Box Beams

- General Shear Stress Equation
- Shear on Common Sections
- Examples

Homework problem – Beam Design
Box Beams

Box beams are built-up beams using dimensioned lumber (2x4 or 2x6) and panels (generally plywood).

The box is framed with the lumber and the sides are skinned with the panels.

The horizontal top and bottom flanges are continuous members as are the vertical ends and internal web stiffeners.

The flanges carry the tension and compression couple (the flexural moment) and the panels carry the shear force.

The internal vertical members brace and stiffen the panels.

Advantages

- low cost – economic alternative to steel or glulam
- good stiffness and strength
- light weight
- minimal shrinkage, warping or twisting
- ease of fabrication
- material availability
- speed of installation
- can be insulated if desired
Box Beams

APA design with tables

- choose span
- run down column to find passing capacity
- find cross section A, B or C
- find Panel Specifications: thickness in x/32" and rating (rafter o.c. / joist o.c.)

### ALLOWABLE LOADS** FOR 12-INCH DEEP ROOF BEAMS OR HEADER (lb/ft)

<table>
<thead>
<tr>
<th>Panel Specification</th>
<th>Cross Section</th>
<th>Approx. Wt. per ft (lb)</th>
<th>Span (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15/32” 32/16’ A</td>
<td>2 x 4</td>
<td>6 8 238* 198* 170*</td>
<td>147 116 94 78 64</td>
</tr>
<tr>
<td>15/32” 32/16’ B</td>
<td>2 x 6</td>
<td>9 12 339* 283* 242*</td>
<td>212 176 143 118 91</td>
</tr>
<tr>
<td>23/32” 48/24’ B</td>
<td>2 x 6</td>
<td>11 14 408* 340* 291*</td>
<td>233 176 143 118 93</td>
</tr>
<tr>
<td>23/32” 48/24’ C</td>
<td>2 x 6</td>
<td>13 17 374* 312* 267*</td>
<td>234 198 160 133 105</td>
</tr>
</tbody>
</table>

### ALLOWABLE LOADS** FOR 16-INCH DEEP ROOF BEAMS OR HEADER (lb/ft)

<table>
<thead>
<tr>
<th>Panel Specification</th>
<th>Cross Section</th>
<th>Approx. Wt. per ft (lb)</th>
<th>Span (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15/32” 32/16’ A</td>
<td>2 x 4</td>
<td>8 10 336* 280* 240*</td>
<td>210 166 134 111 93</td>
</tr>
<tr>
<td>15/32” 32/16’ B</td>
<td>2 x 6</td>
<td>10 13 475* 396* 340*</td>
<td>297 264 219 181 152</td>
</tr>
<tr>
<td>23/32” 48/24’ B</td>
<td>2 x 6</td>
<td>13 16 569* 474* 406*</td>
<td>342 270 219 181 152</td>
</tr>
<tr>
<td>23/32” 48/24’ C</td>
<td>2 x 6</td>
<td>15 19 531* 443* 380*</td>
<td>332 295 266 219 184</td>
</tr>
</tbody>
</table>

### ALLOWABLE LOADS** FOR 20-INCH DEEP ROOF BEAMS OR HEADER (lb/ft)

<table>
<thead>
<tr>
<th>Panel Specification</th>
<th>Cross Section</th>
<th>Approx. Wt. per ft (lb)</th>
<th>Span (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15/32” 32/16’ A</td>
<td>2 x 4</td>
<td>9 11 440* 367* 315*</td>
<td>273 216 175 144 121</td>
</tr>
<tr>
<td>15/32” 32/16’ B</td>
<td>2 x 6</td>
<td>12 15 610* 509* 436* 381*</td>
<td>339 297 246 207</td>
</tr>
<tr>
<td>23/32” 48/24’ B</td>
<td>2 x 6</td>
<td>15 18 728* 607* 520* 455* 367 297 246 207</td>
<td></td>
</tr>
<tr>
<td>23/32” 48/24’ C</td>
<td>2 x 6</td>
<td>17 22 623* 527* 495* 433* 385* 346 312 262</td>
<td></td>
</tr>
</tbody>
</table>

