4x$^1/_2$x8
23	Seat F_y, ksi	36	36	36
24	Seat thickness, in.	0.5	0.625	0.5
25	Seat length, in.	8.0	8.5	8.0
26	Leg area, in.^{2}	4.0	5.3125	4.0
27	Minimum area = 1.25 T_r/F_y, in.^{2}	3.875	5.167	3.875
28	Check	OK	OK	OK
29	Leg yield force, kips	144	191.25	144
30	Bolts to beam	(4) 1"-325X	(4) 1$^1/_8$"-490X	(4) 1"-325X
31	Diameter, in.	1.0	0.875	1.0
32	Bolt design shear capacity, kips (ϕ = 0.75)	141.2	223.6	141.2
33	Check	Close enough	OK	Close enough
	Nominal Positive Moment Capacity			
35	Seat k, fillet length, in.	1.000	1.125	1.000
36	M_p, vertical leg, in.-kips	18.0	29.9	18.0
37	b' (see Figure 8-7), in.	1.00	0.81	1.00
38	Seat tension from bending, kips	31.5	63.8	31.5
39	Seat tension from shear, kips	86.4	114.75	86.4
40	Tension to bottom flange, kips	31.5	63.8	31.5
41	Nominal Positive Moment, M_n^+, ft-kips	67.4	149.9	67.4
42	Percent of Beam M_p	24%	38%	24%
	Demand on Tension Bolts at Nominal Capacity			
44	a' (see Figure 8-7), in.	2.0	2.1	2.0
45	Q (prying), kips	6.8	10.7	6.8
46	Bolt tension, kips	38.3	74.5	38.3
47	Bolts to column	(2) 1"-325X	(2) 1$^1/_8$"-490X	(2) 1"-325X
48	Bolt design tension, kips (ϕ = 0.75)	106	168.4	106
49	Check	OK	OK	OK

Line		Girder	Spandrels	
	Compute Total Joint Moment to Column based on Nominal Capacities			
51	Connection nominal $M_n^- + M_n^+$, ft-kips	283	475	283
52	Minimum column M_p (125% of sum)	177	297	177
53	Average as percentage of beam	51%	60%	51%
54	Check	OK	OK	OK
	Concrete Compression Transfer to Column			
56	Rebar T_y + bottom seat T_y, kips	143.10	212.62	143.10
57	0.85 f'_c on two flanges, kips	364.14	367.71	364.14
58	Projection for flange M_p, in.	2.72	3.10	2.72
59	Force from flange M_p, kips	225.92	254.88	225.92
60	Ratio, demand / minimum capacity	0.63	0.83	0.63
	Web Shear Connection (needed for effective stiffness)			
62	Seismic shear demand, kips	11.5	19.9	23.1
63	Web angles	L4x4x$^1/_4$x8.5	L4x4x$^1/_4$x11.5	L4x4x$^1/_4$x8.5
64	A_w, area of two legs, in.^{2}	4.25	5.75	4.25
65	A_w, limit based on area of rebar, in.^{2}	2.79	3.72	2.79
66	A_w, used in M-θ calculation, in.^{2}	2.79	3.72	2.79
	Moment Rotation Values for Analysis of Effective Stiffness			
68	M_{neg} at service level (0.0025 rad), ft-kip	-178.0	-267.8	-178.0
69	M_{neg} at maximum capacity(0.020 rad), ft-kip	-264.5	-397.7	-264.5
70	Secant stiffness for M_{neg} at 0.0025 radian	71.2	107.1	71.2
71	M_{pos} at service level (0.0025 rad), ft-kip	73.7	117.3	73.7
72	M_{pos} at maximum capacity(0.020 rad), ft-kip	208.9	313.9	208.9
73	Secant stiffness for M_{pos} at 0.0025 radian	29.5	46.9	29.5
74	Rotation at nominal M_{neg}	3.03	3.03	3.03
75	Rotation at nominal M_{pos}	2.29	3.70	2.29
	Beam Moments of Inertia			
77	Full composite action force, beam AF_y, kips	515.0	650.0	515.0
78	Y_2, to plastic centroid in concrete, in.	5.65	5.30	4.31
79	Composite beam inertia for pos. bending, in.^{4}	1,593	2,435	1,402
80	Centroid of all steel for negative bending, in.	6.66	7.81	6.66
81	Composite beam inertia for neg. bending, in.^{4}	834	1366	834
82	Equivalent beam for positive and negative, in.^{4}	1,290	2,008	1,175
83	Weighted connection stiffness, ft-kips/radian	61,263	88,105	61,263
84	Eff. prismatic inertia, beam and PRCC, in.^{4}	639	955	412
85	Ratio of eff. prismatic I / I of beam alone	1.25	1.13	0.81
	Check Bottom Bolt Tension at Maximum Deformation			
87	Rotation at $\phi \times$ (rotation at nominal M_{pos}) $\times C_d$	10.7	14.9	10.7
88	Moment at $\phi \times$ (rot. at nom. M_{pos}) $\times C_d$, ft-kips	152.3	268.2	152.3
89	Tension demand, kips	80.5	125.1	80.5
90	Nominal capacity of bolts, kips	141.3	224.5	141.3
	Check Positive Moment Capacity as a Percentage of Beam M_p (50% criterion)			
92	M_{pos} (at 0.020 radians) / M_p beam	75%	79%	75%

Detailed explanation of the computations in Table 8-2:

Step 1: Establish nominal negative moment capacity: (This is a step created in this design example; is not actually an explicit step in the procedures recommended in the references. It appears to be necessary to satisfy the basic *Provisions* strength requirement. See *Provisions* Sec. 5.2.1, Sec. 5.2.7, and ASCE 7 Sec. 2.3.

