Steel construction: materials & beams
Steel Beam Design

• American Institute of Steel Construction
  – Manual of Steel Construction
  – ASD & LRFD
  – combined in 2005
Steel Materials

- smelt iron ore
- add alloying elements
- heat treatments
- iron, carbon
- microstructure

A36 steel, JOM 1998
Steel Materials

- cast into billets
- hot rolled
- cold formed
- residual stress
- corrosion-resistant "weathering" steels
- stainless
Steel Materials

• steel grades
  – ASTM A36 – carbon
    • plates, angles
    • $F_y = 36\ \text{ksi} \ & \ F_u = 58\ \text{ksi}$
  – ASTM A572 – high strength low-alloy
    • some beams
    • $F_y = 60\ \text{ksi} \ & \ F_u = 75\ \text{ksi}$
  – ASTM A992 – for building framing
    • most beams
    • $F_y = 50\ \text{ksi} \ & \ F_u = 65\ \text{ksi}$
Steel Properties

- high strength to weight ratio
- elastic limit – yield ($F_y$)
- inelastic – plastic
- ultimate strength ($F_u$)
- ductile
- strength sensitive to temperature
- can corrode
- fatigue
Structural Steel

- standard rolled shapes ($W, C, L, T$)
- open web joists
- plate girders
- decking
Steel Construction

- welding
- bolts
Steel Construction

- fire proofing
  - cementicious spray
  - encasement in gypsum
  - intumescent – expands with heat
  - sprinkler system
Unified Steel Design

• ASD

\[ R_a \leq \frac{R_n}{\Omega} \]

– bending (braced) \( \Omega = 1.67 \)
– bending (unbraced*) \( \Omega = 1.67 \)
– shear \( \Omega = 1.5 \) or 1.67
– shear (bolts & welds) \( \Omega = 2.00 \)
– shear (welds) \( \Omega = 2.00 \)

* flanges in compression can buckle
Unified Steel Design

• braced vs. unbraced
**LRFD**

- **loads on structures are**
  - not constant
  - can be more influential on failure
  - happen more or less often
  - **UNCERTAINTY**

\[
R_u = \gamma_D R_D + \gamma_L R_L \leq \phi R_n
\]

- \( \phi \) - resistance factor
- \( \gamma \) - load factor for (D)ead & (L)ive load
LRFD Steel Beam Design

- limit state is yielding all across section
- outside elastic range
- load factors & resistance factors

$\varepsilon_y = 0.001724$

$f_y = 50\text{ksi}$
LRFD Load Combinations

- 1.4D
- 1.2D + 1.6L + 0.5(L_r or S or R)
- 1.2D + 1.0W + L + 0.5(L_r or S or R)
- 1.2D + 1.0E + L + 0.2S
- 0.9D + 1.0W
- 0.9D + 1.0E

  - F has same factor as D in 1-5 and 7
  - H adds with 1.6 and resists with 0.9 (permanent)

ASCE-7 (2010)
Beam Design Criteria (revisited)

- **strength design**
  - bending stresses predominate
  - shear stresses occur
- **serviceability**
  - limit deflection
  - stability
- **superpositioning**
  - use of beam charts
  - elastic range only!
  - “add” moment diagrams
  - “add” deflection CURVES (not maximums)
Steel Beams

- lateral stability - bracing
- local buckling – stiffen, or bigger $I_y$
Local Buckling

- steel I beams
- flange
  - buckle in direction of smaller radius of gyration
- web
  - force
  - “crippling”
Local Buckling

- flange
- web

Figure 2-5. Flange Local Bending Limit State
(Beedle, L.S., Christopher, R., 1964)

Figure 2-7. Web Local Buckling Limit State
(SAC Project)
Shear in Web

- panels in plate girders or webs with large shear
- buckling in compression direction
- add stiffeners

![Image of I-beam with stiffeners to prevent lateral buckling](image1)

