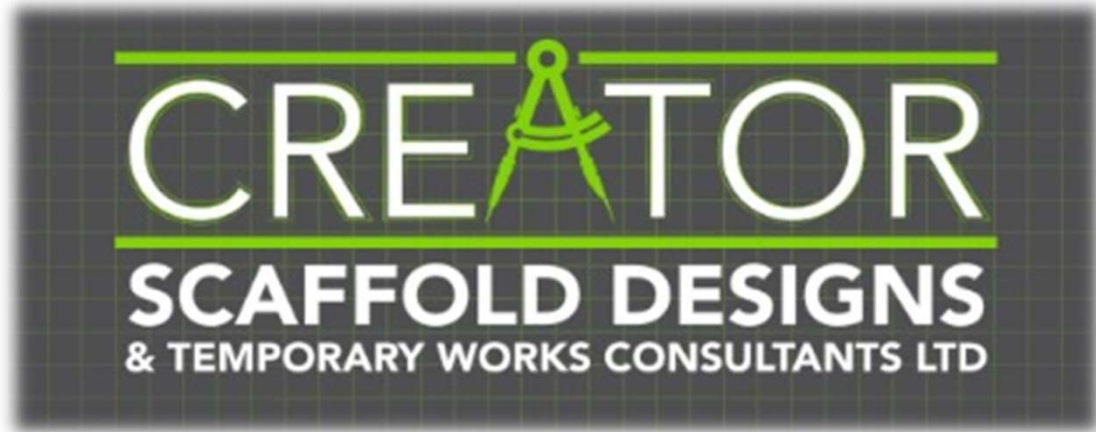




CREATIVE ENGINEERING WITH COMPLETE CUSTOMER FOCUS
IN MIND

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CALCULATIONS

Report Prepared For: MOTION PROJECT MANAGEMENT

Design Reference / Site Reference: MANCHESTER OPERA HOUSE

Site Address: Opera House Manchester 3 Quay Street

Site Address: M3 3HP

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REVISIONS

[illegible]

DOCUMENT SIGNATORIES / CATII 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-MOTION-001-001_BYROM STREET_CALCS_CALCS
APPENDIX A - CRE-MOTION-001-001_BYROM STREET_CALCS-1,2,3,4_DRG
APPENDIX B - CRE-MOTION-001-001 B WIND

Prepared By;

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PROJECT STATUS: FOR INFORMATION ONLY

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 340N/mm² < R_m < 520N/mm²

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, $R_e = 255$ N/mm². Tensile strength, $R_m = 295$ N/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, $V_{pl,Rd} = A_v \times (f_v / \sqrt{3}) / \gamma_{M0}$ where;

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

	British Standard Reference
	BS 1139-1.2
Shear Area, A_v (cm ²)	3.92
Yield Strength, f_v (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	i_y (cm)	1.097

Material Properties of Timber [Table 1 EN 338]

Timber Grade	C16
Bending, $f_{m,k}$ (N/mm ²)	16.0
Shear, $f_{v,k}$ (N/mm ²)	1.80
Mean Modulus of Elasticity Parallel to Grain, $E_{0,mean}$ (kN/mm ²)	8.00

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]	Short Term
Service Class [2.3.1.3 BS EN 1995-1-1]	Service Class 2
Modification Factor, k_{mod} [Table 3.1 BS EN 1995-1-1]	0.9

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 =

Beam not prone to lateral torsional buckling.

Ultimate Bending Moment Resistance

Design Bending Strength, $f_{m,d} = k_{mod} \times k_h \times f_{m,k} / \gamma_M$ (2.4.1 BS EN 1995-1-1)

Where;

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

$$\gamma_M = 1.3 \text{ for solid timber}$$

$$\text{Design Bending Strength} = 14.40 \text{ N/mm}^2$$

$$\text{Ultimate Bending Moment, } M_{ult} = f_{m,d} \times W_{yy}$$

$$\text{Ultimate Bending Moment, } M_{ult} = 0.780 \text{ kNm}$$

Ultimate Shear Force Resistance

$$\text{Design Shear Strength, } f_{v,d} = k_{mod} \times f_{v,k} / \gamma_M \text{ (2.4.1 BS EN 1995-1-1)}$$

$$\text{Design Shear Strength} = 1.246 \text{ N/mm}^2$$

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

$$\text{Shear Stress, } \tau_d = 1.5 \times V / b \times h_{ef}$$

Where;

$$\tau_d = f_{v,d} = 1.246 \text{ N/mm}^2$$

$$\text{Ultimate Shear Force, } V_{ult} = 7.103 \text{ kN}$$

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)
HAKI 450	2.0m Top, 2.0m Bottom	25.9	21.0
Apollo 750 X-Beam	1.0m Top, 1.0m Bottom	67.65	57.75

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	0.010	0.015
Swivel Coupler	kN	0.010	0.015
Single Coupler	kN	0.007	0.011
Sleeve Coupler	kN	0.011	0.017
Band & Plate Coupler	kN	0.020	0.030
Typical Beam Arrangement (obtained below)	kN/m ²	0.223	0.335
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	0.010 kN	0.080
SW Coupler		1.000	0.010 kN	0.010
Total Lineweight =				0.892
Beam Self Weight = Total Lineweight / Area = 0.22 kN/m ² (Unfactored)				

CONSIDER THE WIND LOADING

Peak wind velocity pressure obtained from CADS Velventi as follows:

Revised Peak Velocity Pressure, $Q_p = q_p \times c_c = 0.636 \text{ kN/m}^2$ (Unfactored)
Equivalent wind speed = 72 mph

CONSIDER THE SNOW LOADING

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

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

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)
0.500	0.8	10	0.1	0.8	0.266

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
Other				
Wind Loading;	kN/m ²	0.636	0.954	
Working Wind Loading ^[1] ;	kN/m ²	0.200	0.300	
Snow Loading;	kN/m ²	0.266	0.399	
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 =
$$\frac{0.758 \times 1.200}{2} + 0.000$$

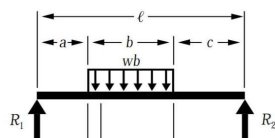
Maximum shear force = 0.455 kN

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.



