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’, 50’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**

<table>
<thead>
<tr>
<th>Girder Span</th>
<th>Joist Span</th>
<th>Splice Dim.</th>
<th>Recommended Wide Flange Member Design (50 ksi)</th>
<th>Joist Selection</th>
<th>Joist Girder Required Simple Span Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;A&quot;</td>
<td>&quot;B&quot;</td>
<td>&quot;C&quot;</td>
<td><strong>Beam &quot;I&quot;</strong></td>
<td><strong>Beam &quot;II&quot;</strong></td>
<td><strong>Beam &quot;III&quot;</strong></td>
</tr>
<tr>
<td>30'</td>
<td>30'</td>
<td>6'-0&quot;</td>
<td>W16x26</td>
<td>W12x19</td>
<td>W14x22</td>
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<td>W24x68</td>
<td>W21x62</td>
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<tr>
<td>50'</td>
<td>50'</td>
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<td>W24x84</td>
<td>W21x62</td>
<td>W24x62</td>
</tr>
</tbody>
</table>
Live Load = 12 psf  
Dead Load = 18 psf  
Wind Uplift = 14 psf

**Exterior Column**

**Interior Column**

**Girder Splice Plates**

<table>
<thead>
<tr>
<th>Girder Span</th>
<th>Joist Span</th>
<th>Splice Dim.</th>
<th>Splice Plates - A36 Mat'1 - (1 Near Side &amp; 1 Far Side)</th>
<th>Connection at Exterior Column</th>
<th>Connection at Exterior Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>30'</td>
<td>30'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 7&quot; x 0'-9&quot;</td>
<td>1/4&quot; x 0'-8 1/2&quot;</td>
<td>1/4&quot; x 0'-8 1/2&quot;</td>
</tr>
<tr>
<td>30'</td>
<td>40'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 7&quot; x 0'-9&quot;</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>40'</td>
<td>30'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 10&quot; x 0'-9&quot;</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>40'</td>
<td>40'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 10&quot; x 0'-9&quot;</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>40'</td>
<td>50'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 10&quot; x 0'-9&quot;</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>50'</td>
<td>40'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 10&quot; x 0'-9&quot;</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>50'</td>
<td>50'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 10&quot; x 0'-9&quot;</td>
<td>3</td>
<td>3</td>
</tr>
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</table>
**Live Load = 20 psf**

**Dead Load = 20 psf**

**Wind Uplift = 14 psf**

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<table>
<thead>
<tr>
<th>Girder Span</th>
<th>Joist Span</th>
<th>Splice Dim.</th>
<th>Wide Flange Member Design (50 ksi)</th>
<th>Joist Selection</th>
<th>Joist Girder Required Simple Span Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;A&quot;</td>
<td>&quot;B&quot;</td>
<td>&quot;C&quot;</td>
<td>Beam &quot;I&quot;</td>
<td>Beam &quot;II&quot;</td>
<td>Beam &quot;III&quot;</td>
</tr>
<tr>
<td>30'</td>
<td>30'</td>
<td>6'-0&quot;</td>
<td>W16x31</td>
<td>W14x22</td>
<td>W16x26</td>
</tr>
<tr>
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<td>40'</td>
<td>6'-0&quot;</td>
<td>W16x36</td>
<td>W14x22</td>
<td>W16x31</td>
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<tr>
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<td>30'</td>
<td>6'-0&quot;</td>
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<td>W18x35</td>
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<tr>
<td>40'</td>
<td>40'</td>
<td>6'-0&quot;</td>
<td>W24x55</td>
<td>W18x40</td>
<td>W21x44</td>
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<tr>
<td>40'</td>
<td>50'</td>
<td>6'-0&quot;</td>
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<td>W21x50</td>
<td>W21x50</td>
</tr>
<tr>
<td>50'</td>
<td>40'</td>
<td>6'-0&quot;</td>
<td>W27x84</td>
<td>W24x55</td>
<td>W24X68</td>
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<td>50'</td>
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<td>W24x62</td>
<td>W24x76</td>
</tr>
</tbody>
</table>
Live Load = 20 psf
Dead Load = 20 psf
Wind Uplift = 14 psf

