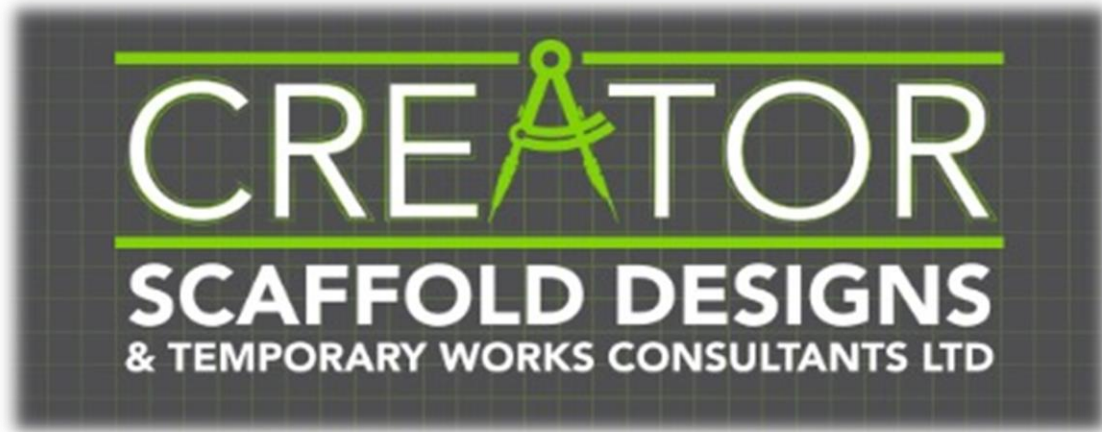




**CREATIVE ENGINEERING WITH COMPLETE CUSTOMER FOCUS
IN MIND**

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CALCULATIONS

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REVISIONS

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-	SLS	03/05/2023	KRB	16/05/2023			FOR CONSTRUCTION
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DOCUMENT SIGNATORIES / CATI CHECK CERTIFICATE

We certify that reasonable professional skill and care has been used in the preparation of this document and in the checking and approval process with a view to securing that:

1. It has been prepared and checked in accordance with the following standards, policies and procedures:

Creator Ltd. Document QM.CRE.2014.02
BS 5975:2019
NASC TG20:21
Full relevant Eurocode suite and other supporting standards

2. All requirements have been accurately translated and incorporated within this document and other associated documents, listed herein, all of which have been checked in accordance with Creator Ltd Quality Management System. The unique reference numbers of the documents which have been checked are:

CRE-IBN-189-001A_CALCS
APPENDIX A - CRE-IBN-189-001A-1,2,3,4_DRG
APPENDIX B - CRE-IBN-189-001A_WIND

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PROJECT STATUS: FOR CONSTRUCTION

GENERAL NOTES - BASIS OF DESIGN

The approach adopted for the analytical analysis of the temporary structure will be the Limit State approach applying partial safety factors for actions and materials in accordance with the Eurocode Suite and BS EN 12811-1. The following partial safety factors will be used in the design;

Partial Safety Factors for Actions, γ_F :

Verification	γ_G (Unfavourable)	γ_G (Favourable)	γ_Q (Unfavourable)	γ_Q (Favourable)
BS EN 12811-1 (10.3.2)	1.50	1.00	1.50	1.00
Ultimate (EN 1990 Eq. 6.10)	1.35	1.00	1.50	1.00
Servicability (EN 1990 Eq. 6.14a/b)	1.00	1.00	1.00	1.00
Accidental (EN 1990 Eq. 6.11a/b)	1.00	1.00	1.00	1.00

Where partial safety factors differ either by requirement of codes or by engineering judgement these will be described and explained within the relevant section.

48.3mm CHS (SCAFFOLD TUBE) CHARACTERISTIC PROPERTIES (STEEL)

The following cross-sectional properties have been calculated for standard 48.3mm CHS steel scaffold tube. These properties will then be applied to enable the ultimate capacities to be calculated for use in design.

Steel Circular Hollow Section Dimensions and Properties

		British Standard Reference	
		BS EN 39 (As New) (Type 3)	BS EN 39 (As New) (Type 4)
Section Designation	d (mm)	48.3	48.3
	t (mm)	3.20	4.00
Mass per Metre (kg/m)		3.56	4.37
Cross-Sectional Area, A (cm ²)		4.53	5.57
Ratio for Local Buckling d/t		15.1	12.1
2 nd Moment of Area, I (cm ⁴)		11.6	13.8
Radius of Gyration, i (cm)		1.60	1.57
Elastic Modulus, W _{el} (cm ³)		4.80	5.70
Plastic Modulus, W _{pl} (cm ³)		6.52	7.87
Modulus of Elasticity, E (N/mm ²)		210000	

BS EN 39 tube of grade S235 steel. Characteristic tensile strength $340\text{N/mm}^2 < R_m < 520\text{N/mm}^2$

All tube is structural HFCHS (hot finished)

For CHS tolerances see BS EN 39

Circular hollow section classification check (EN 1993-1-1);

Class 1 section = $d / t \leq 50\epsilon^2$ where;

$$\epsilon = \sqrt{(235 / f_y)} = 1.000$$

$$50\epsilon^2 = 50$$

Therefore in both cases the scaffold tube satisfies Class 1 section classification

Ultimate bending moment resistance, $M_{pl,Rd} = (W_{pl} \times f_y) / \gamma_{M0}$

		British Standard Reference	
		BS EN 39 (As New) (Type 3)	BS EN 39 (As New) (Type 4)
Yield Strength, f_y (N/mm ²)		355	235
Partial Safety Factor for Material (EN 1993-1-1)		1.00	
Ultimate Bending Resistance, $M_{pl,Rd}$ (kNm)		2.315	1.850

Ultimate shear force resistance, $V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0}$ where;

Shear area, $A_v = 2A / \pi$

		British Standard Reference	
		BS EN 39 (As New) (Type 3)	BS EN 39 (As New) (Type 4)
Shear Area, A_v (cm ²)		2.89	3.54
Yield Strength, f_y (N/mm ²)		355	235
Partial Safety Factor for Material (EN 1993-1-1)		1.00	
Ultimate Shear Force Resistance, $V_{pl,Rd}$ (kN)		59.16	48.08

Any tube that is showing signs of defects and/or is pitted and rusty should not be used. The design herein is based on "as new" with no loss of wall thickness.

As Type 4 tube is most common the following ultimate values of resistance will be used in the design;

$M_{pl,Rd} =$	1.850 kNm	$V_{pl,Rd} =$	48.08 kN
---------------------------------	------------------	---------------------------------	-----------------

48.3mm CHS (SCAFFOLD TUBE) CHARACTERISTIC PROPERTIES (ALLOY)

The following cross-sectional properties have been calculated for standard 48.3mm CHS aluminium scaffold tube. These properties will then be applied to enable the ultimate capacities to be calculated for use in design.

