First Floors (UK) Limited
Unit 1A, The Mill Industrial Park, Kings Coughton, Alcester, Warwickshire B49 5QG.
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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 IN7, Daedulus Park.
Date:	15th February 2021
Reference Drawing Nos.	5712
Design Code:	BS 5950
Design Loading:	2.5 kN/sq.m
Clear Underside Height:	2.992 m
Maximum Span Of Main Beams:	2.810 m
Maximum Span Of Secondary Beams:	4.040 m
Decking Material:	38mm high density particle board.

HADLEY Design Suite PRO

HADLEY GROUP

Company:	Tidebank
Project:	5712
Phase:	Mezzanine Floor
Description:	Unit IN7, Daedulus Park
	-
User:	ROGER

First Floors (UK) Ltd
Unit 1a,The Mill Industrial PArk,Alcester
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UltraMEZZ® Single span Inset

British Standards

Design Parameters

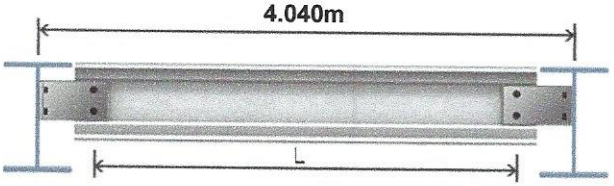
Secondary Beams | **Sheet No:1**

4040 span

Loading inputs			Deflection Criteria
Dead 0.250 kN/m²	Service 0.100 kN/m²	Imposed 2.500 kN/m²	Total Load L/360

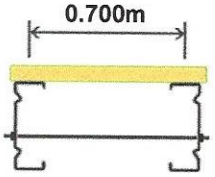
Primary Beam Centres

4.040m



Joist Centres

0.700m



Design Notes

Results

UltraBEAM 2 — **2006312 at 0.700m Centres with 1 Tie Bar at Mid Span.**

Load and Capacities (2006312)

Stress Ratio:	0.69	
Factored Design Load:	4.490	kN/m²
Ultimate Section Capacity:	6.506	kN/m²

Deflection

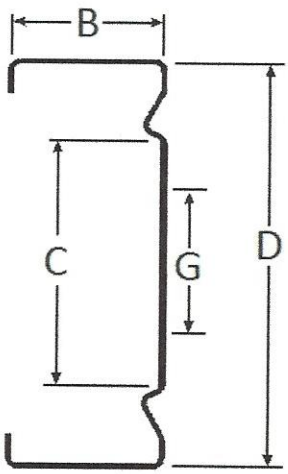
Deflection total load including point load

Deflection	9.55	mm
Span /deflection	L/400	
% of design input	90	

Deflection imposed load only excluding point load

Deflection	8.39	mm
Span L / deflection	L/456	
% of design input	79	

Section Properties



(2006312) Pass

Weight	3.42	kg/m
Depth,D	200	mm
Flange,B	63	mm
Lip	18	mm
Hole Ctrs,G	50	mm

Section modulus

Major axis	25.82	cm³
Minor axis	4.66	cm³

Second moment of area

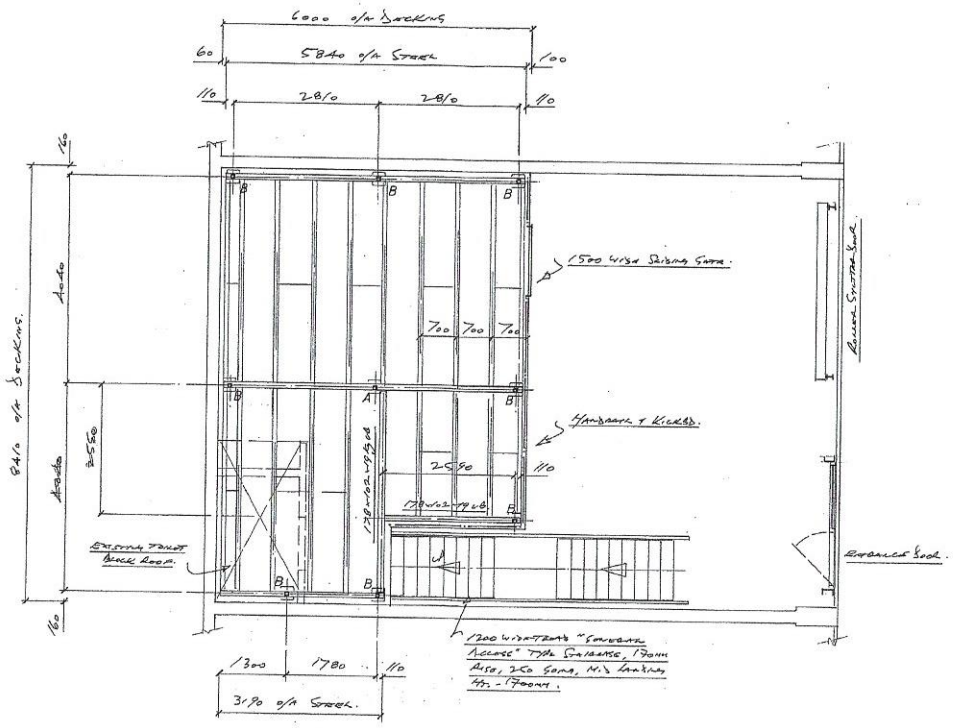
Major axis	258.23	cm⁴
Minor axis	21.29	cm⁴

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.



