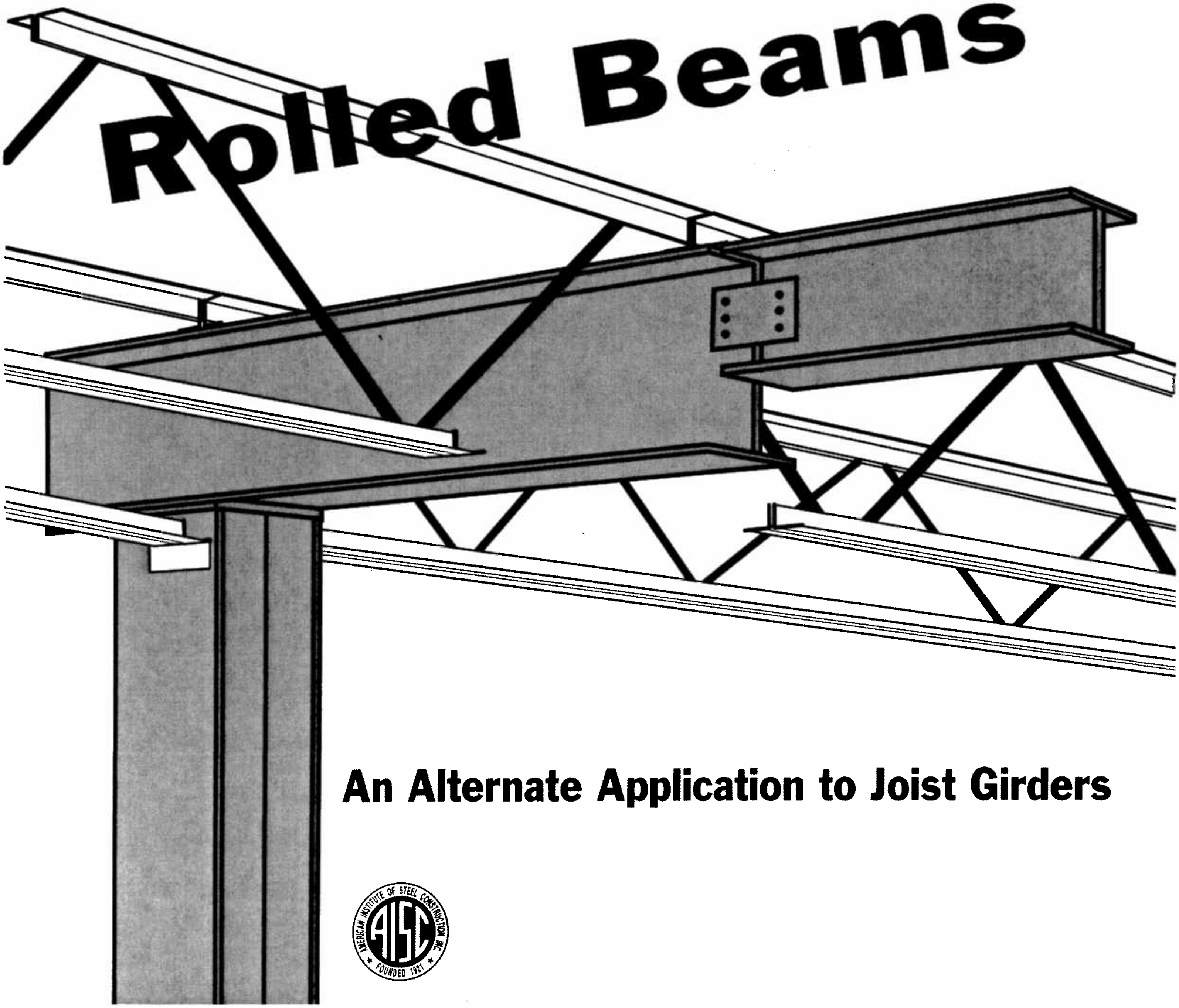
Cantilever Roof Framing Using Rolled Beams



An Alternate Application to Joist Girders

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Preface

In 1994 AISC Marketing, Inc. conducted an extensive market survey of owners, developers, architects, engineers and general contractors. In more than two-hundred interviews conducted, one recurring request was for typical designs for roof framing using rolled beams in lieu of joist-girders. These designs would provide a viable bidding alternative for contractors and in some cases lessen critical delivery problems.

On many projects the roof support system consists of standard "K" series joists closely spaced and supported by joist girders framing into columns. Long lead times in the design and procurement of the joist girders is common in the industry. An alternate roof framing system utilizing the cantilever and suspended structural steel girder is presented through design tables for several common load cases. The primary goal is to make available a preliminary set of design information to make architects, engineers, contractors, and fabricators aware of the system and its potential construction time and cost savings.

There are several advantages to using cantilevered rolled beams for roof framing:

- Greater steel economy over simple-span designs.
- Potential economies of scale by fabricating more wide-flange in a single shop for a given project.
- Potential economies from opening up bidding to more bar-joist suppliers.
- Possibility of improving steel deliveries.
- Reduction in perimeter wall heights with savings in wall material. When electrical and mechanical systems must be accommodated within the roof framing, these systems can pass through web openings or under beams. The resulting wall height with rolled beams will often be less than the joist girder system.
- Less building volume with savings in mechanical and operating costs.
- Greater versatility in carrying concentrated loads.
- Ease in modifying framing to accommodate changed loading conditions.

The tables that follow offer roof framing design solutions, based on a distinct set of parameters of loading, serviceability, bay sizes, and joist spans. These bay studies using cantilevered rolled beams cover five load cases. The designs use the LRFD Specification and Design Manual, 2nd Edition. The designs also parallel the work done by the Canadian Institute of Steel Construction in the publication *Roof Framing with Cantilever (Gerber) Girders & Open Web Steel Joists.*

As with any design problem there are many solutions. Each project will have a unique set of loading and serviceability parameters. The design information and worked example have been prepared accurately and consistent with current structural design practice for several different load cases. All data contained in this publication, are however, preliminary for general information and discussion only and shall not be used or relied upon for any specific application without competent professional examination and verification of its suitability and applicability by a licensed professional structural engineer.

Design Parameters and Limitations

Many specific parameters and limitations go into the design of any structural member. Imposed loadings caused by earthquake, wind, snow, rain, construction methods, etc. vary across the country. Live loads are specified in the applicable building codes. Dead loads are much more variable and require special attention in their computation. Specific requirements for serviceability, strength, lateral stability of individual elements, and the lateral resistance of the building all contribute to the design of a safe and efficient building. The information presented is intended for use in roof framing conditions only without regard to earthquake loading or contributing to lateral resistance of the building.