Aluminium Circular Hollow Section Dimensions and Properties

		British Standard Reference
		BS 1139-1.2
Section Designation	d (mm)	48.3
	t (mm)	4.47
Mass per Metre (kg/m)		1.67
Cross-Sectional Area, A (cm ²)		6.16
Ratio for Local Buckling, d/t		10.8
2 nd Moment of Area, I (cm ⁴)		14.9
Radius of Gyration, i (cm)		1.56
Elastic Modulus, W _{el} (cm ³)		6.18
Plastic Modulus, W _{pl} (cm ³)		8.62
Modulus of Elasticity, E (N/mm ²)		70000

BS 1139-1.2 tube of grade 6082-TF alloy. Yield strength, R_e = 255N/mm². Tensile strength, R_m = 295N/mm²

All tube is structural HFCHS (hot finished)

For CHS tolerances see BS 1139-1.2

Circular hollow section classification check (EN 1993-1-1);

Class 1 section = $d / t \leq 50\epsilon^2$ where;

$$\epsilon = \sqrt{(235 / f_y)} = 0.960$$

$$50\epsilon^2 = 46.08$$

Therefore the scaffold tube satisfies Class 1 section classification

Bending moment resistance, $M_{pl,Rd} = (W_{pl} \times f_y) / \gamma_{M0}$

		British Standard Reference
		BS 1139-1.2
Yield Strength, f _y (N/mm ²)		255
Partial Safety Factor for Material (EN 1999-1)		1.10
Ultimate Bending Resistance, $M_{pl,Rd}$ (kNm)		1.998

Ultimate shear force resistance, $V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0}$ where;

Shear area, A_v = 2A / π

		British Standard Reference
		BS 1139-1.2
Shear Area, A _v (cm ²)		3.92
Yield Strength, f _y (N/mm ²)		255
Partial Safety Factor for Material (EN 1999-1)		1.10
Ultimate Shear Force Resistance, $V_{pl,Rd}$ (kN)		52.44

Any tube that is showing signs of defects and/or is pitted and rusty should not be used. The design herein is based on "as new" with no loss of wall thickness.

The following ultimate values of resistance will be used in the design;

$M_{pl,Rd} =$	1.998 kNm	$V_{pl,Rd} =$	52.44 kN
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SCAFFOLD BOARDS AND TIMBER BATTENS

The table below describes the different timber scaffold boards to be used as per BS 2482.

British Standard Reference	Section Designation		Support Centres (mm)	Ultimate Moment Capacity (kNm)
	b (mm)	h (mm)		
BS 2482	225	38	1200	0.77
BS 2482	225	38	1500	1.19
BS 2482	225	63	2500	2.10

Because of the nature of timber and the factors affecting its performance the values of bending strength obtained by testing in accordance with BS 2482 C.2 ought not be used as design values.

As the above values ought not be used as design values we will check the ultimate resistance values using methods described in EN 1995-1-1 for the most commonly used timber scaffold board currently used in industry.

Timber Section Dimensions and Properties:

Section Designation	b (mm)	225.0
	h (mm)	38.00
Cross-Sectional Area, A (cm ²)		85.50
2 nd Moment of Area	I _{yy} (cm ⁴)	102.9
Elastic Section Modulus	W _{yy} (cm ³)	54.15
Radius of Gyration	iy (cm)	1.097

Material Properties of Timber [Table 1 EN 338]

Timber Grade	C24
Bending, f _{m,k} (N/mm ²)	24.0
Shear, f _{v,k} (N/mm ²)	2.50
Mean Modulus of Elasticity Parallel to Grain, E _{0,mean} (kN/mm ²)	11.00

The tabulated properties are compatible with timber at a moisture content consistent with a temperature of 20°C and a relative humidity of 65%

Duration of Load and Service Class [BS EN 1995-1-1]

Load Duration Class [Table 2.1 & 2.2 BS EN 1995-1-1]	Medium Term
Service Class [2.3.1.3 BS EN 1995-1-1]	Service Class 3
Modification Factor, k _{mod} [Table 3.1 BS EN 1995-1-1]	0.65

Lateral Torsional Buckling Check [Table 4.3 of IStructE Manual for the Design of Timber Buildings to Eurocode 5]

Depth, h = 38.00 mm
 Breadth, b = 225.0 mm
 Depth to Breadth ratio = 0.169
 Lateral Support Scenario = None
Maximum Ratio = 2
 Beam not prone to lateral torsional buckling.

Ultimate Bending Moment Resistance

Design Bending Strength, f_{m,d} = k_{mod} × k_h × f_{m,k} / γ_M (2.4.1 BS EN 1995-1-1)

Where;

k_h = lesser of either (150 / h)^{0.2} or 1.3 = 1.3

γ_M = 1.3 for solid timber

Design Bending Strength = 15.60 N/mm²

Ultimate Bending Moment, M_{ult} = f_{m,d} × W_{yy}

Ultimate Bending Moment, M_{ult} = 0.845 kNm

Ultimate Shear Force Resistance

Design Shear Strength, f_{v,d} = k_{mod} × f_{v,k} / γ_M (2.4.1 BS EN 1995-1-1)

Design Shear Strength = 1.250 N/mm²

Ultimate Shear Force, V_{ult} = f_{v,d} × b × h_{ef} / 1.5 re-arranged from;

Shear Stress, τ_d = 1.5 × V / b × h_{ef}

Where;

τ_d = f_{v,d} = 1.250 N/mm²

Ultimate Shear Force, V_{ult} = 7.125 kN

COUPLERS (ULTIMATE VALUES OF RESISTANCE)

The below table includes resistance values obtained from Table 8 of EN 74-1.

Coupler Type	Parameter	Coupler Class	
		A	B
Right Angled Coupler, RA (Double Coupler)	Slipping Force	10.0 kN	15.0 kN
	Rotational Stiffness	-	7.50 kNm/rad
	Cruciform Bending	-	1.60 kNm
Swivel Coupler (SW)	Slipping Force	10.0 kN	15.0 kN
Sleeve Coupler (SF)	Slipping Force	6.00 kN	9.00 kN
	Bending Moment	-	1.40 kNm
Other / Unclassified			
Gravlock/Girder Coupler	Slipping Force	Along the Flange	16.5 kN
		Against the Flange	10.0 kN
		Away from the Flange	49.5 kN

BEAMS (ULTIMATE VALUES OF RESISTANCE)

The below table includes resistance values obtained from the suppliers for different types of beams.

Beam Type	Restraint Arrangement	Ultimate Bending Moment Resistance (kNm)	Ultimate Shear Force Resistance (kN)
Asterix HD Beam	1.0m Top, 2.0m Bottom	168.6	53.8
HAKI 750	2.0m Top, 2.0m Bottom	68.0	50.0

SELF WEIGHTS / PERMANENT ACTIONS

Tube & Fitting Components

Component	Units	Self Weights (SLS)	Factored Self Weights (ULS)
48.3mm x 4.0mm CHS Steel Scaffold Tube	kN/m	0.043	0.064
38mm Timber Scaffold Board	kN/m ²	0.245	0.368
9mm Plywood	kN/m ²	0.053	0.080
Right Angled Coupler	kN	0.012	0.018
Swivel Coupler	kN	0.013	0.0195
Single Coupler	kN	0.010	0.015
Sleeve Coupler	kN	0.010	0.015
Band & Plate Coupler	kN	0.020	0.030
Typical Beam Arrangement (obtained below)	kN/m ²	0.228	0.342
N/A	N/A	0.000	0.000

Note 1; Ultimate bending moment resistance value predominantly used in design =

1.850 kNm

Note 2; Ultimate shear force resistance value predominantly used in design =

48.1 kN

Beam Configuration Self Weight Calculations

Consider; 2.000 m width x 2.000 m length = 4.000 m ² area				
Component	Length (m)	No.	Unit Self Weight (SLS)	Lineweight (kN)
Prefab. Beams	2.000	2.000	0.050 kN/m	0.200
Lacing Tubes	2.000	3.000	0.043 kN/m	0.258
Bracing Tubes	4.000	2.000	0.043 kN/m	0.344
RA Coupler		8.000	0.012 kN	0.096
SW Coupler		1.000	0.013 kN	0.013
Total Lineweight =				0.911
Beam Self Weight = Total Lineweight / Area = 0.23 kN/m ² (Unfactored)				

CONSIDER THE WIND LOADING

Peak Velocity Pressure = 0.671 kN/m² (Unfactored)
Equivalent wind speed = 74 mph

CONSIDER THE SNOW LOADING

Using both BS EN 1991-1-3 and TG9:current we will obtain the design snow load, S_d.