![Diagram of shear failure and buckling](image2)
Shear in Web

- plate girders and stiffeners

http://nisee.berkeley.edu/godden
Steel Beams

- **bearing**
  - provide adequate area
  - prevent local yield of flange and web

**Figure 9.10** Considerations for bearing in beams with thin webs, as related to web crippling (buckling of the thin web in compression).
LRFD - Flexure

\[ \sum \gamma_i R_i = M_u \leq \phi_b M_n = 0.9 F_y Z \]

- \( M_u \) - maximum moment
- \( \phi_b \) - resistance factor for bending = 0.9
- \( M_n \) - nominal moment (ultimate capacity)
- \( F_y \) - yield strength of the steel
- \( Z \) - plastic section modulus*
Internal Moments - at yield

- material hasn’t failed

\[ M_y = \frac{I}{c} f_y = \frac{bh^2}{6} f_y \]

\[ = \frac{b(2c)^2}{6} f_y = \frac{2bc^2}{3} f_y \]
Internal Moments - ALL at yield

- all parts reach yield
- plastic hinge forms
- ultimate moment
- \( A_{\text{tension}} = A_{\text{compression}} \)

\[
M_p = bc^2 f_y = \frac{3}{2} M_y
\]

\( \sigma_y = 50 \text{ksi} \)

\( \varepsilon_y = 0.001724 \)
n.a. of Section at Plastic Hinge

- cannot guarantee at centroid
- $f_y A_1 = f_y A_2$
- moment found from yield stress times
  moment area

$$M_p = f_y A_1 d = f_y \sum_{n.a} A_i d_i$$
Plastic Hinge Development

(a) $M < M_Y$

(b) $M = M_Y$

(c) $M > M_Y$

(d) $M = M_p$
Plastic Hinge Examples

- stability can be effected
Plastic Section Modulus

• shape factor, \( k \)

\( k = \frac{M_p}{M_y} \)

= 3/2 for a rectangle

\( \approx 1.1 \) for an I

• plastic modulus, \( Z \)

\( Z = \frac{M_p}{f_y} \)
LRFD – Shear (compact shapes)

\[ \sum \gamma_i R_i = V_u \leq \phi_v V_n = 1.0 \left( 0.6 F_{yw} A_w \right) \]

- \( V_u \) - maximum shear
- \( \phi_v \) - resistance factor for shear = 1.0
- \( V_n \) - nominal shear
- \( F_{yw} \) - yield strength of the steel in the web
- \( A_w \) - area of the web = \( t_w d \)
LRFD – Flexure Design

- **limit states for beam failure**
  1. yielding
  2. lateral-torsional buckling*
  3. flange local buckling
  4. web local buckling

- **minimum \( M_n \) governs**

\[
L_p = 1.76 r_y \sqrt{\frac{F_y}{E}}
\]

\[
\Sigma \gamma_i R_i = M_u \leq \phi_b M_n
\]
Compact Sections

• plastic moment can form before buckling

• criteria

\[ - \frac{b_f}{2t_f} \leq 0.38 \sqrt{\frac{E}{F_y}} \]

\[ - \frac{h_c}{t_w} \leq 3.76 \sqrt{\frac{E}{F_y}} \]
Lateral Torsional Buckling

\[ M_n = C_b \left[ \text{moment based on lateral buckling} \right] \leq M_p \]

\[ C_b = \frac{12.5 M_{\text{max}}}{2.5 M_{\text{max}} + 3 M_A + 4 M_B + 3 M_C} \]

- \( C_b \) = modification factor
- \( M_{\text{max}} \) - \(|\text{max moment}|\), unbraced segment
- \( M_A \) - \(|\text{moment}|\), 1/4 point
- \( M_B \) = \(|\text{moment}|\), center point
- \( M_C \) = \(|\text{moment}|\), 3/4 point
Beam Design Charts

Table 3-10 (continued)