### ALLOWABLE LOADS** FOR 24-INCH DEEP ROOF BEAMS OR HEADER (lb/ft)

<table>
<thead>
<tr>
<th>Panel Specification</th>
<th>Cross Section</th>
<th>Approx. Wt. per ft (lb)</th>
<th>Span (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15/32” 32/16’ A</td>
<td>2 x 4</td>
<td>11 13 550* 458* 393* 336 266 215 178 149</td>
<td></td>
</tr>
<tr>
<td>15/32” 32/16’ B</td>
<td>2 x 6</td>
<td>13 16 744* 620* 531* 465* 413 372 312 262</td>
<td></td>
</tr>
<tr>
<td>23/32” 48/24’ B</td>
<td>2 x 6</td>
<td>16 20 885* 738* 632* 553 465 377 312 262</td>
<td></td>
</tr>
<tr>
<td>23/32” 48/24’ C</td>
<td>2 x 6</td>
<td>18 24 854* 711* 610* 533* 474* 427 388 342</td>
<td></td>
</tr>
</tbody>
</table>

(a) Includes 15% snow loading increase.
*Lumber may be No. 2 Douglas-fir or No. 2 southern pine without reduction of tabulated capacity.

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**Table assumptions:**
- DF - No1 or SP - No1
- panels: APA – expos. 1 4- or 5-ply or OSB
- deflection less than L/240 (total load)
- nail size – 8d common
- nail spacing:
  - flanges 1 ½” o.c.
  - may be doubled in middle half of span
  - end stiffeners 1 ½” o.c.
  - mid stiffeners 3” o.c.

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**Box Beams**

APA design with tables (Z416T 2009)

---

**Sheathing or siding over; or set beam out thickness of sheathing and butt sheathing to web.**

*16d common nails [0.162” x 3-1/2”] at 6” o.c.

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**Nailing Layout**

*When end stiffeners extend through the beam, nail spacing is the same as for flanges, except space nails 1 inch on center when double end stiffeners are used in beams with three members per flange (cross-section C). When end stiffeners are inserted between flanges, nails may be spaced 3 inches on center.*
Box Beams

APA design with tables
web panel joint and stiffer layout
flange members are continuous

**WEB JOINT LAYOUTS**

<table>
<thead>
<tr>
<th>Size</th>
<th>Layout</th>
<th>Flanges</th>
<th>Web Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>10' to 10-1/2'</td>
<td>4' to 6-1/2'</td>
<td>6' to 6-1/2'</td>
<td>4'</td>
</tr>
<tr>
<td>12' to 12-1/2'</td>
<td>4' to 4-1/2'</td>
<td>8'</td>
<td>4' to 4-1/2'</td>
</tr>
<tr>
<td>14' to 14-1/2'</td>
<td>4' to 6-1/2'</td>
<td>6' to 6-1/2'</td>
<td>4'</td>
</tr>
<tr>
<td>16' to 16-1/2'</td>
<td>4' to 4-1/2'</td>
<td>8'</td>
<td>4' to 4-1/2'</td>
</tr>
<tr>
<td>18' to 18-1/2'</td>
<td>5' to 5-1/2'</td>
<td>5' to 5-1/2'</td>
<td>8'</td>
</tr>
<tr>
<td>20' to 20-1/2'</td>
<td>5' to 5-1/2'</td>
<td>7'</td>
<td>8'</td>
</tr>
<tr>
<td>22' to 22-1/2'</td>
<td>4' to 4-1/2'</td>
<td>4'</td>
<td>4' to 4-1/2'</td>
</tr>
<tr>
<td>24' to 24-1/2'</td>
<td>6' to 6-1/2'</td>
<td>4'</td>
<td>6'</td>
</tr>
</tbody>
</table>

Example:

APA Z416T design with tables

garage door header using 2x4s
span = 18 ft
factored projected roof load = 35 psf
(DL 10 psf + SL 25 psf)

Load on beam:
35 psf x 20 ft / 2 = 350 plf
Box Beams

APA design with tables

Example:

Load on beam:

\[ 35 \text{ psf} \times 20 \text{ ft} / 2 = 350 \text{ plf} \]

Choose beam:

follow the 18 ft span column

no 12” or 16” deep sections work

20” with 367 > 350+15 plf works

or

24” with 413 > 350+13 plf works

both options use section “B”

20” deep:

23/32” 48/24 panel and DF No1

or

24” deep:

15/32” 32/16 panel and DF No1

In this case the 24” section uses a thinner and lower rated panel and so would be less costly. If the additional depth were a concern the 20” beam could be used.