Design snow load, S_d = S_k x μ

Characteristic ground snow load, S_k (kN/m²) = 0.2 + 0.1Z + (A - 100) / 525

Ground Snow Load at 100m, S _{map} ^[1] (kN/m ²)	Z ^[2]	Site altitude, A (m)	S _k (kN/m ²)	Shape Coefficient, μ ^[3]	Design Snow Load, S _d (kN/m ²)
0.5	3	40.00	0.386	0.8	0.309

Note 1; S_{map} is obtained from Figure D.1 of TG9:current or Figure C.9 of BS EN 1991-1-3

Note 2; Z = 10 x S_{map} - 2

Note 3; μ is obtained from Table D.1 of TG9:current or Table 5.2 of BS EN 1991-1-3

The design snow load, S_d is based on an annual probability of exceedence of 0.02 (1 in 50 years).

Due to the temporary nature of the structure we will take a reduction on this by using expression D.1 from BS EN 1991-1-3 where S_n is the ground snow load with a probability of exceedence of n years.

$$S_n = S_k \left\{ \frac{1 - V \frac{\sqrt{6}}{\pi} [\ln(-\ln(1 - P_n)) + 0.57722]}{(1 + 2.5923V)} \right\}$$

The value of S_n will also be multiplied by the shape coefficient stated above.

S _k (kN/m ²)	Coefficient of Variation, V ^[1]	Return Period, n (years)	Annual Probability of Exceedence, P _n	μ	S _n (kN/m ²)
0.386	0.8	10	0.1	0.8	0.205

Note 1; The coefficient of variation stated above is the average for 18 european countries.

SUMMARY OF ACTIONS

Design Service Loads	Units	SLS	ULS	No. of Levels Loaded
Design Service Load 1;	kN/m ²	2.000	3.000	1.000
Design Service Load 2;	kN/m ²	1.000	1.500	1.000
Inside Board Design Service Load;	kN/m ²	0.750	1.125	2.000
Point Load to Deck;	kN	0.000	0.000	
Fan Design Service Load;	kN/m ²	0.000	0.000	0.000
Other				
Wind Loading;	kN/m ²	0.671	1.007	
Working Wind Loading ^[1] ;	kN/m ²	0.200	0.300	
Snow Loading;	kN/m ²	0.205	0.308	
Horizontal Force in Absence of Wind ^[2] ;	kN	0.300	0.450	
Horizontal Handrail Force ^[2] ;	kN	0.300	0.450	
N/A	N/A	0.000	0.000	0.000

Note 1; Value obtained from part 6.2.7.4.2 of BS EN 12811

Note 2 ; Value obtained from part 6.2.5.2 of BS EN 12811

Accidental Loads	Units	SLS	ULS	No. of Levels Loaded
Vertical Guardrail Load ^[1]	kN	1.250	1.250	
N/A	N/A	0.000	0.000	0.000

Note 1; Value obtained from part 6.2.5.1 of BS EN 12811

CONSIDER THE BOARDS AS DECKING MEMBERS

Consider the Worst Case Loading;

Load Type	Units	SLS	ULS	
Permanent Actions;				
38mm Timber Scaffold Board	kN/m ²	0.245	0.368	SINGLE BOARDS NO COVER
9mm Plywood	kN/m ²	0.000	0.000	
Variable Actions;				
Maximum Design Load	kN/m ²	2.000	3.000	
Point Load to Deck	kN	0.000	0.000	
F ₁ (500mm Along the Length) ^[1]	kN/m	3.000	4.500	1.5kN Total
F ₂ (200mm x 200mm) ^[2]	kN/m	5.000	7.500	1.0kN Total

[1] BS EN 12811-1 Clause 6.2.2 specifies a load to act over a 500mm x 500mm area. Scaffold boards are less than 500mm wide so we have assumed an equivalent UDL acting over 500mm giving the total value specified.

[2] BS EN 12811-1 Clause 6.2.2 specifies a load to act over a 200mm x 200mm area. As scaffold boards are very nearly 200mm wide in total we have assumed an equivalent UDL acting over 200mm giving the total value specified.

BS EN 12811-1 Clause 6.2.2 only gives guidance up to load class 6 (6.0kN/sq.m). For any load greater than load class 6 we will adopt the maximum from the guidance.

Consider the Ultimate Checks;

Design Load;

Maximum moment =
$$\frac{wL^2}{8} + \frac{PL}{4}$$

Single board width = 0.225 m

Total uniformly distributed load, w = 0.758 kN/m (Factored)

Point load to deck, P = 0.000 kN (Factored)

Distance between supports, L = 1.200 m

Maximum moment =
$$\frac{0.758 \times 1.44}{8} + \frac{0.000 \times 1.200}{4}$$

Maximum moment = 0.136 kNm

PASS - Calculated bending moment is less than the ultimate bending moment capacity.

Maximum shear force =
$$\frac{wL}{2} + P$$

Total uniformly distributed load, w = 0.758 kN/m (Factored)

Point load to deck, P = 0.000 kN (Factored)

Distance between supports, L = 1.200 m

Maximum shear force =
$$\frac{0.758 \times 1.200}{2} + 0.000$$

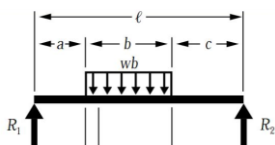
Maximum shear force = 0.455 kN

PASS - Calculated shear force is less than ultimate shear force capacity.

F₁:

Bending moment due to self weight only = 0.066 kNm

We will now assume that, for the worst case bending moment, the UDL will be central to the span.



$$R_1 = V_1 \text{ (max when } a < c) \dots = \frac{wb}{2l}(2c + b)$$

$$M_{\max} \left(\text{at } x = a + \frac{R_1}{w} \right) \dots = R_1 \left(a + \frac{R_1}{2w} \right)$$

Reaction = 1.125 kN

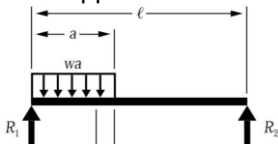
Maximum moment = 0.534 kNm

Maximum moment = 0.601 kNm

PASS - Calculated bending moment is less than the ultimate bending moment capacity.

Shear force due to self weight only = 0.221 kN

We will now assume that, for the worst case shear force, the UDL will act immediately adjacent to the support.



$$R_1 = V_1 \dots \dots \dots = \frac{wa}{2l}(2l - a)$$

Shear force = 2.969 kN

Maximum shear force = 3.189 kN

PASS - Calculated shear force is less than ultimate shear force capacity.

