First Floors (UK) Limited
Unit 1A, The Mill Industrial Park, Kings Coughton, Alcester, Warwickshire B49 5QG.
Tel: 01789 764 172 Fax: 01789 766 964 Email: sales@firstfloors.co.uk

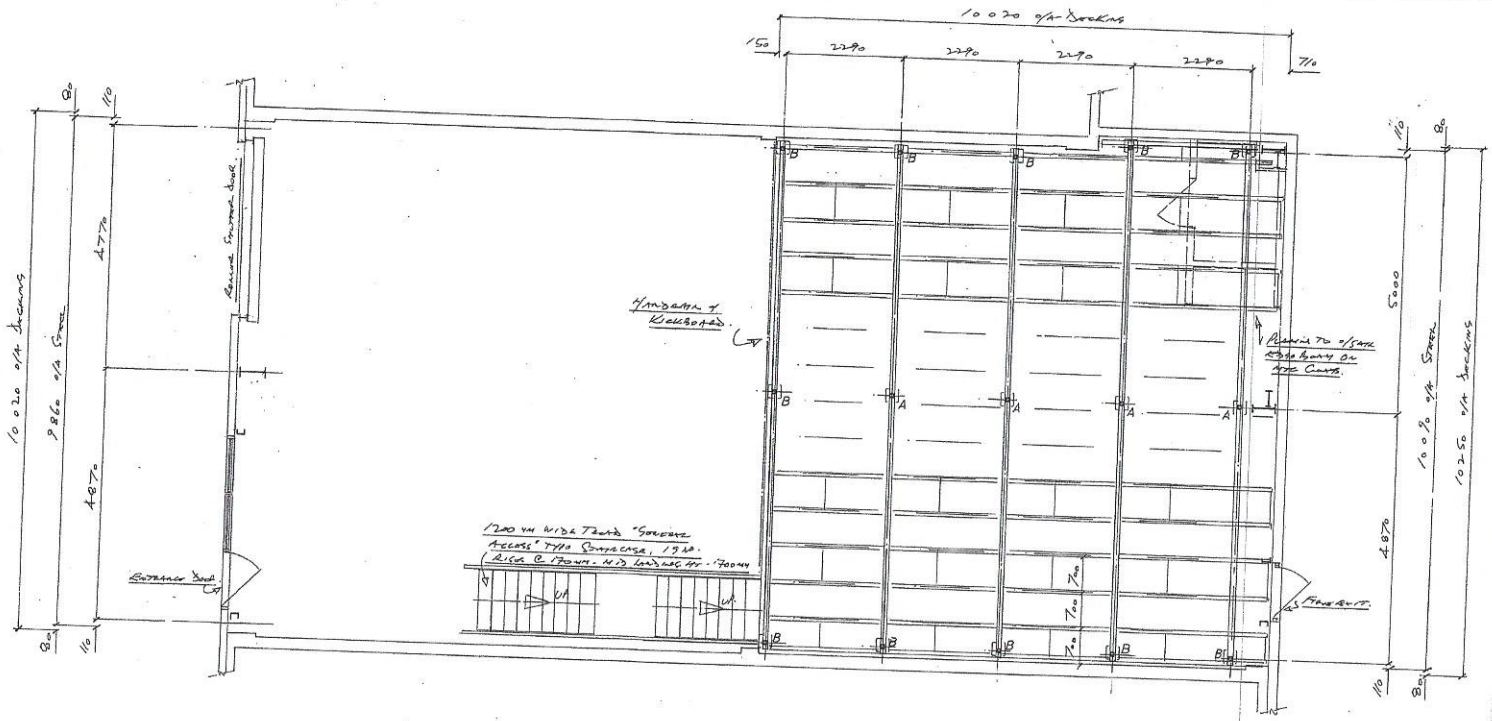


Structural Calculations

Design Method:

The raised platform is designed as pinned at deck level, with the beam to beam, and beam to column connections being pin joints. The stability of the steelwork frame is derived from the columns which will be designed as being fully fixed at concrete slab level, and the load carrying universal beams, which act as moment connections, and provide an effective lateral restraint between the lines of beams and columns.