Proposed Mezzanine Floor Steelwork Layout.

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Revision	Amendments	Date

MEZZANINE FLOOR SPECIFICATION

Design Loading	2.5 Kn/m²
Finished Floor Height	3230 m.
Clear Underside Height	2892 m.
Deck Surface Material	38mm Hi-rib Bonded Phenolic Ply
Mainbeams - Internal	178x102x19 Kg UB
Mainbeams - External	" " "
Columns	100x100x5mm RHS @ "A"
Columns	80x80x5mm RHS @ "B"
Baseplates	300x300x15mm @ "A"
Baseplates	220x220x15mm @ "B".
Secondary Beams	200x63 Universal
Steelwork Colour	Gloss Black
Steelwork Construction	Universal Beams Welded.

FIRE PROTECTION SPECIFICATION

One Hour Fire Rated.

Underside:
600x600x15mm Fireline Plasterboard Ceiling Tiles in an Armstrong Grid System - c/w 4Hr. 2400mm. Min.

Columns:
20mm "Promat" in A Steel Encase in A Matte Anthracite Grey.

Fascia:
15mm "Fireline" Plasterboard.

BUILDING REGULATIONS

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

CUSTOMERS APPROVAL

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

CUSTOMER

TIDEBANK.

Site Address
Unit IN7, Daedulus Park, Daedulus Drive, LEE-ON-THE SOLENT, Hants. PO13 9FX.

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

Date	Scale	Checked by
7.8.17	1-50	
Drawn by Mark.	Drg. no. 5712	

16th March, 2021 Sheet of

Unit IN7: Internal 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 =	0.05 kN m^{-2}	x 1.4 =	0.07 kN m^{-2}	
Total Factored load		=	4.50 kN m^{-2}	

Beam Span = 2.810 m Effective Beam Spacing = 4.040 m

The maximum allowable deflection = $\frac{\text{Span} \times 10^3}{360}$ = 7.8 mm
(clause 2.4.5.1)

The deflection for this section = $\frac{5 \times 2.50 \times 2.810^4 \times 4.040 \times 10^3}{384 \times 205 \times 1360}$

= 2.9 mm
- therefore OK

The total load carried by the beam = 4.50 x 2.810 x 4.040 = 51.08 kN

The maximum bending moment = $\frac{51.08 \times 2.810}{8}$ = 18 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 = $\frac{51.08}{2}$ = 26 kN

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

= 138 kN
- therefore OK

Therefore adopt this 178x102x 19 UB

16th March, 2021 Sheet of .

Unit IN7: Marked A on drg no. 5712

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 =	2.810 m	Beam Spacing Side 1 =	4.040 m
Beam Span Side 2 =	2.810 m	Beam Spacing Side 2 =	4.040 m
Effective Span =	2.810 m	and Effective Spacing =	4.040 m

Column Length to Neutral Axis of Beam = 3.080 m,

therefore, the Effective Length = 1.5 x 3.080 = 4.620 m

The total load carried by the Column = 4.52 x 4.040 x 2.810 = 51.31 kN

The bending moment due to eccentric loading =
51.31 x 100.0 x 2.810 /(4000 x 2.810) = 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 $\frac{F_c}{P_c} + \frac{M}{P_y Z_x} <= 1$

Evenly loaded :

$= \frac{51.31}{226.08} + 0$ = 0.227

- therefore OK

With eccentric loading :

$= 0.5 \times \frac{2.810 \times 51.31}{2.810 \times 226.08} + \frac{1.3 \times 1000}{275 \times 56.6}$ = 0.196 - therefore OK

continued .....

16th March, 2021 Sheet of .