F₂:

Bending moment due to self weight only = 0.066 kNm

We will now assume that, for the worst case bending moment, the UDL will be central to the span.

Reaction = 0.75 kN

Maximum moment = 0.413 kNm

Maximum moment = 0.479 kNm

PASS - Calculated bending moment is less than the ultimate bending moment capacity.

Shear force due to self weight only = 0.221 kN

We will now assume that, for the worst case shear force, the UDL will act immediately adjacent to the support.

Shear force = 1.375 kN

Maximum shear force = 1.596 kN

PASS - Calculated shear force is less than ultimate shear force capacity.

Consider the Serviceability Checks:

$$\text{Maximum deflection, } \delta \text{ (mm)} = \frac{5wL^4}{384EI} + \frac{PL^3}{48EI}$$

Single board width =	225	mm
Total uniformly distributed load, w =	4.5E-04	kN/mm (Unfactored) (neglecting self weights of timber)
Point load to deck, P =	0.000	kN (Unfactored)
Distance between supports, L =	1200	mm
Modulus of elasticity, E =	11.000	kN/mm ²
Second moment of area, I =	1.03E+06	mm ⁴
Deflection, δ =	5	x
	384	x
		4.5E-04 x 2.07E+12
		11.000 x 1.03E+06
		+
	0.000	x
	48	x
		11.000 x 1.73E+09
		1.03E+06
Deflection, δ =		1.074 mm
Deflection limit =	Span	= 12.000 mm
	100	

PASS - Calculated maximum deflection is less than specified deflection limit.

CONSIDER THE BOARD TRANSOMS

Consider the Worst Case Loading;

Load Type	Units	SLS	ULS	
Permanent Actions;				
38mm Timber Scaffold Board	kN/m ²	0.245	0.368	SINGLE BOARDS
9mm Plywood	kN/m ²	0.000	0.000	NO COVER
Variable Actions;				
Maximum Design Load	kN/m ²	2.000	3.000	
Point Load to Deck	kN	0.000	0.000	

Consider the Ultimate Checks;

Maximum moment =

	wL^2	+	PL	
	8		4	
Transom centres =	1.200	m		
Total uniformly distributed load, w =	4.041	kN/m (Factored)		
Point load to deck, P =	0.000	kN (Factored)		
Width of scaffold, L =	1.200	m		
Number of transoms at node =	1.000	no. (Resistance values are multiplied by the number)		
Maximum moment =	4.041	x	1.44	+
	8			0.000
				x
				1.200
				4

Maximum moment = 0.727 kNm

PASS - Calculated bending moment is less than the ultimate bending moment capacity.

Maximum shear force =

	$\frac{wL}{2}$	+	P	
	2			
Total uniformly distributed load, w =	4.041	kN/m (Factored)		
Point load to deck, P =	0.000	kN (Factored)		
Width of scaffold, L =	1.200	m		
Maximum shear force =	4.041	x	1.200	+
				0.000
				2

Maximum shear force = 2.425 kN

PASS - Calculated shear force is less than ultimate shear force capacity.

Consider the Serviceability Checks;

Maximum deflection, δ (mm) =	$\frac{5wL^4}{384EI}$	+	$\frac{PL^3}{48EI}$	
Transom centres =	1200	mm		
Total uniformly distributed load, w =	2.69E-03	kN/mm (Unfactored)		
Point load to deck, P =	0.000	kN (Unfactored)		
Width of scaffold, L =	1200	mm		
Modulus of Elasticity, E =	210	kN/mm ²		
Second moment of area, I =	1.38E+05	mm ⁴		
Deflection, δ =	5	x	2.69E-03	x
	384	x	210	x
				2.07E+12
				1.38E+05
				+
	0.000	x	1.73E+09	
	48	x	210	x
				1.38E+05
Deflection, δ =				2.516 mm (Deflection divided by number of transoms)
Deflection limit =	Span	=	12.000 mm	
	100			

PASS - Calculated maximum deflection is less than specified deflection limit.

CONSIDER THE LEDGERS

Consider the Worst Case Loading;

Load Type	Units	SLS	ULS	
Permanent Actions;				
38mm Timber Scaffold Board	kN/m ²	0.245	0.368	SINGLE BOARDS
9mm Plywood	kN/m ²	0.000	0.000	NO COVER
Variable Actions;				
Maximum Main Board Design Load	kN/m ²	2.000	3.000	
Maximum Inside Board Design Load	kN/m ²	0.750	1.125	
Point Load to Deck	kN	0.000	0.000	

Consider the Ultimate Checks;

Shear forces will be neglected from this section as the bending moment will always be the critical strength property with regards to the ledgers.

Maximum moment due to point load =

PL

4

Scaffold arrangement =

SINGLE WIDTH

Number of Inside Boards =

2.000 No.

N/A

Main Boards

Inside Boards

A	0.000	B	1.200	C	0.45
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Reaction at node (kN) = Transom Centres (m) x Width over Node (m) x Actions over Node (kN/m²)

Reaction at A = 1.200 x 0.000 x 3.368

Reaction at A = 0.000 kN (Factored)

Reaction at B = 1.200 x 0.600 x 3.368

Reaction at B = 2.425 kN (Factored)

Reaction at C = 1.200 x 0.600 x 3.368

1.200 x 0.45 x 1.493

Reaction at C = 3.231 kN (Factored)

Maximum reaction (kN) = Maximum Reaction at Node (kN) + Point Load to Deck (kN)

Maximum reaction = 3.231 + 0.000 = 3.231 kN (Factored)

We will now re-arrange the bending moment equation in terms of L to find the maximum span.

$$1.850 = \frac{3.231 L}{4} \quad \text{Therefore; } L_{\max} = 2.290 \text{ m}$$

$$L_{\text{design}} = 2.200 \text{ m}$$

PASS - Intended ledger span is less than maximum possible calculated ledger span

Consider the Connection Properties;

The maximum load concentrated at the connection is the maximum value at point B or C dependent upon the scaffold arrangement.

Maximum width of main boards over connection = 0.600 m

Maximum width of inside boards over connection = 0.45 m

Total load at the connection = 4.445 + 1.478 + 0.000

Total load at the connection = 5.923 kN (Factored)

PASS - Total load is less than the slipping capacity of an EN74 Class A double coupler.

CONSIDER THE GUARDRAILS

SOLIDLY CLAD

Case 1 - Fall Prevention and Working Wind;

Load Type	Units	SLS	ULS
Permanent Actions;			
48.3mm x 4.0mm CHS Steel Scaffold Tube	kN/m	0.043	0.064
Variable Actions;			
Working Wind Loading	kN/m ²	0.200	0.300
Horizontal Handrail Force	kN	0.300	0.450

Shear forces will be neglected from this section as the bending moment will always be the critical strength property with regards to the guardrails.