Customer:	Tidebank – Unit K4, Daedulus Park.
Date:	15th February 2021
Reference Drawing Nos.	5687/1
Design Code:	BS 5950
Design Loading:	2.5 kN/sq.m
Clear Underside Height:	2.863 m
Maximum Span Of Main Beams:	4.870 m
Maximum Span Of Secondary Beams:	2.290 m
Decking Material:	38mm high density particle board.



Proposed Mezzanine Floor Steelwork Layout.

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Revision	Amendments	Date

MEZZANINE FLOOR SPECIFICATION

Design Loading	2.5 Kn/m²
Finished Floor Height	3.230m.
Clear Underside Height	2.865m.
Deck Surface Material	38mm Hi-Grip Structural Particle Bd.
Mainbeams - Internal	254×102×22 Kg UB.
Mainbeams - External	178×102×19 Kg UB.
Columns	100×100×5mm RHS.
Columns	90×90×5mm RHS.
Baseplates	300×300×15mm.
Baseplates	220×220×15mm.
Secondary Beams	14.56 Kg Universal
Steelwork Colour	Black Gloss
Steelwork Construction	Universally Primed Inside (Concrete Edge Both)

FIRE PROTECTION SPECIFICATION

One Hour Fire Rated.

Underside:
600×600×16mm Perforated Mineral Fibre Ceiling Tiles In An Exposed Grid Metal Suspension System. 4/K HT - 37 Both Affect.

Columns:
20mm Paramount" In A Steel Renders In A Smooth White Finish.

Fascia:
15mm "Supalux" Multiboard.

BUILDING REGULATIONS

Local Authority: ..... Application Date: .....

CUSTOMERS APPROVAL

Signed ..... Date .....

CUSTOMER

TIDEBANK.

Site Address
Unit K4, Daedulus Park, Daedulus Drive, LEE-ON-THE SOLENT, Hants. PO13 9FX.

FIRST FLOORS UK LTD
Unit 1A,
The Mill Industrial Park,
Alcester,
Warwickshire. B49 5QG
Tel: 01789 764172
Fax: 01789 766964
E-mail: sales@firstfloors.co.uk
Web: www.firstfloors.co.uk

Mezzanine Floors · Suspended Ceilings · Office & Industrial Partitioning · Lighting · Electrical Work · Racking & Shelving

Date 26.6.17	Scale 1-50	Checked by
Drawn by RAJOR.	Drg. no. 5687/1	

HADLEY Design Suite PRO

HADLEY GROUP

Company:	Tidebank
Project:	5687
Phase:	Mezzanine Floor
Description:	Unit K4, Daedulus Park
	-
User:	ROGER

First Floors (UK) Ltd
Unit 1a,The Mill Industrial PArk,Alcester Warks,B49 5QG
Tel: 01789 764172
Fax: 01789766964
email: roger@firstfloors.co.uk

UltraMEZZ® Single span Inset

BS EN 1991-1-4

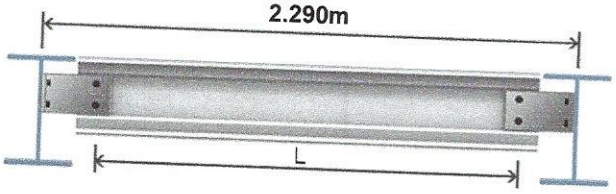
Design Parameters

Secondary Beams

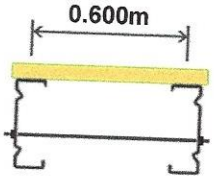
2290 Span | **Sheet No:1**

Loading inputs			Deflection Criteria
Dead 1.250 kN/m²	Service 0.100 kN/m²	Imposed 2.500 kN/m²	Total Load L/360

Primary Beam Centres



Joist Centres



Design Notes

Results

UltraBEAM 2 — **1456312 at 0.600m Centres with 1 Tie Bar at Mid Span.**

Load and Capacities (1456312)

Stress Ratio:	0.31	
Factored Design Load:	5.436	kN/m²
Ultimate Section Capacity:	17.568	kN/m²