Box Beams

NDS analysis

Based on the previous example find the maximum load capacity.

Load on beam:

\[ D+S = 35 \text{ psf} \times 20 \text{ ft} / 2 = 350 \text{ plf} \]

\[ C_D = 1.15 \]

Box Beam Section:

- Douglas Fir No1
- \( G = 0.5 \)

Nails:

- 8d common
- spacing, \( s = 1\frac{1}{2}” \) o.c. flange and ends, \( 3” \) o.c. web stiffeners
- rows of nails, \( R = 2 \) per web per flange (8 total top and bottom)
- allowable load per nail, \( F_n = 74 \text{ lb/nail} \)
- number of webs, \( N_{\text{web}} = 2 \)
Box Beams

Nails:
- 8d common
- allowable load per nail, \( F_n = 74 \text{ lb/nail} \)

Table 12Q COMMON BOX, SINKER, or ROOF SHEATHING RING SHANK (RSRS) STEEL WIRE NAILS:
Reference Lateral Design Values, \( Z \), for Single Shear (two member) Connections

<table>
<thead>
<tr>
<th>Nail Diameter</th>
<th>No. of Shanks</th>
<th>Steel Wire Gauge</th>
<th>Shank Length</th>
<th>( F_n ) (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.8</td>
<td>6d</td>
<td>6d</td>
<td>3.75</td>
<td>47</td>
</tr>
<tr>
<td>4.0</td>
<td>6d</td>
<td>6d</td>
<td>4.0</td>
<td>50</td>
</tr>
<tr>
<td>4.0</td>
<td>6d</td>
<td>6d</td>
<td>4.0</td>
<td>50</td>
</tr>
</tbody>
</table>

NDS analysis
Based on the previous example find the maximum load capacity.

Box Beam Section:
 webs:
- depth of section, \( h = 24 \text{ in} \)
- thickness 15/32 = 0.46875 in
- grade and rating: APA Rated Sheathing 32/16 4-ply Exp. 1
- number of webs, \( N_{\text{web}} = 2 \)
- axial Stiffness, \( E_A = 4,150,000 \text{ lbs/ft width} \) (Panel Design D510)
- bending thickness, \( t_{\text{par}} = \frac{E_A}{(E/A)^2} = \frac{4150}{(1700)^2} = 0.2034 \text{ in} \)
based on transformed area normalizing \( E \) for flange and web.
- shear capacity \( F_{\text{vth}} = 81 \text{ lb/in each web} \)

Flanges:
- 2x4 Douglas Fir – Larch No. 1
- continuous, no but joints. 2 pcs per flange
- depth of flange, \( d = 3" \)
- width of flange, \( b = 3.5" \)
- allowable tension stress, \( F = \frac{175 (1.5)}{2 \times 4} = 1012.5 \text{ psi} \)
- Stiffness, \( E = 1700000 \text{ psi} \)
Box Beams

Douglas Fir – Larch No.1 E = 1700000 psi (NDS Supplement 2018)
15/32" 32/16 panel EA = 4,150,000 lbs/ft width (APA D510) 2020

<table>
<thead>
<tr>
<th>Species and commercial grade</th>
<th>Size classification</th>
<th>Design values in pounds per square inch (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DOUGLAS FIR-LARCH</td>
<td></td>
<td>Bending Tension parallel to grain Shear parallel to grain Compression perpendicular to grain Compression parallel to grain Modulus of Elasticity</td>
</tr>
<tr>
<td>Select Structural</td>
<td></td>
<td>1.500 1.000 180 625 1.700 1.900,000 690,000</td>
</tr>
<tr>
<td>No. 1</td>
<td>2&quot; &amp; wider</td>
<td>1.200 0.800 180 625 1.500 1.800,000 860,000</td>
</tr>
<tr>
<td>No. 2</td>
<td></td>
<td>1.000 0.675 180 625 1.500 1.700,000 620,000</td>
</tr>
<tr>
<td>No. 3</td>
<td></td>
<td>0.900 0.575 180 625 1.350 1.600,000 580,000</td>
</tr>
<tr>
<td>No. 3</td>
<td></td>
<td>0.825 0.535 180 625 1.250 1.500,000 540,000</td>
</tr>
</tbody>
</table>

