APPENDIX D

STRUCTURAL CALCULATIONS
Loading For The Structure:

LOADING

Roof (Existing):
Dead Loads:
- Tiles = 0.75 kN/m²
- Timber Structure = 0.25 kN/m²
- Ceiling & Services = 0.50 kN/m²
  = 1.50 kN/m²

Live Loads:
- Limited access roof = 0.75 kN/m²

Second, First & Upper Ground Floor (Existing):
Dead Loads:
- Finishes = 0.25 kN/m²
- Timber structure = 0.25 kN/m²
- Ceiling & Services = 0.50 kN/m²
  = 1.00 kN/m²

Live Loads:
- Domestic = 1.50 kN/m²

Lower Ground Floor (New):
Dead Loads:
- 50mm screed = 1.10 kN/m²
- 150mm RC Slab = 3.60 kN/m²
- Ceiling & Services = 0.50 kN/m²
  = 5.20 kN/m²

Live Loads:
- Domestic = 1.50 kN/m²

Basement Floor (New):
Dead Loads:
- 50mm screed = 1.10 kN/m²
- 350m RC slab = 8.60 kN/m²
  = 9.70 kN/m²

Live Loads:
- Domestic = 1.50 kN/m²
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<tr>
<th>LOCATION</th>
<th>CALCULATIONS</th>
<th>OPTIONS</th>
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</table>
| Beam LG62 | $q = 27 \text{kN/m}$  
$M = 615 \text{kNm}$  
$N = 3 \text{kN}$  
$V = 0.01 \text{kN}$  
$\Rightarrow$ See Moment Frame 2 |
| Beam LG63 | $q = 27 \text{kN/m}$  
$M = 70 \text{kNm}$  
$N = 3 \text{kN}$  
$V = 0.01 \text{kN}$  
$\Rightarrow$ See Moment Frame 1 |
| Beam LG64 | $q = 9 \text{kN/m}$  
$M = 43.5 \text{kNm}$  
$N = 24 \text{kN}$  
$V = 0.01 \text{kN}$  
$\Rightarrow$ Use $205x153.46 \text{ UC}$  
$\Rightarrow$ Use $205x153.46 \text{ UC}$ |
| Beam LG65 | $q = 6 \text{kN/m}$  
$M = 12 \text{kNm}$  
$N = 1 \text{kN}$  
$V = 0.01 \text{kN}$  
$\Rightarrow$ Use $205x153.46 \text{ UC}$  
$\Rightarrow$ Use $205x153.46 \text{ UC}$ |
| Beam LG66 | $q = 3 \text{kN/m}$  
$M = 12 \text{kNm}$  
$N = 1 \text{kN}$  
$V = 0.01 \text{kN}$  
$\Rightarrow$ Use $205x153.46 \text{ UC}$  
$\Rightarrow$ Use $205x153.46 \text{ UC}$ |
| Beam LG68 | $q = 27 \text{kN/m}$  
$M = 70 \text{kNm}$  
$N = 3 \text{kN}$  
$V = 0.01 \text{kN}$  
$\Rightarrow$ Use $205x153.46 \text{ UC}$  
$\Rightarrow$ Use $205x153.46 \text{ UC}$ |
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<td>Beam B502</td>
<td>( q_i = 16 \text{ kN/m} )</td>
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<td>( q_i = 10.8 \text{ kN/m} )</td>
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<td>( q_i = 11.5 \text{ kN/m} )</td>
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<td>( q_i = 1.4 \text{ kN/m} )</td>
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<td>( q_i = 0.4 \text{ kN/m} )</td>
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**BEAM BB01**

**STEEL BEAM ANALYSIS & DESIGN (BS5950)**

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

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**Support conditions**

| Support A | Vertically restrained  
| Rotationally free  
| Support B | Vertically restrained  
| Rotationally free  

**Applied loading**

| Beam loads | Dead self weight of beam × 1  
| Dead full UDL 9 kN/m  
| Imposed full UDL 2.6 kN/m  

**Analysis results**

| Maximum moment;  
| M_{max} = 83.6 kNm;  
| M_{min} = 0 kNm  
| Maximum shear;  
| V_{max} = 53.9 kN;  
| V_{min} = -53.9 kN  
| Deflection;  
| \delta_{max} = 24.8 mm;  
| \delta_{min} = 0 mm  
| Maximum reaction at support A;  
| R_{A, dead} = 29.3 kN;  
| R_{A, imposed} = 8.1 kN  
| Unfactored dead load reaction at support A;  
| R_{A, Dead} = 29.3 kN  
| Unfactored imposed load reaction at support A;  
| R_{A, Imposed} = 8.1 kN  
| Maximum reaction at support B;  
| R_{B, dead} = 53.9 kN;  
| R_{B, imposed} = 53.9 kN  

---

**Classification of cross sections - Section 3.5**

| Tensile strain coefficient;  
| \varepsilon = 1.00;  
| Section classification;  
| Compact  

**Shear capacity - Section 4.2.3**

| Design shear force;  
| F_v = 53.9 kN;  
| Design shear resistance;  
| P_v = 241.4 kN  

PASS - Design shear resistance exceeds design shear force

**Moment capacity - Section 4.2.5**

| Design bending moment;  
| M = 83.6 kNm;  
| Moment capacity low shear;  
| M_L = 136.8 kNm  

PASS - Moment capacity exceeds design bending moment

**Check vertical deflection - Section 2.5.2**

| Consider deflection due to dead and imposed loads  
| Limiting deflection;  
| \delta_{lim} = 24.8 mm;  
| Maximum deflection;  
| \delta = 24.763 mm  

PASS - Maximum deflection does not exceed deflection limit

---

**Section details**

| Section type;  
| UKC 203x203x46 (Corus Advance);  
| Steel grade;  
| S275

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**Pringuer-James Consulting Engineers Ltd**
STEEL BEAM ANALYSIS & DESIGN (BS5950)
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

Support conditions
Support A  Vertically restrained  Rotationally free
Support B  Vertically restrained  Rotationally free

Applied loading
Beam loads  Dead self weight of beam \times 1
            Dead full UDL 16 kN/m
            Imposed full UDL 1.7 kN/m
            Dead full UDL 2.3 kN/m

Analysis results
Maximum moment;  \( M_{\text{max}} = 5.2 \text{kNm}; \quad M_{\text{min}} = 0 \text{kNm} \)
Maximum shear;  \( V_{\text{max}} = 17.2 \text{kN}; \quad V_{\text{min}} = -17.2 \text{kN} \)
Deflection;  \( \delta_{\text{max}} = 0.1 \text{mm}; \quad \delta_{\text{min}} = 0 \text{mm} \)
Maximum reaction at support A;  \( R_{A,\text{max}} = 17.2 \text{kN}; \quad R_{A,\text{min}} = 17.2 \text{kN} \)
Unfactored dead load reaction at support B;  \( R_{B,\text{dead}} = 11.1 \text{kN} \)
Unfactored imposed load reaction at support B;  \( R_{B,\text{imposed}} = 1 \text{kN} \)

Classification of cross sections - Section 3.5
Tensile strain coefficient;  \( \varepsilon = 1.00 \);  
Design shear force;  \( F_v = 17.2 \text{kN} \)
Design shear resistance;  \( P_v = 191.1 \text{kN} \)
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5
Design bending moment;  \( M = 5.2 \text{kNm}; \)
Moment capacity low shear;  \( M_l = 70.9 \text{kNm}; \)
PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2
Consider deflection due to dead and imposed loads
Limiting deflection;  \( \delta_{\text{lim}} = 4.8 \text{mm} \)
Maximum deflection;  \( \delta = 0.114 \text{mm} \)
PASS - Maximum deflection does not exceed deflection limit
MOMENT FRAME 2

BEAM LGB2

BEAM BB05
**BEAM BB07**

**STEEL BEAM ANALYSIS & DESIGN (BS5950)**

In accordance with BS5950-1:2000 incorporating Corrigendum No. 1

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**Support conditions**

- **Support A**: Vertically restrained, Rotationally free
- **Support B**: Vertically restrained, Rotationally free

**Applied loading**

- **Beam loads**: Dead self weight of beam × 1
- Dead full UDL 12.5 kN/m
- Imposed full UDL 3.6 kN/m

**Analysis results**

- **Maximum moment**: $M_{\text{max}} = 40.9$ kNm; $M_{\text{min}} = 0$ kNm
- **Maximum shear**: $V_{\text{max}} = 44.2$ kN; $V_{\text{min}} = -44.2$ kN
- **Deflection**: $\delta_{\text{max}} = 4.3$ mm; $\delta_{\text{min}} = 0$ mm
- **Maximum reaction at support A**: $R_{A_{\text{dead}}} = 44.2$ kN; $R_{A_{\text{imposed}}} = 44.2$ kN
- **Unfactored dead load reaction at support A**: $R_{A_{\text{dead}}} = 24$ kN
- **Unfactored imposed load reaction at support A**: $R_{A_{\text{imposed}}} = 6.7$ kN
- **Maximum reaction at support B**: $R_{B_{\text{max}}} = 44.2$ kN; $R_{B_{\text{min}}} = 44.2$ kN

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**Classification of cross sections - Section 3.5**

- **Tensile strain coefficient**: $\varepsilon = 1.00$; 
- **Section classification**: Compact

**Shear capacity - Section 4.2.3**

- **Design shear force**: $F_v = 44.2$ kN; 
- **Design shear resistance**: $P_v = 241.4$ kN

**PASS - Design shear resistance exceeds design shear force**

**Moment capacity - Section 4.2.5**

- **Design bending moment**: $M = 40.9$ kNm; 
- **Moment capacity low shear**: $M_c = 136.8$ kNm

**PASS - Moment capacity exceeds design bending moment**

**Check vertical deflection - Section 2.5.2**

- **Consider deflection due to dead and imposed loads**: Limiting deflection $\delta_{\text{lim}} = 14.8$ mm
- **Maximum deflection**: $\delta = 4.314$ mm