Deflection

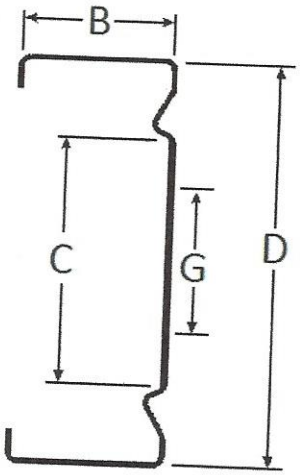
Deflection total load including point load

Deflection	2.04	mm
Span /deflection	L/1014	
% of design input	36	

Deflection imposed load only excluding point load

Deflection	1.33	mm
Span L / deflection	L/1558	
% of design input	23	

Section Properties



(1456312) Pass

Weight	2.70	kg/m
Depth,D	145	mm
Flange,B	63	mm
Lip	16	mm
Hole Ctrs,G	37	mm

Section modulus

Major axis	16.76	cm³
Minor axis	4.34	cm³

Second moment of area

Major axis	121.54	cm⁴
Minor axis	18.89	cm⁴

Section properties are considered in accordance with BS EN 1993-1-3

Capacity shown above is based on decking being screw-fixed to top flange at typically 600mm intervals, but no greater than 1150mm.

The above results are only valid when used in conjunction with the information contained in our current systems manual. All UltraZED.2 and UltraBEAM.2 sections are manufactured from steel gradeS450+Z275 NA-C to BS EN 10346 with tolerances on material to BS EN 10143

The performance of UltraZED2 and UltraBEAM2 products is derived from the interaction of system components, profile design features and extensive testing. Direct comparison of other profiles based upon section properties alone may result in the required minimum design capacities not being achieved. Only the above product as supplied by Hadley Building Products Division may be deemed to satisy this requirement.

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16th March, 2021 Sheet of

Unit K4: Internal Mainbeam

DESIGN OF MAIN BEAMS

with UDL to BS 5950 : Part 1

Try the 254x102x 22 UB

This section is classified as Plastic

	UNFACTORED :		FACTORED :	
Imposed Loading =	2.50 kN m^{-2}	x 1.6 =	4.00 kN m^{-2}
Weight of Decking =	0.25 kN m^{-2}	x 1.4 =	0.35 kN m^{-2}
Weight of Secondary Beams =	0.06 kN m^{-2}	x 1.4 =	0.08 kN m^{-2}
Self-Weight of Beams =	0.10 kN m^{-2}	x 1.4 =	0.13 kN m^{-2}
Total Factored load		=	4.57 kN m^{-2}

Beam Span = 4.870 m Effective Beam Spacing = 2.290 m

The maximum allowable deflection = $\frac{\text{Span} \times 10^3}{360}$ = 13.5 mm
(clause 2.4.5.1)

The deflection for this section = $\frac{5 \times 2.50 \times 4.870^4 \times 2.290 \times 10^9}{384 \times 205 \times 2870}$

= 7.1 mm
- therefore OK

The total load carried by the beam = 4.57 x 4.870 x 2.290 = 50.95 kN

The maximum bending moment = $\frac{50.95 \times 4.870}{8}$ = 31 kN m

The Moment Capacity for this section = $P_y S_x$ or $1.2 P_y Z_x$
which ever is smaller. = 72 kN m
- therefore OK

The maximum shear force = $\frac{50.95}{2}$ = 25 kN

The Shear Capacity for this section = $0.6 P_y t D$
$= 0.6 \times 275 \times 5.8 \times 254.0 \times 10^{-3}$

= 243 kN
- therefore OK

Therefore adopt this 254x102x 22 UB

16th March, 2021 Sheet of .

Unit K4: External Mainbeam

DESIGN OF MAIN BEAMS

with UDL to BS 5950 : Part 1

Try the 178x102x 19 UB

This section is classified as Plastic

	UNFACTORED :		FACTORED :
Imposed Loading =	2.50 kN m^{-2}	x 1.6 =	4.00 kN m^{-2}
Weight of Decking =	0.25 kN m^{-2}	x 1.4 =	0.35 kN m^{-2}
Weight of Secondary Beams =	0.06 kN m^{-2}	x 1.4 =	0.08 kN m^{-2}
Self-Weight of Beams =	0.10 kN m^{-2}	x 1.4 =	0.14 kN m^{-2}
Total Factored load		=	4.58 kN m^{-2}

