Manhattan Loft
Corporation Limited

St Pancras Chambers
and West Wing Hotel

Calculations for Structural
Alterations

Addendum 1

Manhattan Loft Corporation Limited

St Pancras Chambers and West Wing Hotel

Calculations for Structural Alterations

Addendum 1

September 2010

This report takes into account the particular instructions and requirements of our client.
It is not intended for and should not be relied upon by any third party and no responsibility is undertaken to any third party

Job number
118000

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Contents

Addendum 1

1 Barlow House

1.1 New Floors

Two news floors have been introduced into Barlow House, Level One Mezzanine and high level Plant Level. Level One Mezzanine is a new timber floor supported by an array of new steel beams which provides additional hotel space. High level Plant Level replaces the old ceiling to the underside of the roof. This is a concrete deck on Holorib sheeting supported by new steel beams.

Both new floors are supported from the existing masonry structure.

The calculations are as follows which should be read in conjunction with drawings:

Level One Mezzanine: S_6001

Plant Level: S_6002, S_6003, S_6004

OAP20

ARUP | Calculation sheet

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Member/Location

Job Title: ST PANCRAS CHAMBERS.

Drg. Ref.

Made by JKL. Date 08.03.07. Chd.

BARLOW HOUSE LEVEL ONE MEZZANINE LEVEL

Loading

Carpet	0.05
18mm board	0.1
19mm gypsum plank	0.4
battens	0.05
19mm board	0.1
timber joists	0.2
UC steelwork · 2.	0.35
Quilt	0.05
3 layers plasterboard	0.55
	1.85 kN/m²

Partitions
Lightweight on bedrooms and corridor wall lines 3.0 kN/m along beam

LL 2.0 kN/m²

Factored load
= 2.5(1.4×1.8 + 1.6×2.0) + 3.0×1.4 = 18.5 kN/m.

Span ≈ 7.1m ≈ 130 kN

Anchor loads

4 anchors per connection
= 130/(2×4) = 18/16 kN per outer anchor

OAP20

ARUP | Calculation sheet

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Member/Location

Job Title: ST. PANCRAS CHAMBERS

Drg. Ref.

Made by KCo Date 5/3/07 Chd.

BARLOW HOUSE - 1ST FLOOR MEZZ.

- Design steel including vibration checks - ✓
- Detail of timber at opening
- Quick checks for timber design by JB.

Typical Steel Bram:

Max. span = 7.7m

LOADS (from JB; calc. pg 2)

DL = 1.9 kN/m² (includes beam sw)

Assume 1 kN/m² partition weight ← Assume part of dead load } 2.9 kN/m²

LL = 2.0 kN/m² hotel rooms

Trib. width for beams = 2.6 m (at max.)

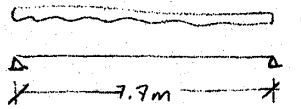
Service load:

w = (2.0 + 2.9) 2.6m = 12.74 kN/m along beam

Factored load:

w = (1.4×2.9 + 1.6×2.0) 2.6m = 18.9 kN/m along beam

- Assume beam is fully laterally restrained at each end (see JB; details)
  Also restrained at 450 o.c. by timber joists:



Service -

$V = \frac{12.74\ kN/m \times 7.7m}{2} = 49\ kN$

$M = \frac{12.74\ kN/m \times 7.7m^2}{8} = 94.4\ kNm$

Factored -

$V = \frac{18.9\ kN/m \times 7.7m}{2} = 72.8\ kN$ (connection force ~18 kN)

$M = \frac{18.9\ kN/m \times 7.7m^2}{8} = 140\ kNm$

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ARUP | Calculation sheet

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Member/Location

Job Title: ST. PANCRAS CHAMBERS

Drg. Ref.

Made by KCo Date 5/3/07 Chd.

BARION HOUSE 1ST MEZZ. CONT.

S355 Steel

Try 203x203x46

n=1 braced at 0.45m o.c.

M_b = 175 kNm > 140 kNm OK

Shear ok by inspection.