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

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

Reaction = 1.125 kN

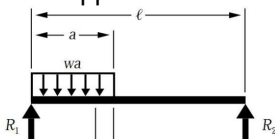
Maximum moment = 0.534 kNm

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.



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

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 =

8.000 kN/mm²

Second moment of area, I =

1.03E+06 mm⁴

Deflection, δ =

5	x	4.5E-04	x	2.07E+12
384	x	8.000	x	1.03E+06
+				
0.000	x	1.73E+09		
48	x	8.000	x	1.03E+06

Deflection, δ =

1.476 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 =

	wL²	+	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	+
				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 =

	wL	+	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) =	5wL⁴	+	PL³	
	384EI		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
				2.07E+12
	384	x	210	x
				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.

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

$$L_{\text{design}} = 2.000 \text{ m}$$

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 = 4.041 + 1.343 + 0.000

Total load at the connection = 5.384 kN (Factored)

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

CONSIDER THE GUARDRAILS



UNCLAD SCAFFOLD

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 =

$$wL^2$$

+

$$PL$$

$$8$$

$$4$$

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 =

$$0.0483 \text{ m}$$

$$1.850 = 0.010 L^2 + 0.1125 L$$

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

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

$$\begin{array}{rcl} & -0.113 & + \quad 0.297 \\ & 0.020 & \\ \hline & -0.1125 & + \quad 0.297 \\ & 0.020 & \end{array} \quad \begin{array}{l} = \\ \\ = \end{array} \quad \begin{array}{l} 9.034 \text{ m} \\ \\ 20.051 \text{ m} \end{array}$$

Maximum guardrail span for fall prevention and working wind = 9.034 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	0.636	0.954

Maximum moment =

$$wL^2$$

$$8$$

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.

$$1.850 = 0.015 L^2$$

Maximum guardrail span for full wind loading = 11.123 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 =

$$PL$$

$$4$$

$$1.850 = 0.313 L$$

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, $P = ((\text{Working wind load (kN/m}^2) \times \text{Pressure coefficient} \times \text{Containment} \times \text{Width over guardrail (m)}) \times \text{Maximum handrail span (m)}) + \text{Horizontal handrail force (kN)}$ where values have been defined above.

Pressure coefficient =	1.200
Containment =	1.000
Width over guardrail =	0.0483 m
Minimum guardrail length =	5.919 m
Point load, $P =$	0.369 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 =	3.16E-10 L ³

Maximum guardrail span when 'In Service' =	4.804 m
--	---------

Maximum allowable guardrail span for use in the design =	4.804 m
Actual guardrail span used in the design =	2.000 m

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

CONSIDER THE UPUNCHEONS

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
0.000	Ledger Brace	1.166	0.000	0.0429	0.000
0.000	Facade Brace	1.414	0.000	0.0429	0.000
1.000	Plan Brace	1.166	1.166	0.0429	0.050
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	0.010	0.020
1.000	Swivel Coupler	0.010	0.010
3.000	Single Coupler	0.007	0.021
1.000	Sleeve Coupler	0.011	0.011

Unfactored tube and coupler self weight per node =

0.464 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

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 Other;

		kN (Unfactored)
--	--	-----------------

Unfactored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.464	0.464
1.000	Board Self Weight	0.496	0.496
1.000	Design Service Load [1]	2.400	2.400
1.000	Inside Board Design Service Load	0.675	0.675
0.000	Load due to Other;	0.000	0.000

Unfactored In Service Legload = 4.035 kN

Unfactored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.464	0.464
1.000	Board Self Weight	0.496	0.496
0.000	Load due to Other;	0.000	0.000

Unfactored Out of Service Legload = 0.960 kN

Factored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.695	0.695
1.000	Board Self Weight	0.744	0.744
1.000	Design Service Load [1]	3.600	3.600
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 = 7.064 kN

Factored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.695	0.695
1.000	Board Self Weight	0.744	0.744
0.000	Load due to Other;	0.000	0.000

Unfactored Out of Service Legload = 1.439 kN

Summary;

Unfactored In Service Legload = 4.035 kN
Factored In Service Legload = 7.064 kN
Unfactored Out of Service Legload = 0.960 kN
Factored Out of Service Legload = 1.439 kN

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
0.000	Ledger Brace	1.166	0.000	0.0429	0.000
0.000	Façade Brace	1.414	0.000	0.0429	0.000
0.000	Plan Brace	1.166	0.000	0.0429	0.000
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	0.010	0.040
0.000	Swivel Coupler	0.010	0.000
3.000	Single Coupler	0.007	0.021
1.000	Sleeve Coupler	0.011	0.011