**PASS - Maximum deflection does not exceed deflection limit**
**BEAM BB08**

**STEEL BEAM ANALYSIS & DESIGN (BS5950)**

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

---

**Support conditions**

Support A  
Vertically restrained  
Rotationally free

Support B  
Vertically restrained  
Rotationally free

**Applied loading**

Beam loads  
Dead self weight of beam \( \times 1 \)  
Dead full UDL 12.5 kN/m  
Imposed full UDL 3.6 kN/m

**Analysis results**

Maximum moment;  
\[ M_{\text{max}} = 4.3 \text{kNm}; \quad M_{\text{min}} = 0 \text{kNm} \]

Maximum shear;  
\[ V_{\text{max}} = 14.3 \text{kN}; \quad V_{\text{min}} = -14.3 \text{kN} \]

Deflection;  
\[ \delta_{\text{max}} = 0 \text{mm}; \quad \delta_{\text{min}} = 0 \text{mm} \]

Maximum reaction at support A;  
\[ R_{A_{\text{dead}}} = 7.8 \text{kN}; \quad R_{A_{\text{imposed}}} = 14.3 \text{kN} \]

Unfactored dead load reaction at support A;  
\[ R_{A_{\text{dead}}} = 7.8 \text{kN} \]

Unfactored imposed load reaction at support A;  
\[ R_{A_{\text{imposed}}} = 14.3 \text{kN} \]

Maximum reaction at support B;  
\[ R_{B_{\text{dead}}} = 7.8 \text{kN}; \quad R_{B_{\text{imposed}}} = 14.3 \text{kN} \]

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**Section details**

Section type;  
UKC 203x203x46 (Corus Advance);  
Steel grade;  
S275

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**Classification of cross sections - Section 3.5**

Tensile strain coefficient;  
\( \varepsilon = 1.00 \);

Section classification;  
Compact

**Shear capacity - Section 4.2.3**

Design shear force;  
\( F_{\nu} = 14.3 \text{kN} \);

Design shear resistance;  
\( P_{\nu} = 241.4 \text{kN} \)

**Moment capacity - Section 4.2.5**

Design bending moment;  
\( M = 4.3 \text{kNm} \);

Moment capacity low shear;  
\( M_{\text{low}} = 136.8 \text{kNm} \)

**Check vertical deflection - Section 2.5.2**

Consider deflection due to dead and imposed loads  
Limiting deflection;  
\( \delta_{\text{lim}} = 4.8 \text{mm} \);

Maximum deflection;  
\( \delta = 0.048 \text{mm} \)

**Load Envelope - Combination 1**

**Bending Moment Envelope**

**Shear Force Envelope**

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Unfactored dead load reaction at support B;  
\[ R_{B_{\text{dead}}} = 7.8 \text{kN} \]

Unfactored imposed load reaction at support B;  
\[ R_{B_{\text{imposed}}} = 2.2 \text{kN} \]
BEAM BB09

STEEL BEAM ANALYSIS & DESIGN (BS5950)
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

Support conditions
Support A
Vertically restrained
Rotationally free
Support B
Vertically restrained
Rotationally free

Applied loading
Beam loads
Dead self weight of beam × 1
Dead full UDL 9 kN/m
Imposed full UDL 2.6 kN/m

Analysis results
Maximum moment;
M_{max} = 83.6 kNm;
M_{min} = 0 kNm
Maximum shear;
V_{max} = 53.9 kN;
V_{min} = -53.9 kN
Deflection;
\delta_{max} = 24.8 mm;
\delta_{min} = 0 mm
Maximum reaction at support A;
R_{A,dead} = 29.3 kN;
R_{A,imposed} = 8.1 kN
Unfactored dead load reaction at support B;
R_{B,dead} = 29.3 kN
Unfactored imposed load reaction at support B;
R_{B,imposed} = 8.1 kN
Maximum reaction at support B;
R_{B,dead} = 53.9 kN;
R_{B,imposed} = 53.9 kN

Classification of cross sections - Section 3.5
Tensile strain coefficient; \epsilon = 1.00;
Section classification; Compact
Shear capacity - Section 4.2.3
Design shear force; F_v = 53.9 kN
Design shear resistance; P_v = 241.4 kN
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5
Design bending moment; M = 83.6 kNm;
Moment capacity low shear; M_{l} = 136.8 kNm
PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2
Consider deflection due to dead and imposed loads
Limiting deflection \delta_{lim} = 24.8 mm
Maximum deflection \delta = 24.763 mm
PASS - Maximum deflection does not exceed deflection limit
**BEAM BB12**

**STEEL BEAM ANALYSIS & DESIGN (BS5950)**  
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

**Section details**  
Section type: UKC 203x203x60 (Corus Advance);  
Steel grade: S275

**Support conditions**  
Support A: Vertically restrained; Rotationally free  
Support B: Vertically restrained; Rotationally free

**Applied loading**  
Beam loads:  
- Dead self weight of beam x 1  
- Dead full UDL 92 kN/m  
- Imposed full UDL 2 kN/m  
- Dead point load 25.8 kN at 1800 mm  
- Imposed point load 7 kN at 1800 mm

**Analysis results**  
- Maximum moment: $M_{\text{max}} = 143.3$ kNm; $M_{\text{min}} = 0$ kNm  
- Maximum shear: $V_{\text{max}} = 195.1$ kN; $V_{\text{min}} = -210.9$ kN  
- Deflection: $\delta_{\text{max}} = 6.1$ mm; $\delta_{\text{min}} = 0$ mm  
- Maximum reaction at support A: $R_{\text{A, max}} = 195.1$ kN; $R_{\text{A, min}} = 0$ kN  
- Unfactored dead load reaction at support A: $R_{\text{A, Dead}} = 133.6$ kN

**Classification of cross sections - Section 3.5**  
Tensile strain coefficient: $\varepsilon = 1.00$; Section classification: Plastic

**Shear capacity - Section 4.2.3**  
Design shear force: $F_v = 210.9$ kN;  
Design shear resistance: $P_v = 325.1$ kN  
**PASS** - Design shear resistance exceeds design shear force

**Moment capacity - Section 4.2.5**  
Design bending moment: $M = 143.3$ kNm;  
Moment capacity high shear: $M = 177.9$ kNm  
**PASS** - Moment capacity exceeds design bending moment

**Check vertical deflection - Section 2.5.2**  
Consider deflection due to dead and imposed loads  
Limiting deflection: $\delta_{\text{lim}} = 7.5$ mm;  
Maximum deflection: $\delta = 6.128$ mm  
**PASS** - Maximum deflection does not exceed deflection limit
Support conditions
Support A
Vertically restrained
Rotationally free
Support B
Vertically restrained
Rotationally free

Applied loading
Beam loads
Dead self weight of beam \times 1
Dead full UDL 92 kN/m
Imposed full UDL 2 kN/m

Analysis results
Maximum moment;
\( M_{\text{max}} = 53.7 \text{kNm} \);
\( M_{\text{min}} = 0 \text{kNm} \)
Maximum shear;
\( V_{\text{max}} = 119.4 \text{kN} \);
\( V_{\text{min}} = -119.4 \text{kN} \)
Deflection;
\( \delta_{x\text{ax}} = 1.4 \text{mm} \);
\( \delta_{x\text{xx}} = 0 \text{mm} \)
Maximum reaction at support A:
\( R_{A,\text{dead}} = 83.2 \text{kN} \);
\( R_{A,\text{imposed}} = 1.8 \text{kN} \)
Unfactored dead load reaction at support A:
\( R_{A,\text{dead}} = 83.2 \text{kN} \)
Unfactored imposed load reaction at support A:
\( R_{A,\text{imposed}} = 1.8 \text{kN} \)
Maximum reaction at support B:
\( R_{B,\text{max}} = 119.4 \text{kN} \);
\( R_{B,\text{min}} = 119.4 \text{kN} \)
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

**Support conditions**
- **Support A**: Vertically restrained
  Rotationally free
- **Support B**: Vertically restrained
  Rotationally free

**Applied loading**
- **Beam loads**
  - Dead self weight of beam × 1
  - Dead full UDL 10.1 kN/m
  - Imposed full UDL 2.9 kN/m
  - Dead point load 18.1 kN at 200 mm
  - Dead point load 18.1 kN at 200 mm

**Analysis results**
- Maximum moment:
  - $M_{max} = 61.1$ kNm; $M_{min} = 0$ kNm
- Maximum shear:
  - $V_{max} = 95.2$ kN; $V_{min} = 48.7$ kN
- Deflection:
  - $\delta_{max} = 11$ mm; $\delta_{min} = 0$ mm
- Maximum reaction at support A:
  - $R_{A, max} = 95.2$ kN; $R_{A, min} = 95.2$ kN
- Unfactored dead load reaction at support A:
  - $R_{A, Dead} = 60$ kN
- Maximum reaction at support B:
  - $R_{B, max} = 48.7$ kN; $R_{B, min} = 48.7$ kN
- Unfactored dead load reaction at support B:
  - $R_{B, Dead} = 26.6$ kN
- Unfactored imposed load reaction at support B:
  - $R_{B, Imp} = 7$ kN

**Section details**
- **Section type**: UKC 203x203x46 (Corus Advance)
- **Steel grade**: S275

**Classification of cross sections - Section 3.5**
- Tensile strain coefficient: $\varepsilon = 1.00$
- Section classification: Compact

**Shear capacity - Section 4.2.3**
- Design shear force: $F_v = 95.2$ kN
- Design shear resistance: $P_v = 241.4$ kN
  - **PASS - Design shear resistance exceeds design shear force**

**Moment capacity - Section 4.2.5**
- Design bending moment: $M = 61.1$ kNm
- Moment capacity low shear: $M_L = 136.8$ kNm
  - **PASS - Moment capacity exceeds design bending moment**

**Check vertical deflection - Section 2.5.2**
- Consider deflection due to dead and imposed loads
- Limiting deflection: $\delta_{lim} = 13.333$ mm
- Maximum deflection: $\delta = 11.044$ mm
  - **PASS - Maximum deflection does not exceed deflection limit**
**BEAM BB17**

