SIMPLE DESIGN CHART & SAMPLE CALCULATIONS

INDEX

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CORCON® is a hybrid multi-ribbed slab + beam system using the best properties of a slab coupled with all the advantages of the beam used to stiffen the slab in deflection.

CORCON® is a flexible system that can be applied to any span and loading configuration by the use of inserts which can be used to vary the depth of the CORCON® rib beams - from a nominal 50 m² depression to 300 m² deep for extra heavy loads and or long spans over 8 to 9m. Also by reducing the (c to c) centre to centre spacing from 1200 m² and continuity the span can be increased to 26m using post-tensioning.

Small spans - say 3m- use CORCON® with "FLAT" insert mould (see the Figure below):

If require structural primary beams could always use upstand beams subject to requirements. (see the Figure below):

This upstand systems allows continuous layting of CORCON® ribs.

Appendix IX. SIMPLE DESIGN CHART & SAMPLE CALCULATIONS
## SIMPLE DESIGN LOAD AND SPAN CHARTS FOR CORCON®

### Single Span

<table>
<thead>
<tr>
<th>Beam Depth d mm</th>
<th>Req'd bar size overlap</th>
<th>Avg Conc. vol m3/m2*</th>
<th>Total depth mm</th>
<th>Allowable superimposed live load (KPa) C to C spans metres</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>4.0</td>
<td>5.0</td>
</tr>
<tr>
<td>50</td>
<td>2Y12 0.12</td>
<td>184</td>
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<td></td>
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<tr>
<td>75</td>
<td>2Y12 0.12</td>
<td>209</td>
<td>5</td>
<td></td>
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<tr>
<td>100</td>
<td>2Y12 0.12</td>
<td>234</td>
<td>5</td>
<td>1.5</td>
</tr>
<tr>
<td>150</td>
<td>2Y12 0.12</td>
<td>284</td>
<td>5</td>
<td>2.5</td>
</tr>
<tr>
<td>200</td>
<td>2Y12 0.12</td>
<td>334</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>250</td>
<td>2Y16 0.13</td>
<td>384</td>
<td>5</td>
<td>5</td>
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<tr>
<td>300</td>
<td>2Y16 0.14</td>
<td>414</td>
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</tbody>
</table>

1.5 hour fire rating, stc 49, min cover 85mm, f62 mesh top & 1/250 deflection
* based on 20m2 room
^ roof loads
** post tensioned
*** continuous post tensioned built in primary beam

### Single Span

<table>
<thead>
<tr>
<th>Beam Depth d mm</th>
<th>Req'd bar size overlap</th>
<th>Avg Conc. vol m3/m2*</th>
<th>Total depth mm</th>
<th>Allowable superimposed live load (KPa) C to C spans metres</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
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<td>3.0</td>
<td>3.5</td>
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<tr>
<td>50</td>
<td>2Y12 0.10</td>
<td>164</td>
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<td>100</td>
<td>2Y12 0.10</td>
<td>214</td>
<td>5</td>
<td>4.5</td>
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<tr>
<td>150</td>
<td>2Y12 0.10</td>
<td>264</td>
<td>5</td>
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<tr>
<td>200</td>
<td>2Y12 0.11</td>
<td>314</td>
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<td>250</td>
<td>2Y16 0.11</td>
<td>364</td>
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<tr>
<td>300</td>
<td>2Y16 0.12</td>
<td>414</td>
<td>5</td>
<td>5</td>
</tr>
</tbody>
</table>

0.5-hour fire rating, min cover 65mm, f62 mesh top & 1/250 deflection
* based on 20m2 room
^ roof loads

### Continuous Span

---

Appendix IX. SIMPLE DESIGN CHART & SAMPLE CALCULATIONS P3
**Appendix IX. SIMPLE DESIGN CHART & SAMPLE CALCULATIONS**

**C to C spans metres**

<table>
<thead>
<tr>
<th>Beam Depth d mm</th>
<th>Reqd bar size overlap</th>
<th>Avg Conc. vol m³/m²</th>
<th>Total depth mm</th>
<th>Allowable superimposed live load (KPa) and deflection to span ratio.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>5.0</td>
</tr>
<tr>
<td>50</td>
<td>2Y12</td>
<td>0.12*</td>
<td>184</td>
<td>2</td>
</tr>
<tr>
<td>75</td>
<td>2Y12</td>
<td>0.12*</td>
<td>209</td>
<td>2</td>
</tr>
<tr>
<td>100</td>
<td>2Y12</td>
<td>0.12*</td>
<td>234</td>
<td>2</td>
</tr>
<tr>
<td>150</td>
<td>2Y12</td>
<td>0.12*</td>
<td>284</td>
<td>2</td>
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<tr>
<td>200</td>
<td>2Y12</td>
<td>0.13*</td>
<td>334</td>
<td>2</td>
</tr>
<tr>
<td>250</td>
<td>2Y16</td>
<td>0.13*</td>
<td>384</td>
<td>2</td>
</tr>
<tr>
<td>300</td>
<td>2Y16</td>
<td>0.13*</td>
<td>434</td>
<td>2</td>
</tr>
</tbody>
</table>

*1.5 hour fire rating, stc 49, min cover 85mm, f62 mesh & y16 @ 1200mm top*
### SELECTED SECTION PROPERTIES

![Diagram of a beam cross-section with dimensions labeled]