Bay sizes presented are 30'x30', 30'x40', 40'x30', 40'x40',x40'x50', 50'x40', and 50'x50'. Five typical conditions for live and dead loads are each tabulated. Live loads address both snow and no snow regions. Dead loads address both built-up and ballasted roof systems. Connection design tables are also included.

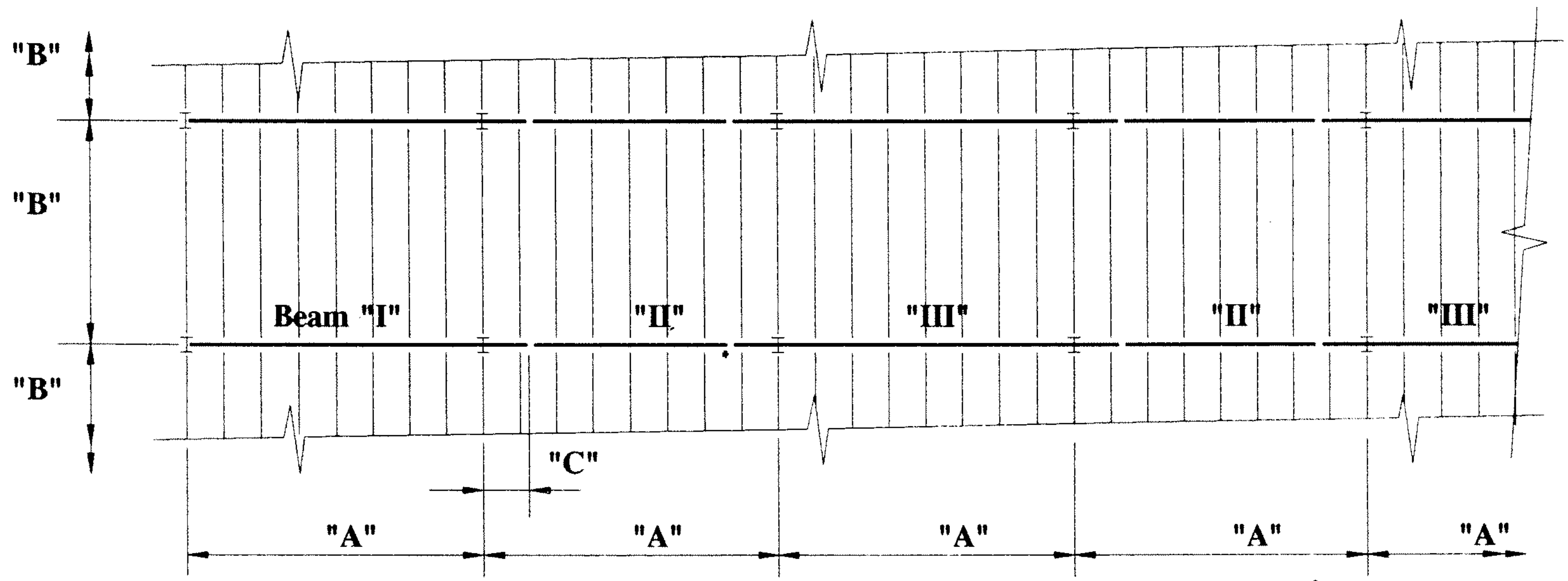
The cantilever and suspended roof girder system design tables which follow are based on the following parameters:

- Load and Resistance Factor Design Specification, December 1, 1993
- Roof loading is uniform on all spans
- Cantilever length selected to provide approximately equal positive and negative moments for a uniformly loaded system
- Column spacing is uniform in each direction
- A "tie joist" is mandatory at each column line. Joist and bottom chord extension are to have sufficient strength and rigidity to provide lateral torsional restraint of the girder
- Joists are uniformly spaced between columns
- Girder webs have been checked for stiffener requirements and noted only if required
- Top of columns are laterally supported by the tie joists/girder
- Columns may be wide flange, pipe, or tube having a rigid cap plate 12" (minimum) in width longitudinal to the girder
- Column design, roof deck selection is not a part of this presentation
- Total load deflection limited to 1/240 of the girder span
- Roof deck/joist system provides lateral support of the girder top flange

Graphically, framing plans indicate joists which are "in-line" across the girders. Not all tabulated member flange widths will allow this condition due to joist bearing criteria. Actual member sizes may be selected with a wider flange or the joists may be staggered for full joist bearing and member economy. Final member selection is the responsibility of the engineer-of-record.

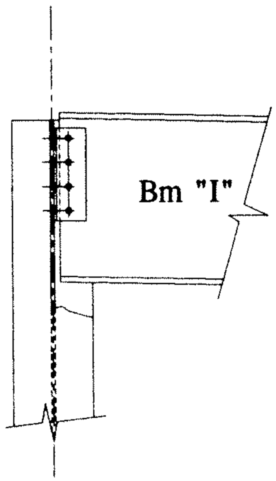
Roof joist selections are included in the design tables to complete the roof framing system. Joist girder design information is tabulated for direct comparison to the alternate cantilever and suspended girder system.

Live Load = 12 psf
Dead Load = 18 psf
Wind Uplift = 14 psf

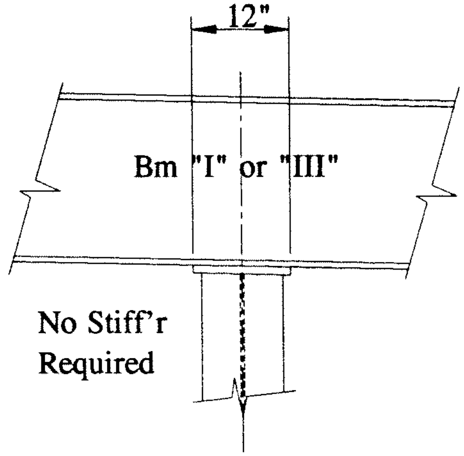


Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Recommended Wide Flange Member Design (50 ksi)			Joist Selection		Joist Girder Required Simple Span Condition	
			Beam "I"	Beam "II"	Beam "III"	Designation	Spacing	Designation	Lbs./Ft.
30'	30'	6'-0"	W16x26	W12x19	W14x22	16K2	5'-0"	20G6N5K	24 ±
30'	40'	6'-0"	W16x31	W12x19	W16x26	20K4	5'-0"	20G6N6.6K	33 ±
40'	30'	6'-0"	W21x44	W16x31	W16x31	16K2	5'-0"	24G8N5K	35 ±
40'	40'	6'-0"	W21x50	W18x40	W18x40	20K4	5'-0"	28G8N6.6K	42 ±
40'	50'	6'-0"	W24x62	W21x44	W21x50	26K6	5'-0"	32G8N8K	43 ±
50'	40'	6'-0"	W24x68	W21x62	W21X62	20K4	5'-0"	32G10N6.8K	68 ±
50'	50'	6'-0"	W24x84	W21x62	W24x62	26K6	5'-0"	36G10N8K	76 ±

