



Newport Civic Centre Clock Tower

Cradle Calculation Package

Project Ref: CC2753

Client: Newport Norse

Report Revision: P01

CAMBRIA

Report Control Sheet

Client	Newport Norse
Project	Newport Civic Centre Clock Tower
Project ref	CC2753
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Prepared by	George Kenyon MEng
Reviewed and authorised by	J Spencer BSc (Hons) CEng MICE

Document naming protocol

Project Ref.	Originator	Vol.	Level	Type	Role	Number
CC2753	CAM	ZZ	XX	RP	SE	0001

Current issue

Revision	Status	Date	Description	Prepared by	Authorised by
P01	S2	14/05/2025	Calc pack for Cradle validation	G Kenyon	J Spencer

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1 Introduction

- 1.1.1 Cambria Consulting have been appointed by Newport Norse – hereafter known as ‘the Client’, to undertake calculations to validate a temporary externally fixed cradle, to allow remediation of the elevations of the Clock Tower located at the Newport Civic Centre, Godfrey Rd, Newport NP20 4UR.
- 1.1.2 Following historic cleaning of the Clock Tower using a MEWP it was identified that the pointing on the stonework of the Tower has begun to fail and needed repointing. Due to the height constraints of the tower, it was concluded that the MEWP could not be used for this remediation and therefore a temporary cradle supported by 2 jibs would be used to provide external access.
- 1.1.3 Calculations for structural members adequacy have been undertaken for the top floor of the Clock tower.
- 1.1.4 This report has been prepared exclusively for Newport Norse and their professional consultees; it may not be relied upon or reproduced by any third party without the written agreement of Cambria Consulting Limited.

2 Limitations

2.1.1 Except where noted otherwise in the report, the following limitations apply.

- ▶ No formal site testing or other similar testing defects were undertaken.
- ▶ The dimensions given in this report are approximate.
- ▶ All dimensions prior to works to be carried out are to be measured and confirmed by the contractor.
- ▶ Measured survey was provided by Newport Norse.
- ▶ Cradle loading and layout provided by contractor – loading assessment of members has been conducted on the loads provided, no loads have been deduced bar permanent slab, beam and wall loading has been deduced by CAMBRIA.

3 Site Location

- 3.1.1 The Newport Civic Centre, located on Godfrey Road in Newport, Wales, is a prominent municipal building known for its distinctive architectural features, including its notable clock tower. Designed by architect Thomas Cecil Howitt in the Art Deco style, the Civic Centre's construction commenced in 1937 but faced delays due to the Second World War. While the main building was completed in the early 1950s, the clock tower was not finished until 1964.
- 3.1.2 The Civic Centre is constructed primarily from Portland stone, with a basement level of brown rock-faced Derbyshire sandstone. The building's design features a U-plan layout, facing southeast, and is set on a sloping site. The clock tower rises above the central block of the building, which contains the council chamber. This tower is characterized by its tall, tapering structure set on a low square base and is capped with a pyramidal roof. The tower's design includes panel-like raised sides and a simple clock face, with three small square openings below the eaves on each side



Figure 3-1 – Site Location Arial view

- 3.1.3 Historical photographs from the early 1960s, during the construction of the Newport Civic Centre's clock tower, reveal a steel frame structure encased in concrete. These images depict the exposed steel framework prior to the application of the Portland stone cladding, indicating that the tower's structural integrity relies on a steel skeleton enveloped in concrete. See Figure 3.2 below.



Figure 3-2 – Historic Photo during Construction

4 Loading

4.1.1 The following loading conditions have been assumed for the calculations:

Dead/Permanent Actions

Floor construction:

- ▶ Concrete Slab 6.0 kN/m²
- ▶ Floor Finish and Ceiling 0 kN/m²
- ▶ Partitions 0 kN/ m²

Total Characteristic SDL **6.0 kN/ m²**

Variable/Imposed Actions

4.1.2 Imposed loads will be in accordance with BS EN 1991-1-1:2002 (British Standards Institute, 2010) and its relevant UK National Annex and PD 6688-1-1:2011 (British Standards Institute, 2011) as follows:

4.1.3 **H:** Maintenance access only (*No access will be permitted into the clock tower during the works only for maintenance of the cradle arms*).

Total Characteristic SDL **0.6 kN/ m²**

Point load from cradle spec sheet

See Appendix B

4.1.4 The point load of the front foot of the cradle frame is +7.25kN. The rear leg has -1.11kN axial load, with an additional dead loading from steel plates acting as ballast against the uplift load totalling 3.5kN.

4.1.5 An accidental load case has also been assessed for if the cradle dropped this would induce a 14.49kN (Which has already been factored by 3) load on the front support.

4.1.6 See Figure 4-1 below for Jib Loading from Spec sheet Appendix B.

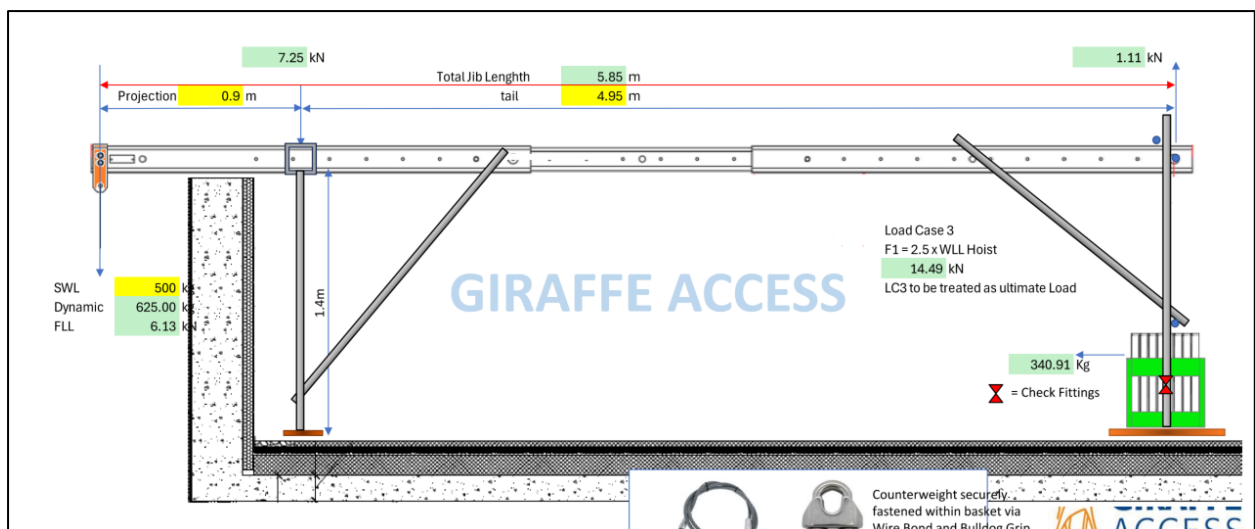


Figure 4-1 Cradle Jib Loading

5 Conclusions

- 5.1.1 In our assessment of the Newport Civic Centre's Clock Tower, we began by analysing provided survey data. This measured survey allowed us to deduce the dimensions of the existing concrete slab and the thickness of the concrete encasement surrounding the steel beams. This information was critical in estimating the size and capacity of the encased steel members.
- 5.1.2 This report does not constitute a structural survey, whereby all structural elements are identified, sized, investigated, and calculations undertaken for adequacy. This report is a structural appraisal calculation report, undertaken specifically to review the temporary additional loads proposed to be applied to clock tower structure. No intrusive opening up works, or visual inspection was undertaken.
- 5.1.3 We have not inspected all areas of the structure which are covered, unexposed or inaccessible and we are therefore unable to report that any such part is free from defects. We cannot be held responsible for unforeseen conditions which may exist that are covered. Conditions may exist beneath the structure that are not detectable from our visual inspection. Localised intrusive opening works were not undertaken during this inspection
- 5.1.4 The loading was calculated including the self-weight of the floor slab and an imposed load representative of maintenance activities. Additionally, point loads from the contractor's specifications for the cradle system were incorporated into the analysis. The combined loading scenarios were applied to the deduced steel beam sizes to verify their structural adequacy. The analysis confirmed that the existing members can safely support the anticipated additional loads.
- 5.1.5 For the cradle support strategy, the front leg of the jib is proposed to rest on the existing window ledge. To ensure the ledge's integrity, a visual inspection will be required prior to installation. If the ledge is found to be in good condition, load-spreading measures will be implemented to distribute the load appropriately along the window ledge. The rear leg will be positioned directly over the primary steel beams that span the length of the room. These beams have been verified to accommodate the point loads imposed by the cradle system.
- 5.1.6 All structural calculations indicate that the existing framework is capable of supporting the proposed maintenance cradle system without compromising safety or integrity. Detailed calculations and analysis are provided in Appendix A.
- 5.1.7 The exact positioning of the cradle support legs is to be determined by the contractor based on site-specific conditions. These proposed locations must be submitted to Cambria for review and approval prior to the commencement of any works. This process ensures that the load paths are appropriate, and that the structural integrity of the supporting elements is maintained as per calculations performed.

Appendix A - Calculations



Client: Newport Norse Engineer: GK Checked By: JS

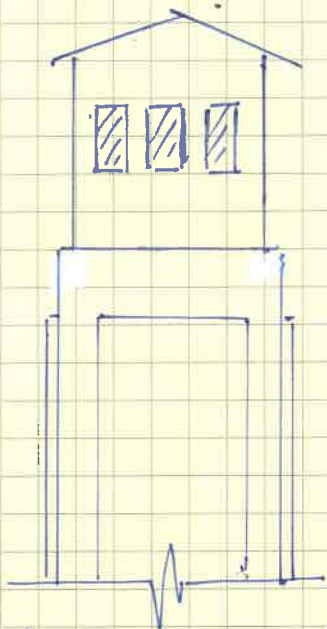
Project: Bell tower Cradle Date: 08/05/25 Approved By: JS

Job No: CC2753 Newport Civic Centre Sheet: 1 Status: POI

CALCULATIONS:

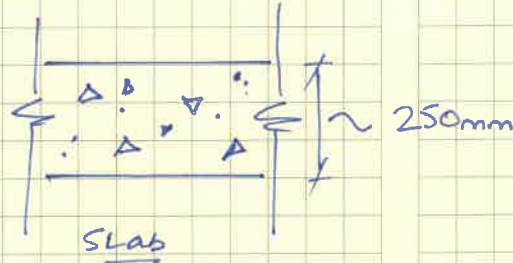
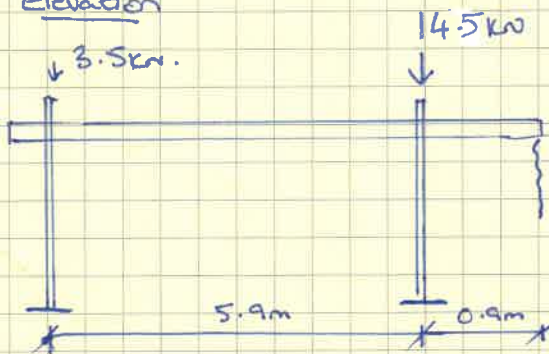
OUTPUT:

Bell Tower Cradle

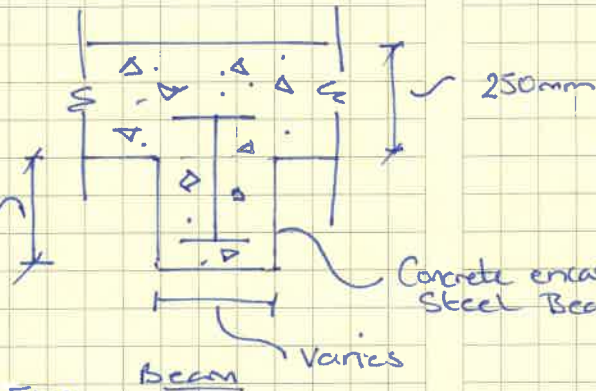


'Constructed in 1965'
Elevation

↓ 3.5kn.



Slab

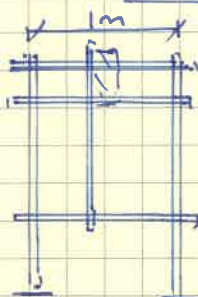


Concrete encased
Steel Beams

Beam

'All dims to be
confirmed by
contractor prior to
commencement of
works'

Hoist elevation



SIDE ELEVATION
Hoist

loading provided
above has
been confirmed
to be split over
the 2 supports.
evenly.