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>b</td>
<td>110mm</td>
</tr>
<tr>
<td>B</td>
<td>155mm</td>
</tr>
<tr>
<td>D</td>
<td>485mm</td>
</tr>
<tr>
<td>d</td>
<td>300mm</td>
</tr>
<tr>
<td>Fck</td>
<td>30N/mm²</td>
</tr>
<tr>
<td>Fy</td>
<td>460N/mm²</td>
</tr>
<tr>
<td>Conc. Cover</td>
<td>25mm</td>
</tr>
<tr>
<td>Rib Spacing</td>
<td>900mm</td>
</tr>
<tr>
<td>Cross sectional Area</td>
<td>177954.57mm²</td>
</tr>
<tr>
<td>Moment of Inertia Ixx</td>
<td>2468.22E6 mm⁴</td>
</tr>
<tr>
<td>Reinforcement at Mid-Span</td>
<td>2*T16 mm Dia.</td>
</tr>
<tr>
<td>Reinforcement at Supports</td>
<td>3*T12 mm Dia.</td>
</tr>
<tr>
<td>Moment Capacity at Mid Span</td>
<td>69.08KN-m</td>
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<tr>
<td>Section Capacity at Support</td>
<td>81.03</td>
</tr>
<tr>
<td>Actual Deflection</td>
<td>1.14mm</td>
</tr>
<tr>
<td>Shear Capacity</td>
<td>0.86N/Sq.mm</td>
</tr>
<tr>
<td>Shear Reinforcement</td>
<td>Please check CORCON test Results.</td>
</tr>
<tr>
<td>Concrete Volume per Sq.m of Plan Area</td>
<td>0.15Cub.m</td>
</tr>
<tr>
<td>Reinforcement per Sq.m of Plan Area</td>
<td>0.26Kg</td>
</tr>
</tbody>
</table>
## SAMPLE CALCULATION

<table>
<thead>
<tr>
<th>PROJECT</th>
<th>The Sukhothai Bangkok</th>
<th>JOB REF</th>
</tr>
</thead>
<tbody>
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<td>CALCULATION BY</td>
<td></td>
<td>CHECKED BY</td>
</tr>
<tr>
<td>Upul Perera</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PART OF STRUCTURE</td>
<td></td>
<td>PRINTED BY</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Decoin Pty Ltd</td>
</tr>
<tr>
<td></td>
<td></td>
<td>DATE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>09/02/2003</td>
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</tbody>
</table>

The Sukhothai Bangkok Project

[Diagram of the Sukhothai Bangkok Project]

UPAL ATTACHMENT

Created with novaPDF Printer (www.novaPDF.com)
<table>
<thead>
<tr>
<th>MEMBER REF</th>
<th>LOAD CALCULATIONS</th>
<th>OUT PUT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loading (Guest Room Area)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(A) Rib Self weight</td>
<td>3.36 kN/m²</td>
</tr>
<tr>
<td></td>
<td>Screed + Finishes</td>
<td>1.177 kN/m²</td>
</tr>
<tr>
<td></td>
<td>Services</td>
<td>0.225 kN/m²</td>
</tr>
<tr>
<td></td>
<td>Ceilings</td>
<td>0.225 kN/m²</td>
</tr>
<tr>
<td></td>
<td>Partitions</td>
<td>3.434 kN/m²</td>
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<tr>
<td></td>
<td>kN/m² - DL</td>
<td>8.421</td>
</tr>
<tr>
<td></td>
<td>Imposed Lead</td>
<td>1.962 kN/m²</td>
</tr>
<tr>
<td></td>
<td>n= 14.93 kN/m²</td>
<td>(ult)</td>
</tr>
<tr>
<td></td>
<td>Udl for Ribs</td>
<td>DL = 8.421 x 1.2 = 10.1 kN/m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LL = 1.962 x 1.2 = 2.4 kN/m</td>
</tr>
<tr>
<td></td>
<td>(B) Plant Room Area</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Rib Self wt.</td>
<td>3.36</td>
</tr>
<tr>
<td></td>
<td>Screed</td>
<td>1.1772</td>
</tr>
<tr>
<td></td>
<td>Services</td>
<td>0.225 kN/m²</td>
</tr>
<tr>
<td></td>
<td>- DL</td>
<td>4.7622</td>
</tr>
<tr>
<td></td>
<td>Live Leads</td>
<td>7.358 kN/m²</td>
</tr>
<tr>
<td></td>
<td>kN/m² - LL</td>
<td></td>
</tr>
<tr>
<td></td>
<td>N= 18.439 kN/m²</td>
<td>(ULT)</td>
</tr>
<tr>
<td></td>
<td>Udl for Ribs</td>
<td>DL = 5.715 kN/m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>LL = 8.829 kN/m</td>
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<tr>
<td></td>
<td>(C ) Storage Room,</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Rib self wt.</td>
<td>3.36 kN/m²</td>
</tr>
<tr>
<td></td>
<td>Screed</td>
<td>1.177</td>
</tr>
<tr>
<td></td>
<td>Services</td>
<td>0.225 kN/m²</td>
</tr>
<tr>
<td></td>
<td>- DL</td>
<td>4.7622</td>
</tr>
<tr>
<td></td>
<td>Imposed Load</td>
<td>4.905 kN/m²</td>
</tr>
<tr>
<td></td>
<td>kN/m² - LL</td>
<td></td>
</tr>
</tbody>
</table>
Appendix IX.