Lines 15-18: M_n is taken as a simple couple of rebar in slab and force at connection of bottom flange of beam; the true maximum moment is larger due to strain hardening in rebar and the bottom connection and due to tension force in the web connection, so long as the bottom connection can handle the additional demand. The nominal capacity is plotted in Figures 8-5 and 8-6 as the break of the bilinear relation. The design capacity, using a resistance factor of 0.85, has two requirements:

1. ϕM_n exceeds demand from seismic load combination: basic *Provisions* requirement

2. ϕM_n exceeds demand from total service gravity loads - simply a good idea to maintain reasonable initial stiffness for lateral loads; by "codes" the factored gravity demand can be checked using plastic analysis

Lines 19-20: M_n exceeds 50 percent (by AISC Seismic, Part II, 8.4) of M_p of the bare steel beam. In this example, the more stringent recommendation of 75 percent contained within the ASCE TC is followed. Note that this check is on nominal strength, not design strength. A larger M_n gives a larger stiffness, thus some drift problems can be addressed by increasing connection capacity.

Step 2: Design bottom seat angle connection for negative moment:

Lines 22-28: Provide nominal yield of angle leg at least 125 percent of nominal yield of reinforcing steel. This allows for increased force due to web shear connection. Strain hardening in the rebar is a factor, but strain hardening the angle would probably be as large. AISC SDGS-8 recommends 120 percent. ASCE TC recommends 133 percent, but then uses 125 percent to check the bolts. This is a check in compression, and the authors elected to use 125 percent.

Lines 29-33: Provide high strength bolts in normal (not oversized) holes to transfer force between beam flange and angle by shear; conventional rules regarding threads in the shear plane apply. The references do apply a resistance factor to the bolts, which may be an inconsistent design methodology. A check based on overstrength might be more consistent. The capacity at bolt slip could be compared against service loads, which would be a good idea for designs subject to strong wind forces.

Step 3: Establish nominal positive moment capacity: This connection is less stiff and less linear for positive moment than for negative moment, and generally weaker. There is not a simple, clear mechanism for a nominal positive moment. The authors of this example suggest the following procedure which follows the normal methods of structural engineering and yields a point relevant to the results of connection tests, in so far as construction of a bilinear approximation is concerned. It significantly underestimates the ultimate capacity.

Lines 35-38: Compute the shear in the vertical leg associated with bending. Figure 8-7 shows the mechanics, which is based on methods in the AISC Manual, for computing prying in hanger-type connections. Compute the nominal plastic moment of the angle leg bending out of plane (line 36) and assume that the location of the maximum moments are at the end of the fillet on the vertical leg (line 35) and at the edge of the bolt shaft (line 37). The moment near the bolt is reduced for the material lost at the bolt hole.

Lines 39-40: Check the shear capacity, compare with the shear governed by moment, and use the smaller. Shear will control if the angle is thick.

Line 41: Compute the nominal positive moment as a couple with the force and the distance from the bottom of the beam to the center of the compression area of the slab on the column. The concrete compression area uses the idealized Whitney stress block (ACI 318). Note that the capacity to transfer concrete compression force to the steel column flange is checked later. The nominal positive moment is also shown on Figures 8-5 and 8-6 at the break point in the bilinear relation.

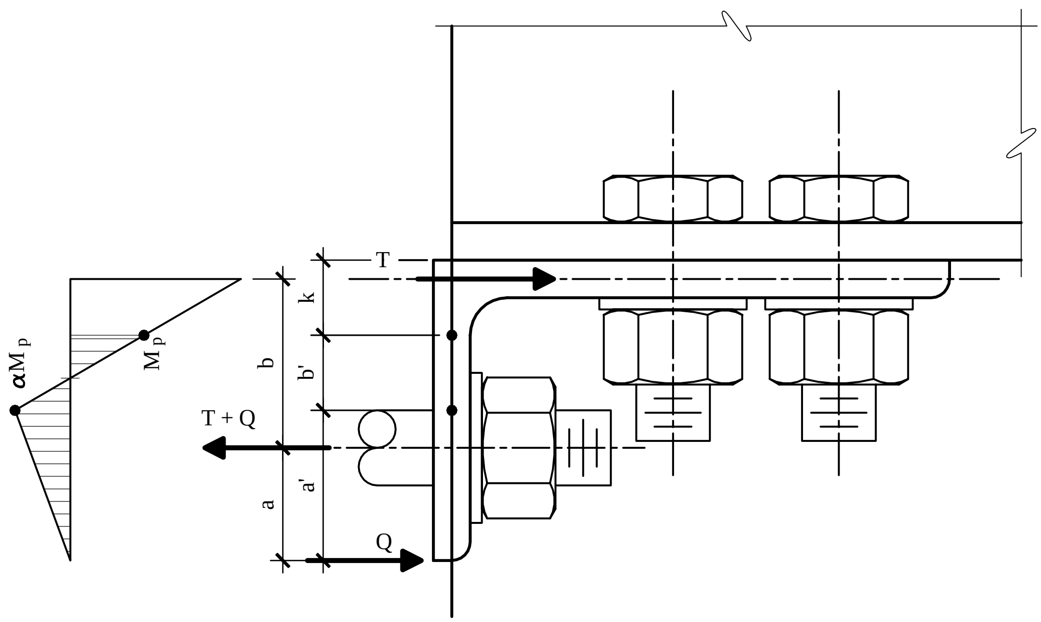

Figure 8-7 Analysis of seat angle for tension.

Step 4: Design the bolts to transfer positive moment tension to the column:

Lines 44-45: Compute the prying force following AISC's recommended method. The moment in the vertical leg is computed as described above, and the moment arm extends from the edge of the bolt shaft (closest to the beam) to the bottom edge of the angle. Refer to Figure 8-7.