**Exterior Column**

**Interior Column**

**Girder Splice Plates**

<table>
<thead>
<tr>
<th>Girder Span</th>
<th>Joist Span</th>
<th>Splice Dim.</th>
<th>Splice Plates - A36 Mat'l - (1 Near Side &amp; 1 Far Side)</th>
<th>Connection at Exterior Column</th>
<th>Connection at Exterior Column</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;A&quot;</td>
<td>&quot;B&quot;</td>
<td>&quot;C&quot;</td>
<td>3/4&quot;ϕ A325N Bolts</td>
<td>A36 - Dbl. Angle / 3/4&quot;ϕ A235N Bolts</td>
<td></td>
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<td>30'</td>
<td>6'-0&quot;</td>
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<td>#/Rows @ 3&quot; o.c.</td>
<td>Double Angles Thickness x Height</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>5/16&quot; x 7&quot; x 0'-9&quot;</td>
<td>2</td>
<td>1/4&quot; x 0'-8 1/2&quot;</td>
</tr>
<tr>
<td>30'</td>
<td>40'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 7&quot; x 0'-9&quot;</td>
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<td>1/4&quot; x 0'-11 1/2&quot;</td>
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<tr>
<td>50'</td>
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<td>6'-0&quot;</td>
<td>5/16&quot; x 13&quot; x 0'-9&quot;</td>
<td>4</td>
<td>1/4&quot; x 0'-11 1/2&quot;</td>
</tr>
<tr>
<td>50'</td>
<td>50'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 13&quot; x 0'-9&quot;</td>
<td>4</td>
<td>1/4&quot; x 0'-11 1/2&quot;</td>
</tr>
</tbody>
</table>
Live Load = 30 psf
Dead Load = 18 psf
Wind Uplift = 14 psf

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<th>Girder Span</th>
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<tbody>
<tr>
<td>&quot;A&quot;</td>
<td>&quot;A&quot;</td>
<td>&quot;C&quot;</td>
<td>Beam &quot;I&quot; Beam &quot;II&quot; Beam &quot;III&quot;</td>
<td>Designation</td>
<td>Spacing</td>
</tr>
<tr>
<td>30'</td>
<td>30'</td>
<td>6'-0&quot;</td>
<td>W16x31 W14x22 W16x26</td>
<td>20K4</td>
<td>5'-0&quot;</td>
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<tr>
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<td>40'</td>
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<tr>
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<td>40'</td>
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<td>24K7</td>
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<td>W30x90 W24x76 W24x84</td>
<td>26K10</td>
<td>5'-0&quot;</td>
</tr>
</tbody>
</table>
Live Load = 30 psf  
Dead Load = 18 psf  
Wind Uplift = 14 psf

### Exterior Column

- Bm "I" or "III"  
- 12"  
- No Stiff'r Required

### Interior Column

- Bm "I" or "III"  
- 2" Typ.

### Girder Splice Plates

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;A&quot;</td>
<td>&quot;B&quot;</td>
<td>&quot;C&quot;</td>
<td>Plate Size t x H x W</td>
<td>Bolts #/Rows @ 3&quot; o.c.</td>
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<tr>
<td>30'</td>
<td>30'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 7&quot; x 0'-9&quot;</td>
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<td>4</td>
</tr>
</tbody>
</table>
**Live Load = 12 psf**
Dead Load = 35 psf

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<th>Girder Span</th>
<th>Joist Span</th>
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<td>Beam &quot;III&quot;</td>
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<tr>
<td>30'</td>
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<td>6'-0&quot;</td>
<td>W16x31</td>
<td>W14x22</td>
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<td>30'</td>
<td>40'</td>
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<td>W24x62</td>
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<td>50'</td>
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<td>W30x90</td>
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<td>W24x84</td>
</tr>
</tbody>
</table>
Live Load = 12 psf  
Dead Load = 35 psf

Exterior Column  
Interior Column  
Girder Splice Plates

<table>
<thead>
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<td>&quot;A&quot;</td>
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<td>&quot;C&quot;</td>
<td>Plate Size t x H x W</td>
<td>3/4&quot;Ø A325N Bolts</td>
<td>Connection at Exterior Column A36 - Dbl. Angle / 3/4&quot;Ø A235N Bolts</td>
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<tr>
<td>30'</td>
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</tr>
<tr>
<td>50'</td>
<td>40'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 10&quot; x 0'-9&quot;</td>
<td>3</td>
<td>1/4&quot; x 0'-11 1/2&quot;</td>
</tr>
<tr>
<td>50'</td>
<td>50'</td>
<td>6'-0&quot;</td>
<td>5/16&quot; x 13&quot; x 0'-9&quot;</td>
<td>4</td>
<td>1/4&quot; x 0'-11 1/2&quot;</td>
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</tbody>
</table>
Live Load = 30 psf
Dead Load = 35 psf