**STEEL BEAM ANALYSIS & DESIGN (BS5950)**

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

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**Support conditions**

Support A
- Vertically restrained
- Rotationally free

Support B
- Vertically restrained
- Rotationally free

**Applied loading**

Beam loads
- Dead self weight of beam × 1
- Dead full UDL 10.3 kN/m
- Imposed full UDL 2.9 kN/m

**Analysis results**

- Maximum moment:
  - \( M_{\text{max}} = 56.7 \text{kNm} \)
  - \( M_{\text{min}} = 0 \text{kNm} \)

- Maximum shear:
  - \( V_{\text{max}} = 47.3 \text{kN} \)
  - \( V_{\text{min}} = -47.3 \text{kN} \)

- Deflection:
  - \( \delta_{\text{max}} = 10.1 \text{mm} \)
  - \( \delta_{\text{min}} = 0 \text{mm} \)

- Maximum reaction at support A:
  - \( R_{A_{\text{max}}} = 47.3 \text{kN} \)
  - \( R_{A_{\text{min}}} = 47.3 \text{kN} \)

- Unfactored dead load reaction at support A:
  - \( R_{A_{\text{dead}}} = 25.8 \text{kN} \)

- Unfactored imposed load reaction at support A:
  - \( R_{A_{\text{imposed}}} = 7 \text{kN} \)

- Maximum reaction at support B:
  - \( R_{B_{\text{max}}} = 47.3 \text{kN} \)
  - \( R_{B_{\text{min}}} = 47.3 \text{kN} \)

---

**Classification of cross sections - Section 3.5**

- Tensile strain coefficient: \( \varepsilon = 1.00 \)
- Section classification: Compact

**Shear capacity - Section 4.2.3**

- Design shear force: \( F_v = 47.3 \text{kN} \)
- Design shear resistance: \( P_v = 241.4 \text{kN} \)

**Moment capacity - Section 4.2.5**

- Design bending moment: \( M = 56.7 \text{kNm} \)
- Moment capacity low shear: \( M_s = 136.8 \text{kNm} \)

**Check vertical deflection - Section 2.5.2**

- Consider deflection due to dead and imposed loads
- Limiting deflection: \( \delta_{\text{lim}} = 19.2 \text{mm} \)
- Maximum deflection: \( \delta = 10.077 \text{mm} \)

**Pass - Design shear resistance exceeds design shear force**

**Pass - Moment capacity exceeds design bending moment**

---

Unfactored dead load reaction at support B: \( R_{B_{\text{dead}}} = 25.8 \text{kN} \)

Unfactored imposed load reaction at support B: \( R_{B_{\text{imposed}}} = 7 \text{kN} \)

---

**Section details**

- Section type: UKC 203x203x46 (Corus Advance)
- Steel grade: S275

---

**Support conditions**

Support A
- Vertically restrained
- Rotationally free

Support B
- Vertically restrained
- Rotationally free

**Applied loading**

Beam loads
- Dead self weight of beam × 1
- Dead full UDL 10.3 kN/m
- Imposed full UDL 2.9 kN/m

**Analysis results**

- Maximum moment:
  - \( M_{\text{max}} = 56.7 \text{kNm} \)
  - \( M_{\text{min}} = 0 \text{kNm} \)

- Maximum shear:
  - \( V_{\text{max}} = 47.3 \text{kN} \)
  - \( V_{\text{min}} = -47.3 \text{kN} \)

- Deflection:
  - \( \delta_{\text{max}} = 10.1 \text{mm} \)
  - \( \delta_{\text{min}} = 0 \text{mm} \)

- Maximum reaction at support A:
  - \( R_{A_{\text{max}}} = 47.3 \text{kN} \)
  - \( R_{A_{\text{min}}} = 47.3 \text{kN} \)

- Unfactored dead load reaction at support A:
  - \( R_{A_{\text{dead}}} = 25.8 \text{kN} \)

- Unfactored imposed load reaction at support A:
  - \( R_{A_{\text{imposed}}} = 7 \text{kN} \)

- Maximum reaction at support B:
  - \( R_{B_{\text{max}}} = 47.3 \text{kN} \)
  - \( R_{B_{\text{min}}} = 47.3 \text{kN} \)
STEEL BEAM ANALYSIS & DESIGN (BS5950)
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

Support conditions
Support A  Vertically restrained  Rotationally free
Support B  Vertically restrained  Rotationally free

Applied loading
Beam loads  Dead self weight of beam × 1
Dead partial UDL 24 kN/m from 900 mm to 1900 mm

Analysis results
Maximum moment;
\[ M_{\text{max}} = 9.4 \text{ kNm}; \quad M_{\text{min}} = 0 \text{ kNm} \]
Maximum shear;
\[ V_{\text{max}} = 9.4 \text{ kN}; \quad V_{\text{min}} = -25.4 \text{ kN} \]
Deflection;
\[ \delta_{\text{max}} = 0.2 \text{ mm}; \quad \delta_{\text{min}} = 0 \text{ mm} \]
Maximum reaction at support A;
\[ R_{A,\text{max}} = 9.4 \text{ kN}; \quad R_{A,\text{min}} = 9.4 \text{ kN} \]
Unfactored dead load reaction at support A;
\[ R_{A,\text{Dead}} = 6.7 \text{ kN} \]
Maximum reaction at support B;
\[ R_{B,\text{max}} = 25.4 \text{ kN}; \quad R_{B,\text{min}} = 25.4 \text{ kN} \]
Unfactored dead load reaction at support B;
\[ R_{B,\text{Dead}} = 18.1 \text{ kN} \]

Classification of cross sections - Section 3.5
Tensile strain coefficient; \( \epsilon = 1.00 \);
Section classification; Compact

Shear capacity - Section 4.2.3
Design shear force; \( F_v = 25.4 \text{ kN} \);
Design shear resistance; \( P_v = 241.4 \text{ kN} \)
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5
Design bending moment; \( M = 9.4 \text{ kNm} \);
Moment capacity low shear; \( M_c = 136.8 \text{ kNm} \)
PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2
Consider deflection due to dead and imposed loads
Limiting deflection \( \delta_{\text{lim}} = 5.278 \text{ mm} \);
Maximum deflection; \( \delta = 0.245 \text{ mm} \)
PASS - Maximum deflection does not exceed deflection limit

Section details
Section type;  UKC 203x203x46 (Corus Advance);
Steel grade;  S275
BEAM BB20

STEEL BEAM ANALYSIS & DESIGN (BS5950)
In accordance with BS5950-1:2000 incorporating Corrigendum No.1
TEDDS calculation version 3.0.04

Maximum reaction at support A:
- Unfactored dead load reaction at support A: $R_{A,\text{Dead}} = 34.8 \text{ kN}$
- Unfactored imposed load reaction at support A: $R_{A,\text{Imposed}} = 10.1 \text{ kN}$
- Maximum reaction at support A: $R_{A_{\text{MAX}}} = 64.9 \text{ kN}$
- Unfactored dead load reaction at support B: $R_{B,\text{Dead}} = 34.8 \text{ kN}$
- Unfactored imposed load reaction at support B: $R_{B,\text{Imposed}} = 10.1 \text{ kN}$
- Maximum reaction at support B: $R_{B_{\text{MAX}}} = 64.9 \text{ kN}$

Support conditions
- Support A: Vertically restrained, Rotationally free
- Support B: Vertically restrained, Rotationally free

Applied loading
- Beam loads
  - Dead self weight of beam $\times 1$
  - Dead full UDL 7.8 kN/m
  - Imposed full UDL 2.3 kN/m
  - Dead point load 9 kN at 200 mm
  - Imposed point load 3 kN at 200 mm
  - Dead point load 9 kN at 6000 mm
  - Imposed point load 3 kN at 6000 mm

Analysis results
- Maximum moment: $M_{\text{MAX}} = 77.1 \text{ kNm}$
- Maximum shear: $V_{\text{MAX}} = 64.9 \text{ kN}$
- Deflection: $\delta_{\text{MAX}} = 20 \text{ mm}$

Classification of cross sections - Section 3.5
- Tensile strain coefficient: $\varepsilon = 1.00$
- Section classification: Plastic

Shear capacity - Section 4.2.3
- Design shear force: $F_v = 64.9 \text{ kN}$
- Design shear resistance: $P_v = 268.8 \text{ kN}$
  - PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5
- Design bending moment: $M = 77.1 \text{ kNm}$
- Moment capacity low shear: $M_L = 156 \text{ kNm}$
  - PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2
- Consider deflection due to dead and imposed loads
- Limiting deflection: $\delta_{\text{lim}} = 24.8 \text{ mm}$
  - PASS - Maximum deflection does not exceed deflection limit

Section details
- Section type: UKC 203x203x52 (Corus Advance)
- Steel grade: S275

Load Envelope - Combination 1

Bending Moment Envelope

Shear Force Envelope
**Beam BB23**

**Steel Beam Analysis & Design (BS5950)**

In accordance with BS5950 - 1:2000 incorporating Corrigendum No.1

---

**Support conditions**

Support A  
Vertically restrained  
Rotationally free

Support B  
Vertically restrained  
Rotationally free

**Applied Loading**

Beam loads  
Dead self weight of beam × 1  
Dead full UDL 13 kN/m  
Imposed full UDL 3.8 kN/m

**Analysis results**

Maximum moment:  
\[ M_{\text{max}} = 4.4 \text{ kNm}; \quad M_{\text{min}} = 0 \text{ kNm} \]

Maximum shear:  
\[ V_{\text{max}} = 14.8 \text{ kN}; \quad V_{\text{min}} = -14.8 \text{ kN} \]

Deflection:  
\[ \delta_{\text{max}} = 0.1 \text{ mm}; \quad \delta_{\text{min}} = 0 \text{ mm} \]

Maximum reaction at support A:  
\[ R_{A,\text{max}} = 14.8 \text{ kN}; \quad R_{A,\text{min}} = 14.8 \text{ kN} \]

Unfactored dead load reaction at support A:  
\[ R_{A,\text{dead}} = 7.9 \text{ kN} \]