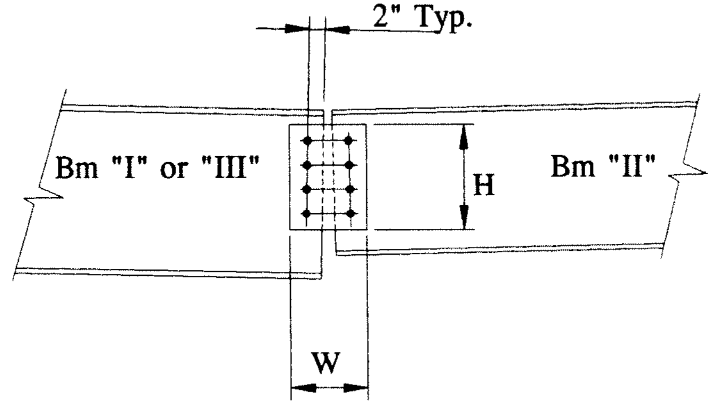
Live Load = 12 psf
Dead Load = 18 psf
Wind Uplift = 14 psf



Exterior Column



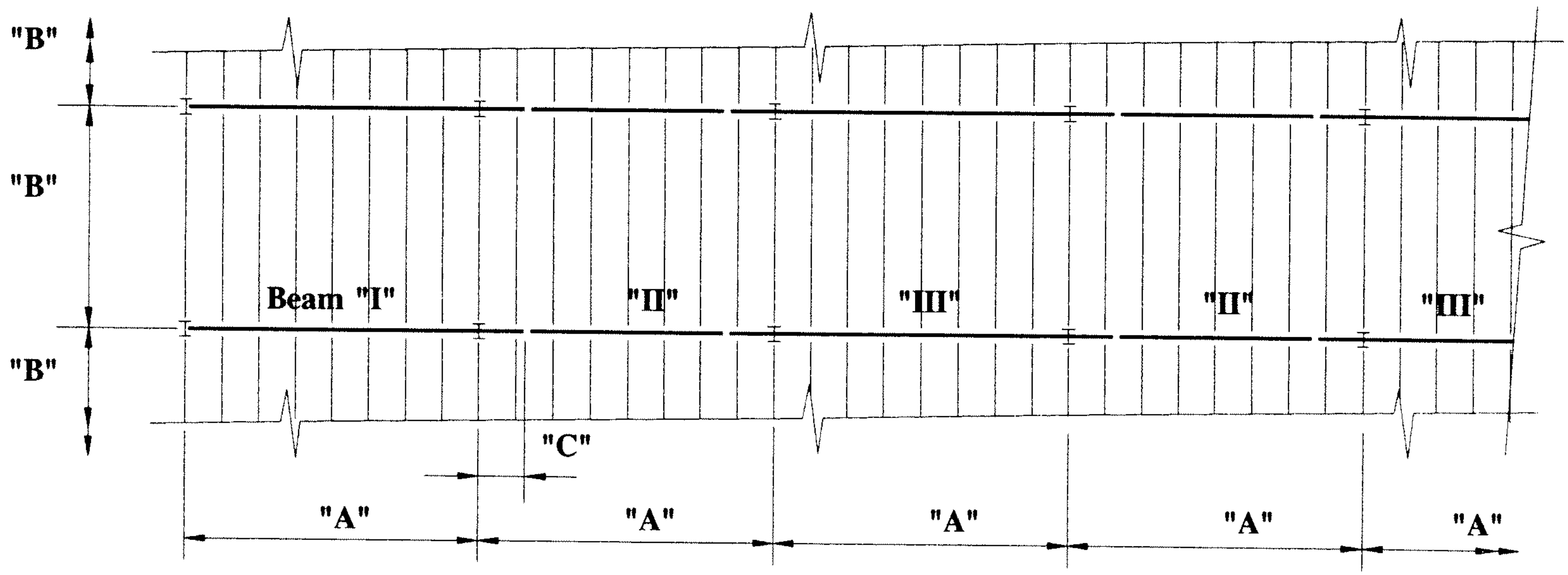
Interior Column



Girder Splice Plates

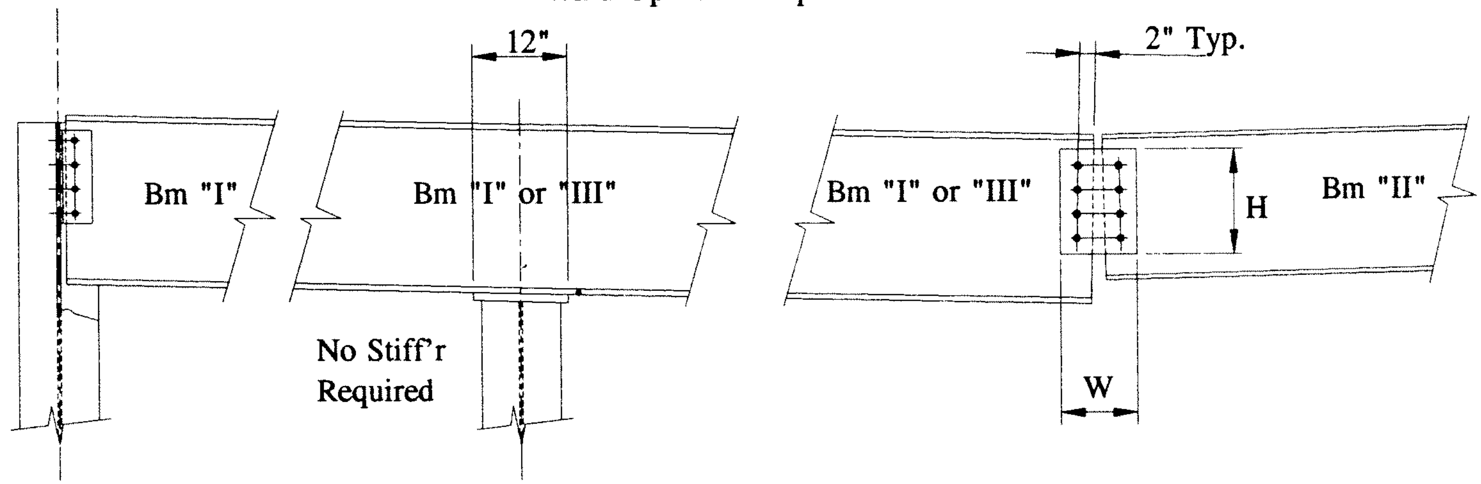
Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Splice Plates - A36 Mat'l - (1 Near Side & 1 Far Side) 3/4"ϕ A325N Bolts		Connection at Exterior Column A36 - Dbl. Angle / 3/4"ϕ A235N Bolts	
			Plate Size t x H x W	Bolts #/Rows @ 3" o.c.	Double Angles Thickness x Height	Bolts #/Rows @ 3" o.c.
30'	30'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
30'	40'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
40'	30'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4