Beam Span = 5.000 m Effective Beam Spacing = 1.855 m

The maximum allowable deflection = $\frac{\text{Span} \times 10^3}{360}$ = 13.9 mm
(clause 2.4.5.1)

The deflection for this section = $\frac{5 \times 2.50 \times 5.000^4 \times 1.855 \times 10^5}{384 \times 205 \times 1360}$

= 13.5 mm
- therefore OK

The total load carried by the beam = 4.58 x 5.000 x 1.855 = 42.46 kN

The maximum bending moment = $\frac{42.46 \times 5.000}{8}$ = 27 kN m

The Moment Capacity for this section = $P_y S_x$ or $1.2 P_y Z_x$
which ever is smaller. = 47 kN m
- therefore OK

The maximum shear force = $\frac{42.46}{2}$ = 21 kN

The Shear Capacity for this section = 0.6 P_y t D
= 0.6 x 275 x 4.7 x 177.8 x 10^{-3}

= 138 kN
- therefore OK

Therefore adopt this 178x102x 19 UB

16th March, 2021 Sheet of .

Unit K4: MArked A on drg no. 5687/1

DESIGN OF COLUMNS

to BS 5950 : Part 1

Try the 100x100x5.0 SHS

This section is classified as Plastic

	UNFACTORED :		FACTORED :
Imposed Loading =	2.50 kN m^{-2}	x 1.6 =	4.00 kN m^{-2}
Dead Loading =	0.37 kN m^{-2}	x 1.4 =	0.52 kN m^{-2}
		Total Factored load =	4.52 kN m^{-2}

Beam Span Side 1 =	4.870 m	Beam Spacing Side 1 =	2.290 m
Beam Span Side 2 =	4.770 m	Beam Spacing Side 2 =	2.290 m
Effective Span =	4.820 m	and Effective Spacing =	2.290 m

Column Length to Neutral Axis of Beam = 2.990 m,

therefore, the Effective Length = 1.5 x 2.990 = 4.485 m

The total load carried by the Column = 4.52 x 2.290 x 4.820 = 49.89 kN

The bending moment due to eccentric loading =
49.89 x 100.0 x 4.870 /(4000 x 4.820) = 1.3 kN m

Local Capacity Check :

for this SHS $\{ M / M_c \}^{5/3} <= 1$

for eccentrically loaded column : $\{ 1.3 / 18 \}^{5/3}$ = 0.012
- therefore OK

Overall Buckling Check :

for this SHS $\frac{F}{P_c} + \frac{M}{P_y Z_x} <= 1$

Evenly loaded :

$= \frac{49.89}{237.43} + 0$ = 0.210
- therefore OK

With eccentric loading :

$= 0.5 \times \frac{4.870 \times 49.89}{4.820 \times 237.43} + \frac{1.3 \times 1000}{275 \times 56.6}$ = 0.187
- therefore OK

continued .....

16th March, 2021 Sheet of .

Unit K4; MArked A on drg no. 5687/1

DESIGN OF COLUMNS

to BS 5950 : Part 1

..... continued

Sway Condition

	UNFACTORED :		FACTORED :
Imposed Loading =	2.50 kN m^{-2}	x 1.3 =	3.25 kN m^{-2}
Dead Loading =	0.37 kN m^{-2}	x 1.4 =	0.52 kN m^{-2}
		Total Factored load =	3.77 kN m^{-2}

The reduced load carried by the Column = 3.77 x 2.290 x 4.820 = 41.61 kN

The notional horizontal force =
2.5 x 0.01 x (0.52 + 1.6 x 2.50) x 2.290 x 4.820 = 1.25 kN
{ a value of 2.5 percent of the total factored load
is adopted to increase the column robustness.}

The bending moment due to the horizontal force = 1.25 x 2.990 = 3.7 kN m

The bending moment due to eccentric loading =
41.61 x 100.0 x 4.870 /(4000 x 4.820) = 1.1 kN m

Therefore, the worst bending moment = 4.8 kN m

Local Capacity Check :

for this SHS $\{ M / M_c \}^{5/3} <= 1$

giving,

for evenly loaded column : $\{ 3.7/ 18 \}^{5/3}$ = 0.073
and for eccentrically loaded column : $\{ 4.8/ 18 \}^{5/3}$ = 0.110
- therefore OK