Unfactored imposed load reaction at support A:  
\[ R_{A,\text{Imposed}} = 2.3 \text{ kN} \]

Maximum reaction at support B:  
\[ R_{B,\text{max}} = 14.8 \text{ kN}; \quad R_{B,\text{min}} = 14.8 \text{ kN} \]

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**Section Details**

Section type: UKB 203x133x25 (Corus Advance)  
Steel grade: S275

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**Classification of Cross Sections - Section 3.5**

Tensile strain coefficient: \( \varepsilon = 1.00 \)

**Shear Capacity - Section 4.2.3**

Design shear force:  
\[ F_v = 14.8 \text{ kN}; \]

Design shear resistance:  
\[ P_v = 191.1 \text{ kN} \]

PASS - Design shear resistance exceeds design shear force

**Moment Capacity - Section 4.2.5**

Design bending moment:  
\[ M = 4.4 \text{ kNm} \]

Moment capacity low shear:  
\[ M_l = 70.9 \text{ kNm} \]

PASS - Moment capacity exceeds design bending moment

**Check Vertical Deflection - Section 2.5.2**

Consider deflection due to dead and imposed loads

Limiting deflection:  
\[ \delta_{\text{lim}} = 4.8 \text{ mm}; \]

Maximum deflection:  
\[ \delta = 0.096 \text{ mm} \]

PASS - Maximum deflection does not exceed deflection limit
COLUMNS C2 PAD FOUNDATION

PAD FOOTING ANALYSIS AND DESIGN (BS8110-1-1997)

**Partial safety factors for loads**
- Dead loads; \( \gamma_G = 1.40 \)
- Imposed loads; \( \gamma_W = 1.60 \)
- Wind loads; \( \gamma_W = 0.00 \)

**Ultimate axial loading on column**
- Ultimate axial load on column; \( P_{uA} = 567.6 \) kN

**Ultimate foundation loads**
- Ultimate foundation load; \( F_y = 48.2 \) kN

**Ultimate horizontal loading on column**
- Ultimate moment on column
  - Ultimate moment on column in x dir; \( M_{ux} = 0.000 \) kNm
  - Ultimate moment on column in y dir; \( M_{uy} = 0.000 \) kNm

**Ultimate pad base reaction**
- Eccentricity of ultimate base reaction in x; \( \phi_{xcu} = 0 \) mm
- Eccentricity of ultimate base reaction in y; \( \phi_{ycu} = 0 \) mm

**Calculate ultimate pad base pressures**
- Maximum ultimate pad base pressure; \( q_{ux} = 190.053 \) kN/m²; \( q_{uy} = 190.053 \) kN/m²; \( q_{ux} = 190.053 \) kN/m²

**Library item: Ultimate pressures summary**
- Ultimate moments on column
  - Ultimate moment in x dir; \( M_x = 127.710 \) kNm
  - Ultimate moment in y dir; \( M_y = 127.710 \) kNm

**Material details**
- Nom. cover to reinforcement; \( c = 40 \) mm

**Moment design in x direction**
- Tens. reinforcement diameter; \( \phi_{rd} = 16 \) mm
- Tens. reinforcement depth; \( d_t = 402 \) mm

**Design formula for rectangular beams (cl 3.4.4.4)**
- Tens. reinforcement provided; \( A_{x,thr,sfr} = 2413 \) mm²

**Moment design in y direction**
- Tens. reinforcement diameter; \( \phi_{rd} = 16 \) mm
- Tens. reinforcement depth; \( d_t = 386 \) mm

---

**Pad footing details**
- Length of pad footing; \( L = 1800 \) mm
- Width of pad footing; \( B = 1800 \) mm
- Depth of pad footing; \( h = 450 \) mm
- Depth of soil over pad footing; \( h_{so} = 0 \) mm
- Density of concrete; \( f_G = 23.6 \) kN/m³

**Column details**
- Column base length; \( l_a = 300 \) mm
- Column base width; \( b_x = 300 \) mm
- Column eccentricity in x; \( \phi_{xu} = 0 \) mm
- Column eccentricity in y; \( \phi_{yu} = 0 \) mm

**Soil details**
- Depth of soil over pad footing; \( h_{so} = 0 \) mm
- Density of soil; \( f_Q = 20.0 \) kN/m²

**Axial loading on column (From Moment Frame 2 Analysis)**
- Dead axial load; \( P_{ux} = 370.0 \) kN
- Total axial load; \( P_A = 435.0 \) kN

**Foundation loads**
- Total base reaction; \( T = 469.4 \) kN
- Base reaction eccentricity in x; \( \phi_{x} = 0 \) mm
- Base reaction eccentricity in y; \( \phi_{y} = 0 \) mm

**Calculate pad base reaction**
- Total base reaction; \( T = 469.4 \) kN
- Base reaction eccentricity in x; \( \phi_{x} = 0 \) mm
- Base reaction eccentricity in y; \( \phi_{y} = 0 \) mm

**Calculate pad base pressures**
- \( q_S = 144.879 \) kN/m²
- \( q_x = 144.879 \) kN/m²
- \( q_y = 144.879 \) kN/m²
- \( q_{ux} = 144.879 \) kN/m²
- \( q_{uy} = 144.879 \) kN/m²
- Minimum base pressure; \( q_{min} = 144.879 \) kN/m²
- Maximum base pressure; \( q_{max} = 144.879 \) kN/m²

**PASS - Maximum base pressure is less than allowable bearing pressure**
Design formula for rectangular beams (cl 3.4.4.4);

- $K_y = 0.016$
- $K_y' = 0.156$

$K_y < K_y'$, compression reinforcement is not required

Tens. reinforcement required: $A_{s, y, req} = 801 \text{ mm}^2$
Minimum tens. reinforcement: $A_{s, y, min} = 1053 \text{ mm}^2$

Tens. reinforcement provided: 12 No. 16 dia. bars btm.
$A_{s, y, prov} = 2413 \text{ mm}^2$

PASS - Tension reinforcement provided exceeds tension reinforcement required

Calculate ultimate shear force at d from top face of column

Ult. pressure for shear: $q_{su} = 190.053 \text{ kN/m}^2$

Area loaded for shear: $A_s = 0.655 \text{ m}^2$
Ult. shear force: $V_{su} = 114.781 \text{ kN}$

Shear stresses at d from top face of column (cl 3.5.5.2)

Design shear stress: $v_{su} = 0.165 \text{ N/mm}^2$
Design concrete shear stress: $v_c = 0.471 \text{ N/mm}^2$
Allowable design shear stress: $v_{max} = 4.382 \text{ N/mm}^2$

PASS - $v_{su} < v_c$ - No shear reinforcement required

Calculate ultimate punching shear force at face of column

Ult. pressure for punching shear: $q_{pu} = 190.053 \text{ kN/m}^2$
Avg. effective rein. depth: $d = 394 \text{ mm}$

Area loaded: $A_{pu} = 0.090 \text{ m}^2$
Length of shear perimeter: $u_{pu} = 1200 \text{ mm}$
Ult. punching shear force: $V_{pu} = 551.833 \text{ kN}$

Punching shear stresses at face of column (cl 3.7.7.2)

Design shear stress: $v_{puA} = 1.167 \text{ N/mm}^2$

PASS - Design shear stress is less than allowable design shear stress

Calculate ultimate punching shear force at perimeter of 1.5 d from face of column

Ult. pressure for punching shear: $q_{puA1.5d} = 190.053 \text{ kN/m}^2$
Avg. effective rein. depth: $d = 394 \text{ mm}$

Area loaded: $A_{puA1.5d} = 2.668 \text{ m}^2$
Length of shear perimeter: $u_{puA1.5d} = 3600 \text{ mm}$
Ult. punching shear force: $V_{puA1.5d} = 125.345 \text{ kN}$

Punching shear stresses at perimeter of 1.5 d from face of column (cl 3.7.7.2)

Design shear stress: $v_{puA1.5d} = 0.088 \text{ N/mm}^2$

PASS - $v_{puA1.5d} < v_c$ - No shear reinforcement required
<table>
<thead>
<tr>
<th>WALL WIDTH</th>
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<td><strong>LEVEL</strong></td>
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<td>1st Floor</td>
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<tr>
<td>Wall</td>
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<td>1.5</td>
</tr>
<tr>
<td>RC Wall</td>
<td>3</td>
<td>-</td>
</tr>
</tbody>
</table>

- **Wall W1:**

- **Wall W2:**

- **Wall W3:**

  - Load = (W1 + W2 + W3) x wall load

  - 5.8
  - 105.7 - 15.13
RAFT SLAB DESIGN AT COLUMN C1

RAFT FOUNDATION DESIGN (BS8110: PART 1 : 1997)

Soil and raft definition

Soil definition
- Allowable bearing pressure: $q_{adm} = 40.0 \text{ kN/m}^2$
- Number of types of soil forming sub-soil: One type only
- Soil density: Firm to loose
- Depth of hardcore beneath slab: $h_{corebtm} = 0 \text{ mm}$ (Dispersal allowed for bearing pressure check)
- Depth of hardcore beneath thickening: $h_{corebtm} = 0 \text{ mm}$ (Dispersal allowed for bearing pressure check)
- Density of hardcore: $\gamma_h = 20.0 \text{ kN/m}^3$
- Basic assumed diameter of local depression: $\phi_h = 2000 \text{ mm}$
- Diameter under slab modified for hardcore: $\phi_h = \phi_h - \phi_h \times h_{corebtm} = 2000 \text{ mm}$
- Diameter under thickening modified for hardcore: $\phi_h = \phi_h - \phi_h \times h_{corebtm} = 2000 \text{ mm}$

Raft slab definition
- Max dimension/max dimension between joints: $l_{max} = 6.200 \text{ m}$
- Slab thickness: $h_{slab} = 250 \text{ mm}$
- Concrete strength: $f_c = 40 \text{ N/mm}^2$
- Poisson's ratio of concrete: $\nu = 0.2$
- Slab mesh reinforcement strength: $f_y = 500 \text{ N/mm}^2$
- Partial safety factor for steel reinforcement: $f_s = 1.15$
- From CACA document 'Concrete ground floors' Table 5