Unit IN7: Marked A on drg no. 5712

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 4.040 x 2.810 = 42.80 kN

The notional horizontal force =
2.5 x 0.01 x (0.52 + 1.6 x 2.50) x 4.040 x 2.810 = 1.28 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 = 1.28 x 3.080 = 4.0 kN m

The bending moment due to eccentric loading =
42.80 x 100.0 x 2.810 /(4000 x 2.810) = 1.1 kN m

Therefore, the worst bending moment = 5.0 kN m

Local Capacity Check :

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

giving,

for evenly loaded column : $\{ 4.0/ 18 \}^{5/3}$ = 0.080
and for eccentrically loaded column : $\{ 5.0/ 18 \}^{5/3}$ = 0.119
- therefore OK

Overall Buckling Check :

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

Evenly loaded :

$= \frac{42.80}{226.08} + \frac{4.0 \times 1000}{275 \times 56.6}$ = 0.443
- therefore OK

With eccentric loading :

$= 0.5 \times \frac{2.810 \times 42.80}{2.810 \times 226.08} + \frac{5.0 \times 1000}{275 \times 56.6}$ = 0.417
- therefore OK

Therefore, adopt this 100x100x5.0 SHS

16th March, 2021 Sheet of .

Unit IN7: Marked B on drg no. 5712

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 = 2.810 m
Beam Span Side 2 = 2.810 m
Effective Span = 2.810 m

Beam Spacing Side 1 = 4.040 m
Length of Cantilever Side 2 = 0.110 m
and Effective Spacing = 2.130 m

Column Length to Neutral Axis of Beam = 3.080 m,

therefore, the Effective Length = 1.5 x 3.080 = 4.620 m

The total load carried by the Column = 4.52 x 2.130 x 2.810 = 27.05 kN

The bending moment due to eccentric loading = 27.05 x 80.0 /2000 = 1.1 kN m

Local Capacity Check :

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

for eccentrically loaded column : $\{ 1.1 / 11 \}^{5/3}$ = 0.021

- therefore OK

Overall Buckling Check :

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

Evenly loaded :

$= \frac{27.05}{117.24} + 0$ = 0.231

- therefore OK

With eccentric loading :

$= \frac{27.05}{117.24} + \frac{1.1 \times 1000}{275 \times 34.7}$ = 0.344

- therefore OK

continued .....

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16th March, 2021 Sheet of

Unit IN7: Marked B on drg no. 5712

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 2.130 x 2.810 = 22.56 kN

The notional horizontal force =
2.5 x 0.01 x (0.52 + 1.6 x 2.50) x 2.130 x 2.810 = 0.68 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 = 0.68 x 3.080 = 2.1 kN m

The bending moment due to eccentric loading = 22.56 x 80.0 /2000 = 0.9 kN m

Therefore, the worst bending moment = 3.0 kN m

Local Capacity Check :

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

giving,

for evenly loaded column : $\{ 2.1/ 11 \}^{5/3}$ = 0.062
and for eccentrically loaded column : $\{ 3.0/ 11 \}^{5/3}$ = 0.114 - therefore OK

Overall Buckling Check :

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

Evenly loaded :

$= \frac{22.56}{117.24} + \frac{2.1 \times 1000}{275 \times 34.7}$ = 0.411 - therefore OK

With eccentric loading :

$= \frac{22.56}{117.24} + \frac{3.0 \times 1000}{275 \times 34.7}$ = 0.505 - therefore OK

Therefore, adopt this 80x80x5.0 SHS

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16th March, 2021 Sheet of .

Unit IN7: Marked A on drg no. 5712

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 = 51.31 kN

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

$= \{ 1750 \times 51.31 \times (0.5 \times (300 - 100))^2 / 265 \times 300^2 \}^{1/2}$ = 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 = 0.57 N mm^{-2}

<= 8.00 N mm^{-2}

- therefore OK

The bearing pressure under the column edge = 0.38 N mm^{-2}

The total moment at the Column edge

$= \{ (0.5 \times 0.38 \times (0.5 \times (300 - 100))^2)$

$+ ((0.57 - 0.38) \times (0.5 \times (300 - 100))^2 / 3) \} / 1000$

= 2.53 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 IN7: Marked A on drg no. 5712

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 = 42.80 kN

The bending moment due to the horizontal force = 0.8 kN m

The eccentricity of the thrust = M / W = 0.8 x 1000 / 42.80 = 18.5 mm

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

Giving, a maximum bearing pressure at the edge of the plate = 0.65 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(300 - 100))^2)
+ ((0.65 - 0.53) x (0.5 x(300 - 100))^2 / 3) } / 1000
= 3.06 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 = 21.40 kN

The resulting combined bending moments = 1.9 kN m

The eccentricity of the thrust = M / W = 1.9 x 1000 / 21.40 = 86.9 mm

Therefore, the width of the baseplate in compression = 189.2 mm

Giving, a maximum bearing pressure at the edge of the plate = 1.42 N mm^{-2}
<= 8.00 N mm^{-2}
- therefore OK

The bearing pressure under the column edge = 0.32 N mm^{-2}

The total moment at the Column edge
= {(0.5 x0.32 x (0.5 x(300 - 100))^2)
+ ((1.42 - 0.32) x (0.5 x(300 - 100))^2 / 3) } / 1000
= 5.25 N m (for a unit strip)
<= 11.92 N m (for a unit strip)
- therefore OK

Therefore, adopt this 300 x 300 x 15 mm plate

16th March, 2021 Sheet of .