Unfactored tube and coupler self weight per node =

0.518 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]	0.000	0.000

Load due to Other;

		kN (Unfactored)
--	--	-----------------

Unfactored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.518	0.518
1.000	Board Self Weight	0.386	0.386
1.000	Design Service Load [1]	2.400	2.400
0.000	Load due to Other;	0.000	0.000

Unfactored In Service Legload = 3.304 kN

Unfactored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.518	0.518
1.000	Board Self Weight	0.386	0.386
0.000	Load due to Other;	0.000	0.000

Unfactored Out of Service Legload = 0.904 kN

Factored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.777	0.777
1.000	Board Self Weight	0.579	0.579
1.000	Design Service Load [1]	3.600	3.600
0.000	Load due to Other;	0.000	0.000

Factored In Service Legload = 4.956 kN

Factored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.777	0.777
1.000	Board Self Weight	0.579	0.579
0.000	Load due to Other;	0.000	0.000

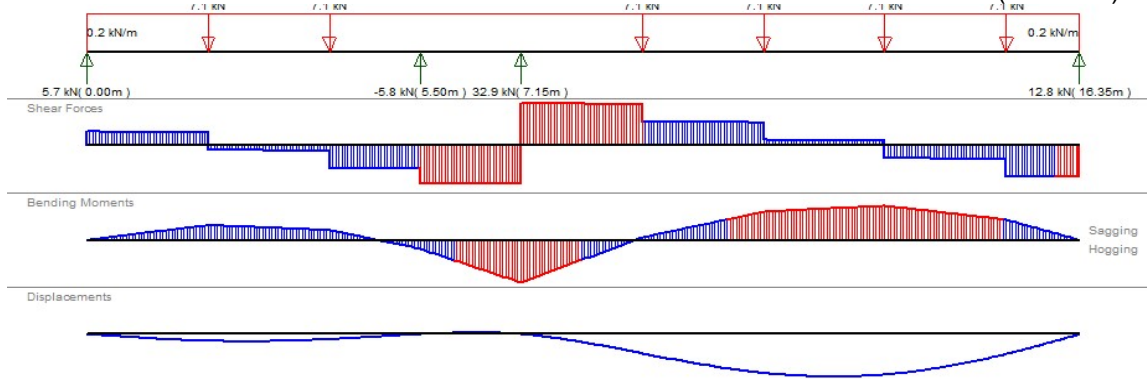
Unfactored Out of Service Legload = 1.356 kN

Summary;

Unfactored In Service Legload = 3.304 kN
 Factored In Service Legload = 4.956 kN
 Unfactored Out of Service Legload = 0.904 kN
 Factored Out of Service Legload = 1.356 kN

CONSIDER THE BEAMS

Legload = 7.064 kN (Factored)
Width over beam = 0.600 m
Beam selfweight = 0.222 kN/m (Unfactored)
Beam selfweight per node = 0.133 kN/m (Unfactored)
0.200 kN/m (Factored)



Beam Length 16.35 m
Second Moment of Area 4483.00 cm⁴
Elastic Modulus 70000 N/mm²

APPLIED LOADING

Point Loads

7.06 kN at 2.00 m
7.06 kN at 4.00 m
7.06 kN at 9.15 m
7.06 kN at 11.15 m
7.06 kN at 13.15 m
7.06 kN at 15.15 m

Distributed Loads

0.20 kN/m at 0.000 m --> 0.20 kN/m at 16.350 m

REACTIONS (+ve Upward)

5.75 kN at 0.00 m
-5.80 kN at 5.50 m
32.91 kN at 7.15 m
12.77 kN at 16.35 m

RESULTS SUMMARY

Max. Reaction 32.91 kN at 7.150 m
Max. Sag Bending Moment 25.72 kN.m at 13.150 m
Max. Hog Bending Moment 32.19 kN.m at 7.150 m
Max. Shear Force 17.31 kN at 7.150 m
Max. Downward Displacement 56.63 mm at 12.300 m
Max. Upward Displacement 2.13 mm at 6.400 m

Max. reaction in the Uprights = 32.910 kN

So, the beam is safe.

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
0.000	Facade Brace	1.414	0.000	0.0429	0.000
0.000	Plan Brace	1.166	0.000	0.0429	0.000
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	0.010	0.020
1.000	Swivel Coupler	0.010	0.010
3.000	Single Coupler	0.007	0.021
1.000	Sleeve Coupler	0.011	0.011

Unfactored tube and coupler self weight per node =

0.464 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 [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 Other;

21.940 kN (Unfactored)

Unfactored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.464	0.464
1.000	Board Self Weight	0.496	0.496
1.000	Design Service Load [2]	1.200	1.200
1.000	Load due to Other;	21.940	21.940

Unfactored In Service Legload = 24.100 kN

Unfactored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.464	0.464
1.000	Board Self Weight	0.496	0.496
1.000	Load due to Other;	21.940	21.940

Unfactored Out of Service Legload = 22.900 kN

Factored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.695	0.695
1.000	Board Self Weight	0.744	0.744
1.000	Design Service Load [2]	1.800	1.800
2.000	Inside Board Design Service Load	1.013	2.025
1.000	Load due to Other;	32.910	32.910

Factored In Service Legload = 38.174 kN

Factored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.695	0.695
1.000	Board Self Weight	0.744	0.744
1.000	Load due to Other;	32.910	32.910

Unfactored Out of Service Legload = 34.349 kN

Summary;

Unfactored In Service Legload = 24.100 kN
Factored In Service Legload = 38.174 kN
Unfactored Out of Service Legload = 22.900 kN
Factored Out of Service Legload = 34.349 kN

Effective Lengths;

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

Open length of upright = 1.750 m

Effective length coefficient = 0.85 (obtained through analysis software).