Lines 46-48: Add the basic tension to the prying force and compare to the factored design capacity of the bolts. Note that the resistance factor is used here to be consistent with step 2. It is common to use the same size and grade of bolt as used for the connection to the beam flange, which generally means that these bolts have excess capacity. Also, for seismic design, another check at maximum positive moment is recommended (see step 9).

Step 5: Compute the flexural demand on the column: AISC Seismic, Part II, 7 and 8, require that the flexural resistance of the column be greater than the demand from the connections, but it does not give any particular margin. ASCE TC recommends a ratio of 1.25.

Lines 51-52: The minimum nominal flexural strength of the column, summed above and below as well as adjusted for the presence of axial load, is set to be 125 percent of the demand from the sum of the nominal strengths of the connections.

Lines 53-54: AISC Seismic, Part II, 8.4 requires that the connection capacity exceed 50 percent of the plastic moment capacity of the beam. In this example, the negative moment connections are designed for 75 percent of the beam plastic moment, and this check shows that the average of negative and positive nominal moment capacities for the connection exceeds 50 percent of the plastic moment for the beam. A later check (step 10) will compare the maximum positive moment resistance to the 50 percent rule.

Step 6: Check the transfer of force from concrete slab to steel column: The tension in the reinforcing steel and the compression couple from positive bending must both transfer. Both flanges provide resistance if concrete fills the space between the flanges, but full capacity of the second flange has probably not been exercised in tests.

Line 56: Add the yield force of the reinforcement and the tension yield force of the seat angle, both previously computed.

Line 57: Compute an upper bound concrete compression capacity as $0.85f'_c$ times the area of concrete bearing on both flanges.

Lines 58-59: Compute the force that would yield the steel column flanges over the thickness of the slab by computing the projection beyond the web fillet that would yield at a load of 0.85f'c. This ignores the capacity of the flange beyond the slab thickness and is obviously conservative.

Line 60: Compare the demand with the smaller of the two capacities just computed.

Step 7: Select the web connection:

Line 62: The seismic shear is computed by assuming beam end moments equal to the nominal capacity of the connections, one in negative moment and one in positive.

Line 63: The gravity demand must be added, and straight gravity demand must also be checked before selecting the actual connection.

Lines 64-66: The web connection influences the overall stiffness and strength of the connection, especially at large rotation angles. The moment-rotation expression include the area of steel in the web angles, but also places a limitation based upon 150 percent of the area of the leg of the seat angle for use in the computation.

Step 8: Determine the effective stiffness of the beam and connection system: Determining the equivalent stiffness for a prismatic beam involves several considerations. Figure 8-8 shows how the moment along the beam varies for gravity and lateral loads as well as composite and non-composite conditions. The moment of inertia for the composite beam varies with the sense of the bending moment. The end connections can be modeled as regions with their own moments of inertia, as illustrated in the figure. Figure 8-9 shows the effective cross section for each of the four stiffnesses: positive and negative bending of the composite beam and positive and negative bending of the composite connection. Given a linear approximation of each connection stiffness expressed as moment per radian, flexural mechanics leads to a simple expression for a moment of inertia of an equivalent prismatic beam.

Lines 68-73: Compute the negative and positive moments at a rotation of 2.5 milliradians, which is the rotation angle that defines the effective stiffness for lateral analysis (per both AISC SDGS-8 and ASCE TC).

Lines 74-75: Using those moments, compute the rotations corresponding to the nominal strength, positive and negative. (This is useful when idealizing the behavior as bilinear, which is plotted in Figures 8-5 and 8-6.)

Lines 77-79: Compute the moment of inertia of the composite beam in positive bending. Note that the system is designed for full composite action, per the recommendations in AISC SDGS-8 and ASCE TC, using the criteria in the AISC manual. The positive bending moment of inertia here is computed using AISC's lower bound method, which uses an area of steel in the flange adequate to replace the Whitney stress block in the concrete flange. This moment of inertia is less than if one used the full concrete area in Figure 8-9.

Lines 80-81: Compute the moment of inertia of the composite beam in negative bending.

Line 82: Compute an equivalent moment of inertia for the beam recognizing that a portion of the span is in positive bending and the remainder is in negative bending. Following the recommendations in AISC SDGS-8 and ASCE TC, this is computed as 60 percent of I_{pos} and 40 percent of I_{neg}.

Lines 83-84: Compute the moment of inertia of a prismatic beam that will give the same total end rotation in a sway condition as the actual system. Gravity loads place both connections in negative moment, so one will be subject to increasing negative moment while the other will be subject to decreasing negative moment. Thus, initially, the negative moment stiffness is the appropriate stiffness, which is what is recommended in the AISC SDGS-8 and ASCE TC. For this example the positive and negative stiffnesses are combined, weighted by the nominal strengths in positive and negative bending, to yield a connection stiffness that is appropriate for analysis up to the nominal strengths defined earlier. Defining this weighted stiffness as K_{conn} and the equivalent composite beam moment of inertia as I_{comp}, the effective moment of inertia is found by:

$$I_{effective} = \frac{I_{comp}}{1 + \dfrac{6EI_{comp}}{L\,K_{conn}}}$$

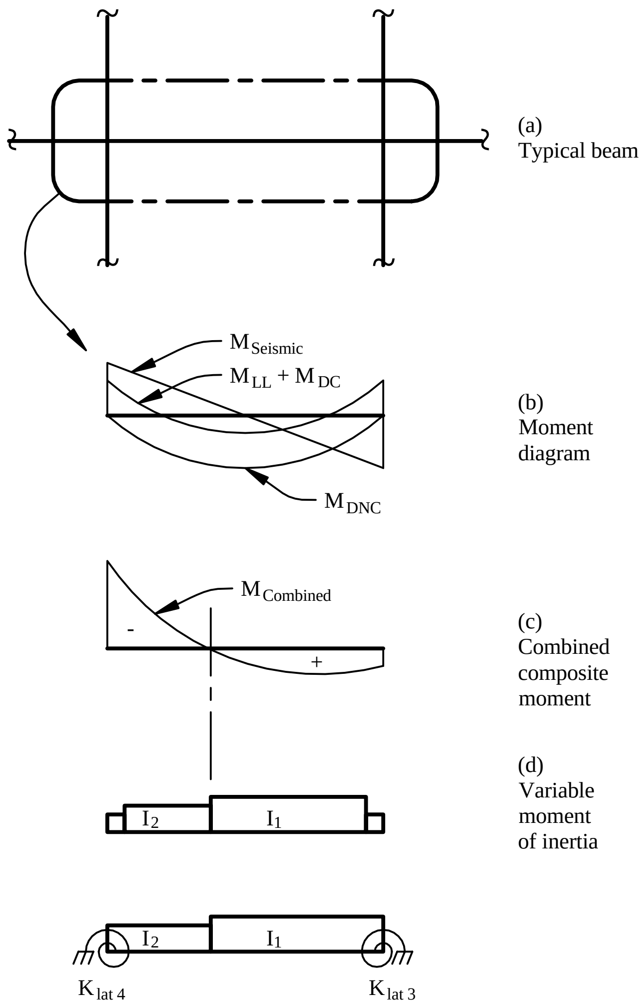

Figure 8-8 Moment diagram for typical beam.

Line 85: compute the ratio of the moment of inertia of the effective prismatic beam to that for the bare steel beam. When using standard computer programs for analysis that have a library of properties of steel cross sections, this ratio is a convenient way to adjust the modulus of elasticity and thus easily compute the lateral drift of a frame. This adjustment could invalidate routines in programs that automatically check various design criteria that depend on the modulus of elasticity.

Step 9: Check the tension bolts at maximum rotation

Line 87: Compute the rotation at total drift as C_d times the drift at the design positive moment.
Line 88: Compute the positive moment corresponding to that drift.
Line 89: Compute the tension force at the bottom seat angle, ignoring any contribution of the web angles, from the moment and a moment arm between the center of the slab thickness and the inflection point in the vertical leg of the seat angle, then add the prying force already calculated for a maximum demand on the tension bolts.
Line 90: Compare with the nominal capacity of the bolts (set $\phi = 1.0$)

Step 10: Check the maximum positive moment capacity:

Line 92: The positive moment at 20 milliradians, already calculated, is compared to the plastic moment capacity of the steel beam. This is the point at which the 50 percent requirement of AISC Seismic, Part II, 8.4 is checked.

Figure 8-10 shows many of the details of the connection for the W18x35. The headed studs shown develop full composite action of the beam between the end and midspan. They do not develop full composite action between the column and the inflection point, but it may be easily demonstrated that they are more than capable of developing the full force in the reinforcing steel within that distance. The transverse reinforcement is an important element of the design, which will be discussed subsequently. Alternating the position above and below is simply a preference of the authors.

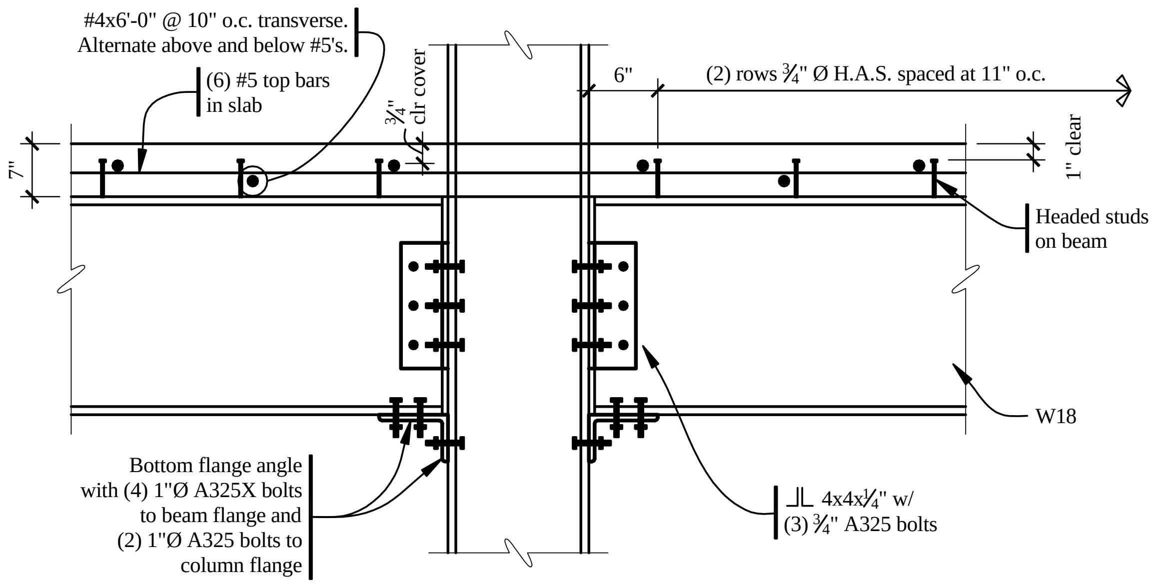

Figure 8-10 Elevation of typical connection (1.0 ft = 0.3048 m, 1.0 in. = 25.4 mm).

8.5 ANALYSIS

8.5.1 Load Combinations

A 3D model using Risa 3D was developed. Non-composite dead loads (steel beams, bar joists, form deck, and concrete) were input as concentrated loads at the columns on each level rather than uniformly distributed to the beams. This was because we want the model for the seismic load combinations to address the moments in the PRC connections. The loads subject to composite action are the composite dead loads, live loads, and seismic loads, not the non-composite dead loads. But the non-composite dead loads still contribute to mass, are subject to ground acceleration, and as such contribute to seismic loads. This gets confusing; so a detailed look at the load combinations is appropriate.