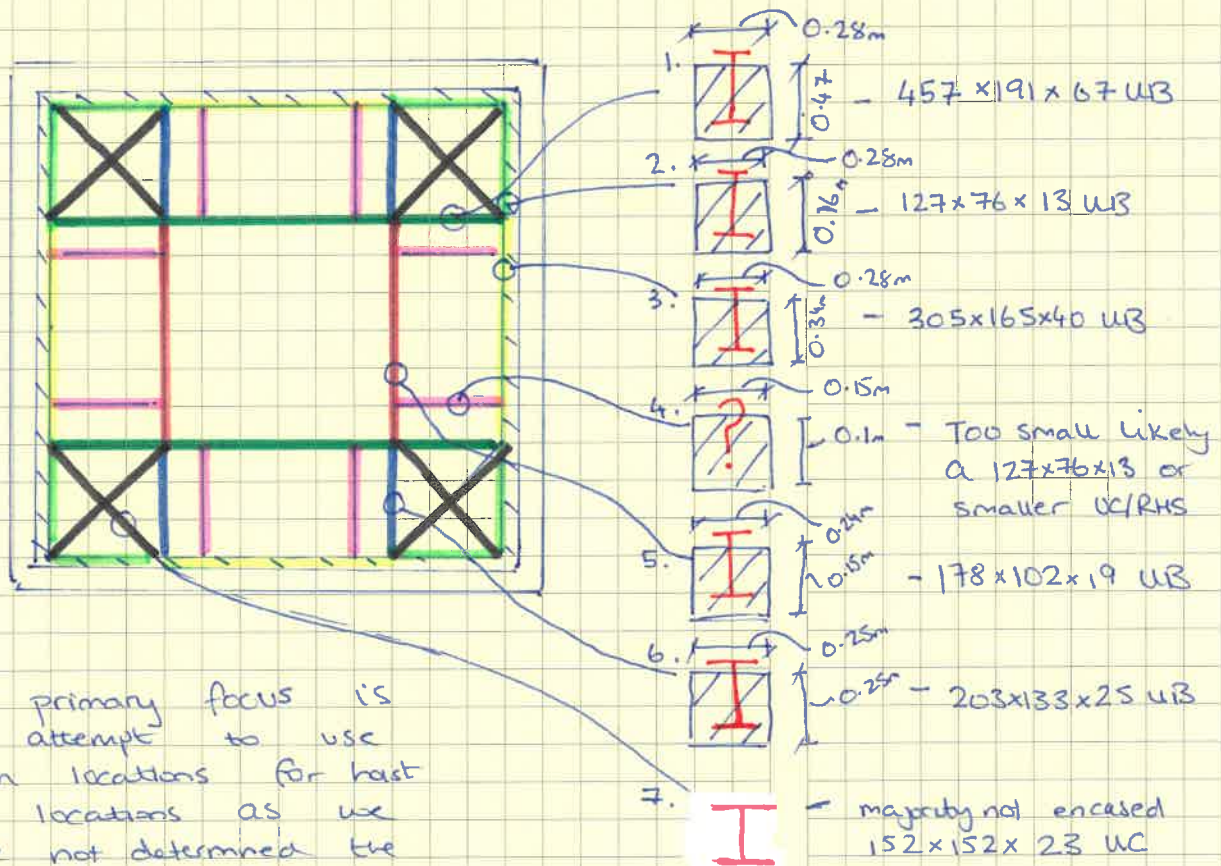
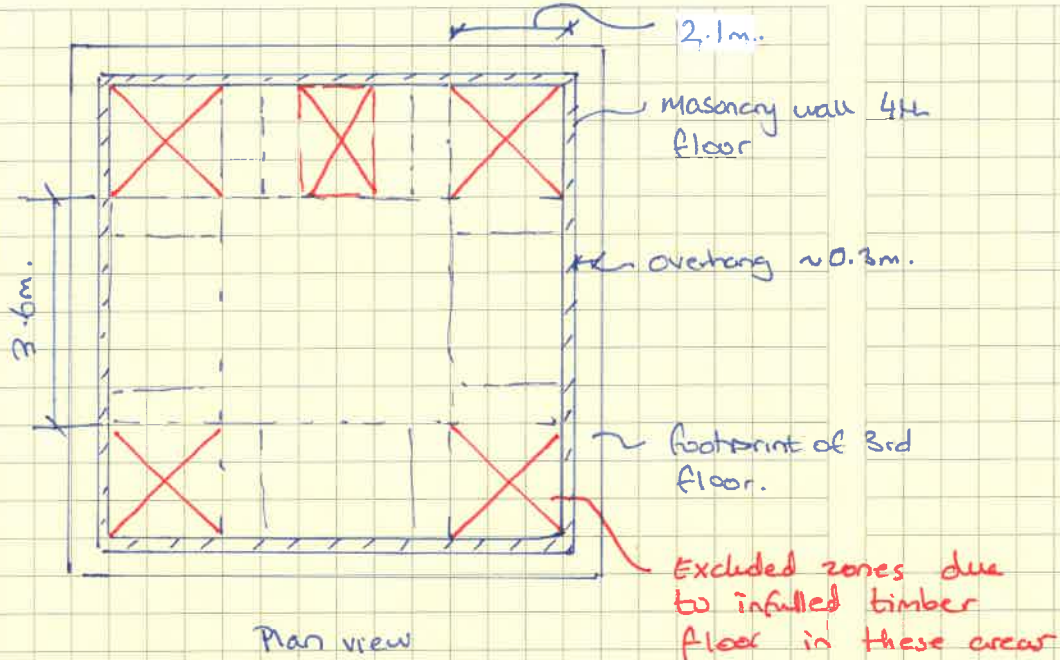
Client: Newport Norse Engineer: AK Checked By: JS

Project: Bell tower Cradle Date: 05/05/25 Approved By: JS

Job No: CC2753 Newport Civic Centre Sheet: 2 Status: POI

CALCULATIONS:

OUTPUT:

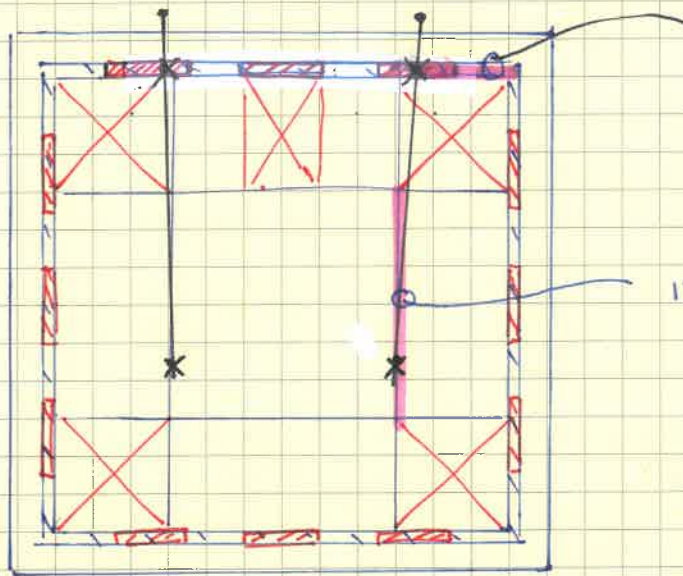


* Our primary focus is to attempt to use beam locations for least leg locations as we have not determined the reinforcement in the slab or punching

Client: Newport Noose Engineer: AK Checked By: JSProject: Bell tower Cradle Date: 08/05/23 Approved By: JSJob No: CC2753 Newport Cradle Centre Sheet: 3 Status: PO1

CALCULATIONS:

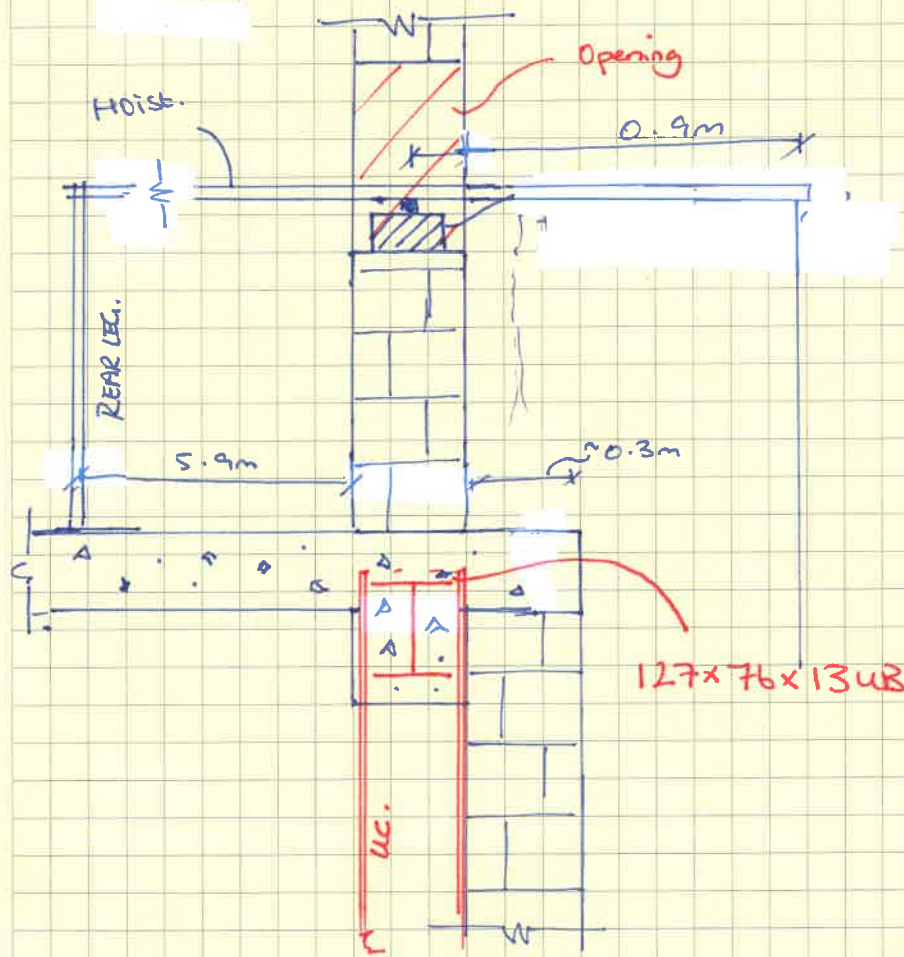
OUTPUT:



Supported on masonry
wall which sits on
a $127 \times 76 \times 13$ UB

$178 \times 102 \times 19$ UB

HOIST SPACING
1m x 0.4m



Client: Newport Norse Engineer: AK Checked By: JSProject: Bell tower Cradle Date: 08/05/25 Approved By: JSJob No: CC2753 Newport Civic Centre Sheet: 4 Status: POI

CALCULATIONS:

OUTPUT:

Building Check historic info

Concrete encased steel Portland stone cladding

BS CP3 - Chapter V-1: 1967 Code design

BS 449 - Use structural steel.

 $\therefore$ Designed for Imposed load Cannot be confirmed.Likely either maintenance 0.6 kN/m^2 or class VI

'Chapels' however as cannot be confirmed

we will design for solely access/maintenance

load 0.6 kN/m^2 DL - Slab = $24 \times 0.25 = 6 \text{ kN/m}^2$ no partitions / ceiling
 $= 6 \text{ kN/m}^2$ DL wall - $4.2 \times 3 \text{ m} = 12.6 \text{ kN/m}$ TOTAL LOADINGDL - 6 kN/m^2
 12.6 kN/m IL - 0.6 kN/m^2 point IL - 15 kN
 3.5 kN



Client: Newport Norse. Engineer: GK Checked By: JS

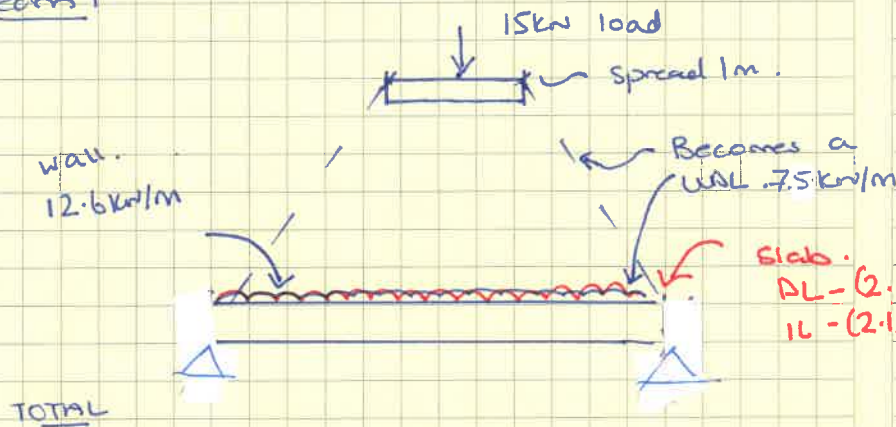
Project: Bell tower Cradle Date: 08/05/25 Approved By: JS

Job No: CE2753 Newport Curve Gate Sheet: 5 Status: PO1

CALCULATIONS:

OUTPUT:

Beam 1



$$\begin{aligned} \text{Slab:} \\ \text{DL} &= (2.1/2) \times 6 = 6.3 \text{ kN/m} \\ \text{IL} &= (2.1/2) \times 0.6 = 0.63 \text{ kN/m} \end{aligned}$$