Overall Buckling Check :

for this SHS $\frac{F}{P_c} + \frac{M}{P_y Z_x} <= 1$

Evenly loaded :

$= \frac{41.61}{237.43} + \frac{3.7 \times 1000}{275 \times 56.6}$ = 0.415
- therefore OK

With eccentric loading :

$= 0.5 \times \frac{4.870 \times 41.61}{4.820 \times 237.43} + \frac{4.8 \times 1000}{275 \times 56.6}$ = 0.396
- therefore OK

Therefore, adopt this 100x100x5.0 SHS

16th March, 2021 Sheet of .

Unit K4: Marked B on drg no. 5687/1

DESIGN OF COLUMNS

to BS 5950 : Part 1

Try the 80x80x5.0 SHS

This section is classified as Plastic

	UNFACTORED :	FACTORED :	
Imposed Loading =	2.50 kN m^{-2}	x 1.6 =	4.00 kN m^{-2}
Dead Loading =	0.37 kN m^{-2}	x 1.4 =	0.52 kN m^{-2}
		Total Factored load =	4.52 kN m^{-2}

Beam Span Side 1 =	2.290 m	Beam Spacing Side 1 =	4.870 m
Beam Span Side 2 =	2.290 m	Length of Cantilever Side 2 =	0.110 m
Effective Span =	2.290 m	and Effective Spacing =	2.545 m

Column Length to Neutral Axis of Beam = 2.990 m,

therefore, the Effective Length = 1.5 x 2.990 = 4.485 m

The total load carried by the Column = 4.52 x 2.545 x 2.290 = 26.34 kN

The bending moment due to eccentric loading = 26.34 x 80.0 /2000 = 1.1 kN m

Local Capacity Check :

for this SHS $\{ M / M_c \}^{8/3} <= 1$

for eccentrically loaded column : $\{ 1.1 / 11 \}^{8/3}$ = 0.020

- therefore OK

Overall Buckling Check :

for this SHS $\frac{F}{P_c} + \frac{M}{P_y Z_c} <= 1$

Evenly loaded :

$= \frac{26.34}{123.75} + 0$ = 0.213

- therefore OK

With eccentric loading :

$= \frac{26.34}{123.75} + \frac{1.1 \times 1000}{275 \times 34.7}$ = 0.323

- therefore OK

continued .....

16th March, 2021 Sheet of .

Unit K4; Marked B on drg no. 5687/1

DESIGN OF COLUMNS

to BS 5950 : Part 1

..... continued

Sway Condition

	UNFACTORED :		FACTORED :
Imposed Loading =	2.50 kN m^{-2}	x 1.3 =	3.25 kN m^{-2}
Dead Loading =	0.37 kN m^{-2}	x 1.4 =	0.52 kN m^{-2}
		Total Factored load =	3.77 kN m^{-2}

The reduced load carried by the Column = 3.77 x 2.545 x 2.290 = 21.97 kN

The notional horizontal force =
2.5 x 0.01 x (0.52 + 1.6 x 2.50) x 2.545 x 2.290 = 0.66 kN
{ a value of 2.5 percent of the total factored load is adopted to increase the column robustness.}

The bending moment due to the horizontal force = 0.66 x 2.990 = 2.0 kN m

The bending moment due to eccentric loading = 21.97 x 80.0 /2000 = 0.9 kN m

Therefore, the worst bending moment = 2.8 kN m

Local Capacity Check :

for this SHS $\{ M / M_c \}^{5/3} <= 1$

giving,

for evenly loaded column : $\{ 2.0 / 11 \}^{5/3}$ = 0.057

and for eccentrically loaded column : $\{ 2.8 / 11 \}^{5/3}$ = 0.105 - therefore OK

Overall Buckling Check :

for this SHS $\frac{F}{P_c} + \frac{M}{P_y Z_x} <= 1$

Evenly loaded :

$= \frac{21.97}{123.75} + \frac{2.0 \times 1000}{275 \times 34.7}$ = 0.384 - therefore OK

With eccentric loading :

$= \frac{21.97}{123.75} + \frac{2.8 \times 1000}{275 \times 34.7}$ = 0.476 - therefore OK