Effective length, L_e = 1.488 m

Compressive Resistance;

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

$\lambda = L_e / i = 94.59$

$1/\lambda = \lambda / ((\pi^2 \times E) / f_y)^{0.5} = 1.01$

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

$\phi = 0.5 (1 + \alpha/(\lambda - 0.2) + 1/\lambda^2) = 1.205$

$X = 1 / (\phi + (\phi^2 - 1/\lambda^2)^{0.5}) = 0.536$

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 = 70.092 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

$$\text{Moment} = 38.174 \times 1.5\% = 0.573 \text{ kNm}$$

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)

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 = 1.750 m

$$\text{Moment} = \frac{0.017 \times 3.063}{10} = 0.005 \text{ kNm}$$

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

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

Total moment in upright = 0.728 kNm

Critical Interaction Check When In Service at Ultimate Limit State;

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

$$\text{Critical interaction} = \frac{0.728}{1.850} + \frac{38.174}{70.092} = 0.938$$

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

$$\text{Moment} = 34.349 \times 1.5\% = 0.515 \text{ kNm}$$

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)

Full wind pressure = 0.954 kN/m² (Factored)

Pressure coefficient = 1.2 CPe

Solidity ratio = 1.0

Tube diameter = 0.0483 m

UDL up the face = 0.055 kN/m (Factored)

Open length of upright = 1.750 m

$$\text{Moment} = \frac{0.055 \times 3.063}{10} = 0.017 \text{ kNm}$$

Total moment in upright = 0.532 kNm

Critical Interaction Check When In Service at Ultimate Limit State;

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

$$\text{Critical interaction} = \frac{0.532}{1.850} + \frac{34.349}{70.092} = 0.778$$

PASS - Upright passes buckling checks made at Ultimate Limit State

Allowable Bearing Pressure;

Maximum calculated legload = 12.050 kN (Unfactored)

If we assume an allowable bearing pressure (or if given one by others use this);

Allowable bearing pressure = 150 kN/m² (Unfactored)

Area required to satisfy allowable bearing pressure = Unfactored calculated legload / Allowable bearing pressure.

Area required = 0.080 m²

Single solepad width = 0.225 m

Number of solepads wide = 2 no.

Total solepad width = 0.45 m
Required solepad length = Area required / Total solepad width = 0.179 m

Required solepad length = 0.450 m

Actual area = 0.203 m²

Actual bearing pressure = 59.505 kN/m² (Unfactored)

PASS - Actual bearing pressure is less than allowable bearing pressure stated

Design of Spreader Boards/Solepads:

In order to design the spreader boards we will first calculate the cantilever distance for the board;

Maximum calculated legload = 38.174 kN (Factored)

Actual bearing pressure = 188.516 kN/m² (Factored)

Solepad length = 0.450 m

Baseplate length = 0.152 m

Actual cantilever length = (Solepad length / 2) - (Baseplate length / 2) = 0.149 m

Load to cantilever = 0.149 x 0.45 x 188.516

Load to cantilever = 12.640 kN (Factored)

Maximum bending moment = 12.640 x $\frac{0.149}{2}$ = 0.942 kNm

Maximum shear stress = Force / (Width x Depth)

Single solepad depth = 0.038 m

Number of solepads deep = 2 no.

Total solepad depth = 0.076 m

Maximum shear stress = $\frac{12.640}{34200}$ = 0.000 N/mm²

Width (mm)	Depth (mm)	Area (mm ²)	Elastic Section Modulus, W_{yy} (mm ³)	Second Moment of Area, I_{yy} (mm ⁴)
450	76	34200	433200	16461600

Timber Grade	C16
Bending, $f_{m,k}$ (N/mm ²)	16.0
Shear, $f_{v,k}$ (N/mm ²)	1.80

Load Duration Class [Table 2.1 & 2.2 BS EN 1995-1-1]	Long 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]	0.55

Design Bending Strength, $f_{m,d} = k_{mod} \times k_h \times f_{m,k} / \gamma_M$ (2.4.1 BS EN 1995-1-1) where;

$k_{mod} = 0.55$

$k_h = \text{lesser of either } (150 / h)^{0.2} \text{ or } 1.3 = 1.145659$

$f_{m,k} = 16.0 \text{ N/mm}^2$

$\gamma_M = 1.3$ (solid timber)

Design Bending Strength = 7.76 N/mm²

Ultimate Bending Moment, $M_{ult} = f_{m,d} \times W_{yy}$ where;

$W_{yy} = 433200 \text{ mm}^3$

Ultimate Bending Moment, $M_{ult} = 3.360 \text{ kNm}$

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

Design Shear Strength, $f_{v,d} = k_{mod} \times f_{v,k} / \gamma_M$ (2.4.1 BS EN 1995-1-1) where;

$k_{mod} = 0.55$

$f_{v,k} = 1.80$

$\gamma_M = 1.3$ (solid timber)