TOTAL

$$\begin{aligned} \text{SLAB} \quad \text{DL} &= 8.19 \text{ kN/m} \\ \text{IL} &= 0.63 \text{ kN/m} \end{aligned}$$

$$\text{Wall} \quad \text{DL} = 12.6 \text{ kN/m}$$

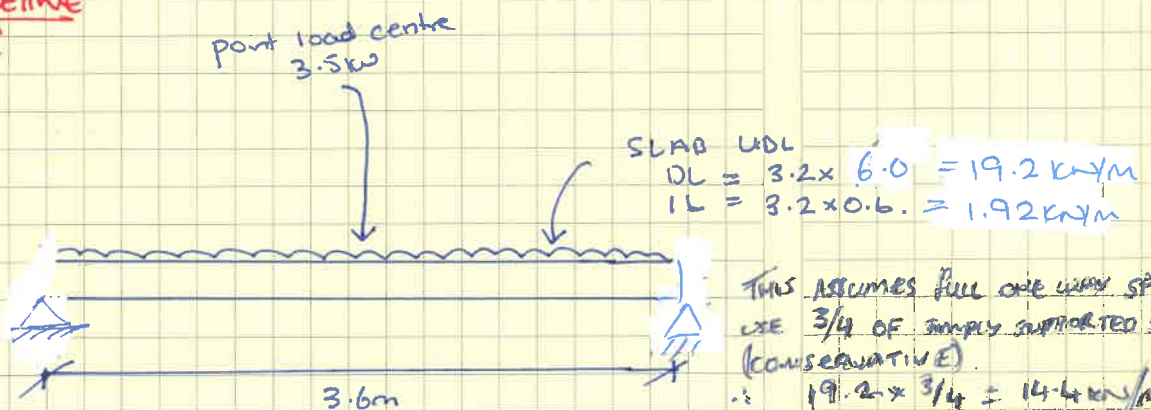
$$\text{Cradle} \quad \text{IL} = 7.5 \text{ kN/m} \quad \text{use } 7.5/1.6 = 4.7 \text{ kN/m}$$

AS the 7.5' is FACTORED FOR 3.0

Beam - 127x102x19 UB Fully Restrained → PASS CHECKS

Beam 2

WORST CASE
CONCRETE
BEAM



$$\begin{aligned} \text{SLAB UDL} \\ \text{DL} &= 3.2 \times 6.0 = 19.2 \text{ kN/m} \\ \text{IL} &= 3.2 \times 0.6 = 1.92 \text{ kN/m} \end{aligned}$$

THIS ASSUMES FULL ONE WAY SPAN
USE 3/4 OF SIMPLY SUPPORTED SPAN
(CONSERVATIVE)
 $\therefore 19.2 \times 3/4 = 14.4 \text{ kN/m}$
 $1.92 \times 3/4 = 1.44$

TOTAL

$$\begin{aligned} \text{SLAB} - \text{DL} &= 19.20 \text{ kN/m} \\ \text{IL} &= 1.92 \text{ kN/m} \end{aligned}$$

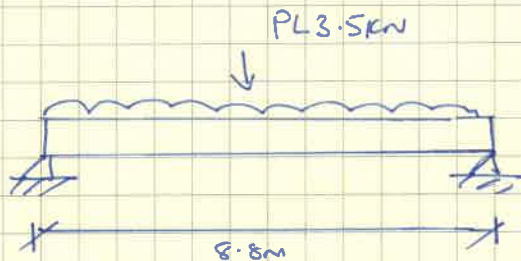
$$\text{Cradle} - \text{IL} = 3.5 \text{ kN point} \quad \text{use } 2.2 \text{ kN as ALREADY FOS APPLIED}$$

Beam - 178x102x19 UB → PASSES CHECKS

Client: Newport Noise Engineer: GK Checked By: JSProject: Poll tower Cradle Date: 08/05/25 Approved By: JSJob No: CC2753 Newport Civil Centre Sheet: 6 Status: PO1

CALCULATIONS:

OUTPUT:

Beam 3

$$D = 6 \text{ kN/m}^2$$
$$L = 0.6 \text{ kN/m}^2$$

Slab WDL

$$DL = 6 \text{ kN/m}^2 \times 6.4 \text{ m} / 2 = 19.2 \text{ kN/m}$$

$$LL = 0.6 \text{ kN/m}^2 \times 6.4 \text{ m} / 2 = 1.92 \text{ kN/m}$$

TOTAL

Slab $DL = 19.2 \text{ kN/m}$

$LL = 1.92 \text{ kN/m}$

Cradle $- LL = 3.5 \text{ kN PL (ALREADY FACTORED} = 3.5 / 1.6) = 2.2 \text{ kN}$

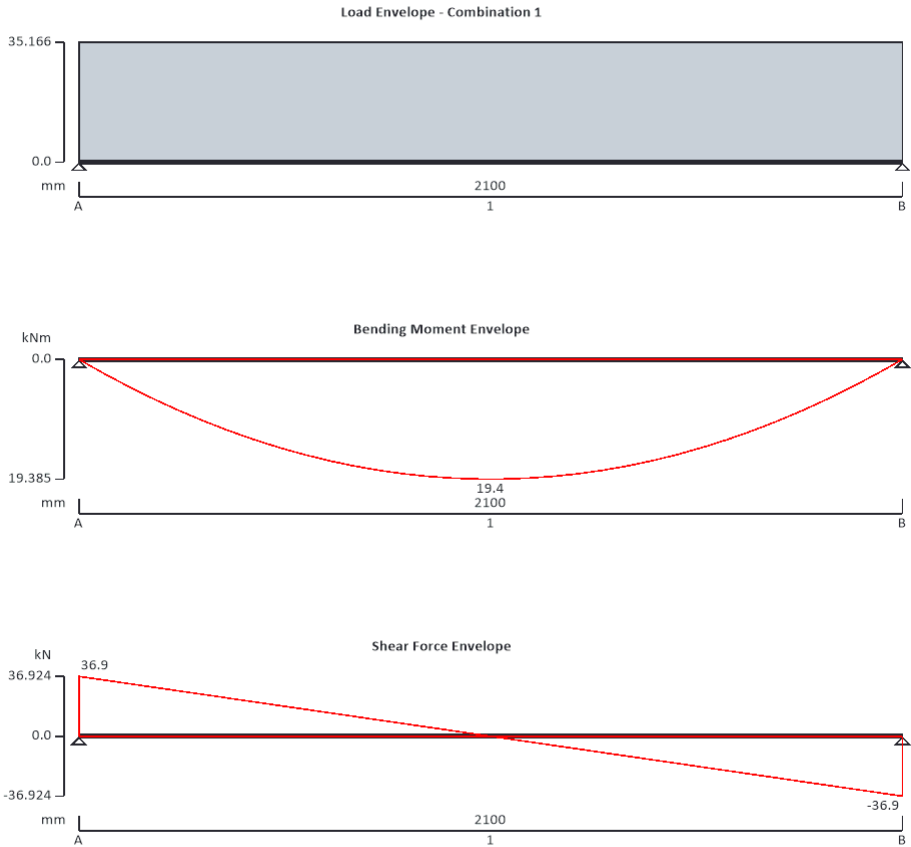
Beam $= 457 \times 19 \text{ UB } 67 \text{ (S355)}$ PASSED

Tekla Tedds Cambria Consulting Ltd Cambria House 16-17a Plas St. Pol de Leon Penarth, CF64 1TR	Project Newport Civic Centre				Job no. CC2753	
	Calcs for Cradle Beam 1				Start page no./Revision 1	
	Calcs by GK	Calcs date 09/05/2025	Checked by	Checked date	Approved by	Approved date

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.08



Support conditions

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

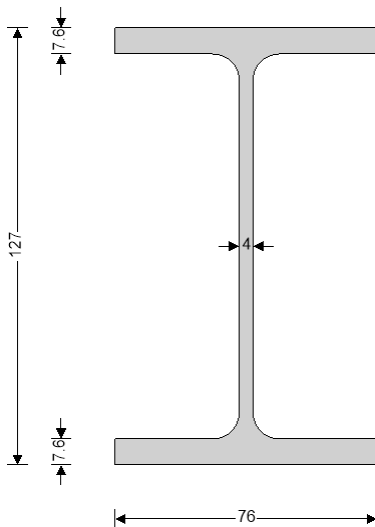
Applied loading

Beam loads	SLAB DL - Dead full UDL 6.3 kN/m SLAB IL - Imposed full UDL 0.63 kN/m WALL DL - Dead full UDL 12.6 kN/m CRADLE IL - Imposed full UDL 4.7 kN/m Dead self weight of beam × 1
------------	--

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

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	Calcs for Cradle Beam 1				Start page no./Revision 2	
	Calcs by GK	Calcs date 09/05/2025	Checked by	Checked date	Approved by	Approved date

			Imposed × 1.60		
Analysis results					
Maximum moment	$M_{\max} = 19.4 \text{ kNm}$		$M_{\min} = 0 \text{ kNm}$		
Maximum shear	$V_{\max} = 36.9 \text{ kN}$		$V_{\min} = -36.9 \text{ kN}$		
Deflection	$\delta_{\max} = 1.4 \text{ mm}$		$\delta_{\min} = 0 \text{ mm}$		
Maximum reaction at support A	$R_{A_{\max}} = 36.9 \text{ kN}$		$R_{A_{\min}} = 36.9 \text{ kN}$		
Unfactored dead load reaction at support A	$R_{A_{\text{Dead}}} = 20 \text{ kN}$				
Unfactored imposed load reaction at support A	$R_{A_{\text{Imposed}}} = 5.6 \text{ kN}$				
Maximum reaction at support B	$R_{B_{\max}} = 36.9 \text{ kN}$		$R_{B_{\min}} = 36.9 \text{ kN}$		
Unfactored dead load reaction at support B	$R_{B_{\text{Dead}}} = 20 \text{ kN}$				
Unfactored imposed load reaction at support B	$R_{B_{\text{Imposed}}} = 5.6 \text{ kN}$				
Section details					
Section type	UB 127x76x13 (British Steel Section Range 2022 (BS4-1))				
Steel grade	S275				
From table 9: Design strength p_y					
Thickness of element	$\max(T, t) = 7.6 \text{ mm}$				
Design strength	$p_y = 275 \text{ N/mm}^2$				
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$				
					
Lateral restraint					
	Span 1 has full lateral restraint				
Effective length factors					
Effective length factor in major axis	$K_x = 1.00$				
Effective length factor in minor axis	$K_y = 1.00$				
Effective length factor for lateral-torsional buckling	$K_{LT,A} = 1.00$ $K_{LT,B} = 1.00$				
Classification of cross sections - Section 3.5					
	$\epsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$				
Internal compression parts - Table 11					
Depth of section	$d = 96.6 \text{ mm}$				
	$d / t = 24.2 \times \epsilon \leq 80 \times \epsilon$				
	Class 1 plastic				

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	Calcs for Cradle Beam 1				Start page no./Revision 3	
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Outstand flanges - Table 11

Width of section

$$b = B / 2 = \mathbf{38 \text{ mm}}$$

$$b / T = 5.0 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{36.9 \text{ kN}}$$

$$d / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = \mathbf{508 \text{ mm}^2}$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = \mathbf{83.8 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{19.4 \text{ kNm}}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{23.1 \text{ kNm}}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{5.833 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{1.391 \text{ mm}}$$

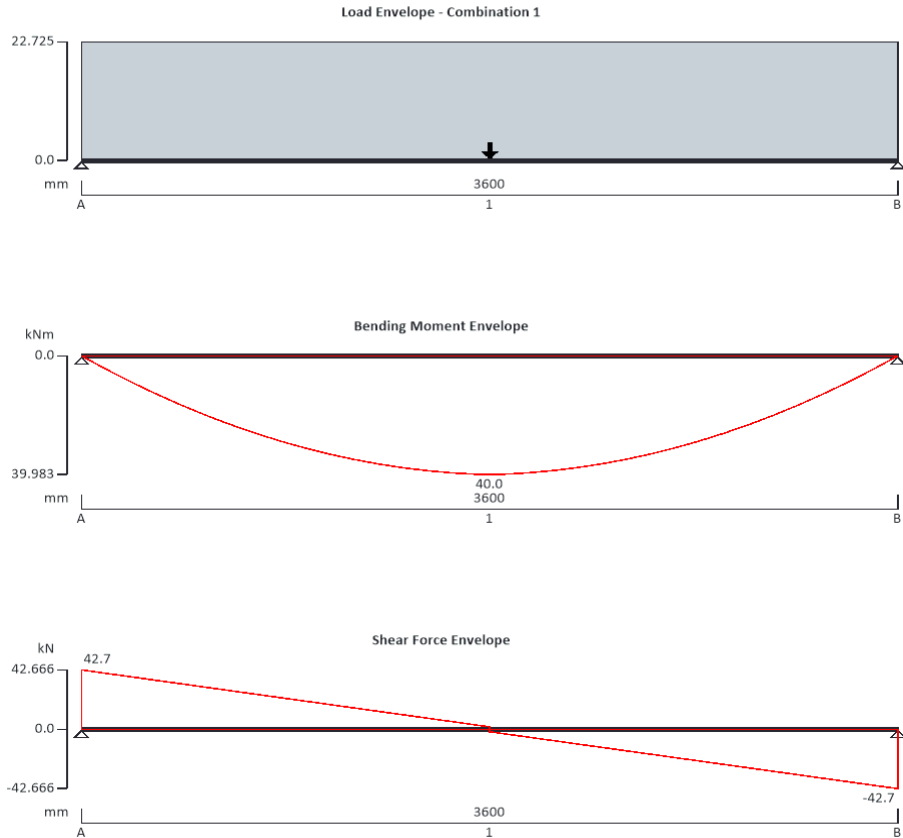
PASS - Maximum deflection does not exceed deflection limit

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STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.08