Therefore, adopt this 80x80x5.0 SHS

16th March, 2021 Sheet of

Unit K4; Marked A on drg no. 5687/1

DESIGN OF BASEPLATES

to BS 5950 : Part 1

Try a 300 x 300 x 15 mm plate(for a 100 x 100 x 5.00 SHS Column)

The foundation bearing strength = $0.4 \times f_{cu}$ = 8.00 N mm^{-2}

The plate bending capacity = $1.2 \times P_y \times t^2 / 6000$ = 11.92 N m (for a unit strip)

Check thickness for concentric forces (clause 4.13.2.2) :

Total Factored Load carried by the Column = 49.89 kN

Minimum thickness = $\{ 2.5 \times W \times 0.7 \times a^2 / P_y \}^{1/2}$

= $\{ 1750 \times 49.89 \times (0.5 \times (300 - 100))^2 / 265 \times 300^2 \}^{1/2}$ = 6 mm
- therefore OK

Check forces due to eccentric loading (clause 4.13.2.3) :

The resulting bending moment = 1.3 kN m

Acting at a distance from the Centre Line of D/2 = 50.0 mm

Therefore, the width of the baseplate in compression = 300.0 mm

Giving, a maximum bearing pressure at the edge of the plate = 0.56 N mm^{-2}
<= 8.00 N mm^{-2}
- therefore OK

The bearing pressure under the column edge = 0.37 N mm^{-2}

The total moment at the Column edge
= $\{(0.5 \times 0.37 \times (0.5 \times(300 - 100))^2)$
$+ ((0.56 - 0.37) \times (0.5 \times(300 - 100))^2 / 3) \} / 1000$
= 2.49 N m (for a unit strip)
<= 11.92 N m (for a unit strip)
- therefore OK

continued .....

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16th March, 2021 Sheet of .

Unit K4; Marked A on drg no. 5687/1

DESIGN OF FOUNDATIONS

For a 300 x 300 x 15 mm plate supporting a 100 x 100 x 5.00 SHS Column

Total Unfactored Load carried by the Column = 31.69 kN

The depth of the foundation (concrete + hardcore) = 0.350 m

The Ground Bearing Capacity = 40.0 kN m^{-2} (0.373 Tons ft^{-2})

Assuming a 45° angle of spread for the pressure through the foundation:

The load will act on the ground over an area = $\{ \frac{300}{1000} + (2 \times 0.350) \}^2$ m^2

giving,

a Ground Bearing Pressure = 31.7 kN m^{-2} (0.295 Tons ft^{-2})

- therefore OK

16th March, 2021 Sheet of .

Unit K4: Marked B on drg no. 5687/1

DESIGN OF BASEPLATES

to BS 5950 : Part 1

Try a 220 x 220 x 15 mm plate{ for a 80 x 80 x 5.00 SHS Column }

The foundation bearing strength = 0.4 x f_{cu} = 8.00 N mm^{-2}

The plate bending capacity = 1.2 x P_y x t^2 / 6000 = 11.92 N m (for a unit strip)

Check thickness for concentric forces (clause 4.13.2.2) :

Total Factored Load carried by the Column = 26.34 kN

Minimum thickness = { 2.5 x W x 0.7 x a^2 / P_y }$^{1/2}$

= { 1750 x 26.34 x (0.5 x (220 - 80))^2 / 265x 220^2 }$^{1/2}$ = 4 mm

- therefore OK

Check forces due to eccentric loading (clause 4.13.2.3) :

The resulting bending moment = 1.1 kN m

Acting at a distance from the Centre Line of D/2 = 40.0 mm

Therefore, the width of the baseplate in compression = 210.0 mm

Giving, a maximum bearing pressure at the edge of the plate = 1.22 N mm^{-2}

<= 8.00 N mm^{-2}

- therefore OK

The bearing pressure under the column edge = 0.79 N mm^{-2}

The total moment at the Column edge

= {(0.5 x0.79 x (0.5 x(220 - 80))^2)

+ ((1.22 - 0.79) x (0.5 x(220 - 80))^2 / 3) } / 1000

= 2.64 N m (for a unit strip)

<= 11.92 N m (for a unit strip)

- therefore OK

continued .....