Design Shear Strength = 0.762 N/mm²

PASS - Calculated shear stress is less than ultimate shear stress capacity

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	1.750	1.750	0.0429	0.075
1.000	Ledger	0.825	0.825	0.0429	0.035
1.000	Transom	0.600	0.600	0.0429	0.026
2.000	Board Transom	0.600	1.200	0.0429	0.051
0.000	Ledger Brace	1.061	0.000	0.0429	0.000
0.000	Facade Brace	0.967	0.000	0.0429	0.000
0.000	Plan Brace	0.728	0.000	0.0429	0.000
2.000	Handrails	0.825	1.650	0.0429	0.071

Coupler Self Weight per Node;

Number	Coupler	Self Weight (kN)	Total Self Weight (kN)
4.000	Right Angled Coupler	0.010	0.040
0.000	Swivel Coupler	0.010	0.000
2.000	Single Coupler	0.007	0.014
1.000	Sleeve Coupler	0.011	0.011

Unfactored tube and coupler self weight per node =

0.323 kN (Unfactored)

Board Self Weight per Node;

Board self weight = 0.245 kN/m² (Unfactored)
 Length of boards = 0.825 m
 Number of boards = 2.500 no. (including toeboard)
 Single board width = 0.225 m
 Area over node = 0.464 m²
 Board self weight per node = 0.114 kN (Unfactored)

Variable Action per Node - Main Boards;

Length over node = 0.825 m
 Width over node = 0.600 m
 Area over node = 0.495 m²

	Unfactored Area Load (kN/m ²)	Total Load per Node (kN)
Design Service Load [2]	1.000	0.495

Load due to Other;

21.940 kN (Unfactored)

Unfactored In Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.323	0.323
1.000	Board Self Weight	0.500	0.500
1.000	Design Service Load [2]	0.495	0.495
1.000	Load due to Other;	21.940	21.940

(Ranch Board included)

Unfactored In Service Legload = 23.258 kN

Unfactored Out of Service Legload;

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.323	0.323
1.000	Board Self Weight	0.114	0.114
1.000	Load due to Other;	21.940	21.940

Unfactored Out of Service Legload = 22.377 kN

Factored In Service Legload:

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.485	0.485
1.000	Board Self Weight	0.500	0.500
1.000	Design Service Load [2]	0.743	0.743
1.000	Load due to Other;	32.910	32.910

Factored In Service Legload = 34.637 kN

Factored Out of Service Legload:

No. of Levels Loaded	Load Type	Load per Node (kN)	Total Load (kN)
1.000	Tube and Coupler Self Weight	0.485	0.485
1.000	Board Self Weight	0.171	0.171
1.000	Load due to Other;	32.910	32.910

Unfactored Out of Service Legload = 33.565 kN

Summary:

Unfactored In Service Legload = 23.258 kN
Factored In Service Legload = 34.637 kN
Unfactored Out of Service Legload = 22.377 kN
Factored Out of Service Legload = 33.565 kN

Effective Lengths:

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

Open length of upright = 1.730 m
Effective length coefficient = 0.85 (obtained through analysis software)
Effective length, L_e = 1.471 m

Compressive Resistance:

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

$\lambda = L_e / i = 93.51$

$1/\lambda = \lambda / ((\pi^2 \times E) / f_y)^{0.5} = 1.00$

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

$\phi = 0.5 (1 + \alpha/(\lambda - 0.2) + 1/\lambda^2) = 1.191$

$\chi = 1 / (\phi + (\phi^2 - 1/\lambda^2)^{0.5}) = 0.542$

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

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

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

Design Buckling Resistance = 70.965 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

Moment = 34.637 x 1.5% = 0.520 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)

Full wind pressure = 0.300 kN/m² (Factored)

Pressure coefficient = 1.2 CPe

Solidity ratio = 1.0

Width over upright = 0.825 m

UDL up the face = 0.297 kN/m (Factored)

Open length of upright = 1.730 m

Moment = $\frac{0.297 \times 2.993}{10} = 0.089 \text{ kNm}$

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 = 1.750 m

$$\text{Moment} = \frac{0.450 \times 1.750}{6} = 0.13125 \text{ kNm}$$

Total moment in upright = 0.740 kNm

Critical Interaction Check When In Service at Ultimate Limit State:

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

$$\text{Critical interaction} = \frac{0.740}{1.850} + \frac{34.637}{70.965} = 0.888$$

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

$$\text{Moment} = 33.565 \times 1.5\% = 0.503 \text{ kNm}$$

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

$$\text{Working wind pressure} = 0.954 \text{ kN/m}^2 \text{ (Factored)}$$

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

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

$$\text{Width over upright} = 0.825 \text{ m}$$

$$\text{UDL up the face} = 0.944 \text{ kN/m (Factored)}$$

$$\text{Open length of upright} = 1.730 \text{ m}$$

$$\text{Moment} = \frac{0.944 \times 2.993}{10} = 0.283 \text{ kNm}$$

$$\text{Total moment in upright} = 0.786 \text{ kNm}$$

Critical Interaction Check When In Service at Ultimate Limit State:

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

$$\text{Critical interaction} = \frac{0.786}{1.850} + \frac{33.565}{70.965} = 0.898$$

PASS - Upright passes buckling checks made at Ultimate Limit State

Allowable Bearing Pressure:

$$\text{Maximum calculated legload} = 11.629 \text{ kN (Unfactored)}$$

If we assume an allowable bearing pressure (or if given one by others use this);

$$\text{Allowable bearing pressure} = 150 \text{ kN/m}^2 \text{ (Unfactored)}$$

Area required to satisfy allowable bearing pressure = Unfactored calculated legload / Allowable bearing pressure.