Let us consider four load cases (illustrated in Figures 8-11 and 8-12):

1. D_c - Composite dead load, which is uniformly distributed and applied to beams (based on 32 psf)

2. D_{nc} - Non-composite dead load, which is applied to the columns (based on 66 psf)

3. L - Composite live load, which is uniformly distributed to beams, using live load reductions

4. E - Earthquake load, which is applied laterally to each level of the building and has a vertical component applied as a uniformly distributed load to the composite beams

We will investigate two load combinations. Recall that composite loads are applied to beams, while non-composite loads are applied to columns. But there is an exception: the $0.2S_{DS}D$ component, which represents vertical acceleration from the earthquake is applied to all the dead load on the beams whether it is composite or non-composite. This is because even non-composite dead load contributes to mass, and is subject to the ground acceleration. Because the non-composite dead load is not distributed on the beams in the computer model, an adjustment to the load factor is necessary. The assignment of loads gets a little complicated, so pay careful attention:

$$
\begin{aligned}
\text{Combination 1} &= 1.2D + 0.5L + 1.0E \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.2S_{DS}D \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.067\,(D_{nc} + D_c) \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.067D_{nc}(D_c/D_c) + 0.067D_c \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.067D_c(D_{nc}/D_c) + 0.067D_c \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.067D_c(66\text{ psf}/32\text{ psf}) + 0.067D_c \\
&= 1.2D_{nc} + 1.2D_c + 0.5L + Q_E + 0.138D_c + 0.067D_c \\
&= 1.2D_{nc} + 1.405D_c + 0.5L + Q_E
\end{aligned}
$$

Q_E will be applied in both the north-south and the east-west directions, so this really represents two load combinations.

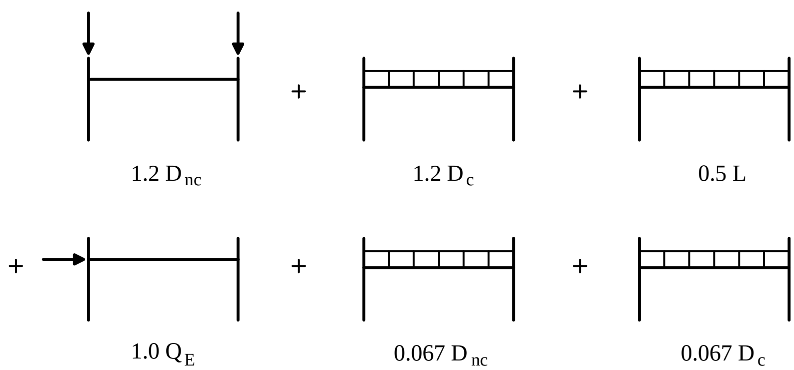

Figure 8-11 Illustration of input for load combination for $1.2D + 0.5L + 1.0Q_E + 0.2S_{DS}D$.

D_{nc} = non-composite dead load.
D_c = composite dead load
L = live load
Q_E = horizontal seismic load

Now consider at the second load combination:

$$
\begin{aligned}
\text{Combination 2} &= 0.9D + 1.0E \\
&= 0.9D_{nc} + 0.9D_c + Q_E - 0.2S_{DS}D \\
&= 0.9D_{nc} + 0.9D_c + Q_E - 0.067\,(D_{nc} + D_c) \\
&= 0.9D_{nc} + 0.9D_c + Q_E - 0.067\,D_{nc}(D_c/D_c) - 0.067D_c \\
&= 0.9D_{nc} + 0.9D_c + Q_E - 0.067\,D_c(D_{nc}/D_c) - 0.067D_c \\
&= 0.9D_{nc} + 0.9D_c + Q_E - 0.067\,D_c(66\text{ psf}/32\text{ psf}) - 0.067D_c
\end{aligned}
$$

$$= 0.9D_{nc} + 0.9D_c + Q_E - 0.138D_c - 0.067D_c$$
$$= 0.9D_{nc} + 0.9D_c + Q_E - 0.205D_c$$
$$= 0.9D_{nc} + 0.695D_c + Q_E$$

Again, Q_E will be applied in both the north-south and the east-west directions, so this represents another two load combinations.

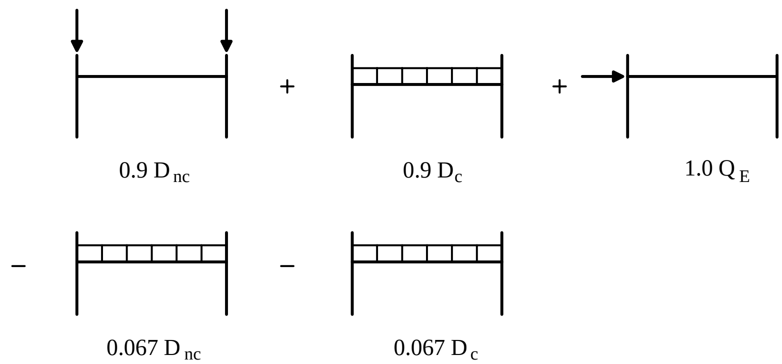

Figure 8-12 Illustration of input for load combination for $0.9D + 1.0Q_E - 0.2S_{DS}D$.