16th March, 2021 Sheet of .

Unit K4; Marked B on drg no. 5687/1

DESIGN OF BASEPLATES

to BS 5950 : Part 1

..... continued

SWAY CONDITION

Check forces due to reduced factored load with horizontal force :

Total Reduced Factored Load carried by the Column = 21.97 kN

The bending moment due to the horizontal force = 0.4 kN m

The eccentricity of the thrust = M / W = 0.4 x 1000 / 21.97 = 17.9 mm

Therefore, the width of the baseplate in compression = 220.0 mm

Giving, a maximum bearing pressure at the edge of the plate = 0.68 N mm^{-2}
<= 8.00 N mm^{-2}
- therefore OK

The bearing pressure under the column edge = 0.53 N mm^{-2}

The total moment at the Column edge
= {(0.5 x0.53 x (0.5 x(220 - 80))^2)
+ ((0.68 - 0.53) x (0.5 x(220 - 80))^2 / 3) } / 1000
= 1.54 N m (for a unit strip)
<= 11.92 N m (for a unit strip)
- therefore OK

Check forces due to reduced factored load acting eccentrically :

Total Reduced Factored Load carried at the Column edge = 21.97 kN

The resulting combined bending moments = 1.3 kN m

The eccentricity of the thrust = M / W = 1.3 x 1000 / 21.97 = 57.9 mm

Therefore, the width of the baseplate in compression = 156.2 mm

Giving, a maximum bearing pressure at the edge of the plate = 2.06 N mm^{-2}
<= 8.00 N mm^{-2}
- therefore OK

The bearing pressure under the column edge = 0.79 N mm^{-2}

The total moment at the Column edge
= {(0.5 x0.79 x (0.5 x(220 - 80))^2)
+ ((2.06 - 0.79) x (0.5 x(220 - 80))^2 / 3) } / 1000
= 4.01 N m (for a unit strip)
<= 11.92 N m (for a unit strip)
- therefore OK

Therefore, adopt this 220 x 220 x 15 mm plate

Portswood, Southampton Tel 01703 586461

16th March, 2021 Sheet of .

Unit K4; Marked B on drg no. 5687/1

DESIGN OF FOUNDATIONS

For a 220 x 220 x 15 mm plate supporting a 80 x 80 x 5.00 SHS Column

Total Unfactored Load carried by the Column = 16.73 kN

The depth of the foundation (concrete + hardcore) = 0.350 m

The Ground Bearing Capacity = 40.0 kN m^{-2} (0.373 Tons ft^{-2})

Assuming a 45° angle of spread for the pressure through the foundations:

The load will act on the ground over an area = $\left(\frac{220}{1000} + (2 \times 0.350)\right)^2$ m^2

giving,

a Ground Bearing Pressure = 19.8 kN m^{-2} (0.184 Tons ft^{-2})

- therefore OK

HANDRAIL AND POST DESIGN

Design to BS. 6399 : Part 1 : 1984.

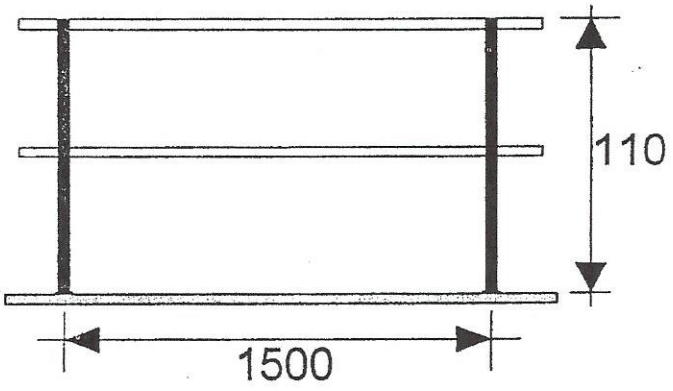
The loadings required are:-

a) a UDL of 0.74 kN m^{-1}

b) a UDL of 1 kN m^{-2}

c) a Point Load of 0.5 kN

of these, case (a) is critical for this geometry.



HANDRAILS

Try a 40 x 40 x 3 mm RHS

This section is classified as plastic.