Live Load = 20 psf
Dead Load = 20 psf
Wind Uplift = 14 psf



Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Recommended Wide Flange Member Design (50 ksi)			Joist Selection		Joist Girder Required Simple Span Condition	
			Beam "I"	Beam "II"	Beam "III"	Designation	Spacing	Designation	Lbs./Ft.
30'	30'	6'-0"	W16x31	W14x22	W16x26	18K3	5'-0"	20G6N6.1K	29 ±
30'	40'	6'-0"	W16x36	W14x22	W16x31	20K7	5'-0"	24G6N8.2K	33 ±
40'	30'	6'-0"	W21x50	W16x31	W18x35	18K3	5'-0"	28G8N6.1K	37 ±
40'	40'	6'-0"	W24x55	W18x40	W21x44	20K7	5'-0"	32G8N8.2K	45 ±
40'	50'	6'-0"	W24x62	W21x50	W21x50	26K9	5'-0"	36G8N10.2K	50 ±
50'	40'	6'-0"	W27x84	W24x55	W24X68	20K7	5'-0"	40G10N8.2K	58 ±
50'	50'	6'-0"	W27x94	W24x62	W24x76	26K9	5'-0"	44G10N10.2K	67 ±

Live Load = 20 psf
Dead Load = 20 psf
Wind Uplift = 14 psf



Exterior Column **Interior Column** **Girder Splice Plates**

Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Splice Plates - A36 Mat'l - (1 Near Side & 1 Far Side) 3/4"ϕ A325N Bolts		Connection at Exterior Column A36 - Dbl. Angle / 3/4"ϕ A235N Bolts	
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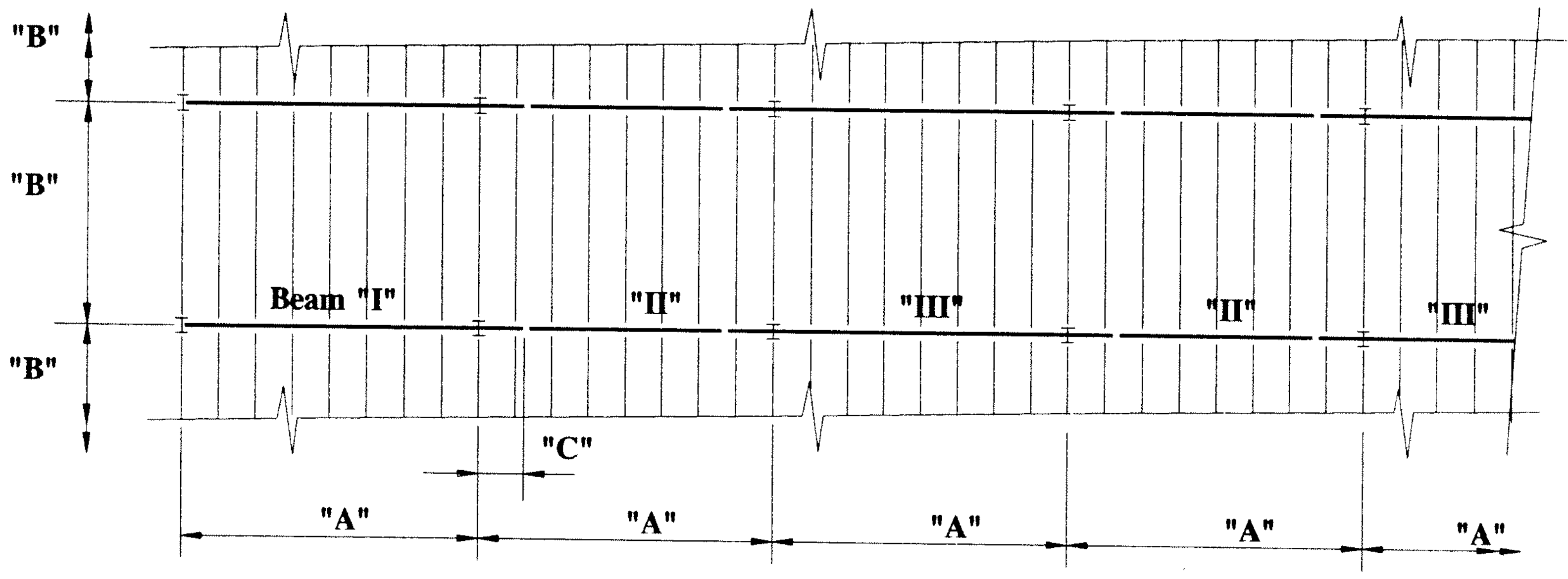
30'	30'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
30'	40'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
40'	30'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	40'	6'-0"	5/16" x 13" x 0'-9"	4	1/4" x 0'-11 1/2"	4
50'	50'	6'-0"	5/16" x 13" x 0'-9"	4	1/4" x 0'-11 1/2"	4