W Shapes

Available Moment vs. Unbraced Length
Charts & Deflections

• beam charts
  – solid line is most economical
  – dashed indicates there is another more economical section
  – self weight is NOT included in $M_n$

• deflections
  – no factors are applied to the loads
  – often governs the design
Design Procedure (revisited)

1. Know unbraced length, material, design method ($\Omega, \phi$)

2. Draw $V$ & $M$, finding $M_{\text{max}}$

3. Calculate $Z_{\text{req'd}}$ \( (M_a \leq M_n / \Omega) \)
\[ (M_u \leq \phi_b M_n) \]

4. Choose (economical) section from section or beam capacity charts
Table 11  Listing of W Shapes in Descending Order of $S_x$ for Beam Design.

<table>
<thead>
<tr>
<th>$S_x$—US (in.$^3$)</th>
<th>Section</th>
<th>$S_x$—SI ($10^3 \times \text{mm}^3$)</th>
<th>$S_x$—US (in.$^3$)</th>
<th>Section</th>
<th>$S_x$—SI ($10^3 \times \text{mm}^3$)</th>
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<tbody>
<tr>
<td>448</td>
<td>W33 x 141</td>
<td>7350</td>
<td>188</td>
<td>W18 x 97</td>
<td>3080</td>
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<td>439</td>
<td>W36 x 135</td>
<td>7200</td>
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<td>411</td>
<td>W27 x 146</td>
<td>6740</td>
<td>176</td>
<td>W24 x 76</td>
<td>2890</td>
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<td></td>
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<tr>
<td>406</td>
<td>W33 x 130</td>
<td>6660</td>
<td>175</td>
<td>W16 x 100</td>
<td>2870</td>
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<tr>
<td>380</td>
<td>W30 x 132</td>
<td>6230</td>
<td>173</td>
<td>W14 x 109</td>
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<td>371</td>
<td>W24 x 146</td>
<td>6080</td>
<td>171</td>
<td>W21 x 83</td>
<td>2800</td>
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<tr>
<td>359</td>
<td>W33 x 118</td>
<td>5890</td>
<td>157</td>
<td>W18 x 86</td>
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<tr>
<td>355</td>
<td>W30 x 124</td>
<td>5820</td>
<td>155</td>
<td>W14 x 99</td>
<td>2570</td>
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<td></td>
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</tr>
<tr>
<td>329</td>
<td>W30 x 116</td>
<td>5400</td>
<td>154</td>
<td>W24 x 68</td>
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<td>151</td>
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<td>W18 x 76</td>
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</tbody>
</table>
# Beam Charts by $Z_x$