SIMPLE DESIGN CHART & SAMPLE CALCULATIONS

<table>
<thead>
<tr>
<th>MEMBER REF</th>
<th>CALCULATIONS</th>
<th>OUT PUT</th>
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</thead>
<tbody>
<tr>
<td>BS8110 Cl 3.3.4</td>
<td><strong>A. Durability and Fire Resistance of Rib Slab</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Exposure conditions: It is assumed that these rib beams are used in areas where the concrete surfaces are protected</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>OUT PUT</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Appendix IX. SIMPLE DESIGN CHART &amp; SAMPLE CALCULATIONS</strong> P8</td>
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</tbody>
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<tbody>
<tr>
<td>Created with novaPDF Printer (<a href="http://www.novaPDF.com">www.novaPDF.com</a>)</td>
</tr>
</tbody>
</table>
against weather or aggressive conditions.

Therefore treat as mild Exposure Conditions.

For Grade 30 Concrete (30 Mpa) the Nominal cover requirement is as 25 mm, for Durability requirement,

Note: Above cover requirement is including links

Nominal cover for Ribbed slab for :

- 01 hr. Fire Period = 20 mm
- 02 hr. Fire Period = 35 mm for continuous beams
  45 mm for Simply Supported beams

:. use 35 mm cover for bottom bars

Minimum member Dimensions regeared for Ribbed beams, is 125 mm and Slab thickness of 95 mm for 1 hr fire rating?

Same for 2-hr fire rating is 125 mm ribs & 125 mm slab thickness.

<table>
<thead>
<tr>
<th>MEMBER REF</th>
<th>CALCULATIONS</th>
<th>OUT PUT</th>
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</thead>
<tbody>
<tr>
<td>BS 8110 Cl.3.4.4.4</td>
<td>Design for bending. ( \text{Max}^M + \text{re moment in the span} = 71.1 \text{ kNm.} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Effective Depth (d) = 400 – (35+6+15) = 344 mm</td>
<td></td>
</tr>
</tbody>
</table>
\[ K = \frac{M}{bd^2 f_{ca}} = \frac{71.1 \times 10^6}{1179 \times 344^2 \times 30} = 0.017 < k' = 0.156 \]

Hence Compression R/F is not required.

\[ Z = d \left[ 0.5 + \sqrt{\frac{0.25 - \frac{k'}{0.9}}{0.9}} \right] < 0.95d \]
\[ = 344 \times 0.5 + \sqrt{\left( 0.25 - \frac{0.017}{0.9} \right)} \]
\[ = 344 \times 0.98 > 0.95d \]

\[ \therefore Z = 0.95d = 0.95 \times 344 = 327\text{mm} \]

Depth to Neutral axis ‘x’ = (d - z)/0.45
\[ = (344 - 327)/0.45 = 37.8\text{mm} < 100\text{mm} \text{ min. flange thickness} \]

Hence, Neutral axis is within flange thickness.

area of tension R/F,
\[ A_s = \frac{M}{0.87 f_{yz}} = \frac{71.1 \times 10^6}{0.87 \times 450 \times 327} = 555\text{mm}^2 \]
\[ \frac{100(A_s)_{\text{min}}}{b_{w}h} = 0.18 \text{ for } \frac{b_{w}}{b} = \frac{95.5}{1179} = 0.08 < 0.4 \]
\[ \therefore (A_s)_{\text{min}} = \frac{0.18 \times 95.5 \times 400}{100} < 68.7\text{mm}^2 < 555\text{mm}^2 \]

Minimum Steel requirement is satisfactory.

<table>
<thead>
<tr>
<th>MEMBER REF</th>
<th>CALCULATIONS</th>
<th>OUT PUT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max(^{\text{in}}) Support (at 1(^{\text{st}}) intention Support) = -63 kNm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Effective depth to tension R/F; (d = 400 - (25 + 6 + 25 + 20/2) = 334\text{mm} )</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Effective depth to compression R/F; (d' = 35 + 6 + 15 = 56\text{mm} )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
M/bd^2 f_{cu} = \frac{63 \times 10^6}{66 \times 334^2 \times 30} = 0.285 > k'=0.156

∴ Compression Steel is required.

Z=d \left[0.5 + \sqrt{\left(0.25 - \frac{k'}{0.9}\right)}\right]
= 334 \left[0.5 + \sqrt{\left(0.25 - \frac{0.156}{0.9}\right)}\right]
= 334 \times 0.776
= 259 \text{ mm}

x= (d-z)/0.45 = (384-259) / 0.45 = 166.6 \text{ mm}

Area of compression Reinforcements:

A'_s=(k-k') f_{cu}bd^2/0.87 f_y (d-d')
= \frac{(0.285 - 0.156) \times 66 \times 334^2}{0.87 \times 450 \times (334 - 56)} = 261.8 \text{ mm}^2

Area of tension R/F :  A_s = k' f_{cu} bd^2 / 0.87 f_y z + A'_s
= \frac{0.156 \times 30 \times 66 \times 334^2}{0.87 \times 450 \times 259} + 261.8 = 601.6 \text{ mm}^2

minimum tension R/F

\frac{100 A_{s\text{min}}}{b_h h} = 0.26

A_{s\text{min}} = \frac{0.26 \times 95.5 \times 400}{100} = 99.3 \text{ mm}^2 < 601.6 \text{ mm}^2

minimum steel requirement is satisfactory.