Maximum moment =

$$\frac{wL^2}{8} + \frac{PL}{4}$$

Uniformly distributed load, w = Working wind load (kN/m²) x Pressure coefficient x Containment x Width over guardrail (m)

Pressure coefficient =

1.200

Containment =

1.000

Width over guardrail =

1.0000 m

$$1.850 = 0.053 L^2 + 0.1125 L$$

Using the quadratic equation we will solve to find the maximum possible guardrail span;

$$\text{if; } x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

-0.113	+	0.636	=	4.940 m
		0.106		
-0.1125	+	0.636	=	7.061 m
		0.106		

Maximum guardrail span for fall prevention and working wind = 4.940 m

Case 2 - Full Wind;

Load Type	Units	SLS	ULS
Permanent Actions;			
48.3mm x 4.0mm CHS Steel Scaffold Tube	kN/m	0.043	0.064
Variable Actions;			
Full Wind Loading	kN/m ²	0.671	1.007

Maximum moment =

$$\frac{wL^2}{8}$$

Uniformly distributed load, w = Full wind load (kN/m²) x Pressure coefficient x Containment x Width over guardrail (m) where values have been defined above.

$$1.850 = 0.159 L^2$$

Maximum guardrail span for full wind loading = 3.411 m

Case 3 - Accidental Vertical Action;

Load Type	Units	SLS	ULS
Accidental Actions;			
Vertical Guardrail Load	kN	1.250	1.250

Only accidental loads will be adopted in this design consideration as the structure is not deemed as serviceable after an accidental event. A full inspection of the area is to be carried out and any amendments required are to be made prior to returning to a serviceable state.

Maximum moment =

$$\frac{PL}{4}$$

$$1.850 = 0.313 L$$

Maximum guardrail span for accidental vertical actions = 5.919 m

Case 4 - Maximum Allowable Deflection;

The deflection limits for an edge protection system will only be relevant during the 'In Service' phase therefore the following serviceable (SLS) loads apply;

Load Type	Units	SLS	ULS
Variable Actions;			
Working Wind Loading	kN/m ²	0.200	0.300
Horizontal Handrail Force	kN	0.300	0.450

Permanent actions have been neglected from this consideration.

$$\text{Maximum deflection, } \delta \text{ (mm)} = \frac{PL^3}{48EI}$$

Point load, P = ((Working wind load (kN/m²) x Pressure coefficient x Containment x Width over guardrail (m)) x Maximum handrail span (m)) + Horizontal handrail force (kN) where values have been defined above.

Pressure coefficient =	1.200
Containment =	1.000
Width over guardrail =	1.0000 m
Minimum guardrail length =	3.411 m
Point load, P =	1.119 kN (Unfactored)
Modulus of Elasticity, E =	210 kN/mm ²
Second moment of area, I =	1.16E+05 mm ⁴
Maximum allowable deflection =	35 mm
35 =	9.58E-10 L ³

Maximum guardrail span when 'In Service' =	3.318 m
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Maximum allowable guardrail span for use in the design =	3.318 m
Actual guardrail span used in the design =	2.200 m

PASS - Actual guardrail span is less than maximum allowable guardrail span

CONSIDER THE UPRIGHTS

When considering the uprights the following combinations apply;

- In service ultimate axial compression = Permanent action @ ULS + Variable action @ ULS
- Out of service ultimate axial compression = Permanent action @ ULS + Variable action @ SLS
- In service actual axial compression = Permanent action @ SLS + Variable action @ SLS
- Out of service actual axial compression = Permanent action @ SLS + Variable action @ SLS

INSIDE UPRIGHT

Inside Boards? **2.000** no.

Tube Self Weight per Node;

Number	Tube	Length (m)	Total Length (m)	Self Weight (kN/m)	Total Self Weight (kN)
1.000	Upright	2.000	2.000	0.0429	0.086
1.000	Ledger	2.000	2.000	0.0429	0.086
1.000	Transom	1.050	1.050	0.0429	0.045
3.000	Board Transom	1.050	3.150	0.0429	0.135
1.000	Ledger Brace	1.166	1.166	0.0429	0.050
1.000	Façade Brace	1.414	1.414	0.0429	0.061
0.000	Handrails	2.000	0.000	0.0429	0.000

Coupler Self Weight per Node;

Number	Coupler	Self Weight (kN)	Total Self Weight (kN)
2.000	Right Angled Coupler	0.012	0.024
2.000	Swivel Coupler	0.013	0.026
3.000	Single Coupler	0.010	0.030
1.000	Sleeve Coupler	0.010	0.010

Unfactored tube and coupler self weight per node =

0.552 kN (Unfactored)

Board Self Weight per Node;

Board self weight = 0.245 kN/m² (Unfactored)
 Length of boards = 2.000 m
 Number of boards = **4.500** no.
 Single board width = 0.225 m
 Area over node = 2.025 m²
 Board self weight per node = 0.496 kN (Unfactored)

Variable Action per Node - Main Boards;

Length over node = 2.000 m
 Width over node = **0.600** m
 Area over node = 1.200 m²

	Unfactored Area Load (kN/m ²)	Total Load per Node (kN)
Design Service Load [1]	2.000	2.400
Design Service Load [2]	1.000	1.200

Variable Action per Node - Inside Boards;

Length over node = 2.000 m
 Number of inside boards over node = **2.000** no.
 Single board width = 0.225 m
 Area over node = 0.900 m²

	Unfactored Area Load (kN/m ²)	Total Load per Node (kN)
Inside Board Design Service Load	0.750	0.675

Load due to bridge span;

0.00 kN (Unfactored)

Unfactored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
10.000	Tube and Coupler Self Weight	0.552	5.522
7.000	Board Self Weight	0.496	3.473
1.000	Design Service Load [1]	2.400	2.400
1.000	Design Service Load [2]	1.200	1.200
2.000	Inside Board Design Service Load	0.675	1.350
0.000	Load due to bridge span;	0.000	0.000

Unfactored In Service Legload = 13.944 kN

Unfactored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
10.000	Tube and Coupler Self Weight	0.552	5.522
7.000	Board Self Weight	0.496	3.473
0.000	Load due to bridge span;	0.000	0.000

Unfactored Out of Service Legload = 8.994 kN

Factored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
10.000	Tube and Coupler Self Weight	0.828	8.282
7.000	Board Self Weight	0.744	5.209
1.000	Design Service Load [1]	3.600	3.600
1.000	Design Service Load [2]	1.800	1.800
2.000	Inside Board Design Service Load	1.013	2.025
0.000	Load due to bridge span;	0.000	0.000

Factored In Service Legload = 20.917 kN

Factored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
10.000	Tube and Coupler Self Weight	0.828	8.282
7.000	Board Self Weight	0.744	5.209
0.000	Load due to bridge span;	0.000	0.000

Factored Out of Service Legload = 13.492 kN

Summary;

Unfactored In Service Legload = 13.944 kN
Factored In Service Legload = 20.917 kN
Unfactored Out of Service Legload = 8.994 kN
Factored Out of Service Legload = 13.492 kN

Effective Lengths;

Effective length, L_e = Open length of upright (m) x Effective length coefficient where;

Open length of upright = 2.000 m
Effective length coefficient = 0.80
Effective length, L_e = 1.600 m

Compressive Resistance;

We will obtain this value using values described herein and BS EN 1993-1-1 Clause 6.3.1.2.