## Table 9.1: Load Factor Resistance Design Selection for Shapes Used as Beams

<table>
<thead>
<tr>
<th>Designation</th>
<th>$Z_x$ in.$^3$</th>
<th>$L_p$ ft</th>
<th>$L_r$ ft</th>
<th>$M_p$ kip-ft</th>
<th>$M_r$ kip-ft</th>
<th>$F_y = 36$ ksi</th>
<th>$F_y = 50$ ksi</th>
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</thead>
<tbody>
<tr>
<td>W 33 × 141</td>
<td>514</td>
<td>10.1</td>
<td>30.1</td>
<td>1,542</td>
<td>971</td>
<td>8.59</td>
<td>23.1</td>
</tr>
<tr>
<td>W 30 × 148</td>
<td>500</td>
<td>9.50</td>
<td>30.6</td>
<td>1,500</td>
<td>945</td>
<td>8.06</td>
<td>22.8</td>
</tr>
<tr>
<td>W 24 × 162</td>
<td>468</td>
<td>12.7</td>
<td>45.2</td>
<td>1,404</td>
<td>897</td>
<td>10.8</td>
<td>32.4</td>
</tr>
<tr>
<td>W 24 × 146</td>
<td>418</td>
<td>12.5</td>
<td>42.0</td>
<td>1,254</td>
<td>804</td>
<td>10.6</td>
<td>30.6</td>
</tr>
<tr>
<td>W 33 × 118</td>
<td>415</td>
<td>9.67</td>
<td>27.8</td>
<td>1,245</td>
<td>778</td>
<td>8.20</td>
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<td>W 30 × 124</td>
<td>408</td>
<td>9.29</td>
<td>28.2</td>
<td>1,224</td>
<td>769</td>
<td>7.88</td>
<td>21.5</td>
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<tr>
<td>W 21 × 147</td>
<td>373</td>
<td>12.3</td>
<td>46.4</td>
<td>1,119</td>
<td>713</td>
<td>10.4</td>
<td>32.8</td>
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<tr>
<td>W 24 × 131</td>
<td>370</td>
<td>12.4</td>
<td>39.3</td>
<td>1,110</td>
<td>713</td>
<td>10.5</td>
<td>29.1</td>
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<tr>
<td>W 18 × 158</td>
<td>356</td>
<td>11.4</td>
<td>56.5</td>
<td>1,068</td>
<td>672</td>
<td>9.69</td>
<td>38.0</td>
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<tr>
<td>W 30 × 108</td>
<td>346</td>
<td>8.96</td>
<td>26.3</td>
<td>1,038</td>
<td>648</td>
<td>7.60</td>
<td>20.3</td>
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<td>W 27 × 114</td>
<td>343</td>
<td>9.08</td>
<td>28.2</td>
<td>1,029</td>
<td>648</td>
<td>7.71</td>
<td>21.3</td>
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<td>921</td>
<td>592</td>
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<td>W 18 × 130</td>
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<td>11.3</td>
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<td>555</td>
<td>9.55</td>
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<td>8.71</td>
<td>24.8</td>
<td>849</td>
<td>531</td>
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<td>19.4</td>
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<tr>
<td>W 24 × 103</td>
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<td>27.0</td>
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<td>20.0</td>
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<td>W 27 × 94</td>
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<td>8.83</td>
<td>25.9</td>
<td>834</td>
<td>527</td>
<td>7.50</td>
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<td>81.6</td>
<td>780</td>
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<td>25.9</td>
<td>762</td>
<td>481</td>
<td>7.00</td>
<td>19.4</td>
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<table>
<thead>
<tr>
<th>$r_y$ in.</th>
<th>$b/2h_f$</th>
<th>$h/l_{x_f}$</th>
<th>$X_1$ ksi</th>
<th>$X_2 \times 10^6$ (1/ksi)$^2$</th>
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<tr>
<td>2.43</td>
<td>6.01</td>
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<td>17,800</td>
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<td>6,270</td>
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<td>3.05</td>
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<td>2,260</td>
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<td>2.95</td>
<td>5.44</td>
<td>26.1</td>
<td>3,140</td>
<td>1,590</td>
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<td>19.8</td>
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<td>3,680</td>
<td>810</td>
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<td>1.99</td>
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<td>41.9</td>
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<td>7,800</td>
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</table>
Beam Design (revisited)

4*. Include self weight for $M_{\text{max}}$
   - it's dead load
   - and repeat 3 & 4 if necessary

5. Consider lateral stability

Unbraced roof trusses were blown down in 1999 at this project in Moscow, Idaho.

Photo: Ken Carper
Beam Design (revisited)

6. Evaluate shear stresses - horizontal

- \((V_a \leq V_n / \Omega)\) or \((V_u \leq \phi_v V_n)\)

- rectangles and W’s

\[
V_n = 0.6 F_{yw} A_w
\]

- general

\[
f_{v-max} = \frac{VQ}{Ib}
\]
Beam Design (revisited)

7. Provide adequate bearing area at supports

\[ P_a \leq \frac{P_n}{\Omega} \]

\[ P_u \leq \phi P_n \]
Beam Design (revisited)

8. Evaluate torsion

\( f_v \leq F_v \)

- circular cross section
  \[ f_v = \frac{T\rho}{J} \]