BS 8110 Table 327

Compression steel area at support  
A'_s = 261.8 \text{ mm}^2

Tension steel area at support  
A_s = 601.6 \text{ mm}^2
### Calculations

#### Check for Shear

Max\(^m\) shear force at 1\(^{st}\) interior support = 60.6 kN,
Take Max\(^m\) shear force at ‘d’ away from face of the support:

\[ V = 54.8 \text{ kN} \]

Tension Steel provided 2 Y 20 (628 mm\(^2\))

\[
\frac{100A_s}{bd} = \frac{100 \times 628}{95.5 \times 334} = 1.968
\]

\[ v = \frac{54.8 \times 10^3}{95.5 \times 334} = 1.718 \text{ N/mm}^2 \]

Design concrete shear stress \( v_c \)

\[
v_c = 0.79 \left( \frac{100A_s}{bd} \right)^{\frac{1}{3}} \left( \frac{400}{d} \right)^{\frac{1}{3}} f_{\gamma_m} \left( \frac{f_{yw}}{25} \right)^{\frac{1}{3}}
\]

\[ = 0.79 \left( \frac{100 \times 628}{95.5 \times 334} \right)^{\frac{1}{3}} \left( \frac{400/334}{1.25} \right)^{\frac{1}{3}} \left( \frac{30}{25} \right)^{\frac{1}{3}}
\]

\[ v_c = 0.79 \times 1.250 \times \left( \frac{1.046}{1.25} \right) \times 1.0625 = 0.878 \text{ N/mm}^2 \]

\[ A_{sv} \geq b_v \left( v - v_c \right) / 0.87 f_{yw} \]

\[ s_v \leq \frac{0.87 f_{yw} A_{sv}}{b_v \left( v - v_c \right)} \leq 0.87 \times 250 \times \left( 28.3 \times 2 \right) / 95.5 \times \left( 1.718 - 0.898 \right)
\]

\[ s_v \leq 153 \text{ mm} \rightarrow \text{provide R6-150 c/c} \]

max shear link spacing \( 0.75d = 0.75 \times 334 = 250 \text{ mm} \)

minimum link requirement \( A_{sv} \geq 0.4 b_v s_v / 0.87 f_{yw} \)

\[ s_v \leq \frac{0.87 f_{yw} A_{sv}}{0.4b_v} \leq \frac{0.87 \times 250 \times \left( 28.3 \times 2 \right)}{0.4 \times 95.5} = 322 \text{ mm} \]

### Output

Provide Shear links at 1\(^{st}\) interior support R6-150 mm c/c.
<table>
<thead>
<tr>
<th>MEMBER REF</th>
<th>CALCULATIONS</th>
<th>OUT PUT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ref. BS 8110 Table 3.10</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Check for Deflection**, (250mm Rib -n = 11.69kN/m²)

\[
\left( \frac{b_w}{b} \right) = \left( \frac{95.5}{1179} \right) = 0.081
\]

Basic span/Depth ratio = 16.76 (Value between simply supported & continuous)

\[
f_s = \frac{5}{8} \times f_y \times \frac{A_{req}}{A_{prov}} \times \frac{1}{b_w} = \frac{5}{8} \times 450 \times \frac{538}{629} = 240 \text{ N/mm²}
\]

\[
\frac{M}{bd^2} = \frac{71.1 \times 10^6}{1179 \times 344^2} = 0.509
\]

Modification factor = \(0.55 + \frac{(477 - f_s)}{120 \times (0.9 + \frac{M}{bd^2})} \leq 2.0\)

\[
\text{M.F. } = 0.55 + \frac{(477 - 240)}{120(0.9 + 0.509)} = 1.95 \leq 2.0
\]

Allowable span = 16.76 x 1.95 x 344 mm

=11,242 mm > actual span 7,258 mm

Hence: Deflection is satisfactory.
CHECK FOR DEFLECTION (BS8110)

DATA
Bending moment at mid span = 114 kNm (Ultimate)
Average web width \((b_w)\) = 95.5 mm
Effective Flange width \((b)\) = 1179 mm
Effective depth at mid span \((d)\) = 394 mm
Actual span of rib = 7525 mm

Strength of reinforcement \((f_y)\) = 450 N/mm\(^2\)
Area of steel required \((A_{s_{req}})\) = 778 mm\(^2\)
Area of steel provided \((A_{s_{pro}})\) = 805 mm\(^2\)
Moment redistribution ratio = 1

CALCULATIONS

Basic Span/Span Depth ratio = 16.76
The design service stress \((f_s)\) = 271.82 N/mm\(^2\)
Modification factor = 1.67

Allowable Span = 11,046 mm

Deflection is Satisfactory

<table>
<thead>
<tr>
<th>b_w/b</th>
<th>Cantilever</th>
<th>Simply Supported</th>
<th>Continuous</th>
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<tbody>
<tr>
<td>0.1</td>
<td>7</td>
<td>20</td>
<td>26</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Flanged Beams with b_w/b &lt;=0.3</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.6</td>
</tr>
<tr>
<td>16</td>
</tr>
<tr>
<td>20.8</td>
</tr>
</tbody>
</table>

Note: Linear Interpolation between values given above can be used.
BEAM DESIGN

BEAM DESIGN [AS3600- Normal Strength Concrete]