<table>
<thead>
<tr>
<th>Girder Span</th>
<th>Joist Span</th>
<th>Splice Dim.</th>
<th>Recommended Wide Flange Member Design (50 ksi)</th>
<th>Joist Selection</th>
<th>Joist Girder Required Simple Span Condition</th>
</tr>
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<tbody>
<tr>
<td>&quot;A&quot;</td>
<td>&quot;B&quot;</td>
<td>&quot;C&quot;</td>
<td>Beam &quot;I&quot; Beam &quot;II&quot; Beam &quot;III&quot;</td>
<td>Designation</td>
<td>Spacing</td>
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<tr>
<td>30'</td>
<td>30'</td>
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<td>W18x35 W16x31 W18x35</td>
<td>24K4</td>
<td>5'-0&quot;</td>
</tr>
<tr>
<td>30'</td>
<td>40'</td>
<td>6'-0&quot;</td>
<td>W21x44 W18x35 W18x40</td>
<td>26K9</td>
<td>5'-0&quot;</td>
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<tr>
<td>40'</td>
<td>30'</td>
<td>6'-0&quot;</td>
<td>W24x62 W21x44 W21x44</td>
<td>24K4</td>
<td>5'-0&quot;</td>
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<tr>
<td>40'</td>
<td>40'</td>
<td>6'-0&quot;</td>
<td>W24x68 W21x50 W24x55</td>
<td>26K9</td>
<td>5'-0&quot;</td>
</tr>
<tr>
<td>40'</td>
<td>50'</td>
<td>6'-0&quot;</td>
<td>W27x84 W24x62 W24x68</td>
<td>32LH6</td>
<td>5'-0&quot;</td>
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<tr>
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<td>6'-0&quot;</td>
<td>W30x99 W24x76 W27X84</td>
<td>26K9</td>
<td>5'-0&quot;</td>
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<tr>
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<td>W30x116 W27x84 W30x99</td>
<td>32LH6</td>
<td>5'-0&quot;</td>
</tr>
</tbody>
</table>
Live Load = 30 psf
Dead Load = 35 psf

Exterior Column

Interior Column

Girder Splice Plates

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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<th></th>
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<tbody>
<tr>
<td>&quot;A&quot;</td>
<td>&quot;B&quot;</td>
<td>&quot;C&quot;</td>
<td>Plate Size t x H x W</td>
<td>Bolts #/Rows @ 3&quot; o.c.</td>
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<td>6'-0&quot;</td>
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<td>3</td>
</tr>
<tr>
<td>40'</td>
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<td>3</td>
</tr>
<tr>
<td>50'</td>
<td>40'</td>
<td>6'-0&quot;</td>
<td>3/8&quot; x 13&quot; x 0'-9&quot;</td>
<td>4</td>
</tr>
<tr>
<td>50'</td>
<td>50'</td>
<td>6'-0&quot;</td>
<td>3/8&quot; x 13&quot; x 0'-9&quot;</td>
<td>4</td>
</tr>
</tbody>
</table>
LRFD Cantilever & Suspended Roof Girder System

DESIGN EXAMPLES

Member Design:

Given:
- Girder span = 40 ft.
- Joist span = 30 ft. @ 5 ft. spacing
- \( F_y = 50 \text{ ksi} \)
- 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, \( +M_{u,max} = 207 \text{ kip-ft, } L_u = 5 \text{ ft} \) and \( -M_{u,max} = 138 \text{ kip-ft, } L_u = 6 \text{ ft} \).
From the LRFD Manual Vol. I, Load Factor Design Selection Table, W18x35 has \( \phi M_p = 249 \text{ kip-ft with } L_p = 4.3 \text{ ft, } BF = 10.7 \text{ kips. By inspection, the positive moment will control. Find capacity of W18x35 for } L_u = 5 \text{ ft.} \)

\[
\phi M_a = 249 - 10.7(5 - 4.3) = 241.5 \text{ kip-ft} > 207 \text{ kip-ft o.k.}
\]

Total service load deflection exceeds \( L/240 \) for W18x35, therefore use W18x40.

Load Combination II
From computer analysis, \( -M_{u,max} = 46.2 \text{ kip-ft, } L_u = 35 \text{ ft.} \) From the Load Factor Design Selection Table, for W18x40, \( \phi M_p = 294 \text{ kip-ft, } L_p = 4.5 \text{ ft, } L_r = 12.1 \text{ ft. Since } L_u \geq L_r, \) calculate \( \phi M_a \) from LRFD Spec. Eqn. (F1-13) with \( I_y = 19.1 \text{ in.}^4, J = 0.81 \text{ in.}^4, C_w = 1440 \text{ in.}^6, \) assume \( C_b = 1: \)
\[ \phi_b M_n = \frac{0.9 \pi}{35 \times 12} \sqrt{29000(19.1)(11200)(0.81) + \left( \frac{\pi 29000}{35 \times 12} \right)^2 (19.1)(1440)} \]

\[ = 44.6 < 46.2 \text{ kip-ft n.g.} \]