The allowable deflection = span / 200 = 1500 / 200 = 7.5 mm

The shear capacity, Pv $= 0.6\ Py\ Av$

$$= \frac{0.6 \times 275 \times 4.4}{2 \times 10} = 36.3 \text{ kN}$$

The moment capacity, Mc $= Py\ S \leq 1.2\ Py\ Z$

$= 1.2 \times 275 \times 4.98 \times 10^{-3} = 1.6$ kNm

for UDL of 0.74 kN m- :-

The maximum deflection

$$= \frac{5 \times 0.74 \times 1.5^4 \times 10^5}{384 \times 205 \times 9.96} = 2.4 \text{ mm}$$

-therefore OK

The maximum shear force

$$= \frac{0.74 \times 1.6 \times 1.5}{2} = 0.9 \text{ kN}$$

-therefore OK

The maximum bending moment

$$= \frac{0.74 \times 1.6 \times 1.5^2}{8} = 0.3 \text{ kN}$$

-therefore OK

Therefore, adopt this 40 x 40 x 3 mm section for handrails.

Continued.....

POSTS

Try a 60 x 40 x 3 mm RHS

This section is classified plastic

The shear capacity, Pv $= 0.6\ Py\ Av$

$$= \frac{0.6 \times 275 \times 5.6 \times 0.6}{10} \qquad = 55.4 \text{ kN}$$

The moment capacity, Mc$= 1.2\ Py\ Z$

$$= 1.2 \times 275 \times 8.96 \times 10\text{-}3 \qquad = 2.96 \text{ kNm}$$

for UDL of 0.74 kN m-1 :-

The maximum shear force$= 0.74 \times 1.6 \times 1.5$ $= 1.8$ kN
-therefore OK

The maximum bending moment
$= 0.74 \times 1.6 \times 1.5 \times 1.1$ $= 2.0$ kN
-therefore OK

The maximum deflection $= \frac{0.74 \times 1.5 \times 1.1^3 \times 10^5}{3 \times 205 \times 26.9}$ $= 8.9$ mm

- Which exceeds the 6.1 mm allowed for a structural cantilever but is acceptable for this application.

Therefore, adopt this 60 x 40 x 3 mm RHS for the posts.

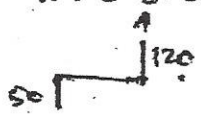
Burman Davison & Associates 40 Bishops Close Stratford upon Avon CV37 9ED Tel: 01789 295577	Project: ASTRO-MEZZ.				Job Ref. 4425	
	Part of Structure				Sheet no./rev. 1	
	Calc. by: RB	Date: 02/10	Check by	Date	App'd by	Date
Ref.	Calculations				Output	

STAIRTREADS 1000 SPAN.

D.L. 0.25 kN/m² | 0.25
L.L. 2.50 | 5.00
2.75 kN/m² | (5.25)

TREAD 275 WIDE UDL = 0.76 kN/m

M = 0.094 (.18) kNm. Zreqd = 5.25 cm³ (10.1) cm³



Total $Z = \frac{4.120^2}{6}(10^3) + \frac{4.50^2}{6}(10^3) = 9.6\,cm^3 + 1.6\,cm^3 = 11.2\,cm^3$

$f_{bc} = \frac{.094}{11.2} = 8.4\ N/mm^2$ ok

$0.18/11.2 = 16.1\ N/mm^2$ ok

USE 4 mm plate.

P.L 4.5 kN M = 1.125 kNm f_{bc} = 100.4 N/mm² ok.

1200 mm wide M = 1.35 kNm f_{bc} = 120.5 N/mm ok.

STRINGERS 4000 horizontal span.

1000 wide UDL = 5.25 x .5 = 2.63 kN/m M = 5.26 kNm

Zreqd = 29.2 cm³. 150 x 10 plt Z = 37.5 cm³.

∴ 150 x 10 plt ok.

5000 horizontal span M = 8.22 kNm Zreqd = 45.66 cm³

150 x 12 plt = 45.00 cm³. USE 160 x 12 plt.

AT 1200 wide UDL = 3.15 kN/m. 4000 span Zreqd = 35 cm³

150 x 10 stringers.

5000 span Zreqd = 55 cm³. 180 x 12 Z = 65 cm³

180 x 12 stringers