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

SLAB DL - Dead full UDL 14.4 kN/m

SLAB IL - Imposed full UDL 1.44 kN/m

CRADLE IL - Imposed point load 2.2 kN at 1800 mm

Dead self weight of beam $\times$ 1

Load combinations

Load combination 1

Support A

Dead $\times$ 1.40

Imposed $\times$ 1.60

Dead $\times$ 1.40

Imposed $\times$ 1.60

Support B

Dead $\times$ 1.40

Imposed $\times$ 1.60

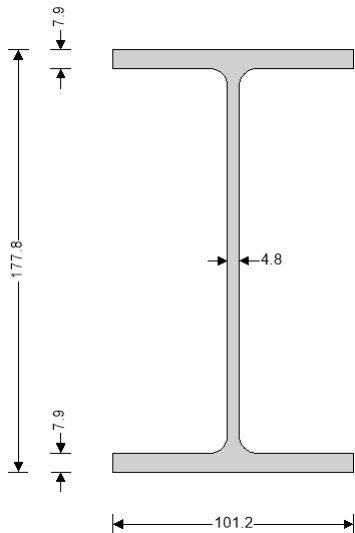
 Cambria Consulting Ltd Cambria House 16-17a Plas St. Pol de Leon Penarth, CF64 1TR	Project Newport Civic Centre				Job no. CC2753	
	Calcs for Cradle Beam 2				Start page no./Revision 2	
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Analysis results

Maximum moment	$M_{max} = 40 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 42.7 \text{ kN}$	$V_{min} = -42.7 \text{ kN}$
Deflection	$\delta_{max} = 1.9 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 42.7 \text{ kN}$	$R_{A_min} = 42.7 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 26.3 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 3.7 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 42.7 \text{ kN}$	$R_{B_min} = 42.7 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 26.3 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 3.7 \text{ kN}$	

Section details

Section type	UB 178x102x19 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 7.9 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT,A} = 1.00$
	$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$$

Internal compression parts - Table 11

Depth of section	$d = 146.8 \text{ mm}$	
	$d / t = 30.6 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

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Outstand flanges - Table 11

Width of section

$$b = B / 2 = \mathbf{50.6 \text{ mm}}$$

$$b / T = 6.4 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{42.7 \text{ kN}}$$

$$d / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = \mathbf{853 \text{ mm}^2}$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = \mathbf{140.8 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{40 \text{ kNm}}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{47.1 \text{ kNm}}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{10 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{1.902 \text{ mm}}$$

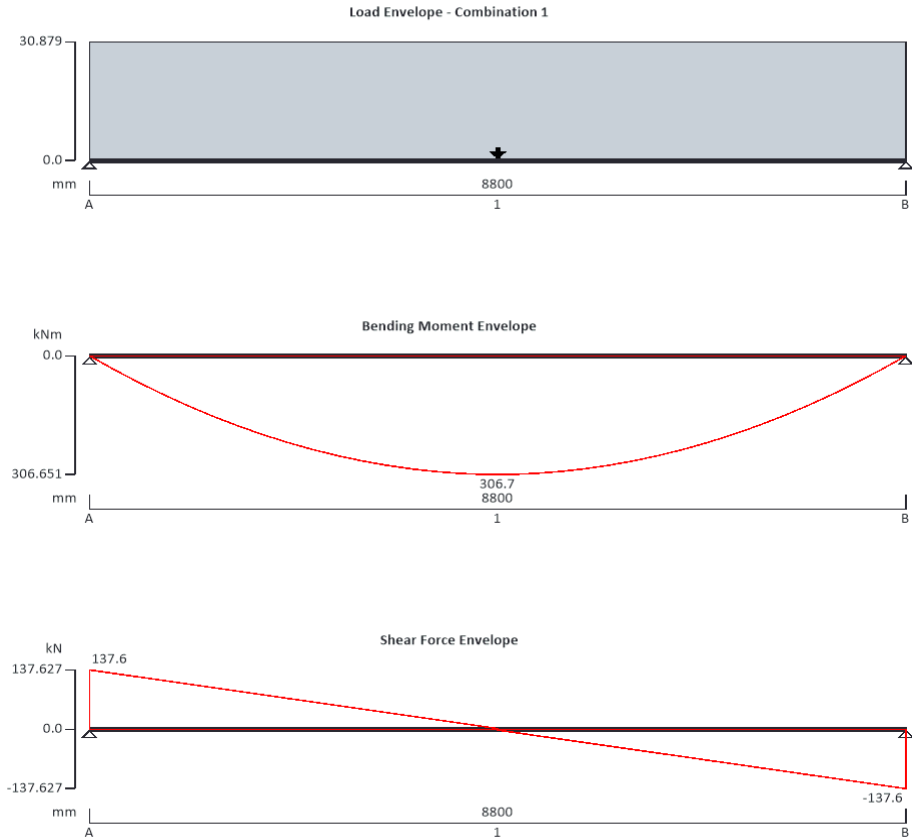
PASS - Maximum deflection does not exceed deflection limit

Tekla Tedds Cambria Consulting Ltd Cambria House 16-17a Plas St. Pol de Leon Penarth, CF64 1TR	Project Newport Civic Centre				Job no. CC2753	
	Calcs for Cradle Beam 3				Start page no./Revision 1	
	Calcs by GK	Calcs date 09/05/2025	Checked by	Checked date	Approved by	Approved date

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.08



Support conditions

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

Applied loading

Beam loads	SLAB DL - Dead full UDL 19.2 kN/m
	SLAB IL - Imposed full UDL 1.92 kN/m
	CRADLE IL - Imposed point load 2.2 kN at 4400 mm
	Dead self weight of beam $\times 1$

Load combinations

Load combination 1	Support A	Dead $\times 1.40$
		Imposed $\times 1.60$
		Dead $\times 1.40$
		Imposed $\times 1.60$
	Support B	Dead $\times 1.40$
		Imposed $\times 1.60$

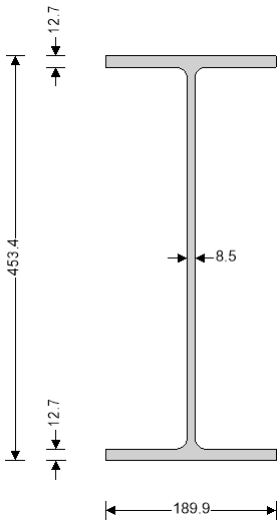
 Cambria Consulting Ltd Cambria House 16-17a Plas St. Pol de Leon Penarth, CF64 1TR	Project Newport Civic Centre				Job no. CC2753	
	Calcs for Cradle Beam 3				Start page no./Revision 2	
	Calcs by GK	Calcs date 09/05/2025	Checked by	Checked date	Approved by	Approved date

Analysis results

Maximum moment	$M_{max} = 306.7 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 137.6 \text{ kN}$	$V_{min} = -137.6 \text{ kN}$
Deflection	$\delta_{max} = 3 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 137.6 \text{ kN}$	$R_{A_min} = 137.6 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 87.4 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 9.5 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 137.6 \text{ kN}$	$R_{B_min} = 137.6 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 87.4 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 9.5 \text{ kN}$	

Section details

Section type	UB 457x191x67 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 12.7 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT,A} = 1.00$
	$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

$\epsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$

Internal compression parts - Table 11

Depth of section	$d = 402.6 \text{ mm}$	
	$d / t = 47.4 \times \epsilon \leq 80 \times \epsilon$	Class 1 plastic

SECTION NO. 5 Loads

5.1 Introduction

The question of the loading for which a structure was designed, or for which it should be designed requires a great deal of consideration. Since the regulations and codes discussed in the previous section became established loading can be assumed with a fair degree of accuracy, but prior to the introduction of such regulations loading often depended upon the whim of the engineer.

Experience has shown that some engineers calculated with some accuracy the amount of loading that a structure or structural element would have to carry in order to serve its purpose and designed exactly to this amount giving no margin whatsoever to possible change of process or occupancy at a later stage!

Loadings will be discussed under three main headings viz:—

- Dead Loads
- Live or Imposed Loads
- Wind Loads.

5.2 Dead Loads

The dead load to be carried by a structural member is the mass per unit area multiplied by the area supported. The area supported is usually easy to determine whilst the mass per unit area must be calculated from an assessment of the construction.

It was mentioned in an earlier section that BS648 which was first published in 1935 was an attempt to rationalise the calculation of dead loads.

Earlier efforts at achieving this object can be found on investigation.

In the introduction to their 1895 Handbook Dorman Long and Company state:— "In planning a floor the first thing to be determined is the load that will be placed upon it. This consists of the weight of the materials composing the floor — or the dead load — and the weight of the persons or goods — being the live load, which together make up the total load. The dead load of a fire-proof floor made of steel joists and 6in coke-breeze concrete may be taken as 70lbs/sq foot . . .".

In their various handbooks Redpath, Brown and Co Limited gave extensive tables of approximate weights of materials for estimating dead and superimposed loads, from which extracts are given in Table 5.1, these were sufficiently comprehensive to enable dead loads to be calculated with a fair degree of accuracy.

Table 5.1 Weights for calculating dead loads

Material	Mass per cubic foot	Mass per sq foot per inch thickness
Asphalt	112 lbs	9 to 10 lbs
Brickwork:—		
Pressed	140 lbs	11 to 12 lbs
Common	120 lbs	10 lbs
Inferior	100 lbs	8 to 9 lbs
Concrete:—		
Breeze	96 lbs	8 lbs
Brick	120 lbs	10 lbs
Stone	144 lbs	12 lbs
Masonry:—		
Dressed freestone	150 lbs	12 to 13 lbs
Dressed granite	165 lbs	13 to 14 lbs
Timber:—		
Jarrah paving	60 lbs	5 lbs
Pine boarding	30 to 45 lbs	2½ to 3½ lbs
Plaster	96 lbs	8 lbs
Slate	180 lbs	15 lbs
Lead sheet	710 lbs	60 lbs
Copper sheet	558 lbs	48 lbs

Note:— The usual allowance for a plaster suspended ceiling was 10 lbs/sq foot to include laths etc, and 5lbs/sq foot per plastered face was added to the weight of brickwork in walls etc.

5.3 Live Loads

The term "live" in this context is intended to cover the use to which a floor is subjected, and is also called "imposed" or "superimposed". It is not intended to cover moving or dynamic loads.

The allowance made for live load has varied appreciably and prior to the turn of the century was very much a factor to be decided by the architect, having regard to the circumstances.

In about 1945 an investigation carried out on the capacity of the floors of a cotton spinning mill in Lancashire showed an allowance for live load of well under 20lbs/sq foot! The floors had been provided to support spinning mules, and a calculation of the weight of such a machine in operation revealed an imposed load of 7lbs/sq foot.

It follows from this that when dealing with the strength of old construction care must be taken to carefully assess the dead load if the margin for live load is to be factual.

Not all buildings were designed to so narrow a margin, and advice on live loads was available from various sources. Some of these are as follows:

5.3.1 Encyclopaedia of Architecture 1881 (first published 1842)

1. Light workshops and factories, public halls and churches and other buildings in which people only accumulate, with warehouses for light goods. An allowance of 168lbs/sq foot (including 40lbs/sq foot for timber floor) ie a live load of 128lbs/sq foot.
2. Storehouses for heavy goods 280lbs/sq foot (including 40lbs/sq foot dead load) ie a live load of 240lbs/sq foot.
3. Ordinary dwelling houses 140lbs/sq foot (including 40lbs/sq foot dead load as before) ie a live load of 100lbs/sq foot.

5.3.2 Dorman Long and Co Handbook 1895

Deducting the dead load allowance already referred to, this handbook makes the following recommendations:—

1. Dwellings or Office Buildings 80lbs/sq foot
2. Public Halls or Schools 110lbs/sq foot
3. Warehouses 120 to 320lbs/sq foot
4. Heavy Machinery 220 to 420lbs/sq foot

In the case of 1 and 2 70lbs/sq foot has been deducted for dead load but in the case of 3 and 4 this has been increased to 80lbs/sq foot as a more realistic amount.

5.3.3 Appleby's Handbook of Machinery 1903

The approximate live loads for various classes of building are as given in Table 5.2

Table 5.2 Live Loads on Floors

Classification	Live Load
Attic floors	34 to 56 lbs/sq ft
Dwelling room floors	56 to 70 lbs/sq ft
Offices, libraries etc	70 to 80 lbs/sq ft
Stairs and passages	80 to 90 lbs/sq ft
Work rooms, light business premises etc	90 to 100 lbs sq ft
Churches, theatres, ballrooms, public halls, stores etc	100 to 125 lbs/sq ft
Workshops and light warehouses	125 to 150 lbs/sq ft
Warehouses for heavy materials	150 to 400 lbs/sq ft
Heavy machinery	250 to 500 lbs/sq ft

5.3.4 London County Council (General Powers) Act 1909

Clause 18(A) of the above Act gave the following requirements for live loads:—

1. For a floor intended to be used wholly for the purpose of human habitation 70lbs/sq foot
2. For a floor intended to be used wholly for office or counting-house etc 100lbs/sq foot
3. For a floor intended to be used wholly for workshop or a retail store 112lbs/sq foot
4. For every floor of a building of the warehouse class not intended to be used wholly for any of the purposes aforesaid not less than 224lbs/sq foot
5. For a roof the plane of which inclines upwards at a greater angle than 20 degrees to the horizontal a live load (deemed to include wind pressure) of 28lbs/sq foot of surface
6. For all other roofs 56lbs/sq foot on plan.