Minimum mesh required in top for shrinkage: $A_{142}$
- Actual mesh provided in top: $A_{393} (h_{slab} = 393 \text{ mm}^2/m)$
- Mesh provided in bottom: $A_{393} (h_{slab} = 393 \text{ mm}^2/m)$
- Top mesh bar diameter: $\phi_{top} = 10 \text{ mm}$
- Bottom mesh bar diameter: $\phi_{top} = 10 \text{ mm}$
- Cover to top reinforcement: $c_{top} = 20 \text{ mm}$
- Cover to bottom reinforcement: $c_{btm} = 35 \text{ mm}$
- Average effective depth of top reinforcement: $d_{htop} = h_{slab} \times c_{top} = 220 \text{ mm}$
- Average effective depth of bottom reinforcement: $d_{hbtm} = h_{slab} \times c_{btm} = 205 \text{ mm}$
- Overall average effective depth: $d_{htop} = (d_{hbtm} + d_{hbtm} + d_{hbtm})/2 = 213 \text{ mm}$
- Minimum effective depth of top reinforcement: $d_{htop} = (d_{hbtm} + d_{hbtm} + d_{hbtm})/2 = 215 \text{ mm}$
- Minimum effective depth of bottom reinforcement: $d_{hbtm} = (d_{hbtm} + d_{hbtm} + d_{hbtm})/2 = 200 \text{ mm}$

Edge beam definition

Overall depth; $h_{edge} = 450 \text{ mm}$
- Width; $b_{edge} = 1000 \text{ mm}$
- Angle of chamfer to horizontal; $\alpha = 45 \text{ deg}$
- Strength of main bar reinforcement; $f_y = 500 \text{ N/mm}^2$
- Strength of link reinforcement; $f_y = 500 \text{ N/mm}^2$
- Reinforcement provided in top; $6 \times 40 \text{ mm}^2$ (Annex E3 $= 393 \text{ mm}$)
- Reinforcement provided in bottom; $6 \times 40 \text{ mm}^2$ (Annex E3 $= 393 \text{ mm}$)
- Link reinforcement provided; $3 \times 12 \text{ mm}^2$ at 100 ctrs ($A_{12} = 3.393 \text{ mm}$)
- Bottom cover to links; $c_{btm} = 35 \text{ mm}$
- Effective depth of top reinforcement; $d_{htop} = h_{slab} \times c_{top} = 220 \text{ mm}$
- Effective depth of bottom reinforcement; $d_{hbtm} = h_{slab} \times c_{btm} = 205 \text{ mm}$

Internal slab design checks

Basic loading
- Slab self weight: $w_{slab} = 24 \text{ kN/m}^2$ (Disn $= 6.0 \text{ kN/m}^2$
- Hardcore: $w_{core} = 0 \text{ kN/m}^2$

Applied loading
- Uniformly distributed dead load: $w_{edl} = 1.5 \text{ kN/m}^2$
- Uniformly distributed live load: $w_{edl} = 1.5 \text{ kN/m}^2$

Internal slab bearing pressure check
- Total uniform load at formation level; $w_{tot} = w_{slab} + w_{core} + w_{edl} + w_{edl} = 9.0 \text{ kN/m}^2$
- $w_{tot} = w_{slab} + w_{core} + w_{edl} + 500 \text{ kN/m}^2$

PASS: $w_{tot} <= q_{adm} - $ Applied bearing pressure is less than allowable

Internal slab bending and shear check

Applied bending moments
- Span of slab; $L = 2220 \text{ mm}$
- Ultimate self weight; $w_{slab} = 1.4 \times w_{slab} = 8.4 \text{ kN/m}^2$
- $M_{edl} = w_{edl} \times L / 6 + 0.8 \text{ kN/m}$
- $M_{edl} = w_{edl} \times L / 6 + 1.3 \text{ kN/m}$
- $V_{edl} = w_{edl} \times L / 4 = 4.7 \text{ kN/m}$

Moments due to applied uniformly distributed loads
- Ultimate applied; $M_{edl} = 1.4 \times M_{edl} = 4.5 \text{ kN/m}^2$
- $M_{edl} = w_{edl} \times L / 2 = 0.7 \text{ kN/m}$
- $V_{edl} = w_{edl} \times L / 4 = 2.5 \text{ kN/m}$

Resultant moments and shears
- Total moment at edge; $M_{edl} = 2.0 \text{ kN/m}$
- Total moment at centre; $M_{edl} = 1.2 \text{ kN/m}$
- Total shear force; $V_{edl} = 7.2 \text{ kN/m}$

Reinforcement required in top
- K factor; $K_{t} = \frac{M_{edl}}{f_y \times A_{1/2}} = 0.001$
- Lever arm; $z_{edl} = A_{1/2} \times \min(0.85, 0.9) = 209.0 \text{ mm}$
- Area of steel required; $A_{edl} = 14 \times 0.02 = 22 \text{ mm}^2$
- Minimum area of steel required; $A_{edl} = 0.0013 \times h_{slab} = 325 \text{ mm}^2$
- Area of steel required; $A_{edl} = 0.0013 \times h_{slab} = 325 \text{ mm}^2$

PASS: $A_{edl} <= A_{slab}$ - Area of reinforcement provided in top to span local depressions is adequate

Reinforcement required in bottom
- K factor; $K_{b} = \frac{M_{edl}}{f_y \times A_{1/2}} = 0.001$
- Lever arm; $z_{edl} = A_{1/2} \times \min(0.85, 0.9) = 194.7 \text{ mm}$
- Area of steel required; $A_{edl} = 14 \times 0.02 = 22 \text{ mm}^2$
- Area of steel required; $A_{edl} = 0.0013 \times h_{slab} = 325 \text{ mm}^2$

PASS: $A_{edl} <= A_{slab}$ - Area of reinforcement provided in bottom to span local depressions is adequate
Shear check
Applied shear stress; 
Tension steel ratio; 
From BS8110-1:1997 - Table 3.8; 
Design concrete shear strength;

Internal slab deflection check
Basic allowable span to depth ratio; 
Moment factor; 
Steel stress service; 
Modification factor; 
Modified allowable span to depth ratio; 
Actual span to depth ratio;

Edge beam design checks
Basic loading; 
Hardcore; 
Edge beam; 
Rectangular beam element; 
Chamber element; 
Slab element; 
Edge beam self weight; 
Edge load number 1; 
Load type; 
Dead load; 
Live load; 
Ultimate load;

Limiting max dispersal to say 6 x beam depth; 
Total dispersal width of point loads; 
Equivalent ultimate udl of edge load 1; 
Equivalent unfactored udl of edge load 1;

Center of longitudinal and equivalent line loads from outside face of raft
Load x distance for edge load 1; 
Load x distance for edge load 2; 
Sum of ultimate longitudinal and equivalent line loads; 
Sum of load x distances; 
Center of loads; 
Initially assume no moment transferred into slab due to load/reaction eccentricity
Sum of unfactored longitudinal and effective line loads; 
Allowable bearing width; 
Pressure bearing due to line/point loads; 
Total applied bearing pressure;

Now assume moment due to load/reaction eccentricity is resisted by slab
Bearing width required; 
Effective bearing width at u/s of slab; 
Load/reaction eccentricity; 
Ultimate moment to be resisted by slab; 
From slab bending check
Moment due to depression under slab (hoggling); 
Total moment to be resisted by slab top steel; 
K factor;

Edge beam bending pressure check
Effective bearing width of edge beam; 
Total uniform load at formation level; 
Longitudinal dispersal of point loads
Total dead point load; 
Total live point load; 
Total ultimate point load; 
Min width of point loads parallel to edge of raft; 
Approx moment capacity of bottom steel; 
Non transverse line loads therefore;

Moment in edge beam due to transverse line loads; 
Residual moment capacity of beam; 
Max allow beam dispersal based on mt cap;

Hand calculation:
M = 596.9kNm 
Slab thickness = 350mm 
[Raft analysis software only allows slab depths up to 250mm, actual depth is 350mm] 
Top cover = 25mm 
Bar Ø = 25mm 
d = 350-25-25/2 = 312.5mm 
f'y = 40N/mm² 
k = 596.9 x 10⁷/(1000x312.5)² = 0.153<0.156 
z = 312.5x(0.5 + (0.25-0.153/0.9)) = 245mm 
A_dreq = 596.9x10⁷/(0.87x500x245) = 5600mm² 
Use H25 @ 100mm centres and H12 @ 150mm centres locally at Column C1 
A_preq = 5663mm²

Edge beam bending check
Diviner for moments due to udl's; 
β inland = 10.0 
Sum for moments due to point loads; 
β inland = 6.0 

Applied bending moments
Span of edge beam; 
Ultimate self weight udl; 
Total ultimate udl (approx); 

Pringuer-James Consulting Engineers Ltd
Self weight and slab bending moment:
\[ M_{edge} = (W_{edge} + W_{ul}) \times l_{edge}^2/2 = 10.0 \, kNm \]
Self weight shear force:
\[ V_{edge} = (W_{edge} + W_{ul}) \times l_{edge}/2 = 20.9 \, kNm \]

**Moments due to applied uniformly distributed loads**

Ultimate udl (approx):
\[ w_{edge} = 1000 \, kNm \]
Bending moment:
\[ M_{edge} = w_{edge} \times l_{edge}/2 = 1.9 \, kNm \]
Shear force:
\[ V_{edge} = w_{edge} \times l_{edge}/2 = 4.0 \, kN \]
Moment and shear due to load number 1
Bending moment:
\[ M_{edge} = w_{edge} \times l_{edge}/2 = 123.9 \, kNm \]
Shear force:
\[ V_{edge} = w_{edge} \times l_{edge}/2 = 310.0 \, kN \]
Moment and shear due to load number 2
Bending moment:
\[ M_{edge} = w_{edge} \times l_{edge}/2 = 95.7 \, kNm \]
Shear force:
\[ V_{edge} = w_{edge} \times l_{edge}/2 = 199.5 \, kN \]