$$\text{Area required} = 0.078 \text{ m}^2$$

$$\text{Single solepad width} = 0.225 \text{ m}$$

$$\text{Number of solepads wide} = 2 \text{ no.}$$

$$\text{Total solepad width} = 0.45 \text{ m}$$

$$\text{Required solepad length} = \text{Area required} / \text{Total solepad width} = 0.172 \text{ m}$$

$$\text{Required solepad length} = 0.450 \text{ m}$$

$$\text{Actual area} = 0.203 \text{ m}^2$$

$$\text{Actual bearing pressure} = 57.427 \text{ kN/m}^2 \text{ (Unfactored)}$$

PASS - Actual bearing pressure is less than allowable bearing pressure stated

Design of Spreader Boards/Solepads:

In order to design the spreader boards we will first calculate the cantilever distance for the board;

$$\text{Maximum calculated legload} = 34.637 \text{ kN (Factored)}$$

$$\text{Actual bearing pressure} = 171.047 \text{ kN/m}^2 \text{ (Factored)}$$

$$\text{Solepad length} = 0.450 \text{ m}$$

$$\text{Baseplate length} = 0.152 \text{ m}$$

$$\text{Actual cantilever length} = (\text{Solepad length} / 2) - (\text{Baseplate length} / 2) = 0.149 \text{ m}$$

$$\text{Load to cantilever} = 0.149 \times 0.45 \times 171.047$$

$$\text{Load to cantilever} = 11.469 \text{ kN (Factored)}$$

$$\text{Maximum bending moment} = 11.469 \times 0.149 = 0.854 \text{ kNm}$$

2

Maximum shear stress = Force / (Width x Depth)
Single solepad depth = 0.038 m
Number of solepads deep = 2 no.
Total solepad depth = 0.076 m
Maximum shear stress = $\frac{11.469}{34200} = 0.000 \text{ N/mm}^2$

Width (mm)	Depth (mm)	Area (mm ²)	Elastic Section Modulus, W_{yy} (mm ³)	Second Moment of Area, I_{yy} (mm ⁴)
450	76	34200	433200	16461600

Timber Grade	C16
Bending, $f_{m,k}$ (N/mm ²)	16.0
Shear, $f_{v,k}$ (N/mm ²)	1.80

Load Duration Class [Table 2.1 & 2.2 BS EN 1995-1-1]	Long 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]	0.55

Design Bending Strength, $f_{m,d} = k_{mod} \times k_h \times f_{m,k} / \gamma_M$ (2.4.1 BS EN 1995-1-1) where;
 $k_{mod} = 0.55$
 $k_h = \text{lesser of either } (150 / h)^{0.2} \text{ or } 1.3 = 1.145659$
 $f_{m,k} = 16.0 \text{ N/mm}^2$
 $\gamma_M = 1.3 \text{ (solid timber)}$
 Design Bending Strength = 7.76 N/mm^2

Ultimate Bending Moment, $M_{ult} = f_{m,d} \times W_{yy}$ where;
 $W_{yy} = 433200 \text{ mm}^3$
 Ultimate Bending Moment, $M_{ult} = 3.360 \text{ kNm}$

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

Design Shear Strength, $f_{v,d} = k_{mod} \times f_{v,k} / \gamma_M$ (2.4.1 BS EN 1995-1-1) where;
 $k_{mod} = 0.55$
 $f_{v,k} = 1.80$
 $\gamma_M = 1.3 \text{ (solid timber)}$
 Design Shear Strength = 0.762 N/mm^2

PASS - Calculated shear stress is less than ultimate shear stress capacity

CONSIDER THE TIES

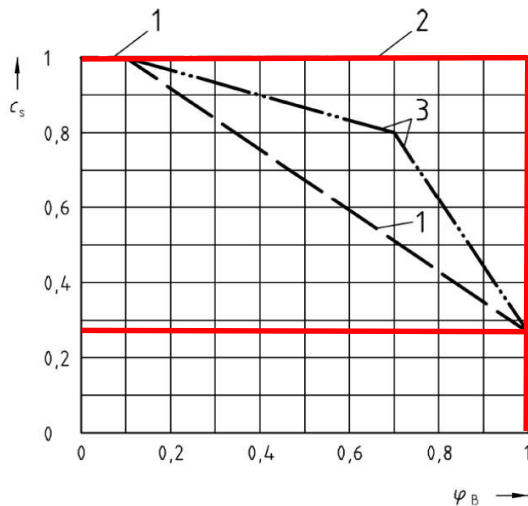
Full wind pressure = 0.636 kN/m² (Unfactored)
Working wind pressure = 0.200 kN/m² (Unfactored)
Horizontal distance over tie = 2.000 m
Vertical distance over tie = 2.000 m
Area over tie = 4.000 m²

Since, the bottom two lifts are ranch boarded, we conservatively assumed the scaffold is fully clad.