Table 5.3 Live loads on floors per I.S.E. 1927

Class and loading	Applications
(a) 40 lb/sq ft	Tenements and upper floors of houses not exceeding 4 storeys in height
(b) 60 lb/sq ft	Upper floors of houses of more than 4 storeys Ground floor of houses not exceeding 4 storeys Bedrooms on upper floors of hotels Private bedrooms in hospitals etc Residential flats
(c) 80 lb/sq ft	Classrooms in schools etc. Dormitories and upper floors in offices
(d) 100 lb/sq ft	Assembly Halls etc including access thereto, churches, chapels, assembly rooms in hospitals, colleges etc. Theatres etc. Dance halls, ball-rooms and reception rooms in hotels and private houses. Ground floors of houses exceeding 4 storeys, Ground floors and below of offices, hospitals etc. Retail shops for display and sale of light-weight goods. Garages for private cars. Light workshops and storage.
(e) 150 lb/sq ft	Garages for vehicles up to 3 ton gross weight, Rooms for storage of medium weight goods and medium workshops, Theatre stages, Drill Halls and Spectators stands.
(f) 200 lb/sq ft	Book stores at libraries, Museums for heavy goods, Heavy storage and workshops etc, Pavements surrounding buildings.
(g) Over 200 lb/sq ft	Storage of extra heavy goods and floors supporting printing machinery and the like
(h) Staircases etc.	Same as floor served but not to exceed 100 lb/sq foot.

5.3.5 Institution of Structural Engineers Report on Steelwork for buildings 1927

This report gave a very comprehensive classification of live loads which is shown in Table 5.3. It also gave loads for flat and sloping roofs. In 1933, after BS449 1932 was issued a revised schedule was published which is shown in Table 5.4. The roof loads in the 1927 report are compared with the 1933 revision in Table 5.5.

An interesting development shown in the 1933 revision was the requirement of minimum loads for the design of slabs in some instances greater than those for the structure. These are given in brackets in Table 5.4.

Table 5.4 Live loads on floors per I.S.E. 1933

Class and loading	Applications
(a) 40 lb/sq ft (50 lb/sq ft for slabs)	Bedrooms, living rooms, toilets, bathrooms and private corridors in private houses, hotels, flats, hospitals, tenements etc; dormitories and wards in hospitals etc.
(b) 50 lb/sq ft (60 lb/sq ft for slabs)	Offices, corridors and passages above entrance floor; public corridors to class (a)
(c) 70 lb/sq ft (80 lb/sq ft for slabs)	Rooms and halls for public assembly with fixed seating including theatres etc. classrooms etc in schools and colleges, reading and writing rooms in libraries, clubs and hotels; churches, chapels and art galleries.
(d) 80 lb/sq ft (ditto for slabs)	Retail shops, offices on entrance floor and below. Garages for cars not exceeding 2 ton gross wt.
(e) 100 lb/sq ft (ditto for slabs)	Rooms and halls for public assembly without fixed seating; drill halls, dance halls and public rooms in hotels, hospitals and restaurants spectators stands, lobbies, corridors and stairs to this class. Light workshops.
(f) 120 lb/sq ft (150 lb/sq ft for slabs)	Garages for vehicles up to 3 tons gross weight when loaded. Storage rooms, workshops, retail shops, bookshops and libraries where it can be shown that the average load/sq foot does not exceed 120 lbs. Staircases etc to this class.
(g) 200 lb/sq ft (ditto for slabs)	Storage rooms workshops and the like where it can be shown that the average load/sq foot does not exceed 200 lbs. Staircases etc to this class. Pavements surrounding buildings but not adjoining roadway.

Table 5.6 Imposed loads from BS449 1948

Class of loading	Floor space occupancy	Superimposed floor load per sq.ft.	Minimum* loads on slabs or floor boards per ft. width, uniformly distributed	Minimum* load on beams uniformly distributed
I	Private dwellings of not more than two storeys	lbs 30	lbs 240	lbs 1920
II	Rooms in private dwellings of more than two storeys, including flats; hospital rooms and wards; bedrooms and private sitting rooms in hotels and tenement houses; and similar occupancies	40	320	2560
III	Rooms used as offices	50	400	3200
IV	Classrooms in schools and colleges; minimum for light workshops	60	480	3840
V	Banking halls and offices where the public may congregate	70	560	4480
VI	Retail shops; places of assembly with fixed seating†; churches and chapels; restaurants; garages for vehicles not exceeding 2½ tons gross weight (private cars, light vans, etc.); circulation space in machinery halls, power stations, pumping stations etc., where not occupied by plant or equipment	80	640	5120
VII	Places of assembly without fixed seating (public rooms in hotels, dance halls, etc.); minimum for filing or record rooms in offices; light, workshops generally, including light machinery	100	800	6400
VIII	Garages to take all types of vehicles	100	Worst combination of actual wheel loads or, if the slab is capable of effective lateral distribution of load, 1.5 x maximum wheel load, but not less than 2000 lb, considered to be distributed over a floor area 2 ft.6 in. square.	
IX	Light storage space in commercial and industrial buildings; medium workshops	150	—	—
X	Minimum for warehouses and general storage space in commercial and industrial buildings; heavy workshops. (The loads imposed by heavy plant and machinery should be determined and allowed for.)	200	—	—

* Minimum load for slabs becomes operative at spans of less than 8 ft. Minimum load for beams becomes operative on areas less than 64 sq. ft. Beams, ribs and joists spaced at not more than 3 ft. centres may be calculated for slab loadings.

The loadings stated in these columns refer to the design of the slabs and individual beams respectively, including their connections.

† Fixed seating implies that the removal of the seating and the use of the space for other purposes are improbable.

Table 5.5 Comparison of roof loads to I.S.E.

Report published 1927	Report revised 1933
The following imposed loads apply to the horizontal and vertical projection of the roof	The following imposed loads deemed to include snow and wind
(i) Flat roofs with inclination not exceeding 15° to horizontal Vertical load 50 lb/sq ft Horizontal load 0 lb/sq ft	(i) On roofs with inclination not exceeding 10° to horizontal:— (a) Where no direct access 15lb/sq foot (b) Where direct access 30 lb/sq foot
(ii) Sloping roofs with inclination not exceeding 40° to horizontal Vertical load 25 lb/sq ft Horizontal load 10 lb/sq ft	(ii) On all roofs inclined at more than 10° to to horizontal, curved and barrel roofs etc. without access. 10 lb/sq foot on plan taken as uniformly distributeol over the slope of the roof plus the normal wind load.
(iii) Sloping roofs with inclination not exceeding 75° to horizontal Vertical load 15 lb/sq ft Horizontal load 15 lb/sq ft	
(iv) Sloping roofs with inclination exceeding 75° to horizontal to be treated as vertical surfaces.	
The above loads are to be considered as inclusive of temporary loads, snow and wind, and only apply to roofs in multi-storey steel frame buildings not exceeding 50 ft in span. However, the imposed load on a flat roof need not exceed the imposed load on the floor below.	

5.3.6 British Standard 449 1932

This specification gave a much simpler schedule of loading viz:—

Imposed loads

Floors of rooms used for domestic purposes	40lb/sq ft.
Offices — floors above entrance floor	50lb/sq ft
Offices — entrance floor and below	80lb/sq ft
Churches, reading rooms etc	70lb/sq ft
Retail stores	80lb/sq ft
Assembly Halls etc	100lb/sq ft
Flat Roofs	30lb/sq ft
Inclined Roofs — wind load only at 15lb/sq ft inwards on windward slope and 10lb/sq ft outwards on leeward slope	

This substantially remained unaltered until 1948.

5.3.7 British Standard 449 1948

Ten categories of imposed load were tabulated. In addition to the distributed loads per square foot of area minimum loads on slabs and on beams were also given.

This is given as Table 5.6.

5.3.8 General

Sufficient information has been given to show the development of live or imposed loads over a long period of time. The publication of CP3 Chapter V 1952 and subsequent amendments transferred loadings from design specifications to separate documents.

CP3 Chapter V should be consulted for any required information subsequent to BS449 1948.

It is interesting to note that E.S. Andrews in his book *Theory and Design of Structures* (Fifth Edition 1932) gives a table comparing live loads in Britain and those concurrently in use in Canada, New York and Germany.

The figures quoted for Canada show substantial reductions on the ISE and LCC values which have already been quoted herein.

5.4 Wind Loads

From the very nature of the subject this has been one of the most difficult matters to resolve.

Prior to the Tay Bridge disaster in 1879 comparatively little attention was given to the subject by engineers, though Smeatons formula, published in 1759, ie 120 years earlier, proves that it had long been appreciated that forces due to wind pressure were both real and important.

Smeatons formula is given as:—

$$p = 0.005V^2$$

where p is the pressure or load in lbs/sq foot and V the velocity of the wind in miles/hour.

Following upon experiments at the National Physical Laboratory it was considered that Smeaton's formula gave results which were too high and the formula was altered to:—

$$p = 0.003V^2$$

Using this latter formula a wind velocity of 100 miles per hour gives a pressure of 30lbs/sq foot, but as such a velocity of wind would not normally be sustained, the average velocity would give a more realistic loading.

Prior to the erection of the Forth Railway Bridge Sir Benjamin Baker conducted experiments on wind loads by measuring the pressure on plates exposed to the wind.

Records of such experiments over a period from 1884 to 1890 gave the following average results:—

(a)	On 1.5sq ft revolving gauge	27.6lb/sq ft
(b)	On 1.5sq ft fixed gauge	29.8lb/sq ft
(c)	On 300sq ft fixed gauge	16.9lb/sq ft

Subsequent to the erection of the Forth Railway Bridge records were kept on small fixed gauges 1.5sq ft in area placed at different heights above high water level. The average pressures recorded between 1901 and 1906 were:—

(a)	At 50ft above high water level	13lb/sq ft
(b)	At 163ft above high water level	23lb/sq ft
(c)	At 214ft above high water level	29lb/sq ft
(d)	At 378ft above high water level	50lb/sq ft.

It is on record that wind loads of 30, 40, 50 and even 56lb/sq ft have been specified by various authorities prior to the introduction of the L.C.C. loading in 1909.

In 1930 a volume, *Wind Stresses in Buildings* by Robins Fleming was published in USA. This details the investigation into wind loads at length and was accepted at that period as the most authoritative work on the subject.

Various specifications for wind loads are as follows:—

5.4.1 London County Council (General Powers) Act 1909

The requirements for loads on roofs which were deemed to include wind forces were given in item 5.3.4. and will not be repeated. The most important requirement is:—

"All buildings shall be so designed as to resist safely a wind pressure in any horizontal direction of not less than 30lb/sq ft of the upper two-thirds of the surface of such buildings exposed to wind pressure".

This requirement was strictly imposed in the design of buildings in London. It was also used outside London prior to the publication of the ISE Report.

5.4.2 Institution of Structural Engineers — Report on Steelwork for Buildings Part 1 Loads and Stresses 1927

Apart from the loads on roofs which have already been specified (see 5.3.5.) the following simple requirement was given:—

"The whole building must be designed to resist a horizontal wind load equivalent to wind pressure on a projected vertical surface of:—

- (a) Up to a level of 30ft above ground 15lb/sq ft
- (b) At levels more than 30ft above ground 20lb/sq ft
- (c) Towers, turrets and like structures projecting above roof 30lb/sq ft

The negative wind pressure on the leeward side may be ignored for ordinary buildings".

5.4.3 Institution of Structural Engineers — Report on Steelwork for Buildings Part 1 Loads and Stresses 1938

This report made drastic amendments to wind loads on the complete structure, as well as on roofs which have already been reported upon (see 5.3.5.)

All wind forces must be calculated on the whole area of the exposed surface irrespective of length and height as follows:—

- (a) On vertical exposed surfaces normal to the direction of the wind.
A horizontal wind force of 5lb/sq ft at mean ground level increasing by 1lb/sq ft for each 10ft increase in height up to a maximum of 15lb/sq ft at a height of 100ft and then a constant value of 15lb/sq ft for all heights exceeding 100ft.
- (b) Wind forces equal to 50% in excess of the above values for exposed chimneys, isolated ventilators and similar projections above general roof level.
- (c) Wind forces normal to inclined windward surfaces transverse to the direction of the wind equal to the mean intensity of the wind between eaves and ridge, calculated as (a) above and multiplied by coefficient C from Table 5.7 below.

Table 5.7 List of Coefficients C for calculating wind forces normal to surface

θ	C	θ	C	θ	C
5°	0.173	30°	0.800	65°	0.995
10°	0.337	35°	0.863	70°	0.998
15°	0.486	40°	0.910	75°	0.999
20°	0.612	45°	0.943	80°	1.00
25°	0.718	50°	0.965	85°	1.00
26.34°	0.745	55°	0.980	90°	1.00
		60°	0.990		

NB. θ is the angle of inclination to the horizontal

- (d) Wind Forces normal to inclined leeward surfaces transverse to the direction of the wind equal to 50% of the values given in (c) acting outward, to be considered as acting simultaneously with the downward acting force on the windward face specified in (c).
- (e) For open sided buildings the wind load on the leeward slope to be 100% that on the windward slope.
- (f) Wind loads on gable ends to be calculated from (a) except that between eaves and ridge the mean value to be used.
- (g) A dragging force on the full area of surfaces parallel to the direction of the wind of $2\frac{1}{2}\%$ of the value of the wind at the appropriate height from (a) above.
- (h) Reduction of wind loads on multi-span roofs, viz
Span next windward span 50% reduction
Next span 75% reduction
Remaining spans 87½% reduction
these to be used to calculate the overall effect of the wind on the structure.