**Resultant moments and shears**
Total moment (hoggling and sagging):
\[ M_{edge} = 231.6 \, kNm \]
Maximum shear force:
\[ V_{edge} = 534.5 \, kN \]

**Reinforcement required in top**
Width of section in compression zone:
\[ b_{wedge} = b_{edge} = 1000 \, mm \]
Average web width:
\[ b_w = b_{edge} \times \tan(\alpha/2) = 1225 \, mm \]
K factor:
\[ K_{edge} = M_{edge}/(b_w \times h_{edge} \times f_y) = 0.037 \]
Lever arm:
\[ z_{edge} = (d_{edge} / 2 \times h_{edge}) / \sin(\alpha/2) = 378 \, mm \]
Area of steel required for bending:
\[ A_{edge} = 0.0013 \times 1.0 \times b_w \times h_{edge} = 717 \, mm^2 \]
Minimum area of steel required:
\[ A_{edge} = 0.0013 \times 1.0 \times b_w \times h_{edge} = 717 \, mm^2 \]
Area of steel required:
\[ A_{edge} = \max(A_{edge, min}, A_{edge, required}) = 1409 \, mm^2 \]

**PASS** - A_{edge, required} < A_{edge} - Area of reinforcement provided in top of edge beams is adequate

**Reinforcement required in bottom**
Width of section in compression zone:
\[ b_{wedge} = b_{edge} = 1000 \, mm \]
Average web width:
\[ b_w = b_{edge} \times \tan(\alpha/2) = 1440 \, mm \]
K factor:
\[ K_{edge} = M_{edge}/(b_w \times h_{edge} \times f_y) = 0.268 \]
Lever arm:
\[ z_{edge} = (d_{edge} / 2 \times h_{edge}) / \sin(\alpha/2) = 373 \, mm \]
Area of steel required for bending:
\[ A_{edge} = 0.0013 \times 1.0 \times b_w \times h_{edge} = 1427 \, mm^2 \]
Minimum area of steel required:
\[ A_{edge} = 0.0013 \times 1.0 \times b_w \times h_{edge} = 1427 \, mm^2 \]
Area of steel required:
\[ A_{edge} = \max(A_{edge, min}, A_{edge, required}) = 1427 \, mm^2 \]

**PASS** - A_{edge, required} < A_{edge} - Area of reinforcement provided in bottom of edge beams is adequate

**Edge beam shear check**
Applied steel stress:
\[ V_{edge} = V_{ul} \times (b_w \times d_{edge}) = 1.096 \, N/mm^2 \]
Tension steel ratio:
\[ f_{edge} = 100 \times A_{edge}/(b_w \times d_{edge}) = 0.387 \]
From BS8110-1:1997 - Table 3.8
Design concrete shear strength:
\[ V_{concrete} = 0.539 \, N/mm^2 \]
\[ V_{edge} > V_{concrete} + 0.4N/mm^2 - Therefore designed links required \]
Link area to spacing ratio required:
\[ A_{u, upon, \, edge} = (V_{edge} - V_{concrete}) / ((1.0/y) / f_y) = 1.570 \, mm \]
Link area to spacing ratio provided:
\[ A_{u, upon, \, edge} = N_{edge} \times (d_{edge} / 4 \times s_{edge})^2 = 3.393 \, mm \]

**PASS** - A_{u, upon, \, edge} < A_{u, upon, \, edge, required} - Shear reinforcement provided in edge beams is adequate

**Corner design checks**

**Basic loading**
Total uniform load at formation level:
\[ W_{ul} = W_{load} + W_{dead} + W_{bar} + W_{stabilizer} = 13.4 \, kN/m^2 \]

**PASS** - W_{load} < Q_{allow} - Applied bearing pressure is less than allowable
RAFT SLAB DESIGN (TYPICAL)

RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)

Soil and raft definition

Soil definition

Allowable bearing pressure; qallow = 40.0 kN/m²
Number of types of soil forming sub-soil; One type only
Soil density; Firm to loose
Depth of hardcore beneath slab; hcorethick = 0 mm
Depth of hardcore beneath thickenings; hcorethick = 0 mm
Density of hardcore; ρ = 20.0 kN/m³
Basic assumed diameter of local depression; dallow = 2000 mm
Diameter under slab modified for hardcore; d = 2000 mm
Diameter under thickenings modified for hardcore; d = 2000 mm

Raft slab definition

Max dimension/max dimension between joints; lbas = 6.200 m
Slab thickness; hbas = 250 mm
Concrete strength; fck = 40 N/mm²
Poison ratio of concrete; ν = 0.2
Slab mesh reinforcement strength; fsteel = 500 N/mm²
Partial safety factor for steel reinforcement; ksteel = 1.15
Partial safety factor for concrete; kConcrete = 1.0

From CACA document ‘Concrete ground floors’ Table 5
Minimum mesh required in top for shrinkage; A142;
Actual mesh provided in top; A393 (Aallow ≤ A393 = 393 mm²/m)
Mesh provided in bottom; A393 (Aallow ≤ A393 = 393 mm²/m)
Top mesh bar diameter; østeel = 10 mm
Bottom mesh bar diameter; østeel = 10 mm
Cover to top reinforcement; ctop = 20 mm
Cover to bottom reinforcement; cbottom = 35 mm
Average effective depth of top reinforcement; dsteel = hbas - ctop - østeel = 220 mm
Average effective depth of bottom reinforcement; dsteel = hbas - cbottom - østeel = 205 mm
Overall average effective depth; dsteel = (dsteel + dsteel)/2 = 213 mm
Minimum effective depth of top reinforcement; dsteel = hbas - ctop - østeel/2 = 215 mm
Minimum effective depth of bottom reinforcement; dsteel = hbas - cbottom - østeel/2 = 200 mm

Edge beam definition

Overall depth;
Width;
Angle of chamfer to horizontal;
Strength of main bar reinforcement;
Strength of link reinforcement;
Reinforcement provided in top;
Reinforcement provided in bottom;
Link reinforcement provided;
Bottom cover to links;
Effective depth of top reinforcement;
Effective depth of bottom reinforcement;

Internal slab design checks

Basic loading
Slab self weight;
Hardcore;

Applied loading
Uniformly distributed dead load;
Uniformly distributed live load;

Internal slab bearing pressure check
Total uniform load at formation level;

Internal slab bending and shear check

Applied bending moments
Span of slab;
Ultimate self weight udl;
Self weight moment at centre;
Self weight moment at edge;
Self weight shear force at edge;

Moments due to applied uniformly distributed loads
Ultimate applied udl;
Moment at centre;
Moment at edge;
Shear force at edge;

Resultant moments and shears
Total moment at edge;
Total moment at centre;
Total shear force;

Reinforcement required in top
K factor;
Lever arm;
Area of steel required for bending;
Minimum area of steel required;
Area of steel required;

Reinforcement required in bottom
K factor;
Lever arm;
Area of steel required for bending;
Area of steel required;

PASS - Aallow ≤ Asteel - Area of reinforcement provided in top to span local depressions is adequate
PASS - Aallow ≤ Asteel - Area of reinforcement provided in bottom to span local depressions is adequate
Shear check

Applied shear stress;  v = V/vsdef = 0.033 N/mm²
Tension steel ratio;  ρ = 100 × A_steel/vsdef = 0.183
From BS8110:1.1997 - Table 3.8;
Design concrete shear strength;  \( v_c = 0.490 \text{ N/mm}^2 \)

**PASS** -  v < v_c - Shear capacity of the slab is adequate

Internal slab deflection check
Basic allowable span to depth ratio;  \( R \) = 26.0
Moment factor;  \( M_{fact} = M_{fact,allow} / M_{def} = 0.028 \text{ N/mm}^2 \)
Steel service stress;  \( f_s = 2 / 3 \times f_{steel,allow} \)
Modification factor;  \( M_{fact,allow} = M_{fact,allow} / M_{def} = 0.007 \text{ kN/m} \)
Modified allowable span to depth ratio;  \( R_{fact,allow} = R_{fact,allow} / R = 5.1 \text{ kN/m} \)
Actual span to depth ratio;  \( R_{fact,allow} = R_{fact,allow} / R = 7.1 \text{ kN/m} \)

**PASS** -  R < R - Slab span to depth ratio is adequate

Edge beam design checks
Basic loading
Hardcore;  \( w_{core} = 1.6 \text{ kN/m} \)
Edge beam;  \( w_{edge} = 1.2 \text{ kN/m} \)
Rectangular beam element;  \( w_{rbe} = 2 \text{ kN/m} \)
Chamber element;  \( w_{chamber} = 1.0 \text{ kN/m} \)
Slab element;  \( w_{slab} = 1.0 \text{ kN/m} \)
Edge beam self weight;  \( w_{edge} = 1.0 \text{ kN/m} \)
Edge load number 1
Load type;  \( w_{load} = 50 \text{ kN/m} \)
Dead load;  \( w_{dead} = 20 \text{ kN/m} \)
Live load;  \( w_{live} = 20 \text{ kN/m} \)
Ultimate load;  \( w_{ult} = 20 \text{ kN/m} \)
Longitudinal line load width;  \( b_{edge} = 25 \text{ mm} \)
Centroid of load from outside face of raft;  \( A_{edge} = 150 \text{ mm}^2 \)
Effective bearing width of edge beam;  \( A_{effective,edge} = 1200 \text{ mm}^2 \)
Effective bearing width of edge beam;  \( A_{effective,edge} = 1200 \text{ mm}^2 \)
Total uniform load at formation level;  \( A_{effective,edge} = 13.4 \text{ kN/m}^2 \)

Centroid of longitudinal and equivalent lines loads from outside face of raft
Load x distance for edge load 1;  \( S_{edge} = 28.6 \text{ kN/m} \)
Sum of ultimate load and equivalent load lines;  \( S_{ult,edge} = 190.7 \text{ kN/m} \)
Sum of load x distances;  \( A_{edge} = 150 \text{ mm} \)
Initial assumed moment transferred into slab due to load/reaction eccentricity