Live Load = 30 psf
Dead Load = 18 psf
Wind Uplift = 14 psf



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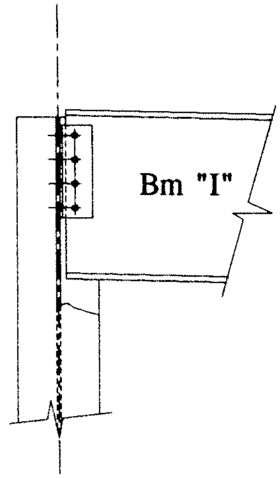
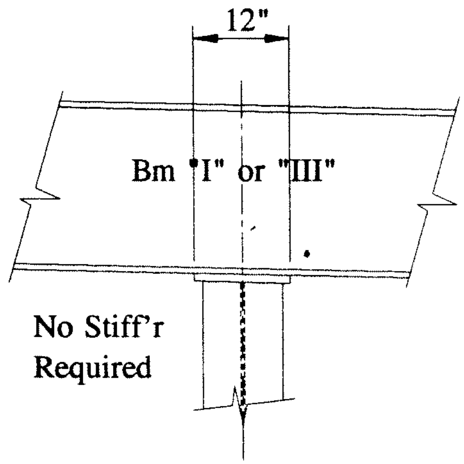
			Beam "I"	Beam "II"	Beam "III"	Designation	Spacing	Designation	Lbs./Ft.
30'	30'	6'-0"	W16x31	W14x22	W16x26	20K4	5'-0"	24G6N7.7K	32 ±
30'	40'	6'-0"	W18x35	W16x26	W18x35	24K7	5'-0"	24G6N10K	39 ±
40'	30'	6'-0"	W24x55	W18x35	W18x40	20K4	5'-0"	28G8N8K	47 ±
40'	40'	6'-0"	W24x62	W18x40	W21x50	24K7	5'-0"	32G8N10K	51 ±
40'	50'	6'-0"	W24x68	W21x50	W24x55	26K10	5'-0"	40G8N13K	56 ±
50'	40'	6'-0"	W27x84	W24x68	W24x68	24K7	5'-0"	40G10N10.1K	69 ±
50'	50'	6'-0"	W30x90	W24x76	W24x84	26K10	5'-0"	48G10N13K	77 ±

Live Load = 30 psf
Dead Load = 18 psf
Wind Uplift = 14 psf



Exterior Column



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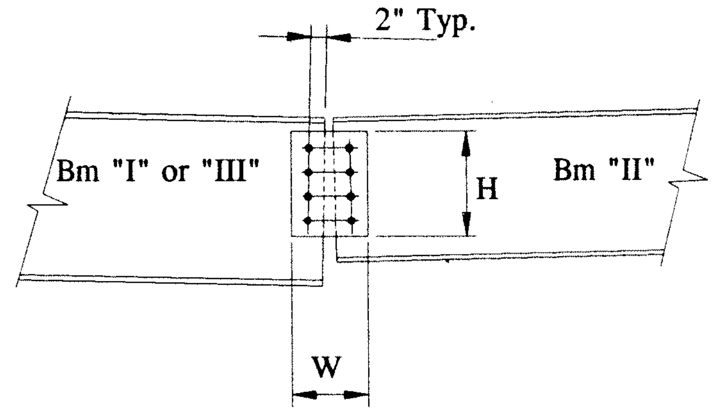

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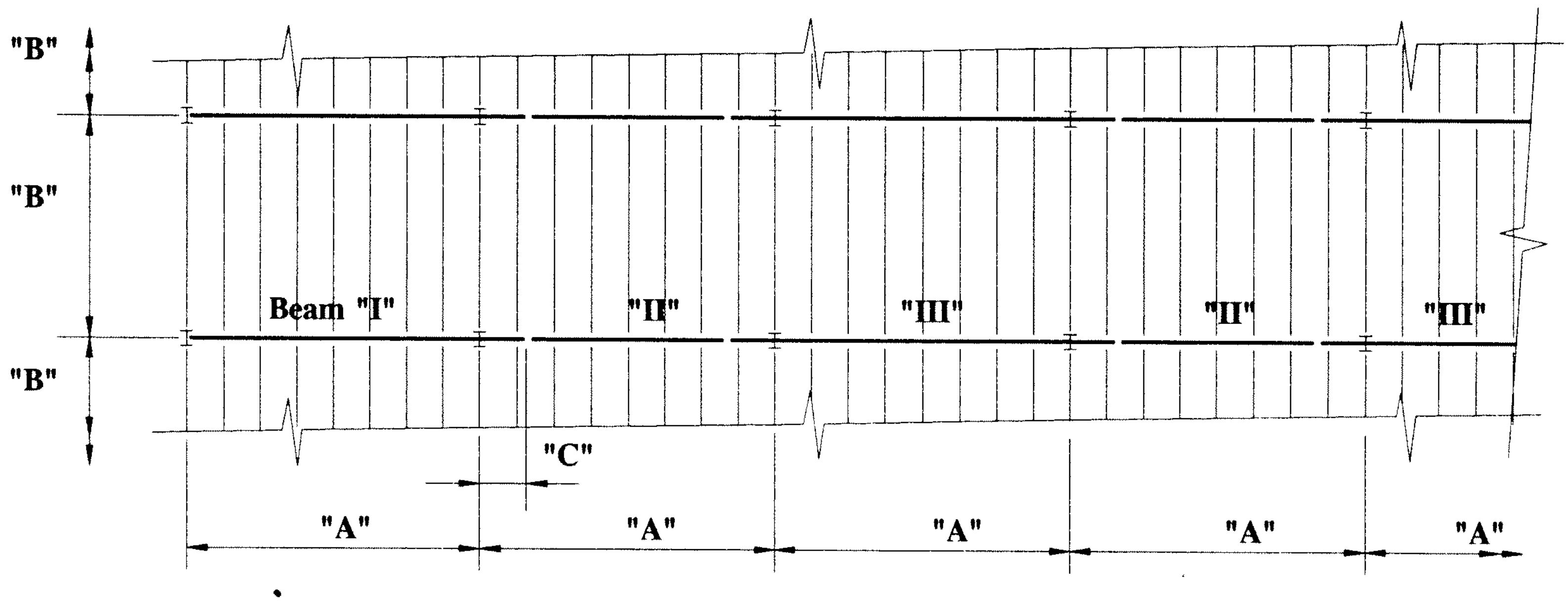
30'	30'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
30'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	30'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	50'	6'-0"	5/16" x 13" x 0'-9"	4	1/4" x 0'-11 1/2"	4