Table: Rated Panels Design Capacities

<table>
<thead>
<tr>
<th>Span Rating</th>
<th>3-ply</th>
<th>4-ply</th>
<th>5-ply</th>
<th>OSB</th>
</tr>
</thead>
<tbody>
<tr>
<td>24/0</td>
<td>3,350,000</td>
<td>3,350,000</td>
<td>3,350,000</td>
<td>3,350,000</td>
</tr>
<tr>
<td>24/16</td>
<td>3,800,000</td>
<td>3,800,000</td>
<td>3,800,000</td>
<td>3,800,000</td>
</tr>
<tr>
<td>32/16</td>
<td>4,150,000</td>
<td>4,150,000</td>
<td>4,150,000</td>
<td>4,150,000</td>
</tr>
<tr>
<td>40/20</td>
<td>5,000,000</td>
<td>5,000,000</td>
<td>5,000,000</td>
<td>5,000,000</td>
</tr>
<tr>
<td>48/24</td>
<td>NA</td>
<td>5,850,000</td>
<td>5,850,000</td>
<td>5,850,000</td>
</tr>
<tr>
<td>16 oc</td>
<td>4,500,000</td>
<td>4,500,000</td>
<td>4,500,000</td>
<td>4,500,000</td>
</tr>
<tr>
<td>20 oc</td>
<td>5,000,000</td>
<td>5,000,000</td>
<td>5,000,000</td>
<td>5,000,000</td>
</tr>
<tr>
<td>24 oc</td>
<td>NA</td>
<td>5,850,000</td>
<td>5,850,000</td>
<td>5,850,000</td>
</tr>
<tr>
<td>32 oc</td>
<td>NA</td>
<td>NA</td>
<td>7,500,000</td>
<td>7,500,000</td>
</tr>
<tr>
<td>48 oc</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>8,200,000</td>
</tr>
</tbody>
</table>

Box Beams

15/32" 32/16 rated panel, 4-ply $F_{t,v}$ lbs/in width (APA D510)

$F_{t,v} = 81$ lbs/in

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TABLE (Continued)

RATED PANELS DESIGN CAPACITIES

<table>
<thead>
<tr>
<th>Span Rating</th>
<th>Plywood</th>
<th>Stress Parallel to Strength Axis</th>
<th>Stress Perpendicular to Strength Axis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3-ply</td>
<td>4-ply</td>
<td>5-ply</td>
</tr>
<tr>
<td></td>
<td>3-ply</td>
<td>4-ply</td>
<td>5-ply</td>
</tr>
<tr>
<td>PANEL SHEAR THROUGH THE THICKNESS, $F_{t,h}$ (lbf/in. of shear-resisting panel length)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24/0</td>
<td>53</td>
<td>69</td>
<td>80</td>
</tr>
<tr>
<td>24/16</td>
<td>57</td>
<td>74</td>
<td>86</td>
</tr>
<tr>
<td>32/16</td>
<td>62</td>
<td>81</td>
<td>93</td>
</tr>
<tr>
<td>40/20</td>
<td>68</td>
<td>88</td>
<td>100</td>
</tr>
<tr>
<td>48/24</td>
<td>NA</td>
<td>98</td>
<td>115</td>
</tr>
<tr>
<td>16 oc</td>
<td>58</td>
<td>75</td>
<td>87</td>
</tr>
<tr>
<td>20 oc</td>
<td>67</td>
<td>87</td>
<td>100</td>
</tr>
<tr>
<td>24 oc</td>
<td>68</td>
<td>88</td>
<td>100</td>
</tr>
<tr>
<td>32 oc</td>
<td>NA</td>
<td>NA</td>
<td>120</td>
</tr>
<tr>
<td>48 oc</td>
<td>NA</td>
<td>NA</td>
<td>160</td>
</tr>
</tbody>
</table>