D_{nc} = non-composite dead load.
D_c = composite dead load
L = live load
Q_E = horizontal seismic load

8.5.2 Drift and P-delta

As defined by the *Provisions*, torsional irregularity is considered to exist when the maximum displacement computed including accidental torsion at one end of the structure transverse to an axis is more than 1.2 times the average of the displacements at the two ends of the structure (*Provisions* Sec. 5.4.4.3). For this building the maximum displacement at the roof including accidental torsion, is 1.65 in. The displacement at the other end of the building in this direction is 1.43 in. The average is 1.54 in. Because 1.65 in. < 1.85 in. = (1.2)(1.54 in.), the structure is not torsionally irregular. Consequently, it is not necessary to amplify the accidental torsion nor to check the story drift at the corners. A simple check at the center of the building suffices. [In the 2003 *Provisions*, the maximum limit on the stability coefficient has been replaced by a requirement that the stability coefficient is permitted to exceed 0.10 if and only "if the resistance to lateral forces is determined to increase in a monotonic nonlinear static (pushover) analysis to the target displacement as determined in Sec. A5.2.3. P-delta effects shall be included in the analysis." Therefore, in this example, the stability coefficient should be evaluated directly using 2003 *Provisions* Eq. 5.2.-16.]

The elastic story drifts were computed by the RISA 3D analysis for the required load combinations. Like most modern computer programs for structural analysis, a P-delta amplification can be automatically computed, but to illustrate the effect of P-delta in this structure and to check the limit on the stability index, two computer runs have been performed, one without the P-delta amplifier and one with it. The allowable story drift is taken from *Provisions* Table 5.2.8. The allowable story drift is 0.025 h_{sx} = (0.025)(13 ft × 12 in./ft) = 3.9 in. With a C_d of 5.5, this corresponds to a drift 0.71 in. under the equivalent elastic forces. At this point design for wind does influence the structure. A drift limit of h/400 (= 0.39 in.) was imposed, by office practice, to the service level wind load. In order to achieve the desired stiffness, the seismic story drift at elastic forces is determined thus:

Elastic story drift limit = (wind drift limit)(total seismic force)/(service level wind force)
Elastic story drift limit = (0.39 in.)(170 kip)/123 kip = 0.54 in.

The structure complies with the story drift requirements, but it was necessary to increase the size of the spandrel beams from the preliminary W18x35 to W21x44 to meet the desired wind stiffness. This is summarized in Table 8-3. The structure also complies with the maximum limit on the stability index (Provisions 5.4.6.2-2):

$$\theta_{\max} = \frac{0.5}{\beta C_d} = \frac{0.5}{0.5 * 5.5} = 0.18 \le 0.25$$

β is the ratio of demand to capacity for the story shear, and has not yet been computed. Maximum demand and design capacity are tabulated in the following section; the average is about two-thirds. The preceding data show that the maximum resistance is higher, especially for positive moment, than the value suggested here for design capacity. The average ratio of demand to maximum capacity with a resistance factor is well below 0.5, so that value is arbitrarily used to show that the actual stability index is within the limits of the *Provisions*.

Table 8-3 Story Drift (in.) and P-delta Analysis

Story	North-south (X direction)				East-west (Z direction)			
	without P-delta	with P-delta	P-delta amplifier	Stability index	without P-delta	with P-delta	P-delta amplifier	Stability index
1	0.358	0.422	1.179	0.152	0.312	0.360	1.154	0.133
2	0.443	0.517	1.167	0.143	0.410	0.471	1.149	0.130
3	0.449	0.513	1.143	0.125	0.402	0.453	1.127	0.113
4	0.278	0.304	1.094	0.086	0.239	0.259	1.084	0.077

8.5.3 Required and Provided Strengths

The maximum beam end moments from the frame analysis for the seismic load combinations are as follows:

Table 8-4 Maximum Connection Moments and Capacities (ft-kips)

Quantity	W18 Girders		W21 Spandrels	
	Negative	Positive	Negative	Positive
Demand (level 2), M_u	143	36.6	118	103
Nominal, M_n	216	67.2	325	149
Design capacity, ϕM_n	184	57.1	276	127

The capacities, using a resistance factor of 0.85, are well in excess of the demands. The girder member sizes are controlled by gravity load in the construction condition. All other member and connection capacities are controlled by the design for drift. The negative moment demands are somewhat larger than would result from a more careful analysis, because the use of a prismatic member overestimates the end moments due to distributed load (composite gravity load) along the member. The higher stiffness of the portion of the beam in positive bending with respect to the connections would result in higher positive moments at midspan and lower negative moments at the supports. This conservatism has no real effect on this design example. (The above demands and capacities do not include the girders supporting the storage bays, which are required to be W18x40 for the gravity load condition. The overall analysis does not take that larger member into account.)

Snow load is not included in the seismic load combinations. (According to the *Provisions*, snow load equal to or less than 30 psf does not have to be included in the mass.) Further, as a designer's judgment call, it was considered that the moments from 0.2S (= 6 psf) were so small, considering that the roof was designed with the same connections as the floors, that it would make no significant difference in the design analysis.

The maximum column forces are shown in Table 8-5; the particular column does support the storage load. The effective length of the columns about their weak axis will be taken as 1.0, because they are braced by perpendicular frames acting on the strong axes of the columns, and the P-delta analysis captures the secondary moments due to the "leaning" column effect. The effective length about their strong axis will exceed 1.0. The ratio of column stiffness to beam stiffness will use the same effective beam stiffness computed for the drift analysis, thus for the W10x77 framed into the W18x35 beams:

$I_{col} / L_{col} = 455/(13 \times 12) = 2.92$

$I_{beam} / L_{beam} = 1.25 \times 510 / (25 \times 12) = 2.12$

and the ratio of stiffnesses, $G = 2.92 / 2.12 = 1.37$

Although the column in the lowest story has greater restraint at the foundation, and thus a lower K factor, it is illustrative to determine K for a column with the same restraint at the top and bottom. From the nomographs in the AISC Manual or from equivalent equations, K = 1.45. It turns out that the effective slenderness about the strong axis is less than that for the weak axis, so the K factor does not really control this design.