Live Load = 12 psf

Dead Load = 35 psf



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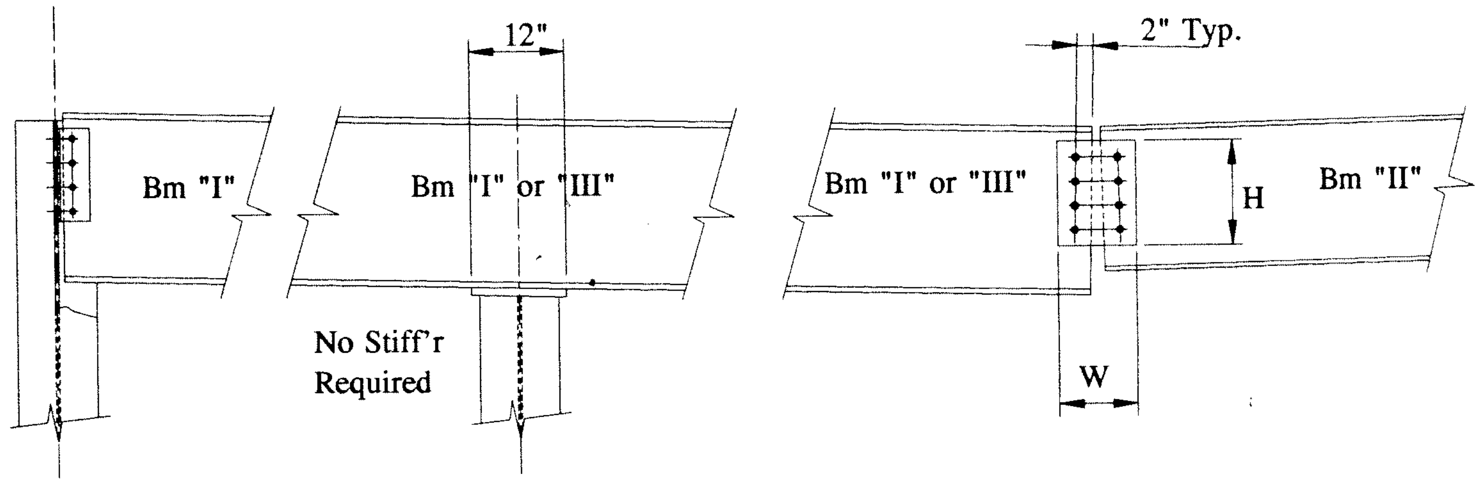
			Beam "I"	Beam "II"	Beam "III"	Designation	Spacing	Designation	Lbs./Ft.
30'	30'	6'-0"	W16x31	W14x22	W16x26	18K4	5'-0"	28G6N7.1K	26 ±
30'	40'	6'-0"	W18x35	W16x26	W18x35	24K7	5'-0"	28G6N9.5K	35 ±
40'	30'	6'-0"	W24x55	W18x35	W18x40	18K4	5'-0"	32G8N7.1K	37 ±
40'	40'	6'-0"	W24x62	W18x40	W21x50	24K7	5'-0"	36G8N9.5K	48 ±
40'	50'	6'-0"	W24x68	W21x50	W24x55	26K10	5'-0"	40G8N12K	52 ±
50'	40'	6'-0"	W27x84	W24x68	W24x68	24K7	5'-0"	44G10N9.5K	65 ±
50'	50'	6'-0"	W30x90	W24x76	W24x84	26K10	5'-0"	52G10N12K	68 ±

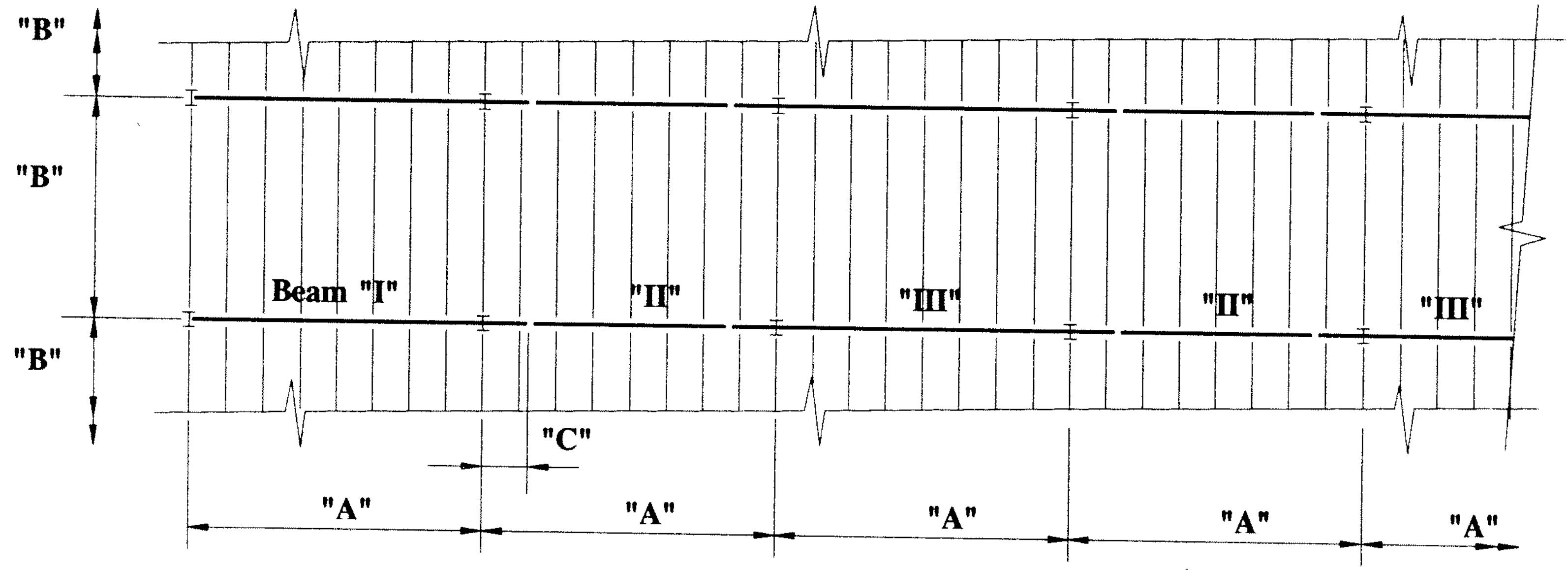
Live Load = 12 psf
Dead Load = 35 psf



Exterior Column **Interior Column** **Girder Splice Plates**

Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Splice Plates - A36 Mat'l - (1 Near Side & 1 Far Side) 3/4"ϕ A325N Bolts		Connection at Exterior Column A36 - Dbl. Angle / 3/4"ϕ A235N Bolts	
			Plate Size t x H x W	Bolts #/Rows @ 3" o.c.	Double Angles Thickness x Height	Bolts #/Rows @ 3" o.c.
30'	30'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
30'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	30'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	50'	6'-0"	5/16" x 13" x 0'-9"	4	1/4" x 0'-11 1/2"	4