PROJECT: CORCON RIBBED BEAMS FOR TEST IN CHINA

BEAM: RIB BEAM 250-100

DESIGN DATA

- $f_c = 32$ MPa
- $f_y = 500$ MPa
- $f_p = 20000$ MPa
- $D = 400$ mm
- $d_a = 60$ mm
- $b = 1200$ mm
- $d_p = 366$ mm
- $A_w = 402$ mm$^2$
- $A_{ec} = 0$ mm$^2$

DESIGN FOR BENDING

Compressive strain

$\varepsilon_c = 0.003$

$\gamma = 0.85 - 0.007(f_c - 28)$

$0.65 \leq \gamma \leq 0.85$

Neutral axis depth

$d_a = f_y (A_w - A_p) \left( \frac{1}{0.85 f_p b \gamma} \right)$

$\gamma = 7.5$ mm

Lever arm distance

$z = d_a - 0.5 \gamma d_a$

$z = 362.9$ mm

$k_a = \frac{d_a}{d_p}$

$k_a = 0.02$

Check yield Assumptions

Yield strain of steel

$\varepsilon_y = \frac{f_y}{E_s}$

$\varepsilon_y = 0.0025$

Strain in the tensile steel

$\varepsilon_d = \frac{f_p}{E_s}$

$\varepsilon_d = 0.1436$

Strain in the compressive steel

$\varepsilon_w = \frac{f_y}{E_s} - \frac{d_a}{d_p}$

$\varepsilon_w = -0.021$

Concrete Compressive Force

$C_t = 0.85 f_p \gamma b d_a 10^{-3}$

$C_t = 201$ kN

Composite steel force

$C_s = A_w f_y 10^{-3}$

$C_s = 0$ kN

Tensile steel force

$T = A_p f_p 10^{-3}$

$T = 201$ kN

Moment Capacity

$M_u = [f_y A_w (d_a - d_p) + 0.85 (f_p b \gamma d_a) z] 10^{-6}$

$M_u = 72.9$ kNm

Note: if steel not yielded go to next page

$\phi = 0.8$

Design Bending strength

$\phi M_u = 86.4$ kNm
Appendix IX. SIMPLE DESIGN CHART & SAMPLE CALCULATIONS

BEAM DESIGN [AS3600- Normal Strength Concrete]

PROJECT: CORCON RIBBED BEAMS FOR TEST IN CHINA

BEAM: RIB BEAM 150-85

DESIGN DATA

- \( f_c = 32 \) MPa
- \( f_y = 500 \) MPa
- \( E_b = 200000 \) MPa
- \( D = 285 \) mm
- \( d_c = 60 \) mm
- \( b = 1200 \) mm
- \( d_a = 251 \) mm
- \( A_{se} = 0.02 \) mm²
- \( A_{sc} = 0 \) mm²

DESIGN FOR BENDING

Compressive strain

\[ e_{ac} = 0.003 \]

Neutral axis depth

\[ \gamma = 0.85 - 0.007(f_c - 28) \]

\[ \gamma = 0.822 \quad 0.65 \leq \gamma \leq 0.85 \]

\[ d_n = f_y \left( A_{sl} - A_{sc} \right) \left( \frac{1}{0.85 E_b \gamma} \right) \]

\[ d_n = 7.5 \text{ mm} \]

Lever arm distance

\[ z = d_a - 0.5 \gamma d_n \]

\[ z = 247.9 \text{ mm} \]

Check yield Assumptions

Yield strain of steel

\[ e_{sy} = \frac{f_y}{E_b} \]

\[ e_{sy} = 0.0025 \]

Strain in the tensile steel

\[ e_{at} = \frac{d_a - d_n}{d_a} \]

\[ e_{at} = 0.0975 \]

Strain in the compressive steel

\[ e_{sc} = \frac{d_n - d_{sc}}{d_a} \]

\[ e_{sc} = -0.021 \]

Concrete Compressive Force

\[ C_c = 0.85 f_c \gamma b d_n \times 10^{-3} \]

\[ C_c = 201 \text{ kN} \]

Compressive steel force

\[ C_s = A_{sc} f_y \times 10^{-3} \]

\[ C_s = 0 \text{ kN} \]

Tensile steel force

\[ T = A_{at} f_t \times 10^{-3} \]

\[ T = 201 \text{ kN} \]

Moment Capacity

\[ M_b = \left[ f_y \gamma A_{at} \left( d_n - d_a \right) + 0.85 f_c \gamma b d_n e_a^2 \right] \times 10^{-6} \]

\[ M_b = 49.8 \text{ kNm} \]

Note: if steel not yielded go to next page

\[ \phi = 0.8 \]

Design Bending strength

\[ 6\phi M_b = 39.9 \text{ kNm} \]
BEAM DESIGN [AS3600- Normal Strength Concrete]

PROJECT : CORCON RIBBED BEAMS FOR TEST IN CHINA

BEAM : RIB BEAM 90-50

DESIGN DATA

\( f_c = 32 \) MPa
\( f_y = 500 \) MPa \( E_s = 20,000 \) MPa

\( D = 190 \) mm \( d_{cu} = 60 \) mm

\( b = 900 \) mm \( d_{ct} = 164 \) mm

\( A_{se} = 226 \) mm²

\( A_{st} = 0 \) mm²

DESIGN FOR BENDING

Compressive strain
\( \varepsilon_c = 0.003 \)

\( \gamma = 0.85 - 0.007(f_c - 28) \)

Neutral axis depth
\( d_n = f_y (A_{et} - A_{se}) \left( \frac{1}{0.85 f_y b \gamma} \right) \)