Use: W21x44 for Member I

**Member II Design:**

**Load Combination I**
From computer analysis, \(+M_{u,max} = 138 \text{ kip-ft and } L_u = 5 \text{ ft. From Load Factor Design Selection Table, the W12x26 with } \phi M_p = 140 \text{ kip-ft and } L_p = 5.3 \text{ 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 \text{ kip-ft and } L_u = 28 \text{ ft } > L_r = 11.0 \text{ ft for W16x31. Check W16x31 using LRFD Spec. Eqn. (F1-13): } \phi_b M_n = 35.2 > 30.8 \text{ kip-ft o.k.}**

Use: W16x31 for Member II

**Member III Design:**

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

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

Total service load deflection exceeds \(L/240\) for W14x30, therefore use W16x31.

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

\( \phi r_v = 31.8 \) Kips / Bolt, Double Shear

Eccentrically Loaded Bolt Group (*LRFD Manual, Volume II*, Table 8 - 18):
- \( e_s = e/2 = 5/2 = 2.5 \) inches
- \( n = 3 \) (first trial)
- \( C = 1.99 \)

\( \phi R_u = 1.99 \times 31.8 = 63.28 > > 16 \) kips o.k.

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

\[ \phi R_u = C \times (2.4d t F_e) = 1.99 \times (2.4 \times 0.75 \times 0.275 \times 65) \]

\( \phi R_u = 64.03 \) kips > 16 kips o.k.

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

\[ \phi R_u = \phi 0.6 F_y A_w = 0.90 \times 0.6 \times 50 \times (15.88 \times 0.275) \]

\( \phi R_u = 117.9 \) kips > > 16 kips o.k.
Net shear on splice plates (*LRFD Specification*, Equation J4-1):

Try splice plates 5/16" x 10"

\[
\phi R_n = \phi 0.6 F_u A_s = 0.75 \times 0.6 \times 58 \times ((10-(3 \times 0.8125)) \times 2 \times 0.3125)
\]

\[
\phi R_n = 123.36 \text{ kips} \gg 16 \text{ kips o.k.}
\]

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

\[
\phi R_n = \phi 0.6 F_y A_s = 0.90 \times 0.6 \times 36 \times (10 \times 2 \times 0.3125)
\]

\[
\phi R_n = 121.50 \text{ kips} \gg 16 \text{ kips o.k.}
\]

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 coefficient = 137
\[
\phi R_n = (68 + 137) \times 2 \times 0.3125 = 128.13 \text{ kips}
\]

Table 8 - 48a coefficient = 152
Table 8 - 48b coefficient = 54
\[
\phi R_n = (152 + 54) \times 2 \times 0.3125 = 128.75 \text{ kips o.k.}
\]

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

\[
M_y = R_s e/2 = 2.5 \times 16 = 40 \text{ kip-in}
\]

\[
S_x = (t \times H^2) / 6 = (100 \times 2 \times 0.3125) / 6 = 10.42 \text{ in}^3
\]

\[
\phi M_n = \phi F_y S_x = 0.90 \times 36 \times 10.42
\]

\[
\phi M_n = 337.50 \text{ kip-in} \gg 40 \text{ kip-in o.k.}
\]

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

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

\[
\phi M_n = \phi F_y S_n = 0.75 \times 58 \times 6.25
\]

\[
\phi M_n = 271.87 \text{ kip-in} \gg 40 \text{ kip-in o.k.}
\]
Double Angle Connection at Exterior Column:

Given:

- \( l_e = 1\ 1/4 \) inches - on connecting angles
- \( l_n = 1\ 1/2 \) inches - minimum
- 1/4" minimum connection angles
- ASTM A36 plate material, \( F_y = 36 \) ksi, \( F_u = 58 \) ksi
- 3" bolt spacing
- 3/4"\( \phi \) A325N bolts
- Minimum connection depth \( T/2 \) of connected members
- Minimum 2 bolt connection
- Bolted / Bolted design
- \( R_n = 25 \) 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"\( \phi \) A325N bolts
- \( \phi R_n = 104 \) kips \( >> \) 25 kips o.k.

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

- \( \phi R_n = \phi 0.6 F_y A_w = 0.90 \times 0.6 \times 50 \times (20.66 \times 0.350) \)
- \( \phi R_n = 195.23 \) kips \( >> \) 25 kips o.k.

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

- \( \phi R_n = 87.8 \times t \times n = 87.8 \times 0.35 \times 4 \)
- \( \phi R_n = 122.92 \) kips \( >> \) 25 kips o.k.