Unit IN7: Marked A on drg no. 5712

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 = 32.60 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 = $\left(\frac{300}{1000} + (2 \times 0.350)\right)^2$ m^2

giving,

a Ground Bearing Pressure = 32.6 kN m^{-2} (0.304 Tons ft^{-2})

- therefore OK

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16th March, 2021 Sheet of

Unit IN7: Marked B on drg no. 5712

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 = $0.4 \times f_{cu}$ = 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 = 27.56 kN

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

$= \{ 1750 \times 27.56 \times (0.5 \times (220 - 80))^2 / 265 \times 220^2 \}^{1/2}$ = 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 = 1.28 N mm^{-2}

<= 8.00 N mm^{-2}

- therefore OK

The bearing pressure under the column edge = 0.82 N mm^{-2}

The total moment at the Column edge

$= \{(0.5 \times 0.82 \times (0.5 \times(220 - 80))^2)$

$+ ((1.28 - 0.82) \times (0.5 \times(220 - 80))^2 / 3) \} / 1000$

= 2.76 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 IN7: Marked B on drg no. 5712

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 = 22.99 kN

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

The eccentricity of the thrust = M / W = 0.4 x 1000 / 22.99 = 18.5 mm

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

Giving, a maximum bearing pressure at the edge of the plate = 0.71 N mm^{-2}
<= 8.00 N mm^{-2}
- therefore OK

The bearing pressure under the column edge = 0.56 N mm^{-2}

The total moment at the Column edge
= {(0.5 x0.56 x (0.5 x(220 - 80))^2)
+ ((0.71 - 0.56) x (0.5 x(220 - 80))^2 / 3) } / 1000
= 1.63 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 = 22.99 kN

The resulting combined bending moments = 1.3 kN m

The eccentricity of the thrust = M / W = 1.3 x 1000 / 22.99 = 58.5 mm

Therefore, the width of the baseplate in compression = 154.6 mm

Giving, a maximum bearing pressure at the edge of the plate = 2.21 N mm^{-2}
<= 8.00 N mm^{-2}
- therefore OK

The bearing pressure under the column edge = 0.82 N mm^{-2}

The total moment at the Column edge
= {(0.5 x0.82 x (0.5 x(220 - 80))^2)
+ ((2.21 - 0.82) x (0.5 x(220 - 80))^2 / 3) } / 1000
= 4.28 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

16th March, 2021 Sheet of .

Unit IN7: Marked B on drg no. 5712

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 = 17.51 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 = $\{ \frac{220}{1000} + (2 \times 0.350) \}^2$ m^2

giving,

a Ground Bearing Pressure = 20.7 kN m^{-2}
(0.193 Tons ft^{-2})
- therefore OK

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HANDRAIL AND POST DESIGN

Design to BS. 6399 : Part 1 : 1984.

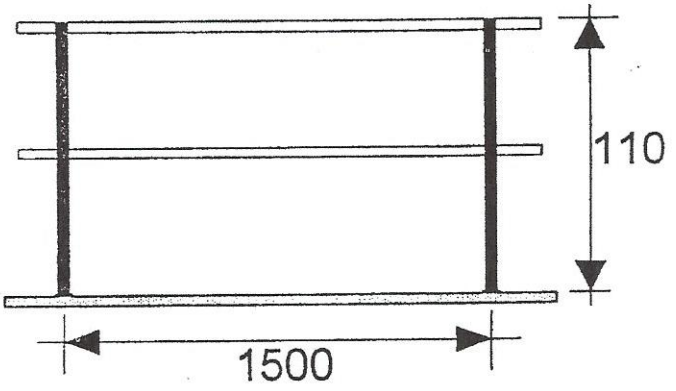
The loadings required are:-

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 $= 0.6\ Py\ Av$

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

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

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

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 49 Bishops Close Stratford upon Avon CV37 9ED Tel: 01789 299577	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 $Z = \frac{4 \cdot 120^2}{6}(10^3) + \frac{4 \cdot 50^2}{6}(10^3) = 9.6\,cm^3 + 1.6\,cm^3 = 11.2\,cm^3$

$f_{bc} = \frac{.094}{11.2} = 8.4$ N/mm² ok

0.18/11.2 = 16.1 N/mm² 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 × .5 = 2.63 kN/m M = 5.26 kNm

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

∴ 150 × 10 plt ok.

5000 horizontal span M = 8.22 kNm Zreqd = 45.66 cm³

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

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

150 × 10 stringers.

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

180 × 12 stringers