\( d_n = 5.6 \) mm

Lever arm distance
\( z = d_n - 0.5 \gamma d_b \)

\( z = 161.7 \) mm

\( k_n = \frac{d_n}{d_b} \)

\( k_n = 0.034 \)

Check yield Assumptions

Yield strain of steel
\( \varepsilon_y = \frac{f_y}{E_s} \)

\( \varepsilon_y = 0.0025 \)

Strain in the tensile steel
\( \varepsilon_{st} = \varepsilon_c \frac{d_{st} - d_{et}}{d_n} \)

\( \varepsilon_{st} = 0.0846 \)

Strain in the compressive steel
\( \varepsilon_{sc} = \varepsilon_c \frac{d_{sc} - d_{et}}{d_n} \)

\( \varepsilon_{sc} = -0.0291 \)

Concrete Compressive Force
\( C_c = 0.85 f_y \gamma b d_b 10^{-3} \)

\( C_c = 113 \) kN

Compressive steel force
\( C_s = A_{se} f_y 10^{-3} \)

\( C_s = 0 \) kN

Tensile steel force
\( T = A_{et} f_y 10^{-3} \)

\( T = 113 \) kN

Moment Capacity
\( M_p = \left[ f_y A_{st} (d_{st} - d_{et}) + 0.85 f_y \gamma d_b \right] \times 10^{-3} \)

\( M_p = 18.3 \) kNm

Note: if steel not yielded go to next page

\( \phi = 0.8 \)

Design Bending strength
\( \delta M_p = 14.6 \) kNm
Appendix IX. SIMPLE DESIGN CHART & SAMPLE CALCULATIONS

BEAM DESIGN [AS3600- Normal Strength Concrete]

PROJECT : CORCON RIBBED BEAMS FOR TEST IN CHINA

BEAM : RIB BEAM 00-05

DESIGN DATA

- $f_c = 32$ MPa
- $f_y = 500$ MPa
- $E_i = 20000$ MPa
- $D = 115$ mm
- $d_w = 60$ mm
- $b = 600$ mm
- $d_a = 89$ mm
- $A_{sh} = 226$ mm$^2$
- $A_{sl} = 0$ mm$^2$

DESIGN FOR BENDING

Compressive strain

$$e_{cm} = 0.003$$

$$\gamma = 0.85 - 0.007(f_c - 28)$$

Neutral axis depth

$$d_n = \frac{f_y}{E_i} \left( A_{sl} - A_{sh} \right) \left( \frac{1}{0.85f_y - \gamma} \right)$$

$$d_n = 8.4$$ mm

Lever arm distance

$$z = d_a - 0.5 \gamma d_h$$

$$z = 85.5$$ mm

$$k_u = \frac{d_a}{d_n}$$

$$k_u = 0.095$$

Check yield Assumptions

Yield strain of steel

$$e_{sy} = \frac{f_y}{E_i}$$

$$e_{sy} = 0.0025$$

Strain in the tensile steel

$$e_{st} = e_{cy} = d_a - d_n$$

$$e_{st} = 0.0287$$

Strain in the compressive steel

$$e_{sc} = e_{cm}$$

$$e_{sc} = -0.0184$$

Concrete Compressive Force

$$C_e = 0.85 f_y - \gamma b - d_a 10^{-3}$$

$$C_e = 113$$ kN

Compressive steel force

$$C_s = A_{sh} f_p 10^{-3}$$

$$C_s = 6$$ kN

Tensile steel force

$$T = A_{sh} f_p 10^{-3}$$

$$T = 113$$ kN

Moment Capacity

$$M_e = \left[ \frac{f_y - A_{sh} (d_n - d_a)}{0.85 f_y - \gamma d_a 2} \right] \cdot 10^{-6}$$

$$M_e = 9.7$$ kNm

Note: if steel not yielded go to next page

$$\phi = 0.8$$

Design Bending strength

$$6.5M_e^{0.875}$$ kNm

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BEAM DESIGN [AS3600- Normal Strength Concrete]

PROJECT: CORCON RIB DESIGN FOR TEST IN CHINA

BEAM: RIB BEAM 300-120

DESIGN DATA

\( f_c = 32 \text{ MPa} \)
\( f_y = 500 \text{ MPa} \)
\( f_{cu} = 700000 \text{ MPa} \)
\( f_{ctf} = 750 \text{ MPa} \)

\( D = 470 \text{ mm} \)
\( d_y = 422 \text{ mm} \)
\( \Delta_{uy} = 56 \text{ mm} \)
\( \Delta_y = 0 \text{ mm} \)

\( b = 75 \text{ mm} \)
\( b_y = 206 \text{ mm} \)

\( A_y = 402 \text{ mm}^2 \)
\( \phi = 0.7 \)

\( N = 0 \text{ kN} \)
\( V = 48.5 \text{ kN} \)

(Factored shear force \( V^* \))

**Design for Shear**

(a) Calculation of \( V_u \)

\[ \beta_1 = 1.1 \left( 1.6 - \frac{d_y}{1000} \right) \]

\[ \beta_1 = 1.206 \]

\[ \beta_2 = 1 + \frac{2 \times 10^3}{1.4 \times A_y} \]

For members with Compressive axial force

\[ \beta_2 = 1 - \frac{3 \times 10^3}{3.5 \times A_y} \]

For members with tensile axial force

\[ \beta_3 = 1 \]

\[ \beta_4 = 1 \]

(As there is no Concentrated load near the support)

\[ b_y = 0.5 (b - b_y) \]

\[ V_u = \frac{1}{\beta_1 \beta_2 \beta_3 \beta_4} \left( \frac{A_{xy}}{b_y d_y} \right) \times 10^{-3} \]

\[ V_u = 46.4 \text{ kN} \]

\[ 0.5 \phi V_u = 16.2 \text{ kN} \]

Greater than applied shear force.
Therefore, it is necessary to design shear reinforcements.