- (i) Refers to wind on cylindrical surfaces to be taken at 60% of values in (a) and (b).
- (j) All the forces specified to apply where velocity V of wind at a height of 50ft above mean ground level was not expected to exceed 80 miles/hour. On sea coast and other exposed positions the values given to be increased in ratio of:—
(Anticipated Velocity V)²

$$80^2$$

As will be seen the determination of wind loads to this report was very comprehensive, and much more related to fact than previous requirements as will be appreciated by reference to CP3 Chapter V 1972.

5.4.4. British Standard 449 1932

This substantially repeated the LCC requirements with some additions and alterations viz:—

Wind on vertical surfaces	15lb/sq ft horizontal on upper $\frac{2}{3}$ rds
Wind on inclined surfaces	
Windward	15lb/sq ft inward
Leeward	10lb/sq ft outward

Wind on projections above main roof	10lb/sq ft horizontal
-------------------------------------	-----------------------

5.4.5 British Standard 449 1948

Ten years after the Institution of Structural Engineers issued their amended Part 1 — Loads and Stresses 1938 of their Report on Steelwork for Buildings, which so drastically altered the whole approach to wind loading, British Standard 449 was also altered to give a more rational view of wind loading.

This specification now related the wind pressure to the assumed wind velocity in miles per hour and the exposed height of the building.

For assuming the wind velocity the United Kingdom was divided into three zones and three conditions of exposure as shown in Table 5.8.:

Table 5.8 Wind Velocities in miles/hour

District	With natural protection	Open country inland	Condition of max. exposure
West Scotland			
N W Ireland			
N W England	60	70	80
S W England			
Wales			
East Scotland			
N E Ireland			
N E England	55	65	75
E England			
S E England			
S England			
Central England including Severn Estuary	50	60	70

The unit wind pressure (p) in lbs/sq ft on the vertical face of a building applied uniformly over the whole surface was given by the formula:—

$$p = \frac{V^2}{600} \sqrt{1 + 0.06 (h-s)}$$

where:—

- h — the height of the windward face above general ground level
s — the assumed sheltered height from permanent obstacles (not to be taken at more than 0.5h).

This unit wind pressure (p) was used to determine the applied forces as follows:—

- (i) The external wind load on vertical surfaces of buildings was divided into a pressure of 0.5p on the windward face and a suction of 0.5p on the leeward face.
N.B. A surface inclined to the horizontal of 70° or more was considered to be vertical.
(ii) The external wind load on roofs was as shown in Table 5.9:—

Table 5.9 Wind Pressure on roofs

Slope of roof in degrees	Windward slope or half of flat roof	Leeward slope or half of flat roof
0	— 1.0p	} — 0.5p
22½	— 0.25p	
30	0	
45	+ 0.25p	
70 and over	+ 0.5p	

The specification required that the supporting framework and foundation of a building must be designed to resist the accumulative effect of the wind on the sides and roof plus any projections above the main roof, with certain allowance being made for the stiffening effect of walls and floors, when stability calculations could be neglected. It also called for an examination of the permeability of the cladding and allowances depending thereon for internal pressure and suction.

These notes, which do not give a fully comprehensive repeat of the wind load requirements contained in the 1948 edition of BS449, are intended to indicate the extensive change in requirements for wind loading which occurred shortly after the end of the wartime relaxations. Though the ISE report was issued before the war commenced in 1939 this was not mandatory whilst the use of BS449 was called for in the by-laws of a large number of local authorities.

5.4.6. C.P.3. Chapter V 1952 etc

This "code of functional requirements of buildings", Chapter V Loading gave very detailed information for the determination of wind loads on structures.

Due to extensive investigation at the Building Research Station, thinking on wind loads constantly altered, and therefore it is not intended giving any detailed explanation.

In 1965 a reset edition of CP3 was published, the title being amended to "code of basic data for the design of buildings". In the introduction to this edition it reported that the recommendations in the Code, and, in particular, those on wind loading were the subject of further investigation.

In the event, CP3 Chapter V was shortly re-issued in two parts viz:—

Part 1 Dead and Imposed Loads (1967)

Part 2 Wind Loads (1970)

Both parts gave information in metric and imperial units.

5.4.7 Code of basic data for the design of buildings CP3 Chapter V Part 2 Wind Loads 1972:

This is a reprint of the 1970 version, with additional information contained therein. The loads and other data apply to any type of building including steel framed hence the current design codes do not give details of loadings but specify the use of CP3 Chapter V to determine these.

Using this code the assessment of the wind loads on a structure can be shown by a series of steps thus:—

1. The basic wind speed V applicable to the district where the structure is to be erected is found from a map or table
2. The design wind speed V_s is found by multiplying V by factors S1, S2 and S3.

$$\text{Hence } V_s = V \times S1 \times S2 \times S3$$

where S1 is topography factor varying between 0.9 and 1.1

S2 is ground roughness, building size and height above ground factor and can vary between 0.47 and 1.27 for a building up to 200m in height

S3 is building life factor based upon a statistical probability of winds of exceptional violence occurring during the life of the building

This is usually taken as 1.0.

3. The design wind speed V_s is converted to dynamic pressure q using the formula

$$q = kV_s^2$$

where k is a constant based on the units being used viz:—

k = 0.613 in S.I. Units (N/m² and m/sec)

or k = 0.0625 in technical metric units (kg/m² and m/sec)

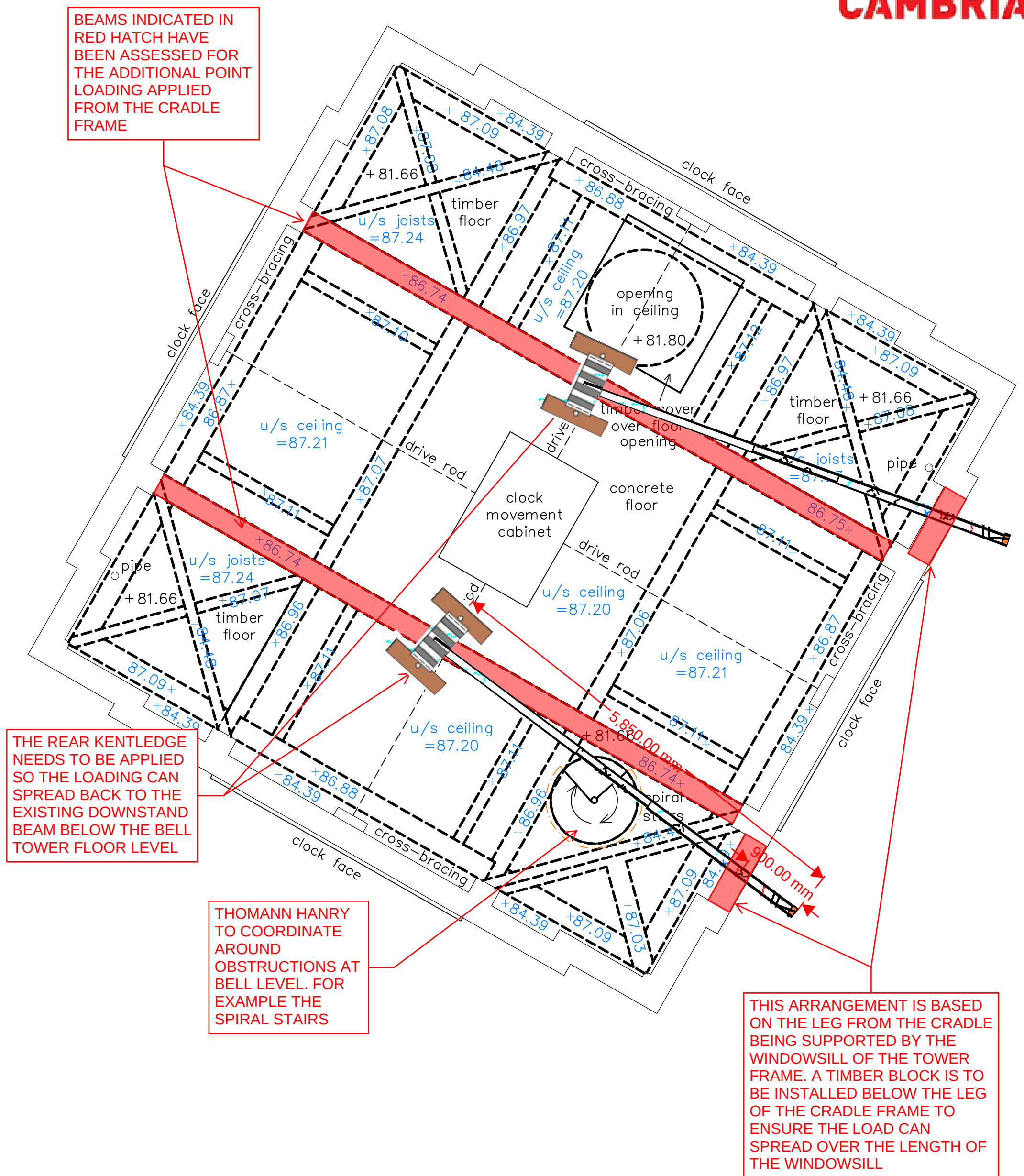
or k = 0.00256 in imperial units. (lb/sq ft and miles/hour)

4. The dynamic pressure q is multiplied by pressure coefficients Cpe and Cpi or by force co-efficients Cf to find the pressure p exerted at any point on the surface of a building.

Numerous examples exist of the application of the provisions of CP3 Chapter V Part 2 Wind loading including Constrado Publication "Workshop with EOT Crane" published in 1977

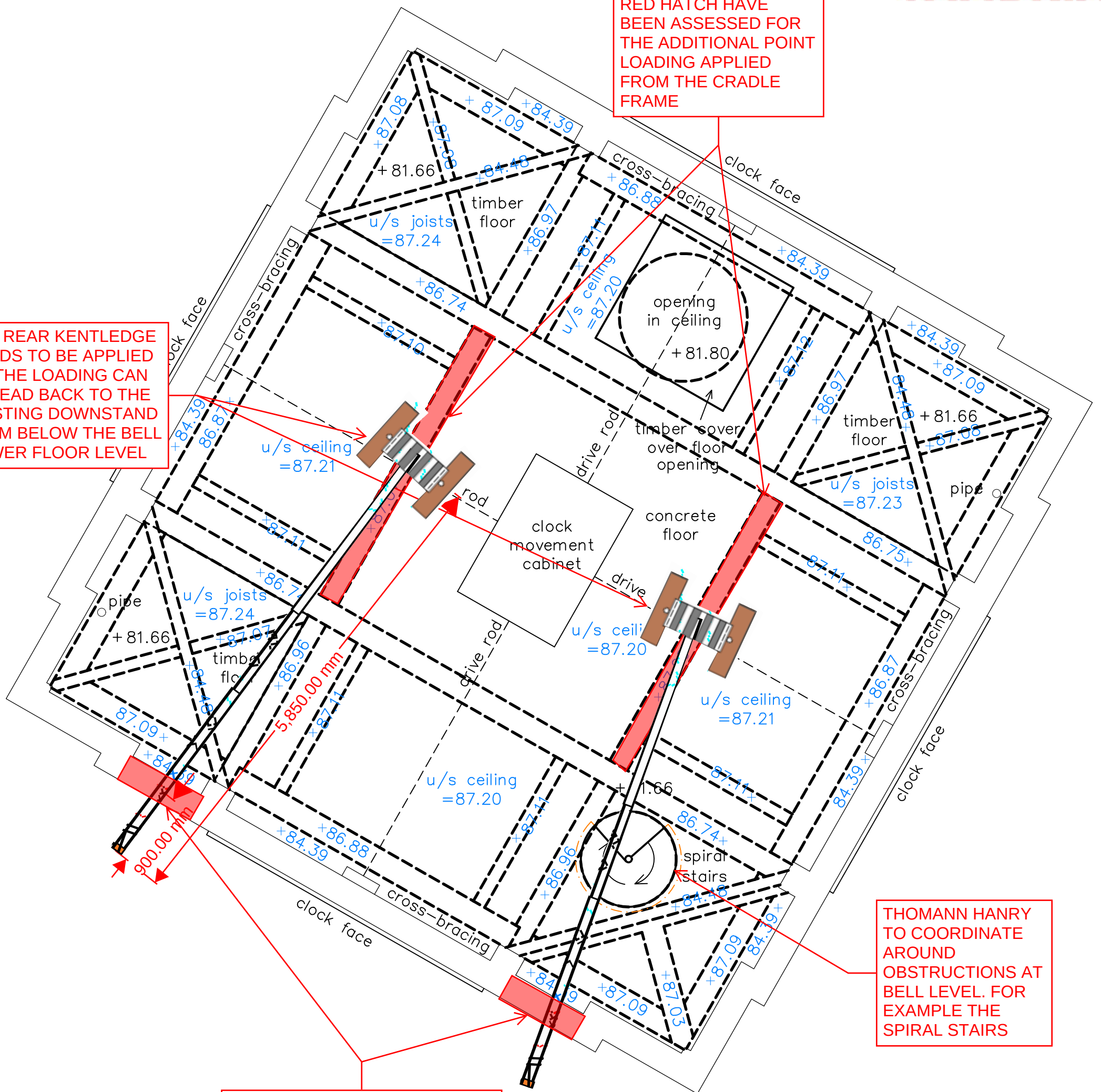
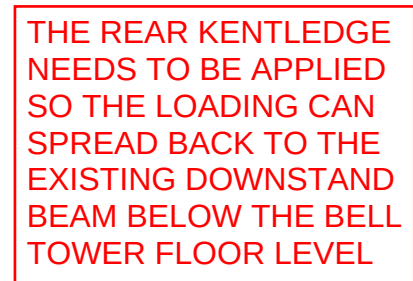
5.5 General

The three basic loadings, ie dead, live and wind have been covered in some depth in this section. There are, of course, many others which need to be considered from time to time such as plant loads, dynamic loads from crane gantries etc which are of too variable a nature to be easily summarised in this publication.