Table 8-5 Column Strength Check, for W10x77

	Seismic Load Combination	Gravity Load Combination
Axial force, P_u	391 kip	557 kip
Moment, M_u	76.3 ft-kip	52.5 ft-kip
Interaction equation	0.72	0.89

8.6 DETAILS OF THE DESIGN

8.6.1 Overview

The requirements in AISC Seismic for C-PRMF systems are brief. Some of the requirements are references to Part I of AISC Seismic for the purely steel components of the system. A few of those detail checks are illustrated here. For this example, more attention is paid to the details of the joint.

8.6.2 Width-Thickness Ratios

The width-thickness ratio of the beam flanges, $b_f/2t_f$ is compared to λ_p given in AISC Seismic, Part I, Table I-9-1. Both beam sizes, W18x35 and W21x44 are found to be acceptable. The W21x44 is illustrated below:

$$\lambda_p = \frac{52}{\sqrt{F_y}} = \frac{52}{\sqrt{50}} = 7.35$$ (AISC Seismic, Table I-9-1)

$$\frac{b_f}{2t_f} = 7.22$$ (AISC Manual)

7.22 < 7.35 OK

The limiting h/t ratios for columns is also given in AISC Seismic, Part I, Table I-9-1. A W10x77 column from the lower level of an interior bay with storage load is illustrated (the axial load from the seismic load combination is used):

$$\frac{P_u}{\phi_b P_y} = \frac{391\text{ kips}}{(0.9)(22.6\text{ in.}^2\text{ x }50\text{ ksi})} = 0.385 > 0.125$$ (AISC Seismic, Table I-9-1)

$$\lambda_p = \frac{191}{\sqrt{F_y}}\left[2.33 - \frac{P_u}{\phi_b P_y}\right] = \frac{191}{\sqrt{50}}\left[2.33 - 0.385\right] = 52.5$$ (AISC Seismic, Table I-9-1)

Check:

$$\lambda_p = 52.5 > 35.7 = \frac{253}{\sqrt{F_y}}$$ OK

$$\frac{h}{t_w} = 13.0$$ (AISC Manual)

13.0 < 52.5 OK

8.6.3 Column Axial Strength

AISC Seismic, Part I, 8.2 requires that when $P_u/\phi P_n > 0.4$ (in a seismic load combination), additional requirements be met. Selecting the same column as above for our illustration:

$$\frac{P_u}{\phi P_n} = \frac{391\text{ kips}}{(0.85)(22.6\text{ in.}^2)(38.4\text{ ksi})} = 0.53 > 0.4$$ (AISC Seismic, Part I, 8.2)

Therefore the requirements of AISC Seismic, Part I, 8.2a, 8.2b, and 8.2c apply. These necessitate the calculation of axial loads using the System Overstrength Factor, $\Omega_0 = 3$. Analysis needs to be run for two additional load combinations:

$1.2D + 0.5L + 0.2S + \Omega_0 Q_E$ (AISC Seismic, Part I, Eq. 4.1)

and

$0.9D - \Omega_0 Q_E$ (AISC Seismic, Part I, Eq. 4.2)

The axial seismic force in this column is only 7.5 kips, therefore P_u becomes 397 kips, obviously much less than ϕP_n. The low seismic axial load is common for a moment-resisting frame system. Given that this requirement is a check ignoring bending moment, it does not control the design.

[The special load combinations have been removed from the 2002 edition of AISC Seismic to eliminate inconsistencies with other building codes and standards. Therefore, 2003 *Provisions* Eq. 4.2-3 and 4.2-4 should be used in conjunction with the load combinations in ASCE 7.]

8.6.4 Details of the Joint

Figure 8-13 shows a plan view at an edge column, concentrating on the arrangement of the steel elements. Figure 8-14 shows a section at the same location, showing the arrangement of the reinforcing steel. It is not required that the reinforcing bars be equally distributed on the two sides of the column, but it is necessary to place at least some of the bars on each side. This means that some overhang of the slab beyond the column flange is required. This example shows two of the six bars on the outside face. Figure 8-15 shows a plan view at a corner column. U shaped bent bars are used to implement the negative moment connection at such a location. Threaded bars directly attached to the column flange are also illustrated. Note the close spacing of the headed anchor studs for composite action. The reason for the close spacing at this location is that the beam span is half the normal span, yet full composite action is still provided.

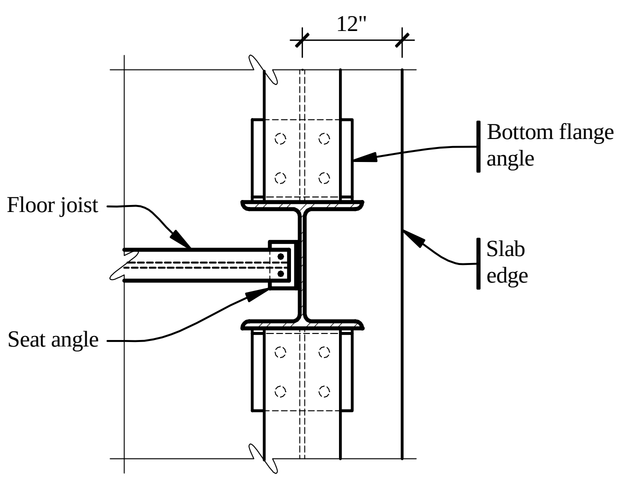

Figure 8-13 Detail at column.