$F = \frac{p}{A}$

$p = FA$
Box Beams

Section Properties:

Moment of Inertia, \( I \)
\[
I_f = \frac{3.5 \times (24^3 - (24 - 6)^3)}{12} \quad I_f = 2331 \text{ in}^4
\]
\[
I_w = \frac{2(0.2034 \times 24^3)}{12} \quad I_w = 468.6 \text{ in}^4
\]
\[
I_{\text{total}} = I_f + I_w = 2331 + 468.6 = 2800 \text{ in}^4
\]

Statical Moment of Area, \( Q \)
\[
Q_f = \frac{3.5 \times 3 \times (24 - 3)}{2} \quad Q_f = 110.25 \text{ in}^3
\]
\[
Q_w = \frac{2 \times (0.2034 \times 24^2)}{8} \quad Q_w = 29.29 \text{ in}^3
\]
\[
Q_{\text{total}} = Q_f + Q_w = 110.25 + 29.29 = 139.5 \text{ in}^3
\]

Transformed Section

Box Beams

Section Capacities:

Max. Moment
\[
F = \frac{Mc}{I} \quad M = F \cdot \frac{h}{2} (12) \quad M = 1012.5 \text{ lb/in}^2 \times (2331 \text{ in}^4) / ((24 \text{ in} / 2) (12 \text{ in/ft})) \quad M = 16390 \text{ ft-lb}
\]

Max Web Shear
\[
F_v = \frac{Q}{I} \quad V_h = (F_{vtv} I_{t} (N_{\text{webs}}) / Q_t) \quad V_h = (81 \text{ lb/in}) \times 2800 \text{ in}^4 (2) / 139.5 \text{ in}^3 \quad V_h = 3252 \text{ lbs}
\]

Max Nail Shear
\[
V_n = F_{nt} I_{t} (N_{\text{webs}}) ( \text{Rows} ) / (\text{spacing } Q_t) \quad V_n = 74 \text{ lb/nail} \times 2800 \text{ in}^4 (2) / (1.5 \text{ in/nail}) \times 110.25 \text{ in}^3 \quad V_n = 5011 \text{ lbs}
\]

Stiffness
\[
E I = \frac{EF}{E I} = 1700000 \times 2800 = 4,760,000,000
\]
Box Beams

Allowable Uniform Load (D+S)

**Bending**

\[ M = \frac{w_b L^2}{8} \]

\[ w_b = \frac{M (C_D) 8}{L^2} \]

\[ w_b = \frac{16390 \, \text{ft-lb} \times (1.15) 8}{18 \, \text{ft}^2} = 465 \, \text{plf} \]

**Web Shear**

\[ V_h = \frac{w_v L}{2} \]

\[ w_v = \frac{V_h (C_D) 2}{L} \]

\[ w_v = \frac{3252 \, \text{lbs} \times (1.15) 2}{18 \, \text{ft}} = 415 \, \text{plf} \]

**Nail Shear**

\[ V_n = \frac{w_n L}{2} \]

\[ w_n = \frac{V_n (C_D) 2}{L} \]

\[ w_n = \frac{5011 \, \text{lbs} \times (1.15) 2}{18 \, \text{ft}} = 640 \, \text{plf} \]

**Deflection**

\[ \Delta = \frac{5KLwL^4}{384EI} \]

\[ \Delta = \frac{5 \times (1.5) (415) (18)^4 (1728)}{(384 \times 4,760,000,000)} = 0.31 \, \text{"} \]

\[ L/360 = \frac{18(12)}{360} = 0.6 \, \text{"} \]

K factor for deflection in composite panel section (from APA testing)

for L < 14 ft K = 2.0, else K = 1.5

---

**Box Beams**

span: 8 ft  depth: 12"
flanges: 2x4 S-P-F No2
webs: 1/4" 3-ply plywood
nails: 8d common at 3" o.c.

**Capacity:**

4000 lbs first cracking
8000 ft-lbs
5000 lbs ultimate
10000 ft lbs
Box Beams

Initial failure: web shear
Further failure: nail pull out and head pull through
Ultimate failure: tension flange