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 = 1.0 (both directions)(BS EN 1991-1-4)
Reference area = 4.000 m²
Cladding solidity ratio, ϕ_B = 1.0
Area of sheeting with openings deducted = 4.000 m²
Site coefficient (obtained from following graph) = 1.0 (abutting force)
-0.29 (tensile force)



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	0.800	1.200	(not critical to the design)
Abutting Value When Out of Service	2.544	3.816	(critical to the design)
Tie Value When In Service	0.232	0.348	(not critical to the design)
Tie Value When Out of Service	0.738	1.107	(critical to the design)

LOADING SUMMARY

Tie Values

Solidly Clad Scaffold Structure:

Maximum Calculated Tie Value =	0.738 kN (Unfactored)
Maximum Calculated Tie Value =	1.107 kN (Factored)
Maximum Calculated Abutting Value =	2.544 kN (Unfactored)
Maximum Calculated Abutting Value =	3.816 kN (Factored)

Pull-Test/Proof Test Loads:

Maximum Calculated Tie Value = 7.682 kN (Factored)

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

Loads to Ground:

Maximum Calculated Legload =	24.100 kN (Unfactored)
Maximum Calculated Legload =	38.174 kN (Factored)
Maximum Calculated Bearing Pressure =	71.111 kN/m ² (Unfactored)

Loads to BEAMS:

Maximum Calculated Legload =	21.940 kN (Unfactored)
Maximum Calculated Legload =	32.910 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 =	0.636 kN/m ² =	72 mph
Working Wind Pressure =	0.200 kN/m ² =	40 mph
Maximum Calculated Snow Loading =	0.266 kN/m ²	

Number of Boarded Levels

Maximum Number of Boarded Levels = 1.000 no. levels

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

DESIGNERS RISK ASSESSMENT

DESIGNERS RISK ASSESSMENT

Likelihood		Severity		Risk Level						Risk Assessment Notes
Probability of Occurrence	Probability Index	Hazard Severity / Potential Maximum Consequence	Severity Index	Hazard Severity Index	Probability Index					
					A	B	C	D	E	
So unlikely that probability is close to zero	A	Minor injury/illness resulting in lost time of 3 days or less	1	1	L	L	M	H	H	• Risk assessment in accordance with the Construction (Design and Management) Regulations 2015 • All control measures to be managed on site by party indicated in 'action to be taken by:' column • All residual risks will be indicated on the design as follows; Description of control Measure to be adopted Description of Risk RISK - [.....] Control Measure: [.....]
Unlikely though conceivable	B	Injury/illness resulting in lost time of more than 3 days	2	2	L	M	H	H	H	
Likely to occur sometime	C	Major injury causing long term loss of work	3	3	L	M	H	S	S	
Occurrence not surprising. May occur more than once.	D	Major illness / Disability / Single Fatality	4	4	M	H	S	S	S	
Occurrence inevitable. May occur many times.	E	Multiple Fatality	5	5	M	H	S	S	S	

Hazard:	Initial Risk			Risk Control Measures:	Residual Risk		
	P	R	L		P	R	L
Inaccuracies in the visual representation and detailing of existing 'structure' and ground based obstacles etc.	C	1	M	Accuracies to be checked on site and any discrepancies made aware of to Creator Ltd as erection progresses.	C	1	M
Excessive bay sizes installed during erection process causing structural failure of various members.	C	4	S	Bay sizes indicated to be maintained with the tolerances level indicated on the design. This tolerance should not be the norm for all bays	C	1	M
Failure of the scaffold during the in-use phase caused by overloading.	A	4	M	The maximum design service load indicated to be managed by the principal contractor and all loads indicated on the Scaffold	A	3	L
Excessive lift heights installed during erection causing buckling of the uprights.	C	4	S	Maximum lift heights to be installed by the scaffold contractor as detailed in the design ±100mm.	A	3	L
Scaffold conductivity causing electric shock or fatality	C	5	S	No OHLE lines present on the immediate site however earthing and bonding should be carried out by the principal contractor as necessary. This should be carried out by a suitably competent person.	A	1	L
Unauthorised access onto/into the scaffold structure	D	3	S	Where appropriate, ladder access will be installed and dismantled when required. Where indicated on the design, hoarding will be installed by the principal contractor.	A	3	L
Damage to the scaffold structure caused by on-site works or otherwise.	B	4	H	The principal contractor is to ensure all relevant procedures are in place when site vehicles are operating in close proximity of the scaffold structure.	A	3	L
Overloading of existing structure due to the need to support the scaffold structure.	B	4	H	All loads to the existing structure(s) will be checked by a structural engineer before erecting the scaffold.	A	3	L
Clearance from tracks to scaffold structure causing insufficient sighting to signals, collision with scaffolding etc.	C	5	S	All scaffold to be kept an agreed distance from tracks and platform edges.	A	3	L
Failure of the existing bearing stratum by means of: exceeding the bearing stratum, immediate settlement, consolidation settlement, saturated soils etc.	C	4	S	The maximum legloads and reactions should be appraised by the principal contractor. All founded on hardstanding.	A	3	L
Instability of scaffold boards to main boarded deck(s) during all stages i.e. in-use and out-of-use.	A	4	M	All boards to be secured down against uplift using board clips or similar.	A	4	M
Instability of ladder access and the ability for persons to fall as a result. Also, falls as a result of no handrails adjacent to ladder access.	B	4	H	Ladder to be effectively secured against movement and access gate provided adjacent to the ladder access	A	1	L
Insufficient board bearers (transoms) causing boards to fall under maximum design service load	A	4	M	Transoms to be installed at positions / centres indicated on the design	A	3	L
Boards of insufficient 'quality' to sustain maximum design service load (defects in boards such as rotting, 'cut' etc)	C	4	S	Boards should be visually inspected regularly to ensure they are of sufficient 'quality' to sustain the loads. Boards of insufficient quality should be replaced.	A	3	L
Materials / objects falling from boarded deck level / persons passing through void between intermediate guardrail and deck	A	5	M	Toeboards to be secured to leading edges and double guardrails installed where falls from height are possible	A	1	L
Materials / objects falls through scaffold into a pedestrianised area or other area causing damage / injury to persons / objects.	B	5	H	Toeboards to be secured to leading edges and double guardrails installed where falls from height are possible. A protective polythene layer is to be installed between boards. Double guardrails installed where falls from height are possible. Gaps to be a maximum of 25mm except between any inside board and main boards which should be 50mm	B	1	L
Falls from height / gaps in decking members	B	5	H		B	1	L
Coupler of insufficient tightness, grade and quality	C	5	S	Couplers to be manufactured to EN74 and should be tightened to a 50Nm torque. Couplers should be undamaged	C	1	M
Collision of vehicles with scaffold	C	5	S	Demarcation to be implemented. To prevent collision temporary vehicle crash barriers to be installed by the principal contractor.	C	1	M