Live Load = 30 psf
Dead Load = 35 psf



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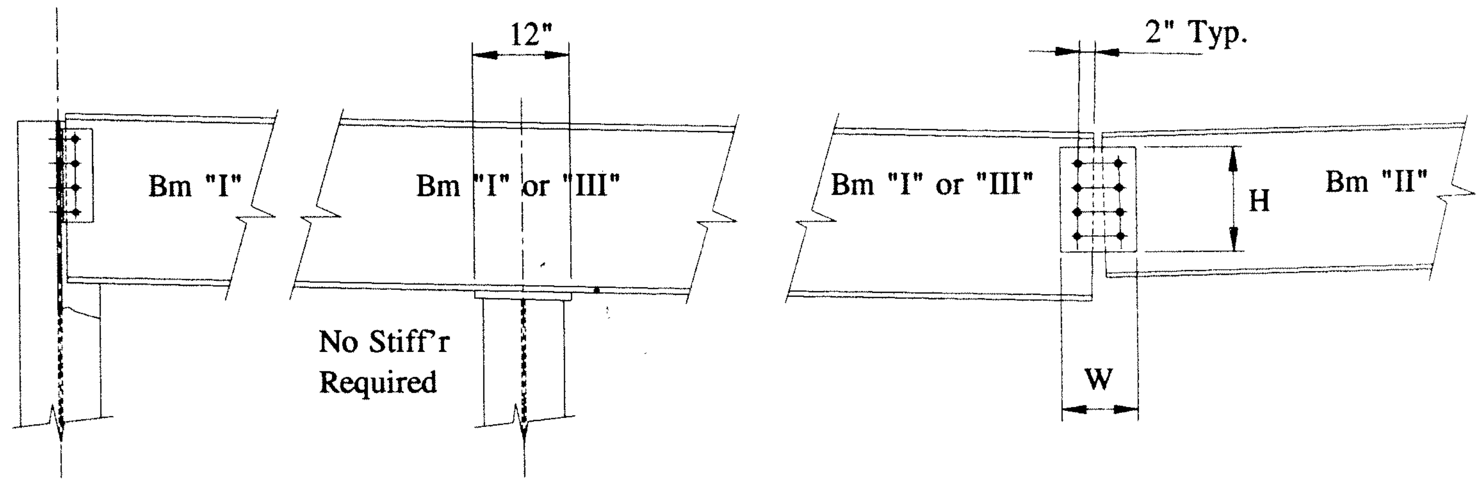
			Beam "I"	Beam "II"	Beam "III"	Designation	Spacing	Designation	Lbs./Ft.
30'	30'	6'-0"	W18x35	W16x31	W18x35	24K4	5'-0"	28G6N10K	35 ±
30'	40'	6'-0"	W21x44	W18x35	W18x40	26K9	5'-0"	32G6N13.2K	42 ±
40'	30'	6'-0"	W24x62	W21x44	W21x44	24K4	5'-0"	32G8N10K	51 ±
40'	40'	6'-0"	W24x68	W21x50	W24x55	26K9	5'-0"	32G8N13.2K	66 ±
40'	50'	6'-0"	W27x84	W24x62	W24x68	32LH6	5'-0"	40G8N16.4K	77 ±
50'	40'	6'-0"	W30x99	W24x76	W27X84	26K9	5'-0"	52G10N13.2K	75 ±
50'	50'	6'-0"	W30x116	W27x84	W30x99	32LH6	5'-0"	60G10N16.4K	79 ±

Live Load = 30 psf
Dead Load = 35 psf



Exterior Column **Interior Column** **Girder Splice Plates**

Girder Span "A"	Joist Span "B"	Splice Dim. "C"	Splice Plates - A36 Mat'l - (1 Near Side & 1 Far Side) 3/4"ϕ A325N Bolts		Connection at Exterior Column A36 - Dbl. Angle / 3/4"ϕ A235N Bolts	
			Plate Size t x H x W	Bolts #/Rows @ 3" o.c.	Double Angles Thickness x Height	Bolts #/Rows @ 3" o.c.
30'	30'	6'-0"	5/16" x 7" x 0'-9"	2	1/4" x 0'-8 1/2"	3
30'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-8 1/2"	3
40'	30'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	40'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
40'	50'	6'-0"	5/16" x 10" x 0'-9"	3	1/4" x 0'-11 1/2"	4
50'	40'	6'-0"	3/8" x 13" x 0'-9"	4	5/16" x 1'-2 1/2"	5
50'	50'	6'-0"	3/8" x 13" x 0'-9"	4	5/16" x 1'-2 1/2"	5

LRFD Cantilever & Suspended Roof Girder System

DESIGN EXAMPLES

Member Design:

Given:

Girder span = 40 ft.
Joist span = 30 ft. @ 5 ft. spacing
F_y = 50 ksi
Live Load = 12 psf
Dead Load = 18 psf
Wind Uplift = 14 psf

Solution:

Calculate factored loading: Live load = 12 x 5 = 60 plf
Dead load = 18 x 5 = 90 plf
Wind Uplift = 14 x 5 = 70 plf
Min. dead load (excl. HVAC, Elec., etc.) = 8 x 5 = 40 plf
(self-weight of girder included in the computer analysis)

Load Combination I (1.2D + 1.6L): (LRFD Spec. Sect. A4.1)
Factored loading = 1.2(90) + 1.6(60) = 204 plf on joists
Point loads on Girder = .204 x 30 = 6.12 kips ↓

Load Combination II (.9D + 1.3W):
Factored loading = .9(40) - 1.3(70) = -55 plf
Point loads on Girder = 0.055 x 30 = 1.65 kips ↑

Member I Design:

Load Combination I

From computer analysis, $+M_{u,max}$ = 207 kip-ft, L_u = 5 ft and $-M_{u,max}$ = 138 kip-ft, L_u = 6 ft. From the *LRFD Manual Vol. I*, Load Factor Design Selection Table, W18x35 has ϕM_p = 249 kip-ft with L_p = 4.3 ft, BF = 10.7 kips. By inspection, the positive moment will control. Find capacity of W18x35 for L_u = 5 ft.

$$\phi M_n = 249 - 10.7(5 - 4.3) = 241.5 \text{ kip-ft} > 207 \text{ kip-ft} \quad \textbf{o.k.}$$

Total service load deflection exceeds $L/240$ for W18x35, therefore use W18x40.

Load Combination II

From computer analysis, $-M_{u,max}$ = 46.2 kip-ft, L_u = 35 ft. From the Load Factor Design Selection Table, for W18x40, ϕM_p = 294 kip-ft, L_p = 4.5 ft., L_r = 12.1 ft. Since $L_u \geq L_r$, calculate ϕM_n from LRFD Spec. Eqn. (F1-13) with I_y = 19.1 in.^{4}, J = 0.81 in.^{4}, C_w = 1440 in.^{6}, assume C_b = 1:

$$\phi_b M_n = \frac{0.9\pi}{35x12}\sqrt{29000(19.1)(11200)(0.81) + \left[\frac{\pi 29000}{35x12}\right]^2 (19.1)(1440)}$$

$= 44.6 < 46.2$ kip-ft **n.g.**

Use: W21x44 for Member I

Member II Design:

Load Combination I
From computer analysis, $+M_{u,max} = 138$ kip-ft and $L_u = 5$ ft. From Load Factor Design Selection Table, the W12x26 with $\phi M_p = 140$ kip-ft and $L_p = 5.3$ ft. is **o.k.**

Total service load deflection exceeds $L/240$ for W12x26 and W16x26, therefore use W16x31.