$\lambda = L_e / i = 101.74$
 $\lambda = \lambda / ((\pi^2 \times E) / f_y)^{0.5} = 1.08$
Factor for the buckling curve, $\alpha = 0.49$
 $\phi = 0.5 (1 + \alpha(\lambda - 0.2) + \lambda^2) = 1.303$

COLD FORMED

$$X = 1 / (\phi + (\phi^2 - \lambda^2)^{0.5}) = 0.493$$

Design Buckling Resistance, $N_{Rd} = X \times f_y \times A$ where;

Yield strength, $f_y = 235 \text{ N/mm}^2$

Cross-sectional area, $A = 557 \text{ mm}^2$

Design Buckling Resistance = 64.518 kN

Bending Moments in Standard When In Service at Ultimate Limit State:

Case 1; Maximum compression in standard x % out of plumb = Moment due to compression

Moment = 20.917 x 1.0% = 0.209 kNm

Case 2; ((Working wind pressure x Width over upright) x Open length of upright²) / 10 = Moment due to wind (10 is used to account for continuity in the upright)

Working wind pressure = 0.300 kN/m² (Factored)

Pressure coefficient = 1.2 CPe

Solidity ratio = 1.0

Tube diameter = 0.0483 m

UDL up the face = 0.017 kN/m (Factored)

Open length of upright = 2.000 m

Moment = $\frac{0.017 \times 4.000}{10} = 0.007 \text{ kNm}$

Case 3; (Horizontal handrail load x Open length of upright) / 6 = Moment due to fall arrest (6 is used to account for continuity in the upright)

Horizontal handrail load = 0.450 kN (Factored)

Open length of upright = 2.000 m

Moment = $\frac{0.450 \times 2.000}{6} = 0.15 \text{ kNm}$

Total moment in upright = 0.366 kNm

Critical Interaction Check When In Service at Ultimate Limit State:

$1.0 > (\text{Calculated moment} / \text{Moment capacity}) + (\text{Calculated compression} / \text{Buckling resistance})$

Critical interaction = $\frac{0.366}{1.850} + \frac{20.917}{64.518} = 0.522$

PASS - Upright passes buckling checks made at Ultimate Limit State

Bending Moments in Standard When Out of Service at Ultimate Limit State:

Case 1; Maximum compression in standard x % out of plumb = Moment due to compression

Moment = 13.492 x 1.0% = 0.135 kNm

Case 2; ((Full wind pressure x Width over upright) x Open length of upright²) / 10 = Moment due to wind (10 is used to account for continuity in the upright)

Full wind pressure = 1.007 kN/m² (Factored)

Pressure coefficient = 1.2 CPe

Solidity ratio = 1.0

Tube diameter = 0.0483 m

UDL up the face = 0.058 kN/m (Factored)

Open length of upright = 2.000 m

Moment = $\frac{0.058 \times 4.000}{10} = 0.023 \text{ kNm}$

Total moment in upright = 0.158 kNm

Critical Interaction Check When In Service at Ultimate Limit State:

$1.0 > (\text{Calculated moment} / \text{Moment capacity}) + (\text{Calculated compression} / \text{Buckling resistance})$

Critical interaction = $\frac{0.158}{1.850} + \frac{13.492}{64.518} = 0.295$

PASS - Upright passes buckling checks made at Ultimate Limit State

OUTSIDE UPRIGHT

Tube Self Weight per Node;

Number	Tube	Length (m)	Total Length (m)	Self Weight (kN/m)	Total Self Weight (kN)
1.000	Upright	2.000	2.000	0.0429	0.086
1.000	Ledger	2.000	2.000	0.0429	0.086
1.000	Transom	0.600	0.600	0.0429	0.026
3.000	Board Transom	0.600	1.800	0.0429	0.077
1.000	Ledger Brace	1.166	1.166	0.0429	0.050
1.000	Façade Brace	1.414	1.414	0.0429	0.061
2.000	Handrails	2.000	4.000	0.0429	0.171

Coupler Self Weight per Node;

Number	Coupler	Self Weight (kN)	Total Self Weight (kN)
4.000	Right Angled Coupler	0.012	0.048
2.000	Swivel Coupler	0.013	0.026
3.000	Single Coupler	0.010	0.030
1.000	Sleeve Coupler	0.010	0.010

Unfactored tube and coupler self weight per node =

0.670 kN (Unfactored)

Board Self Weight per Node;

Board self weight = 0.245 kN/m² (Unfactored)
 Length of boards = 2.000 m
 Number of boards = 3.500 no. (including toeboard)
 Single board width = 0.225 m
 Area over node = 1.575 m²
 Board self weight per node = 0.386 kN (Unfactored)

Variable Action per Node - Main Boards;

Length over node = 2.000 m
 Width over node = 0.600 m
 Area over node = 1.200 m²

	Unfactored Area Load (kN/m ²)	Total Load per Node (kN)
Design Service Load [1]	2.000	2.400
Design Service Load [2]	1.000	1.200

Load due to bridge span;

0.00 kN (Unfactored)

Unfactored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
10.000	Tube and Coupler Self Weight	0.670	6.705
7.000	Board Self Weight	0.386	2.701
1.000	Design Service Load [1]	2.400	2.400
1.000	Design Service Load [2]	1.200	1.200
1.000	Load due to bridge span;	0.000	0.000

Unfactored In Service Legload = 13.006 kN

Unfactored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
10.000	Tube and Coupler Self Weight	0.670	6.705
7.000	Board Self Weight	0.386	2.701
0.000	Load due to bridge span;	0.000	0.000

Unfactored Out of Service Legload = 9.406 kN

Factored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
10.000	Tube and Coupler Self Weight	1.006	10.057
7.000	Board Self Weight	0.579	4.052
1.000	Design Service Load [1]	3.600	3.600
1.000	Design Service Load [2]	1.800	1.800
1.000	Load due to bridge span;	0.000	0.000

Factored In Service Legload = 19.509 kN

Factored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
10.000	Tube and Coupler Self Weight	1.006	10.057
7.000	Board Self Weight	0.579	4.052
0.000	Load due to bridge span;	0.000	0.000

Unfactored Out of Service Legload = 14.109 kN

Summary:

Unfactored In Service Legload = 13.006 kN
Factored In Service Legload = 19.509 kN
Unfactored Out of Service Legload = 9.406 kN
Factored Out of Service Legload = 14.109 kN

Effective Lengths;

Effective length, L_e = Open length of upright (m) x Effective length coefficient where;

Open length of upright = 2.000 m

Effective length coefficient = 1.00

Effective length, L_e = 2.000 m

Compressive Resistance;

We will obtain this value using values described herein and BS EN 1993-1-1 Clause 6.3.1.2.

$\lambda = L_e / i = 127.18$

$1/\lambda = \lambda / ((\pi^2 \times E) / f_y)^{0.5} = 1.35$

Factor for the buckling curve, $\alpha = 0.49$

$\phi = 0.5 (1 + \alpha/(\lambda - 0.2) + 1/\lambda^2) = 1.700$

$X = 1 / (\phi + (\phi^2 - 1/\lambda^2)^{0.5}) = 0.367$

Design Buckling Resistance, $N_{Rd} = X \times f_y \times A$ where;

Yield strength, $f_y = 235 \text{ N/mm}^2$

Cross-sectional area, $A = 557 \text{ mm}^2$

Design Buckling Resistance = 47.975 kN

COLD FORMED

Bending Moments in Standard When In Service at Ultimate Limit State:

Case 1; Maximum compression in standard x % out of plumb = Moment due to compression

$$\text{Moment} = 19.509 \times 1.0\% = 0.195 \text{ kNm}$$

Case 2; ((Working wind pressure x Width over upright) x Open length of upright²) / 10 = Moment due to wind (10 is used to account for continuity in the upright)

$$\text{Full wind pressure} = 0.300 \text{ kN/m}^2 \text{ (Factored)}$$

$$\text{Pressure coefficient} = 1.2 \text{ CPe}$$

$$\text{Solidity ratio} = 1.0$$

$$\text{Width over upright} = 2.2000 \text{ m}$$

$$\text{UDL up the face} = 0.792 \text{ kN/m (Factored)}$$

$$\text{Open length of upright} = 2.000 \text{ m}$$

$$\text{Moment} = \frac{0.792 \times 4.000}{10} = 0.317 \text{ kNm}$$

Case 3; (Horizontal handrail load x Open length of upright) / 6 = Moment due to fall arrest (6 is used to account for continuity in the upright)

$$\text{Horizontal handrail load} = 0.450 \text{ kN (Factored)}$$

$$\text{Open length of upright} = 2.000 \text{ m}$$

$$\text{Moment} = \frac{0.450 \times 2.000}{6} = 0.15 \text{ kNm}$$

$$\text{Total moment in upright} = 0.662 \text{ kNm}$$

Critical Interaction Check When In Service at Ultimate Limit State:

1.0 > (Calculated moment / Moment capacity) + (Calculated compression / Buckling resistance)

$$\text{Critical interaction} = \frac{0.662}{1.850} + \frac{19.509}{47.975} = 0.764$$

PASS - Upright passes buckling checks made at Ultimate Limit State

Bending Moments in Standard When Out of Service at Ultimate Limit State:

Case 1; Maximum compression in standard x % out of plumb = Moment due to compression

$$\text{Moment} = 14.109 \times 1.0\% = 0.141 \text{ kNm}$$

Case 2; ((Full wind pressure x Width over upright) x Open length of upright²) / 10 = Moment due to wind (10 is used to account for continuity in the upright)

$$\text{Working wind pressure} = 1.007 \text{ kN/m}^2 \text{ (Factored)}$$

$$\text{Pressure coefficient} = 1.2 \text{ CPe}$$

$$\text{Solidity ratio} = 1.0$$

$$\text{Width over upright} = 2.2000 \text{ m}$$

$$\text{UDL up the face} = 2.657 \text{ kN/m (Factored)}$$

$$\text{Open length of upright} = 2.000 \text{ m}$$

$$\text{Moment} = \frac{2.657 \times 4.000}{10} = 1.063 \text{ kNm}$$

$$\text{Total moment in upright} = 1.204 \text{ kNm}$$

Critical Interaction Check When In Service at Ultimate Limit State:

1.0 > (Calculated moment / Moment capacity) + (Calculated compression / Buckling resistance)

$$\text{Critical interaction} = \frac{1.204}{1.850} + \frac{14.109}{47.975} = 0.945$$

PASS - Upright passes buckling checks made at Ultimate Limit State

CONSIDER THE TIES

Full wind pressure = 0.671 kN/m² (Unfactored)
Working wind pressure = 0.200 kN/m² (Unfactored)
Horizontal distance over tie = 4.000 m
Vertical distance over tie = 4.000 m
Area over tie = 16.000 m²

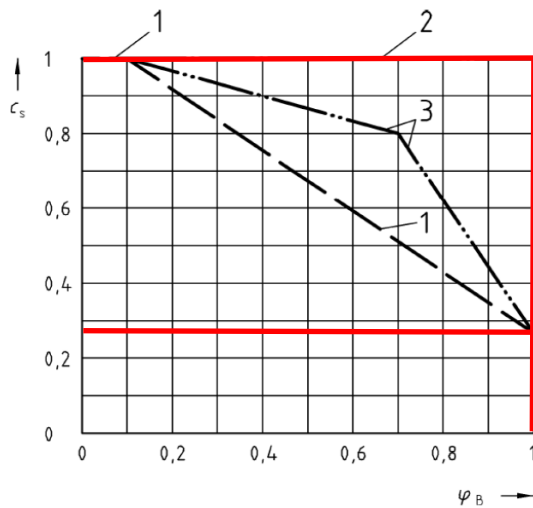
For Solidly Clad Scaffold Structures:

Tie value perpendicular to façade = Site coefficient x Σ (Aerodynamic force coefficient x Reference area x Full wind pressure) where;

Aerodynamic force coefficient = 1.3
Reference area = 16.000 m²
Building solidity ratio, ϕ_B = 1.0

Site coefficient (obtained from following graph) =

1.0 (abutting force) (Line #2)
-0.29 (tensile force) (Line #3 based upon building solidity)



Key

- 1 netting in both normal and parallel directions;
- 2 sheeting in both normal and parallel directions;
- 3 for sheeting, but only for calculation of the tensile anchoring forces normal to the façade;
- c_s site coefficient;
- ϕ_B the solidity ratio.

	SLS (kN)	ULS (kN)	
Abutting Value When In Service	4.160	6.240	(not critical to the design)
Abutting Value When Out of Service	13.957	20.935	(critical to the design)
Tie Value When In Service	1.206	1.810	(not critical to the design)
Tie Value When Out of Service	4.047	6.071	(critical to the design)

For Netted Scaffold Structures:

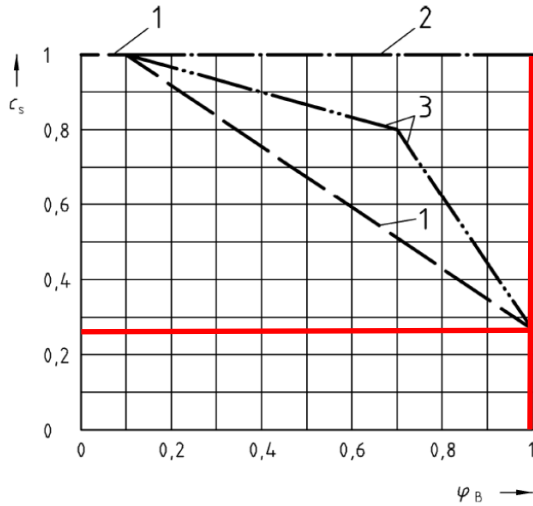
Tie value perpendicular to façade = Site coefficient x Σ (Aerodynamic force coefficient x Reference area x Full wind pressure) where;

Aerodynamic force coefficient = 0.65 (if TG20 high permeability debris netting is used)

Reference area = 16.000 m²

Building solidity ratio, ϕ_B = 1.0

Site coefficient (obtained from following graph) = 0.29 (both directions)



Key

- 1 netting in both normal and parallel directions;
- 2 sheeting in both normal and parallel directions;
- 3 for sheeting, but only for calculation of the tensile anchoring forces normal to the façade;
- c_s site coefficient;
- ϕ_B the solidity ratio.

	SLS (kN)	ULS (kN)	
Abutting Value When In Service	0.603	0.905	(not critical to the design)
Abutting Value When Out of Service	2.024	3.036	(critical to the design)
Tie Value When In Service	0.603	0.905	(not critical to the design)
Tie Value When Out of Service	2.024	3.036	(critical to the design)

For Unclad Scaffold Structures;

Tie value perpendicular to façade = [(Area of tube x Cpe) + (Area of boards x Cpe) + (Area of 'other' x Cpe)] x Full wind pressure where;

Area of tube = Total length of scaffold tube (m) x Tube diameter (m) where;

Scaffold tube diameter = 0.0483 m

Σ Length of scaffold tube =

Number of tubes	Member	Individual Length (m)	Total Length (m)
4	Uprights	4.000	16.000
4	Ledgers	4.000	16.000
2	Ledger Brace	2.332	4.665
2	Façade Brace	2.973	5.946
6	Handrails	4.000	24.000
0	Other	0.000	0.000

Total = 66.611 m

Area of tube = 66.611 x 0.0483 = 3.217 m²

Area of scaffold boards = Length of scaffold boards between ties x No. of levels of boards over ties x

Overall depth of scaffold boards where;

Length of scaffold boards between ties = 4.000 m

No. of levels of boards over ties = 2.000 no.