Where the risks identified are not applicable these are to be struck out by the scaffold/principal contractor.

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

CONTRACTORS NOTES

DESIGN APPROACH

This 'design' has been carried out using the limit state approach in accordance with BS EN 12811-1, NASC TG20:(Current) and the Eurocode Suite.

PROPRIETARY EQUIPMENT

All proprietary equipment should be installed and used in accordance with the manufacturer's instructions.

DESIGN SPECIFICATION AND INTERPRETATION

The design has been produced using information provided to us by yourself and/or by others involved in the project. It is your responsibility to ensure that the 'services' provided by us are correct and the specification for the works has been interpreted correctly. This includes but is not limited to; loading, dimensions, lift heights etc. It is your responsibility to ensure that you produce a Risk Assessment and Method Statement (RAMS) in line with the design and that all items detailed are practicable for your company prior to starting on site.

ANCHORS (TIES)

The anchors / ties used in design are based on correct installation in accordance with manufacturer's details and the implementation of proof tests in accordance with NASC TG4:(Current) to ensure suitability of building strata. A minimum of 5% of the total number of anchors/ties should be tested to the values indicated in 'LOADINGS' notes. It is the responsibility of the Principal Contractor or Project Management Team to ensure approval of loads is sought from a structural engineer - CREATOR Ltd cannot take responsibility for the existing structure and its load capacity/integrity. It is the responsibility of the scaffold contractor to ensure that load bearing couplers are used for all ties, that they are installed correctly and fully in accordance with the design. Under no circumstances should ties be removed once installed as per the design. Should a tie require removal Creator Ltd should be contacted for advice.

FOUNDATIONS (SOIL AS BEARING STRATA)

It is the responsibility of the Principal Contractor to prepare all ground prior to erection. The forces/pressures to the ground, as detailed in 'SUMMARY LOADINGS', should be approved by the Principal Contractor prior to erection ensuring that settlement is kept to an absolute minimum. All foundations should be regularly inspected and as a minimum during the statutory weekly inspection. If at any stage the Principal Contractor / Contractor has concerns with the ground CREATOR Ltd should be contacted immediately.

FOUNDATIONS (EXISTING STRUCTURAL MEMBER AS BEARING STRATA)

It is the responsibility of the Principal Contractor to ensure that all loads applied to the existing structure are approved by a competent structural engineer prior to erection of the scaffold structure as this involves matters beyond our remit. See 'SUMMARY LOADINGS' for summary of loads to the existing structure. Should the existing structure be unsuitable to support the applied loads contact CREATOR Ltd prior to erection to obtain new proposals. Once approval of loads has been made and scaffold has been erected, the existing structure should be regularly inspected by the Principal Contractor.

DESIGN SERVICE LOADS (LIVE LOADS)

It is the responsibility of the Principal Contractor and Scaffold Contractor should ensure that the loads allowed in 'SUMMARY LOADINGS' notes are suitable for the work being undertaken and that the loading can be effectively managed by the Principal Contractor during the in-use phase.

BOARDED LEVELS

The maximum number of boarded levels is as indicated on the design and in the 'SUMMARY LOADINGS' section of the design. It is the responsibility of the scaffold contractor to ensure that the correct number of boarded levels are installed, that boards are in good condition showing no signs of defects and that they are effectively secured against movement.

MATERIALS

All materials should be in good condition showing no signs of defects. The design is based on the use of BS EN 2482 visually graded scaffold boards, EN 74 couplers and EN 39 good quality galvanised scaffold tube. All beams and prefabricated members should have a manufacturers identifier and should be in good condition showing no signs of defects.

DIMENSIONS

All written dimensions should take precedence over scaled dimensions. Any dimensional discrepancies on the design should be notified to Creator Ltd. It is the responsibility of the Scaffold Contractor and/or Scaffold Contractor to ensure the scaffold is set out correctly and in accordance with the design.

PERMITS AND PERMISSIONS

It is the responsibility of the Principal Contractor/Scaffold Contractor to obtain all permits and permissions prior to erection of the scaffold.

PROPERTY / MODIFICATION

This document remains the exclusive property of Creator Ltd and should be returned immediately upon request. No modification or alteration should be made to the design without written permission from Creator Ltd.

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