Tower Level 3

Clock Room

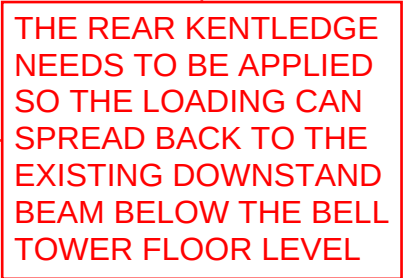


THOMANN HANRY
TO COORDINATE
AROUND
OBSTRUCTIONS AT
BELL LEVEL. FOR
EXAMPLE THE
SPIRAL STAIRS

THIS ARRANGEMENT IS BASED ON THE LEG FROM THE CRADLE BEING SUPPORTED BY THE WINDOWSILL OF THE TOWER FRAME. A TIMBER BLOCK IS TO BE INSTALLED BELOW THE LEG OF THE CRADLE FRAME TO ENSURE THE LOAD CAN SPREAD OVER THE LENGTH OF THE WINDOWSILL

Tower Level 3

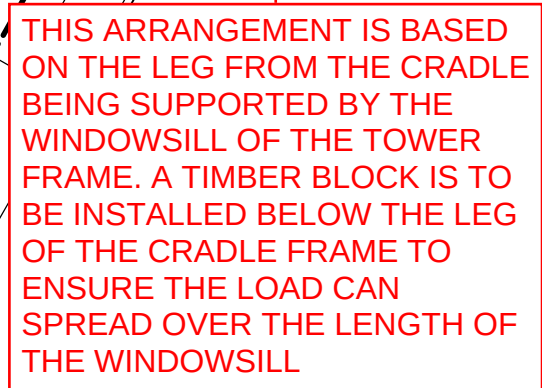
Clock Room



BEAMS INDICATED IN
RED HATCH HAVE
BEEN ASSESSED FOR
THE ADDITIONAL POINT
LOADING APPLIED
FROM THE CRADLE
FRAME

Tower Level 3

Clock Room



BEAMS INDICATED IN
RED HATCH HAVE
BEEN ASSESSED FOR
THE ADDITIONAL POINT
LOADING APPLIED
FROM THE CRADLE
FRAME

Tower Level 3

Clock Room

Appendix B – Cradle Spec Sheet

Newport Civic Centre

Giraffe Access Temporary Suspended Access (TSAE)



Specialists in Suspended Access

Giraffe Access Company Limited
7 Hungate, Pickering, North
Yorkshire, YO18 7DL
t - 01751 475290
m - 07725242913
e - ian@giraffeaccess.com
w - www.giraffeaccess.com

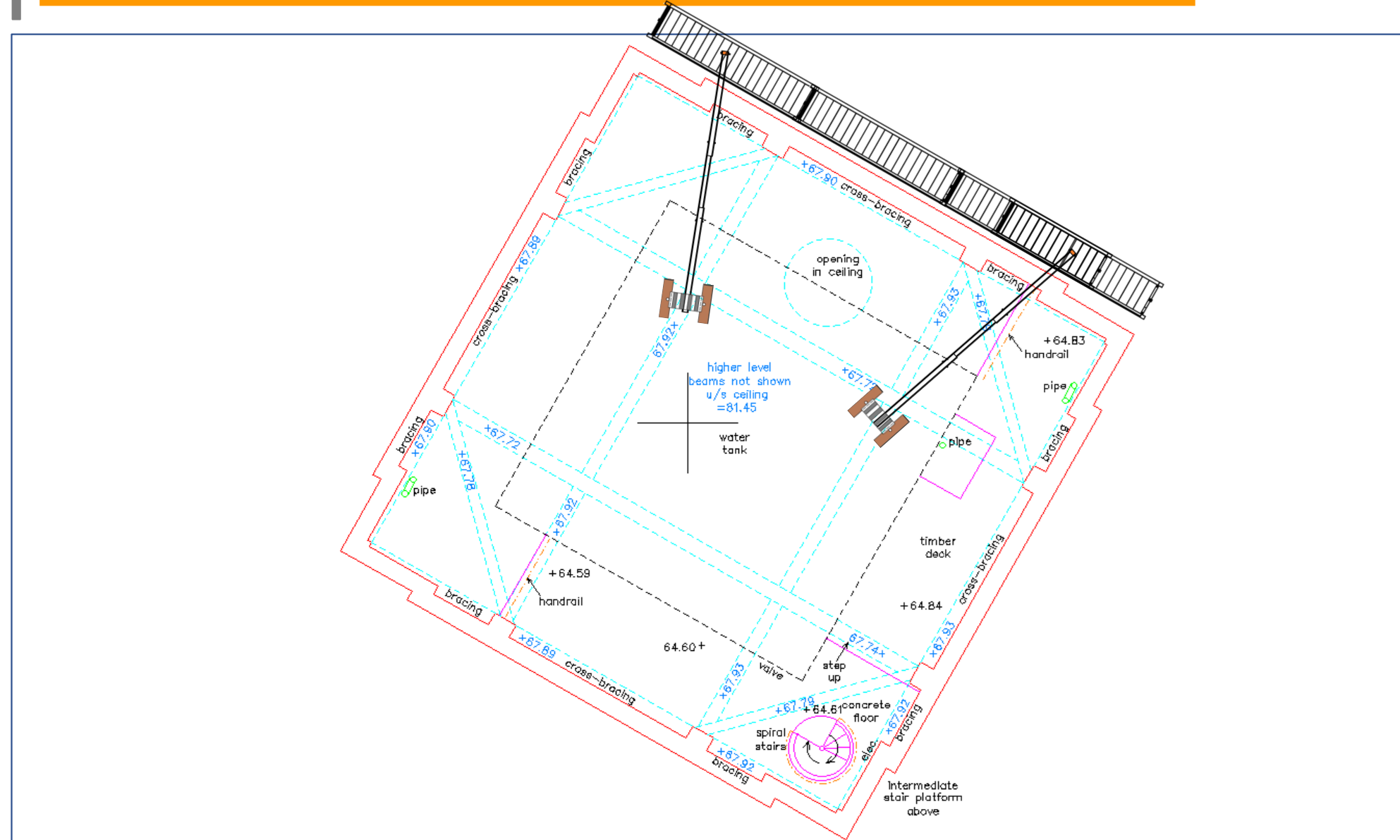


GIRAFFE ACCESS

Notes:



Plan View Cradle Location

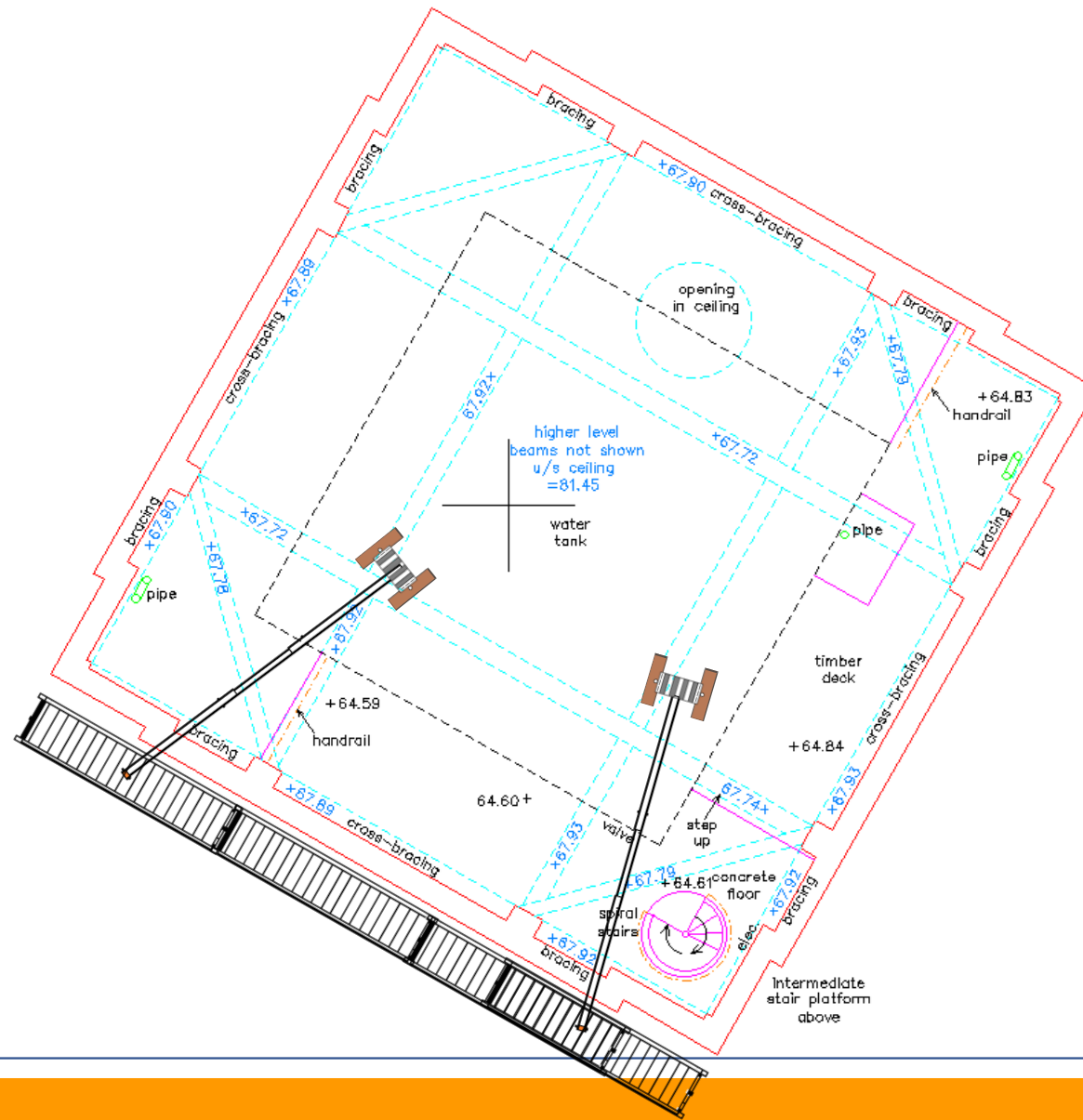


Notes:

Notes:



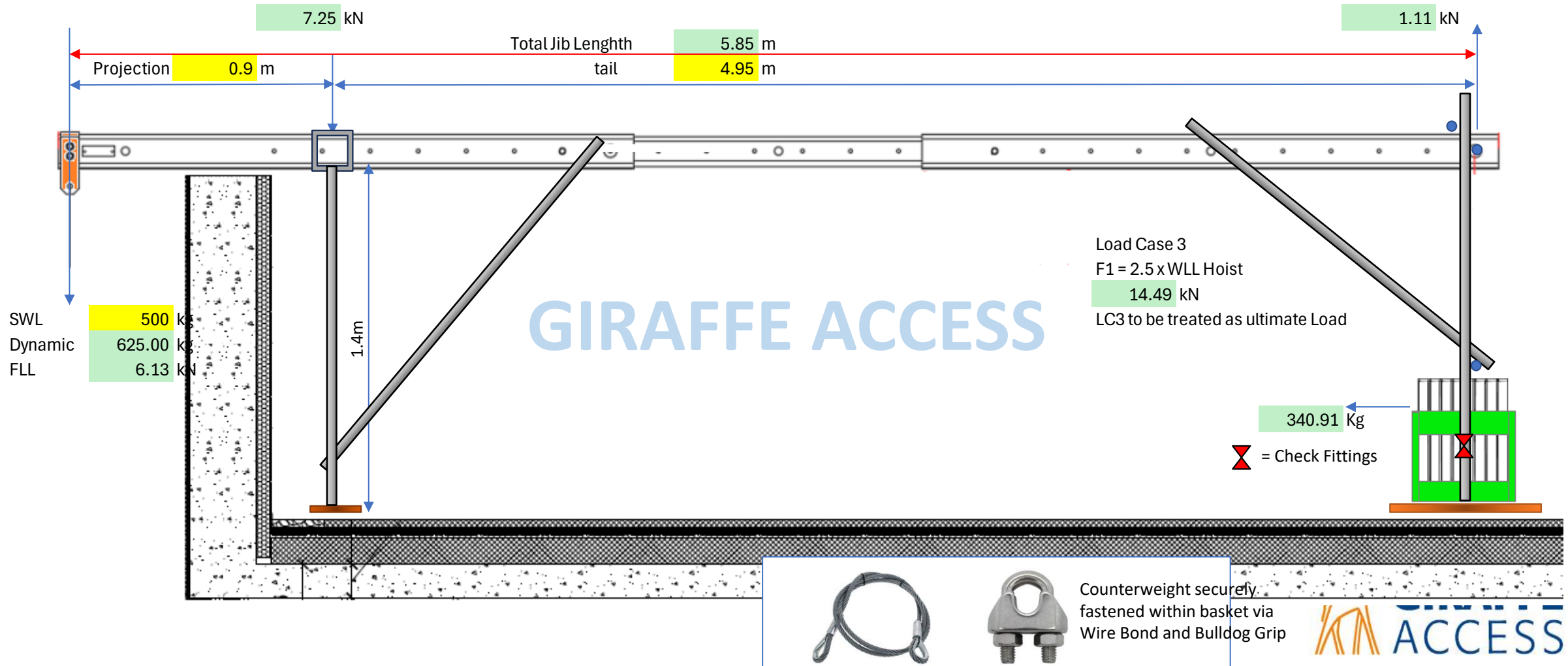
Plan View Cradle Location



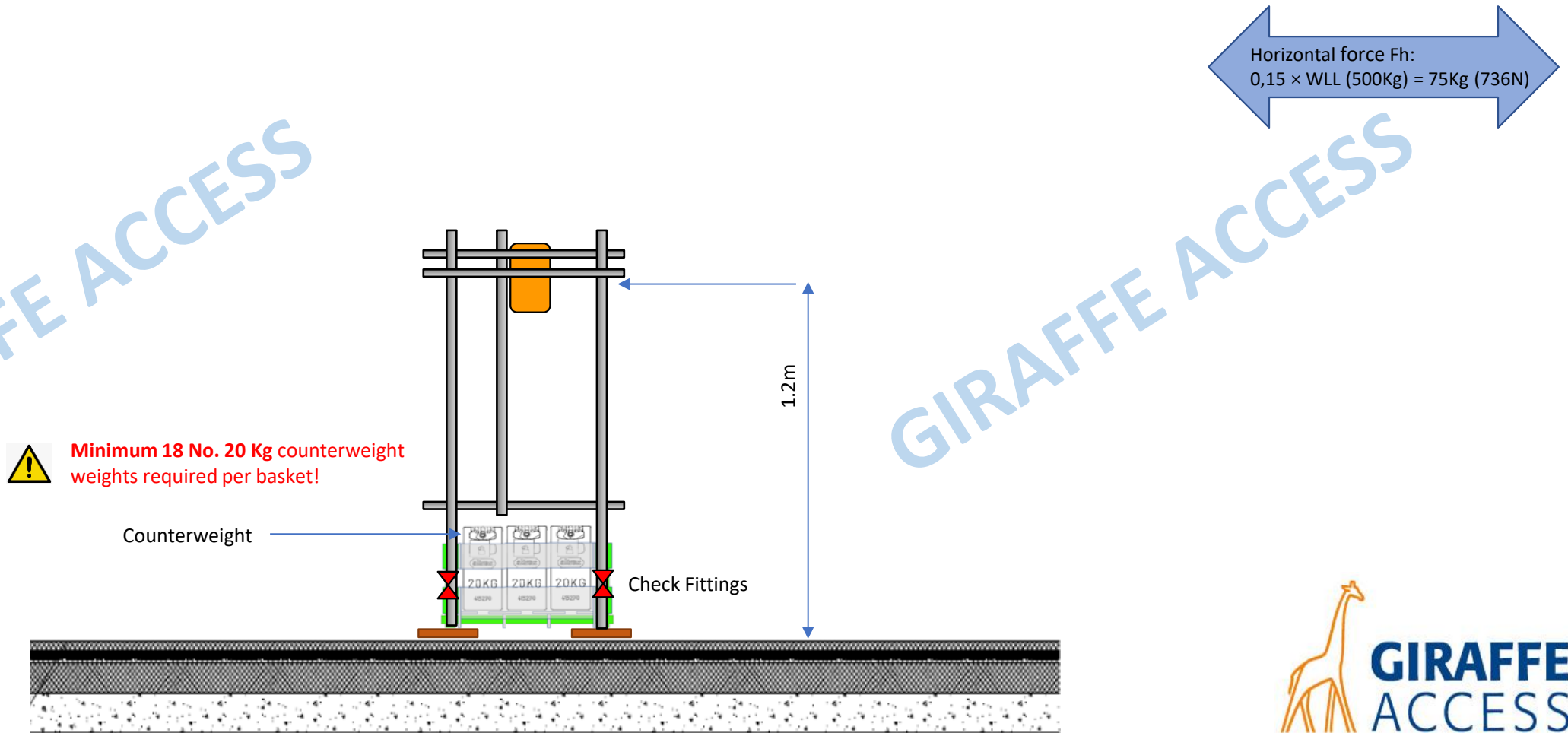
Notes:



ZHS Jib 3 Part Jib Configuration

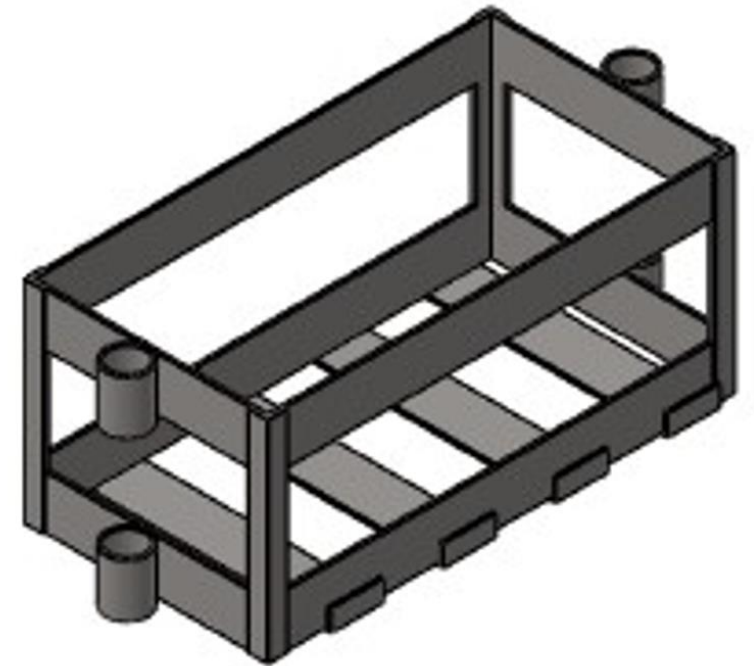
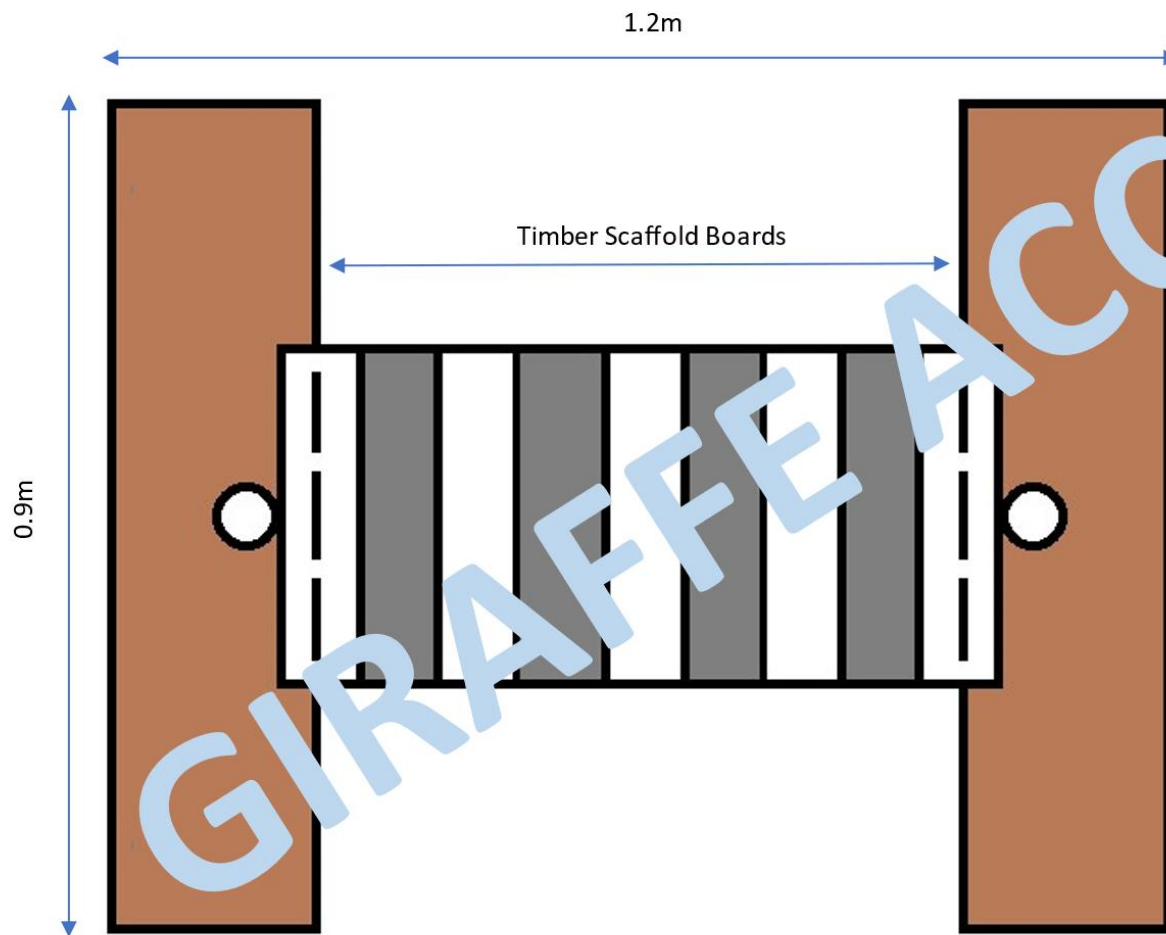


Jib Configuration: Rear View



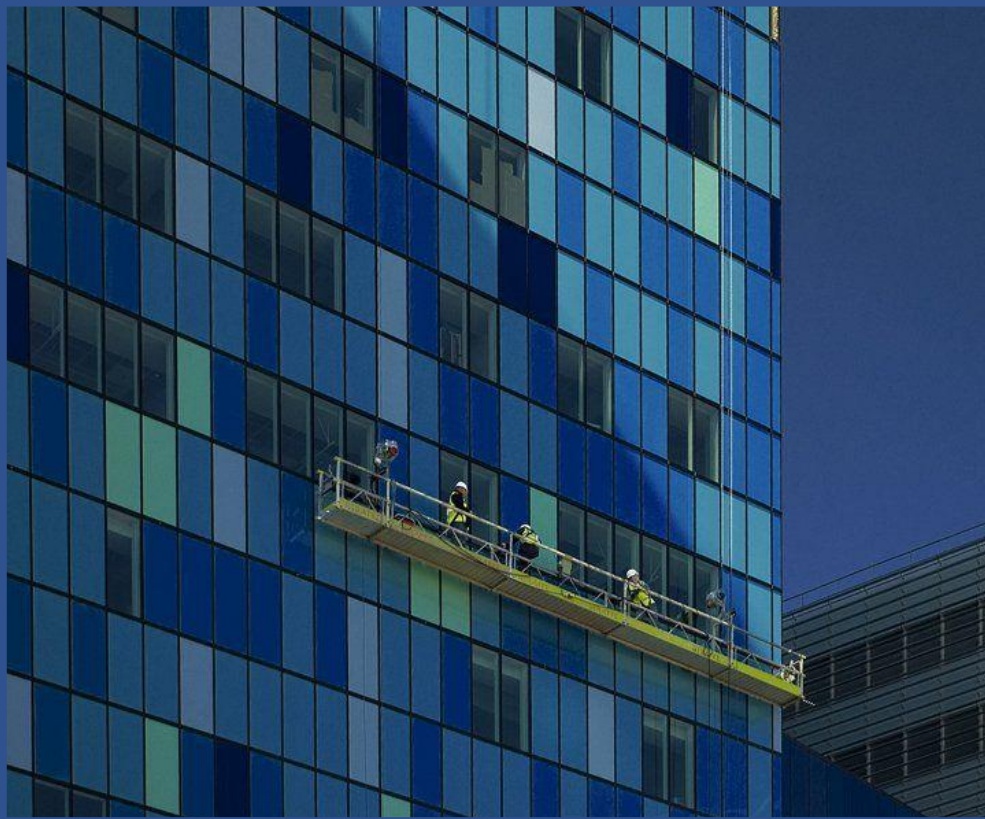
Not to Scale

Counterweight [F2] Load-Spread Detail:





GIRAFFE ACCESS



Specialists in **Suspended Access**