Load Combination II
From computer analysis, $-M_{u,max} = 30.8$ kip-ft and $L_u = 28$ ft $> L_r = 11.0$ ft for W16x31. Check W16x31 using LRFD Spec. Eqn. (F1-13): $\phi_b M_n = 35.2 > 30.8$ kip-ft **o.k.**

Use: W16x31 for Member II

Member III Design:

Load Combination I
From computer analysis, $+M_{u,max} = 138$ kip-ft, $L_u = 5$ ft and $-M_{u,max} = 138$ kip-ft, $L_u = 8.33$ ft. From the Load Factor Design Selection Table, for a W14x30, $\phi_b M_p = 177$ kip-ft, $L_p = 5.3$ ft, $L_r = 13.7$ ft and $BF = 6.06$ kips. Negative moment controls and $L_p < 8.33 < L_r$, therefore

$$\phi M_n = 177 - 6.06(8.33 - 5.3) = 158.6 > 138 \text{ kip-ft} \quad \textbf{o.k.}$$

Total service load deflection exceeds $L/240$ for W14x30, therefore use W16x31.

Load Combination II
From computer analysis, $-M_{u,max} = 30.8$ kip-ft, $L_u = 28.75$ ft. From Member II Design, $\phi_b M_n = 35.2$ kip-ft > 30.8 **o.k.**

Use: W16x31 for Member III

Splice Connection Design:

Given:

l_v = 2 inches - on all elements
l_h = 2 inches - on all elements
1"± maximum between member ends
5/16" minimum plate thickness
ASTM A36 plate material, F_y = 36 ksi, F_u = 58 ksi
3" bolt spacing
3/4"ϕ A325N bolts
Minimum connection depth T/2 of connected members
Minimum 2 bolt connection
Bolted / Bolted design
R_u = 16 kips (from computer analysis)
W21x44 cantilevered member
W16x31 suspended member

Solution:

Check Bolts:

Minimum connection plate depth to meet T/2 criteria = 9 1/8"±
Minimum 3 bolt connection

ϕr_v = 31.8 Kips / Bolt, Double Shear

Eccentrically Loaded Bolt Group (*LRFD Manual, Volume II*, Table 8 - 18):

$e_x = e/2 = 5/2 = 2.5$ inches
n = 3 (first trial)
C = 1.99

$\phi R_n = 1.99 \times 31.8 = 63.28$ >> 16 kips **o.k.**

Bearing on W16x31 web material (*LRFD Manual, Volume II*, Table 8 - 13):

$\phi R_n = C \times (2.4dtF_u) = 1.99 \times (2.4 \times 0.75 \times 0.275 \times 65)$

$\phi R_n = 64.03$ kips > 16 kips **o.k.**

Shear on W16x31 (*LRFD Specification*, Equation F2-1):

$\phi R_n = \phi 0.6 F_y A_w = 0.90 \times 0.6 \times 50 \times (15.88 \times 0.275)$

$\phi R_n = 117.9$ kips >> 16 kips **o.k.**

Net shear on splice plates (*LRFD Specification*, Equation J4-1):

Try splice plates 5/16" x 10"

$\phi R_n = \phi 0.6 F_u A_n = 0.75 \times 0.6 \times 58 \times ((10-(3 \times .8125)) \times 2 \times .3125)$

$\phi R_n = 123.36$ kips >> 16 kips **o.k.**

Gross shear on splice plates (*LRFD Specification*, Equation F2-1):

$\phi R_n = \phi 0.6 F_y A_g = 0.90 \times 0.6 \times 36 \times (10 \times 2 \times .3125)$

$\phi R_n = 121.50$ kips >> 16 kips **o.k.**

Block Shear Rupture on splice plates (*LRFD Manual, Volume II*, Table 8 - 47a,b & Table 8 - 48a,b):

Table 8 - 47a coefficient = 68
Table 8 - 47b coeffecient = 137
$\phi R_n = (68 + 137) \times 2 \times 0.3125 = 128.13$ kips

Table 8 - 48a coefficient = 152
Table 8 - 48b coeffecient = 54
$\phi R_n = (152 + 54) \times 2 \times 0.3125 = 128.75$ kips **o.k.**

Flexural Yield on splice plates (*LRFD Specification*, Chapter F):

$M_u = R_u e/2 = 2.5 \times 16 = 40$ kip-in
$S_x = (t \times H^2) / 6 = (100 \times 2 \times .3125) / 6 = 10.42 \text{ in}^3$

$\phi M_n = \phi F_y S_x = 0.90 \times 36 \times 10.42$

$\phi M_n = 337.50$ kip-in >> 40 kip-in **o.k.**

Flexural Rupture on splice plates (*LRFD Manual, Volume II*, Table 12 - 1):

$S_n = 6.25 \text{ in}^3$ from Table 12 - 1 (conservative by H = 9" in table)

$\phi M_n = \phi F_u S_n = 0.75 \times 58 \times 6.25$

$\phi M_n = 271.87$ kip-in >> 40 kip-in **o.k.**

Double Angle Connection at Exterior Column:

Given:

l_v = 1 1/4 inches - on connecting angles
l_h = 1 1/2 inches - minimum
1/4" minimum connection angles
ASTM A36 plate material, F_y = 36 ksi, F_u = 58 ksi
3" bolt spacing
3/4"ϕ A325N bolts
Minimum connection depth T/2 of connected members
Minimum 2 bolt connection
Bolted / Bolted design
R_u = 25 kips (from computer analysis)
W21x44 member

Solution:

Double angles (*LRFD Manual, Volume II*, Table 9 - 2):

11 1/2" long connection required to meet T/2 criteria

2 - 1/4" angles with 4 rows 3/4"ϕ A325N bolts

ϕR_n = 104 kips >> 25 kips **o.k.**

Shear on W21x44 (*LRFD Specification*, Equation F2-1):

$\phi R_n = \phi 0.6 F_y A_w = 0.90 \times 0.6 \times 50 \times (20.66 \times 0.350)$

ϕR_n = 195.23 kips >> 25 kips **o.k.**

Bearing on W21x44 web material (*LRFD Manual, Volume II*, Table 8 - 13):

$\phi R_n = 87.8 \times t \times n = 87.8 \times 0.35 \times 4$

ϕR_n = 122.92 kips > 25 kips **o.k.**