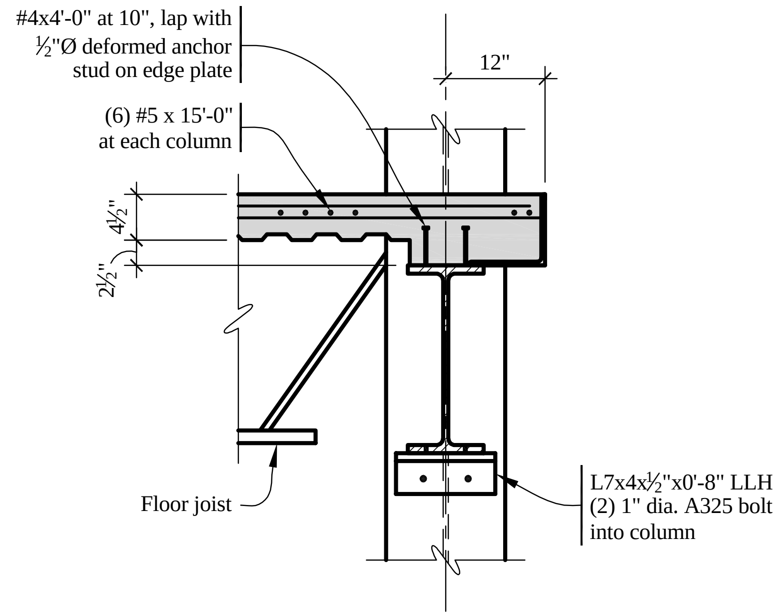

Figure 8-14 Detail at spandrel.

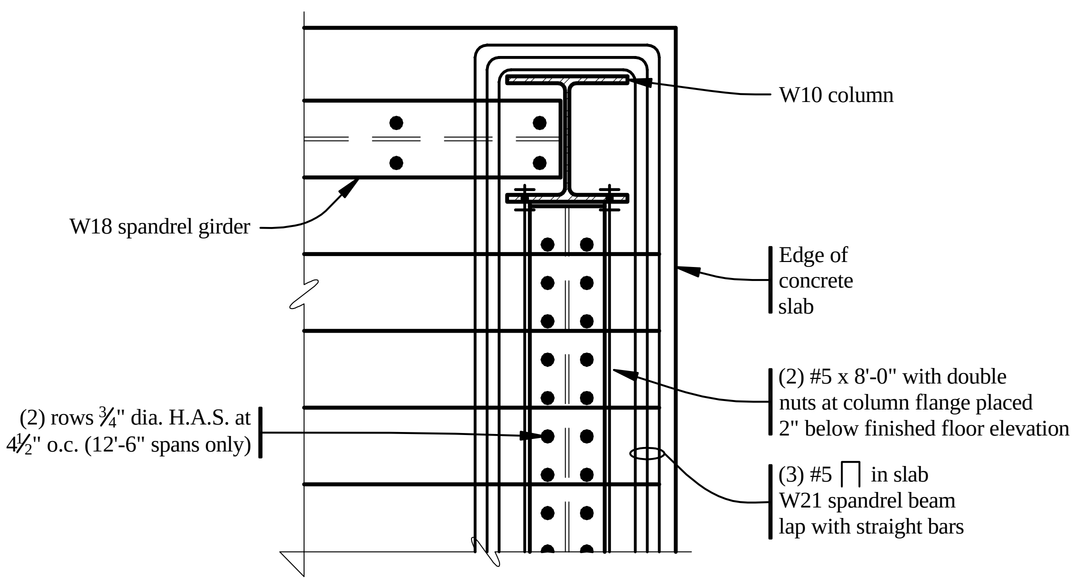

Figure 8-15 Detail at building corner.

The compressive force in the deck is transferred to both flanges of the column. This is shown in Figure 8-16. Note that both flanges can accept compressive forces from the concrete. Also note that the transverse reinforcement will carry tension as force is transferred from the principal tension reinforcement through the concrete to bearing on the column flange. Strut and tie models can be used to compute the appropriate tension.

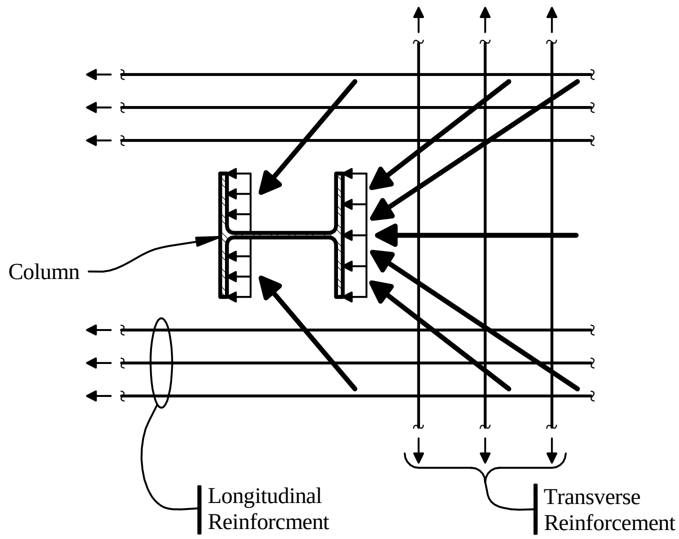

Figure 8-16 Force transfer from deck to column.

AISC SDGS-8 and ASCE TC include the following recommendations regarding the reinforcing steel:

1. Place the principal tension reinforcement within a strip of width equal to 7 times the width of the column flange (or less)

2. Use at least 6 bars for the principal reinforcement, extend it one quarter of the span from the column, but at least 24 bar diameters beyond the inflection point, and extend at least two of the bars over the full span

3. Do not use bars larger than number 6 (0.75 in. diameter)

4. Provide transverse reinforcement consistent with a strut and tie model to enable the transfer of forces (in the authors' observation such reinforcement is also necessary to preserve the capacity of the headed studs for shear transfer)