---

**Calculate \( V_u \)**

\[ V_u = V_{uc} + 0.6 b_y d_y \times 10^{-3} \]

\[ V_u = 82.2 \text{ kN} \]

\[ \phi V_u = 57.5 \text{ kN} \]

\[ > V^* \text{ Therefore, provide min. shear r/f.} \]

\[ 250 \text{ mm} < D \]

\[ \delta_{min} = \frac{A_{xy} f_{xy}}{0.35 b_y} \]

\[ \delta_{min} = 565 \text{ mm} \]

Maximum shear link spacing is

\[ \delta_{max} = 0.75 \times D \]

\[ \delta_{max} = 352.5 \text{ mm} \]

Hence provide R6-350 g/g shear links
BEAM DESIGN [AS3600- Normal Strength Concrete]
PROJECT: CORCON RIB DESIGN FOR TEST IN CHINA.
BEAM: 1 RIB BEAM 250-100

DESIGN DATA
- \( f_c = 32 \) MPa
- \( f_y = 500 \) MPa
- \( f_{y,	ext{ef}} = 670 \) MPa
- \( f_y = 650 \) MPa
- \( D = 400 \) mm
- \( d_y = 565 \) mm
- \( A_{sh} = 56 \) mm²
- \( b_1 = 75 \) mm
- \( t_1 = 211 \) mm
- \( \phi = 0.7 \)
- \( N = 0 \) kN
- \( V = 47.4 \) kN

(Totaled shear force \( V^* \))

Design for Shear

(a) Calculation of \( V_{uc} \)

\[
\beta_1 = 1.357
\]

\[
\beta_2 = 1 + \frac{N \cdot 10^6}{14.8 \cdot A_y}
\]

For members with compressive axial force.

\[
\beta_2 = 1 - \frac{N \cdot 10^6}{3.5 \cdot A_y}
\]

For members with tensile axial force.

\[
\beta_2 = 1
\]

(As there is no concentrated load near the support)

\[
b_y = 0.5(b + t_1)
\]

\[
V_{uc} := \beta_1 \beta_2 \beta_3 b_y \frac{1}{d_y} \left( \frac{A_{sh} f_y}{V_{uc} d_y} \right) \cdot 10^{-3}
\]

\[
0.5 \cdot \phi \cdot V_{uc} = 15.6 \ \text{kN}
\]

Greater than applied shear force.

Therefore, it is necessary to design shear reinforcement.

---

**Calculate \( V_{ul,\text{min}} \)**

\[
V_{ul,\text{min}} := V_{uc} + 0.6 b_y d_y 10^{-3}
\]

\[
V_{ul,\text{min}} = 75.9 \ \text{kN}
\]

\[
\phi \cdot V_{ul,\text{min}} = 53.1 \ \text{kN}
\]

\[
> V^* \ \text{Therefore, provide min. shear r/f.}
\]

250 mm < \( D \)

\[
s_{\text{min}} := \frac{A_{sh} f_y}{0.35 d_y}
\]

\[
s_{\text{min}} = 559 \ \text{mm}
\]

Maximum shear link spacing is

\[
s_{\text{max}} := 0.75 \cdot D
\]

\[
s_{\text{max}} = 300 \ \text{mm}
\]

Hence provide R0-300 c/c shear links.
BEAM DESIGN [AS3600- Normal Strength Concrete]
PROJECT: CORCON RIB DESIGN FOR TEST IN CHINA
BEAM: RIB BEAM 150-85

DESIGN DATA

\( f_c = 32 \) MPa
\( f_y = 500 \) MPa
\( f_k = 700000 \) MPa
\( f_{ref} = 750 \) MPa

\( D = 285 \) mm  \( \phi_k = 251 \) mm  \( A_{w} = 56 \) mm²

\( b = 75 \) mm  \( b_k = 236 \) mm  \( A_{k} = 0 \) mm²

\( A_{d} = 402 \) mm²  \( \phi = 0.7 \)

\( N = 0 \) kN  \( V = 41.7 \) kN  \( (\text{Factored shear force } V^*) \)