Resultant moments and shears
Total moment to be resisted by slab top steel;  \( M_{allow} = M_{ult} + M_{dead} = 453.0 \text{ kN/m} \)
K factor;  \( K = M_{allow} / (K_w \times A_{effective,edge}) = 0.245 \)
**PASS** -  K > 0.156 - Therefore compression reinforcement is required
The design is outside the scope of this calculation

Hand calculation:

M = 453.0 kN/m
Slab thickness = 350 mm
[Raft analysis software only allows slab depths up to 250mm, actual depth is 350mm]
Top cover = 25mm
Bar Ø = 25mm
d = 350-25-25 = 312.5mm

Load and reaction eccentricity on raft slab:
Load type;  \( w_{load} = 50 \text{ kN/m} \)

Effective bearing width of edge beam;  \( A_{edge} = 150 \text{ mm}^2 \)
Effective bearing width of edge beam;  \( A_{edge} = 1200 \text{ mm}^2 \)
Effective bearing width of edge beam;  \( A_{edge} = 13.4 \text{ kN/m}^2 \)

Moment due to load/reaction eccentricity is resisted by slab

Bearing width required;  \( b_{req} = 3 \times S_{ult,edge} / (w_{dead} + w_{edge}) = 5030 \text{ mm} \)
Effective bearing width at u/s of slab;  \( b_{ueffective} = 3 \times S_{ult,edge} / (w_{dead} + w_{edge}) = 3030 \text{ mm} \)

**PASS** -  A_{effective,edge} < A_{edge} - Area of reinforcement provided in top of edge beams is adequate
Width of section in compression zone:

\[ b_{\text{edge}} = b_{\text{edge}} + (h_{\text{edge}} - h_{\text{slab}}) \tan(\alpha_{\text{edge}}) + 0.1 \times l_{\text{edge}} = 1440 \text{ mm} \]

K factor:

\[ K_{\text{edge}} = \frac{M_{\text{edge}}}{f_{\text{cu}} \times b_{\text{edge}} \times d_{\text{edge}}} = 0.011 \]

Lever arm:

\[ z_{\text{edge}} = d_{\text{edge}} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{\text{edge}}/0.9)}) = 373 \text{ mm} \]

Area of steel required for bending:

\[ A_{\text{s,edge}} = \frac{M_{\text{edge}}}{(1.0/\gamma_{s}) \times f_{y} \times z_{\text{edge}}} = 625 \text{ mm}^2 \]

Minimum area of steel required:

\[ A_{\text{s,min}} = 0.0013 \times b \times h_{\text{edge}} = 717 \text{ mm}^2 \]

Area of steel required:

\[ A_{\text{s,req}} = \max(A_{\text{s,edge}}, A_{\text{s,min}}) = 717 \text{ mm}^2 \]

**PASS** - \( A_{\text{s,req}} \leq A_{\text{s,edge}} \) - Area of reinforcement provided in bottom of edge beams is adequate

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**Edge beam shear check**

Applied shear stress:

\[ \nu_{\text{edge}} = \frac{V_{\text{edge}}}{b_{\text{w}} \times d_{\text{edge}}} = 0.520 \text{ N/mm}^2 \]

Tension steel ratio:

\[ \nu_{\text{edge}} = \frac{100 \times A_{\text{s,edge}}}{b_{\text{w}} \times d_{\text{edge}}} = 0.387 \]

From BS8110-1:1997 - Table 3.8

Design concrete shear strength:

\[ \nu_{\text{edge}} = \frac{0.539 \text{ N/mm}^2}{V_{\text{edge}}} \leq 0.4 \text{ N/mm}^2 \] - Therefore minimum links required

Link area to spacing ratio required:

\[ A_{\text{s,upon,edge}} = 0.4 \times (1.0/\gamma_{s}) \times f_{y} = 1.127 \text{ mm} \]

Link area to spacing ratio provided:

\[ A_{\text{s,upon,edge}} = \frac{N_{\text{edgelink}} \times \pi \times \phi_{\text{edgelink}}^2}{4 \times s_{\text{edge}}} = 3.393 \text{ mm} \]

**PASS** - \( A_{\text{s,upon,edge}} \leq A_{\text{s,upon,edge}} \) - Shear reinforcement provided in edge beams is adequate
PROPPED RETAINING WALL DESIGN

RETAINING WALL ANALYSIS (BS 8002:1994)

Wall details
Retaining wall type; Cantilever
Height of wall stem; h_{stem} = 2700 mm
Length of toe; l_{toe} = 1000 mm
Overall length of base; l_{base} = 1200 mm
Height of retaining wall; h_{wall} = 3150 mm
Depth of downstand; d_{ds} = 0 mm
Position of downstand; l_{ds} = 750 mm
Depth of cover in front of wall; d_{cover} = 0 mm
Height of ground water; h_{water} = 1700 mm
Density of wall construction; \gamma_{wall} = 23.6 kN/m^3
Density of base construction; \gamma_{base} = 23.6 kN/m^3
Angle of soil surface; \beta = 0.0 deg
Mobilisation factor; M = 1.5
Moist density; \gamma_{mb} = 18.0 kN/m^3
Design shear strength; \phi_d = 24.2 deg
Design shear strength; \phi_b = 24.2 deg
Moist density; \gamma_{md} = 18.0 kN/m^3

Using Coulomb theory
Active pressure; K_a = 0.369
At-rest pressure; K_0 = 0.590

Loading details
Surcharge load; Surcharge = 0.0 kN/m^2
Vertical dead load; W_{dead} = 116.9 kN/m
Horizontal dead load; F_{dead} = 0.0 kN/m
Position of vertical load; l_{load} = 1100 mm

Mobilisation factor; M = 1.5
Moist density; \gamma_{mb} = 18.0 kN/m^3
Design shear strength; \phi_d = 24.2 deg
Design shear strength; \phi_b = 24.2 deg
Moist density; \gamma_{md} = 18.0 kN/m^3

Using Coulomb theory
Active pressure; K_a = 0.369
At-rest pressure; K_0 = 0.590

Loading details
Surcharge load; Surcharge = 0.0 kN/m^2
Vertical dead load; W_{dead} = 116.9 kN/m
Horizontal dead load; F_{dead} = 0.0 kN/m
Position of vertical load; l_{load} = 1100 mm

Wall details
Retaining wall type; Cantilever
Height of wall stem; h_{stem} = 2700 mm
Length of toe; l_{toe} = 1000 mm
Overall length of base; l_{base} = 1200 mm
Height of retaining wall; h_{wall} = 3150 mm
Depth of downstand; d_{ds} = 0 mm
Position of downstand; l_{ds} = 750 mm
Depth of cover in front of wall; d_{cover} = 0 mm
Height of ground water; h_{water} = 1700 mm
Density of wall construction; \gamma_{wall} = 23.6 kN/m^3
Density of base construction; \gamma_{base} = 23.6 kN/m^3
Angle of soil surface; \beta = 0.0 deg
Mobilisation factor; M = 1.5
Moist density; \gamma_{mb} = 18.0 kN/m^3
Design shear strength; \phi_d = 24.2 deg
Design shear strength; \phi_b = 24.2 deg
Moist density; \gamma_{md} = 18.0 kN/m^3

Using Coulomb theory
Active pressure; K_a = 0.369
At-rest pressure; K_0 = 0.590

Loading details
Surcharge load; Surcharge = 0.0 kN/m^2
Vertical dead load; W_{dead} = 116.9 kN/m
Horizontal dead load; F_{dead} = 0.0 kN/m
Position of vertical load; l_{load} = 1100 mm

Wall details
Retaining wall type; Cantilever
Height of wall stem; h_{stem} = 2700 mm
Length of toe; l_{toe} = 1000 mm
Overall length of base; l_{base} = 1200 mm
Height of retaining wall; h_{wall} = 3150 mm
Depth of downstand; d_{ds} = 0 mm
Position of downstand; l_{ds} = 750 mm
Depth of cover in front of wall; d_{cover} = 0 mm
Height of ground water; h_{water} = 1700 mm
Density of wall construction; \gamma_{wall} = 23.6 kN/m^3
Density of base construction; \gamma_{base} = 23.6 kN/m^3
Angle of soil surface; \beta = 0.0 deg
Mobilisation factor; M = 1.5
Moist density; \gamma_{mb} = 18.0 kN/m^3
Design shear strength; \phi_d = 24.2 deg
Design shear strength; \phi_b = 24.2 deg
Moist density; \gamma_{md} = 18.0 kN/m^3

Using Coulomb theory
Active pressure; K_a = 0.369
At-rest pressure; K_0 = 0.590

Loading details
Surcharge load; Surcharge = 0.0 kN/m^2
Vertical dead load; W_{dead} = 116.9 kN/m
Horizontal dead load; F_{dead} = 0.0 kN/m
Position of vertical load; l_{load} = 1100 mm
Calculate propping force

Propping force;

\[ F_{\text{prop}} = 0.0 \text{kN/m} \]

Check bearing pressure

Total vertical reaction;

\[ R = 159.3 \text{kN/m} \]

Distance to reaction;

\[ x_{\text{bar}} = 600 \text{ mm} \]

Eccentricity of reaction;

\[ e = 0 \text{ mm} \]

\text{Reaction acts within middle third of base}

Bearing pressure at toe;

\[ p_{\text{toe}} = 132.7 \text{kN/m}^2 \]

Bearing pressure at heel;

\[ p_{\text{heel}} = 132.7 \text{kN/m}^2 \]

\text{PASS - Maximum bearing pressure is less than allowable bearing pressure}

Calculate propping forces to top and base of wall

Propping force to top of wall;

\[ F_{\text{prop, top}} = -5.404 \text{kN/m} \]

Propping force to base of wall;

\[ F_{\text{prop, base}} = 5.404 \text{kN/m} \]
RETYING WALL DESIGN (BS 8002:1994)

Ultimate limit state load factors
- Dead load factor; \( \gamma_D = 1.4 \)
- Live load factor; \( \gamma_L = 1.6 \)
- Earth pressure factor; \( \gamma_E = 1.4 \)