- rectangular
  \[ f_v = \frac{T}{c_1ab^2} \]

**TABLE 3.1. Coefficients for Rectangular Bars in Torsion**

<table>
<thead>
<tr>
<th>a/b</th>
<th>c_1</th>
<th>c_2</th>
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<td>1.0</td>
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<td>1.2</td>
<td>0.219</td>
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<td>4.0</td>
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<td>0.281</td>
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<tr>
<td>5.0</td>
<td>0.291</td>
<td>0.291</td>
</tr>
<tr>
<td>10.0</td>
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<td>0.312</td>
</tr>
<tr>
<td>∞</td>
<td>0.333</td>
<td>0.333</td>
</tr>
</tbody>
</table>
Beam Design (revisited)

9. Evaluate deflections – NO LOAD FACTORS

\[ y_{\text{max}}(x) = \Delta_{\text{actual}} \leq \Delta_{\text{allowable}} \]
Load Tables & Equivalent Load

- uniformly distributed loads
- equivalent “w” \[ M_{\text{max}} = \frac{W_{\text{equivalent}} L^2}{8} \]

Load for live load deflection limit in **RED**, total in **BLACK**

<table>
<thead>
<tr>
<th>Joint Designation</th>
<th>10K1</th>
<th>12K1</th>
<th>12K2</th>
<th>12K3</th>
<th>12K4</th>
<th>14K1</th>
<th>14K2</th>
<th>14K3</th>
<th>14K4</th>
<th>14K5</th>
<th>16K2</th>
<th>16K3</th>
<th>16K4</th>
<th>16K5</th>
<th>16K6</th>
<th>16K7</th>
<th>16K8</th>
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</thead>
<tbody>
<tr>
<td>Depth (in.)</td>
<td>10</td>
<td>12</td>
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<td>12</td>
<td>12</td>
<td>12</td>
<td>14</td>
<td>14</td>
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<tr>
<td>Approx. Wt (lbs./ft.)</td>
<td>5.0</td>
<td>5.0</td>
<td>5.7</td>
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<td>8.1</td>
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<tr>
<td>Span (ft)</td>
<td>10</td>
<td>11</td>
<td>12</td>
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</tbody>
</table>
| Load for live load deflection limit in **RED**, total in **BLACK**
Sloped Beams

- stairs & roofs
- projected live load
- dead load over length

- perpendicular load to beam:
  \[ W_\perp = w \cdot \cos \alpha \]

- equivalent distributed load:
  \[ W_{\text{adj.}} = \frac{w}{\cos \alpha} \]
Steel Arches and Frames

- solid sections
  or open web

http://nisee.berkeley.edu/godden
Steel Shell and Cable Structures
Approximate Depths

<table>
<thead>
<tr>
<th>Structure</th>
<th>Span</th>
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<tbody>
<tr>
<td>Decking</td>
<td>L/30–L/50</td>
</tr>
<tr>
<td>Wide flanges</td>
<td>L/18–L/28</td>
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<tr>
<td>Plate girders</td>
<td>L/15–L/20</td>
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<tr>
<td>Open-web joists</td>
<td>L/18–L/22</td>
</tr>
<tr>
<td>Fink truss</td>
<td>L/4–L/5</td>
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<tr>
<td>Howe truss</td>
<td>L/4–L/5</td>
</tr>
<tr>
<td>Bowstring truss</td>
<td>L/6–L/10</td>
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<tr>
<td>Special truss</td>
<td>L/4–L/15</td>
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<tr>
<td>Arches</td>
<td>L/3–L/5</td>
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<tr>
<td>Ribbed domes</td>
<td>L/3–L/5</td>
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<tr>
<td>Cables</td>
<td>L/5–L/11</td>
</tr>
<tr>
<td>Space frame (column-supported)</td>
<td>L/12–L/20</td>
</tr>
<tr>
<td>Space frame (wall-supported)</td>
<td>L/12–L/20</td>
</tr>
</tbody>
</table>