**Design for Shear**

(a) Calculation of \( V_u \)

\[ \beta_1 = 1.1 \left( 1.6 - \frac{d_k}{1000} \right) \]

\[ \beta_1 = 1.484 \]

\[ \beta_2 = 1 + \frac{N \cdot 10^3}{14 \cdot A_k} \]  \( \text{For members with Compressive axial force} \)

\[ \beta_2 = 1 - \frac{N \cdot 10^3}{3.5 \cdot A_k} \]  \( \text{For members with tensile axial force} \)

\[ \beta_3 = 1 \]

\[ \beta_4 = 1 \]  \( \text{(As there is no Concentrated load near the support)} \)

\[ b_s = 0.5(b + b_k) \]

\[ V_u = \beta_1 \beta_2 \beta_3 \beta_4 \left( \frac{A_{w} f_c}{b_s d_k} \right) \cdot 10^{-3} \]

\[ V_u = 40 \] kN  \( \text{Greater than applied shear force} \)

\[ 0.5 \phi V_u = 14 \] kN  \( \text{Therefore, it is necessary to design shear reinforcements} \)

\[ \text{Calculate } V_u \text{min } \]

\[ V_u \text{min} := V_u + 0.6 b_s \cdot d_k \cdot 10^{-3} \]

\[ V_u \text{min} = 63.4 \] kN  \( \text{Therefore, provide min. shear reinforcement} \)

\[ \phi \cdot V_u \text{min} = 44.4 \] kN  \( \text{Therefore, provide min. shear reinforcement} \)

\[ \frac{A_{w} f_c}{0.35 b_s} \]

\[ \phi_{\text{min}} = 214 \] mm  \( \text{Minimum shear link spacing is} \)

\[ s_{\text{max}} := \frac{0.75 D}{\phi} \]

\[ s_{\text{max}} = 213.75 \] mm  \( \text{Hence provide R6-200 c/c shear links} \)
BEAM DESIGN [AS3600- Normal Strength Concrete]
PROJECT: CORCON RIB DESIGN FOR TEST IN CHINA
BEAM: RIB BEAM 90-50

DESIGN DATA

- $f_c = 32$ MPa
- $f_y = 500$ MPa
- $f'_{cu} = 2800$ MPa
- $E_f = 25000$ MPa

- $d_p = 164$ mm
- $A_{s} = 56$ mm
- $A_{s} = 0$ mm

- $b = 75$ mm
- $t = 234$ mm

- $A_d = 226$ mm
- $\phi = 0.7$

- $N = 0$ kN
- $V = 20$ kN

(Factored shear force $V^*$)

Design for Shear

(a) Calculation of $V_{uc}$

$$
\beta_1 = 1.1 \left(1.6 \cdot \frac{d_p}{1000}\right) \quad \beta_1 = 1.58
$$

$$
\beta_2 = 1 + \frac{2 \cdot 10^3}{4 \cdot A_{s}} \quad \text{For members with Compressive axial force}
$$

$$
\beta_2 = 1 - \frac{3 \cdot 10^4}{3.5 \cdot A_{s}} \quad \text{For members with tensile axial force}
$$

$$
\beta_3 = 1
$$

$$
\beta_4 = 1 \quad \text{(As there is no Concentrated load near the support)}
$$

$$
b_v = 0.5(b + h_v)
$$

$$
V_{uc} = \beta_1 \beta_2 \beta_3 \beta_4 d_p \left(\frac{A_d}{b_v d_p}\right) \cdot 10^{-3}
$$

$$
V_{uc} = 27.5 \quad \text{kN}
$$

$$
0.5 \cdot \phi \cdot V_{uc} = 9.6 \quad \text{kN}
$$

Greater than applied shear force

Therefore, it is necessary to design shear reinforcements.

Calculate $V_{u,min}$

$$
V_{u,min} = V_{uc} + 0.6b_v d_p 10^{-3}
$$

$$
V_{u,min} = 43.7 \quad \text{kN}
$$

$$
\phi \cdot V_{u,min} = 30.6 \quad \text{kN}
$$

$\geq V^*$ and Depth $< 250$ Therefore, provision of min. shear is not required

Shear links not required.
Appendix IX. SIMPLE DESIGN CHART & SAMPLE CALCULATIONS

BEAM DESIGN [AS3600- Normal Strength Concrete]
PROJECT: CORCON RIB DESIGN FOR TEST IN CHINA
BEAM: RIB BEAM 00-65

DESIGN DATA
- $f_y = 32$ MPa
- $f_p = 500$ MPa
- $E_s = 204000$ MPa
- $f_{ct} = 75$ MPa
- $D = 115$ mm
- $d_h = 89$ mm
- $A_{sh} = 56$ mm²
- $b = 200$ mm
- $b_h = 332$ mm
- $A_g = 0$ mm²
- $A_d = 226$ mm²
- $\phi = 0.7$

$N = 0$ kN  $V = 20$ kN  (Factored shear force $V^*$)

Design for Shear

(a) Calculation of $V_{uc}$

$\beta_1 = 1.1 \left(1 - \frac{d_h}{1000}\right)$

$\beta_1 = 1.662$

$\beta_2 = 1 + \frac{N \cdot 10^3}{14 \cdot A_g}$  

For members with Compressive axial force

$\beta_2 = 1 - \frac{N \cdot 10^3}{3.5 \cdot A_g}$  

For members with tensile axial force

$\beta_2 = 1$

$\beta_3 = 1$  

(As there is no Concentrated load near the support)

$b_v = 0.5(b + b_h)$

$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_h \left(\frac{A_{sch} f_s}{b_v d_h}\right) \cdot 10^{-3}$

$V_{uc} = 26.5$ kN

Greater than applied shear force $V^*$  

Therefore, it is necessary to design shear reinforcements.

$0.5 \phi V_{uc} = 9.3$ kN

Calculate $V_{u,min}$

$V_{u,min} = V_{uc} + 0.6 b_v d_h \cdot 10^{-4}$

$V_{u,min} = 40.7$ kN

$V > V^*$ and Depth $< 250$ Therefore, provision of min. shear reinforcement is not required
REFERENCE FIGURES FOR DESIGN