Calculate propping force
- Propping force; \( F_{\text{prop}} = 0.0 \) kN/m

Calculate propping forces to top and base of wall
- Propping force to top of wall; \( F_{\text{prop\_top\_f}} = -7.527 \) kN/m
- Propping force to base of wall; \( F_{\text{prop\_base\_f}} = 15.806 \) kN/m

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties
- Strength of concrete; \( f_{\text{cu}} = 40 \) N/mm²
- Strength of reinforcement; \( f_y = 500 \) N/mm²

Base details
- Minimum reinforcement; \( k = 0.13 \) %
- Cover in toe; \( c_{\text{toe}} = 40 \) mm

Design of retaining wall toe
- Shear at heel; \( V_{\text{toe}} = 173.8 \) kN/m
- Moment at heel; \( M_{\text{toe}} = 105.1 \) kN/m

Check toe in bending
- Reinforcement provided; 12 mm dia. bars @ 150 mm centres
- Area required; \( A_{\text{s\_toe\_req}} = 629.7 \) mm²/m
- Area provided; \( A_{\text{s\_toe\_prov}} = 754 \) mm²/m

Check shear resistance at toe
- Design shear stress; \( V_{\text{toe\_des}} = 0.430 \) N/mm²
- Allowable shear stress; \( V_{\text{adm\_toe}} = 5.000 \) N/mm²
- Concrete shear stress; \( V_{\text{c\_toe}} = 0.422 \) N/mm²

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties
- Strength of concrete; \( f_{\text{cu}} = 40 \) N/mm²
- Strength of reinforcement; \( f_y = 500 \) N/mm²

Wall details
- Minimum reinforcement; \( k = 0.13 \) %
- Cover in stem; \( c_{\text{stem}} = 40 \) mm
- Cover in wall; \( c_{\text{wall}} = 30 \) mm

Design of retaining wall stem
- Shear at base of stem; \( V_{\text{stem\_base}} = 48.3 \) kN/m
- Moment at base of stem; \( M_{\text{base}} = 23.4 \) kN/m

Check wall stem in bending
- Reinforcement provided; 12 mm dia. bars @ 150 mm centres
- Area required; \( A_{\text{s\_stem\_req}} = 367.4 \) mm²/m
- Area provided; \( A_{\text{s\_stem\_prov}} = 754 \) mm²/m

Check shear resistance at wall stem
- Design shear stress; \( V_{\text{stem\_des}} = 0.313 \) N/mm²
- Allowable shear stress; \( V_{\text{adm\_stem}} = 5.000 \) N/mm²
- Concrete shear stress; \( V_{\text{c\_stem}} = 0.740 \) N/mm²

Design of retaining wall at mid height
- Moment at mid height; \( M_{\text{wall\_mid}} = 11.8 \) kN/m

Check retaining wall deflection
- Max span/depth ratio; \( \text{Ratio}_{\text{max}} = 38.80 \)
- Actual span/depth ratio; \( \text{Ratio}_{\text{act}} = 17.53 \)

PASS - Span to depth ratio is acceptable
CONTINUOUS BEAM ANALYSIS - RESULTS

Unfactored support reactions

<table>
<thead>
<tr>
<th>Support</th>
<th>Dead (kN)</th>
<th>Imposed (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.7</td>
<td>-1.6</td>
</tr>
<tr>
<td>B</td>
<td>-27.0</td>
<td>-6.8</td>
</tr>
<tr>
<td>C</td>
<td>-20.4</td>
<td>-2.3</td>
</tr>
</tbody>
</table>

See Tadd's Analysis for prop loads & reinforcement design.
CONCRETE SLAB DESIGN – HOGGING – OUTER LAYER OF STEEL (CL 3.5.4)

Design hogging moment (per m width of slab); \( m_{u\text{h}} = 8.1 \text{kNm/m} \)

Moment Redistribution Factor; \( \gamma = 1.0 \)

Area of reinforcement required
\[
A_{\text{req,h}} = \frac{\text{abs}(m_{u\text{h}})}{(0.5 + \gamma)} = 0.009
\]
\[
K_{s} = \min (0.156 , (0.402 \times (f_{c} - 0.4) - (0.18 \times (f_{c} - 0.4)^{2})) = 0.156
\]

Slab requiring outer tension steel only - bars (hogg)
\[
z_{\text{req,h}} = \min ((0.95 \times d_{\text{d}})/(d_{\text{d}} - d_{\text{h}} - h_{\text{c}} - h_{\text{c}})) = 146 \text{ mm}
\]

Neutral axis depth; \( x_{\text{h}} = (d_{\text{d}} - z_{\text{h}})/0.45 = 17 \text{ mm} \)

Area of tension steel required
\[
A_{\text{req,h}} = \frac{\text{abs}(m_{u\text{h}})}{(1/\gamma \times f_{c} z_{\text{h}})} = 127 \text{ mm}^{2}/\text{m}
\]

Tension steel
\[
\text{Provide 10 dia bars @ 150 centres; outer tension steel resisting hogging}
A_{\text{req,h,prov}} = A_{\text{req,h}} = 524 \text{ mm}^{2}/\text{m}
\]

CONCRETE SLAB DESIGN – SAGGING – OUTER LAYER OF STEEL (CL 3.5.4)

Design sagging moment (per m width of slab); \( m_{u\text{s}} = 9.6 \text{kNm/m} \)

Moment Redistribution Factor; \( \gamma = 1.0 \)

Area of reinforcement required
\[
A_{\text{req,s}} = \frac{\text{abs}(m_{u\text{s}})}{(0.5 + \gamma)} = 0.008
\]
\[
K_{c} = \min (0.156 , (0.402 \times (f_{c} - 0.4) - (0.18 \times (f_{c} - 0.4)^{2})) = 0.156
\]

Slab requiring outer tension steel only - bars (sag)
\[
z_{\text{req,s}} = \min ((0.95 \times d_{\text{d}})/(d_{\text{d}} - d_{\text{h}} - h_{\text{c}} - h_{\text{c}})) = 161 \text{ mm}
\]

Neutral axis depth; \( x_{\text{s}} = (d_{\text{d}} - z_{\text{s}})/0.45 = 19 \text{ mm} \)

Area of tension steel required
\[
A_{\text{req,s}} = \frac{\text{abs}(m_{u\text{s}})}{(1/\gamma \times f_{c} z_{\text{s}})} = 138 \text{ mm}^{2}/\text{m}
\]

Tension steel
\[
\text{Provide 10 dia bars @ 150 centres; outer tension steel resisting sagging}
A_{\text{req,s,prov}} = A_{\text{req,s}} = 524 \text{ mm}^{2}/\text{m}
\]
Shear resistance of concrete slabs (CL 3.5.5)

Outer tension steel resisting sagging moments
- Depth to tension steel from compression face; \( d_t = 169 \text{ mm} \)
- Area of tension reinforcement provided (per m width of slab); \( A_{sx,prov} = 754 \text{ mm}^2/\text{m} \)
- Design ultimate shear force (per m width of slab); \( V_u = 33 \text{ kN/m} \)
- Characteristic strength of concrete; \( f_{cu} = 40 \text{ N/mm}^2 \)

Applied shear stress
\[
\nu = \frac{V_u}{d_t} = 0.19 \text{ N/mm}^2
\]

Check shear stress to clause 3.5.5.2

\[
\nu_{allowable} = \min (0.8 \sqrt{f_{cu}}, 5 \text{ N/mm}^2) = 5.00 \text{ N/mm}^2
\]

Shear stresses to clause 3.5.5.3

Design shear stress
\[
f_{u,cx,req} = \frac{f_{cu} \times A_{sx,req}}{2 \times A_{sx,prov}} = 40 \text{ N/mm}^2
\]
\[
\nu_{cx} = \frac{0.79 \times A_{sx,req} \times \min(3, \frac{A_{sx,req}}{d_t})^{1/3} \times \max(0.67, \frac{400}{d_t})^{1/4}}{1.25 \times f_{u,cx,req}^{1/3}} = 0.70 \text{ N/mm}^2
\]

Applied shear stress
\[
\nu = 0.19 \text{ N/mm}^2
\]

No shear reinforcement required

Concrete slab deflection check (CL 3.5.7)
- Slab span length; \( l = 1.900 \text{ m} \)
- Design ultimate moment in shorter span per m width; \( M_{ux} = 10 \text{ kNm/m} \)
- Depth to outer tension steel; \( d_t = 169 \text{ mm} \)

Tension steel
- Area of outer tension reinforcement provided; \( A_{sx,prov} = 754 \text{ mm}^2/\text{m} \)
- Area of tension reinforcement required; \( A_{sx,req} = 138 \text{ mm}^2/\text{m} \)
- Moment Redistribution Factor; \( \beta_{bx} = 1.00 \)

Modification factors
- Basic span / effective depth ratio (Table 3.9); \( \frac{\text{ratio}_{span,depth}}{20} \)

The modification factor for spans in excess of 10m (ref cl 3.4.6.4) has not been included.
\[
\nu = 2 \times f_y \times A_{sx,req} / (3 \times A_{sx,prov} \times \beta_{ux}) = 60.8 \text{ N/mm}^2
\]
\[
\text{factor}_{css} = \min \left( 2, 0.55 + \frac{477 \text{ N/mm}^2}{\nu}, \frac{\nu}{120}, \frac{0.9 \text{ N/mm}^2}{120} \right) = 2.000
\]

Calculate maximum span
This is a simplified approach and further attention should be given where special circumstances exist. Refer to clauses 3.4.6.4 and 3.4.6.7.

Maximum span; \( l_{max} = \frac{\text{ratio}_{span,depth} \times \text{factor}_{css} \times d_t}{6.76} = 6.76 \text{ m} \)

Check the actual beam span

Actual span/depth ratio; \( l / d_t = 11.24 \)
Span depth limit; \( \frac{l}{d_t} \times \text{factor}_{css} = 40.00 \)

Span/depth ratio check satisfied