Depth of scaffold boards = 0.263 m

Area of scaffold boards = 2.104 m²

Wind pressure coefficients are as follows;

Wind pressure coefficient for tube = 1.2 Cpe

Wind pressure coefficient for boards = 1.3 Cpe

Wind pressure coefficient for 'Other' = 1.3 Cpe

	SLS (kN)	ULS (kN)	
Tie Value When In Service	1.319	1.979	(not critical to the design)
Tie Value When Out of Service	4.426	6.639	(critical to the design)

CONSIDER THE BRIDGE SPAN

PUNCHEON LEGLOAD

Inside Boards? **2.000** no.

Tube Self Weight per Node;

Number	Tube	Length (m)	Total Length (m)	Self Weight (kN/m)	Total Self Weight (kN)
1.000	Upright	2.000	2.000	0.0429	0.086
1.000	Ledger	2.000	2.000	0.0429	0.086
1.000	Transom	1.050	1.050	0.0429	0.045
3.000	Board Transom	1.050	3.150	0.0429	0.135
1.000	Ledger Brace	1.166	1.166	0.0429	0.050
1.000	Façade Brace	1.414	1.414	0.0429	0.061

Unfactored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
8.000	Tube and Coupler Self Weight	0.552	4.417
8.000	Board Self Weight	0.496	3.969
1.000	Design Service Load [1]	2.400	2.400
1.000	Design Service Load [2]	1.200	1.200
2.000	Inside Board Design Service Load	0.675	1.350

Unfactored In Service Legload = 13.336 kN

Unfactored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
8.000	Tube and Coupler Self Weight	0.552	4.417
8.000	Board Self Weight	0.496	3.969

Unfactored Out of Service Legload = 8.386 kN

Factored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
8.000	Tube and Coupler Self Weight	0.828	6.626
8.000	Board Self Weight	0.744	5.954
1.000	Design Service Load [1]	3.600	3.600
1.000	Design Service Load [2]	1.800	1.800
2.000	Inside Board Design Service Load	1.013	2.025
0.000	Load due to Other;	0.000	0.000

Factored In Service Legload = 20.004 kN

Factored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
8.000	Tube and Coupler Self Weight	0.828	6.626
8.000	Board Self Weight	0.744	5.954
0.000	Load due to Other;	0.000	0.000

Factored Out of Service Legload = 12.579 kN

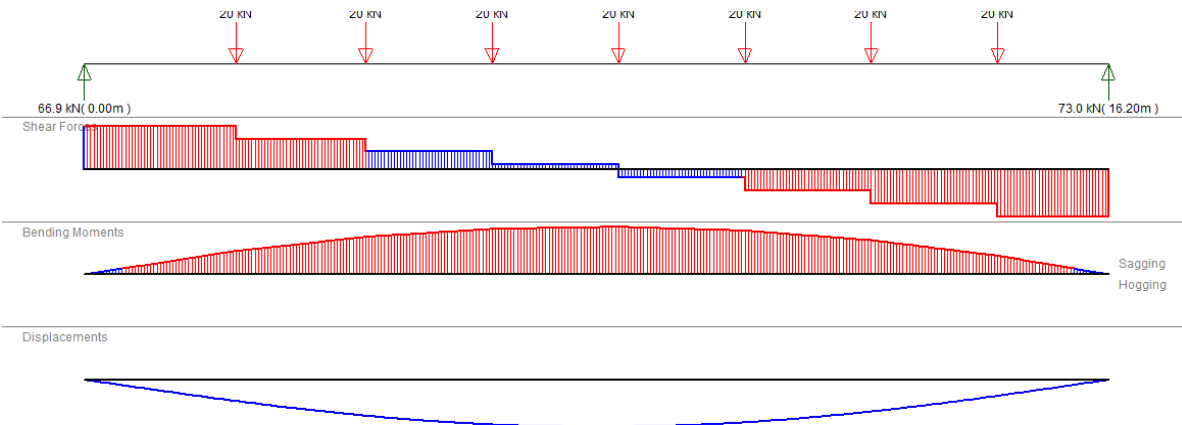
Summary;

Unfactored In Service Legload = 13.336 kN
Factored In Service Legload = 20.004 kN
Unfactored Out of Service Legload = 8.386 kN
Factored Out of Service Legload = 12.579 kN

Legload to be applied to Purflex =

20.004 kN

UDL not applicable as puncheon legload is conservative and accounts for 8 lifts of boards.



Beam Length 16.20 m
Second Moment of Area 17182.00 cm⁴
Elastic Modulus 70000 N/mm²

APPLIED LOADING

Point Loads

20.00 kN at 2.40 m
20.00 kN at 4.45 m
20.00 kN at 6.45 m
20.00 kN at 8.45 m
20.00 kN at 10.45 m
20.00 kN at 12.45 m
20.00 kN at 14.45 m

REACTIONS (+ve Upward)

66.89 kN at 0.00 m
72.98 kN at 16.20 m

RESULTS SUMMARY

Max. Reaction 72.98 kN at 16.200 m
Max. Sag Bending Moment 325.35 kN.m at 8.450 m
Max. Shear Force 73.14 kN at 14.450 m

Maximum reaction from bridge span = **72.98 kN**

Pairs of beams to be installed on inside and outside line of standards meaning beam arrangement is adequate by inspection.

LOADING SUMMARY

Tie Values

Unclad Scaffold Structure:

Maximum Calculated Tie/Abutting Value = 4.426 kN (Unfactored)
Maximum Calculated Tie/Abutting Value = 6.639 kN (Factored)

Pull-Test/Proof Test Loads:

Maximum Calculated Tie Value = 6.639 kN (Factored)

Legload(s), Bearing Pressure(s), Reaction Forces

Loads to Ground:

Maximum Calculated Legload = 13.944 kN (Unfactored)
Maximum Calculated Legload = 20.917 kN (Factored)

Accumulative Tower Loads:

Maximum Calculated Tower Load = 62.598 kN (Unfactored)
Maximum Calculated Tower Load = 93.897 kN (Factored)

Reaction Forces:

Maximum Calculated Reaction Force(s) = As indicated on the design.

Design Actions/Variable Actions

All design actions/variable actions stated below are unfactored (SLS) values;

Allowable Design Actions:

Design Service Load 1 =	2.000 kN/m ² on	1.000 no. levels
Design Service Load 2 =	1.000 kN/m ² on	1.000 no. levels
Inside Board Design Service Load =	0.750 kN/m ² on	2.000 no. levels

Allowable Variable Actions:

Maximum Calculated Wind Pressure =	0.671 kN/m ² =	74 mph
Working Wind Pressure =	0.200 kN/m ² =	40 mph
Maximum Calculated Snow Loading =	0.205 kN/m ²	

Number of Boarded Levels

Maximum Number of Boarded Levels = 7.000 no. levels

Where the maximum number of boarded levels is different this is indicated on the design.

APPENDIX A - DESIGN(S), DRAWING(S), SKETCH(ES)