Check deflection:

$$\Delta_{TL} = \frac{5wL^4}{384EI} = \frac{5 \times 12.74\ kN/m \times (7.7\ m)^4 \times 10^5}{384 \times 205 \times 10^3\ N/mm^2 \times 4568\ cm^4 \times 10^4} = 62.3\ mm \text{ or } L/124 \text{ NG}$$

Try 203x203x52 I_x = 5259 cm⁴

Δ_{TL} = 54.1 mm or $L/142$ $L/240$ = 32 mm

Would have to use I_x = 8890 min or 203x203x86 I_x = 9449

or 254x254x73 I_x = 11410

Check frequency

for L/240 Δ_{TL} x 3/5 to get Δ_{DL}

$$f = \frac{18}{\sqrt{y}} = \frac{18}{\sqrt{19.2mm}} = 4.4\ Hz$$

pretty good

for live load only $L/360$ = 21 mm

Δ_{LL} = 0.408 Δ_{TL} = 0.408 x 54.1 mm = 22 mm OK

not governed by LL deflection.

USE 203x203x86 $\left(f = \frac{18}{\sqrt{y}} = \frac{18}{\sqrt{15mm}} = 4.65\ Hz\right)$

Even though DL has increased due to beam SW, OK by inspection.

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ARUP | Calculation sheet

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Member/Location

Job Title: ST. PANCRAS CHAMBERS

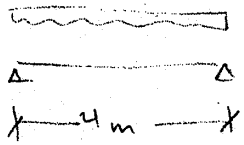
Drg. Ref.

Made by KCo Date 5/3/07 Chd.

BARLOW HOUSE 1ST MEZZ. CONT.

Shorter span case: $L < 4m$

Assume same loading as before



Service -

$V = \frac{12.74 \times 4}{2} = 25.5\ kN$

$M = \frac{12.74 \times 4^2}{8} = 25.5\ kNm$

Factored -

$V = \frac{18.9 \times 4}{2} = 37.8\ kN$

$M = \frac{18.9 \times 4^2}{8} = 37.8\ kNm$

Try 203x203x46 $I_x = 5259$

$A_{TL} = \frac{5 \times 12.74 \times (4m)^4 \times 10^5}{384 \times 205 \times 5259} = 3.94\ mm$ or $L/1000$ OK

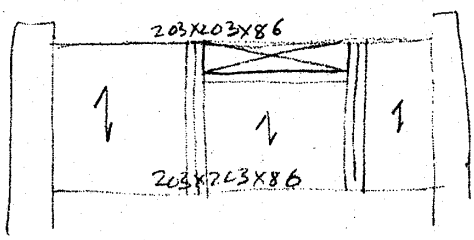
not governed by deflection

$M_b = 175\ kNm >> 37.8\ kNm$

$f = \frac{18}{\sqrt{2.72_{mm}}} = 10.9\ Hz$ OK

USE 203x203x46

Typical Opening



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Member/Location

Job Title: ST. PANCRAS CHAMBERS

Drg. Ref.

Made by KC Date 5/3/07 Chd.

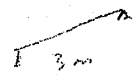
Quick Timber Checks –

JB design

joists at ~18" o.c. ✓

8" deep by 3" wide

what max. span for this design? 2.5m (8.2') ? 3m (~10') 1/15



Design of channel at wall

Trib. width = $\frac{2.6}{2}$ = 1.3m or half of previous loads

Max. span between wall connections = 4m (conservative)

Service	Strength
V = 25.5/2 = 13 kN	V = 37.8/2 = 18.9 kN
M = 25.5/2 = 13 kNm	M = 37.8/2 = 18.9 kNm

Try 203x76x23.82 I_x = 1955 cm^4

M_b = 75 kNm >> 18.9 kNm OK

Δ_{TL} = 11 mm or L/377 OK

Use 203x76x23.82 PFC (min connection @ 4m o.c.)

616

$\frac{5\ kN/m \times 3m^2}{8}$ = 5.63 kNm = 5625 Nm

$\frac{bh^3}{12}$ 16.6

75 × 200 $\frac{5625\ Nm}{5.3\ N/mm^2}$?

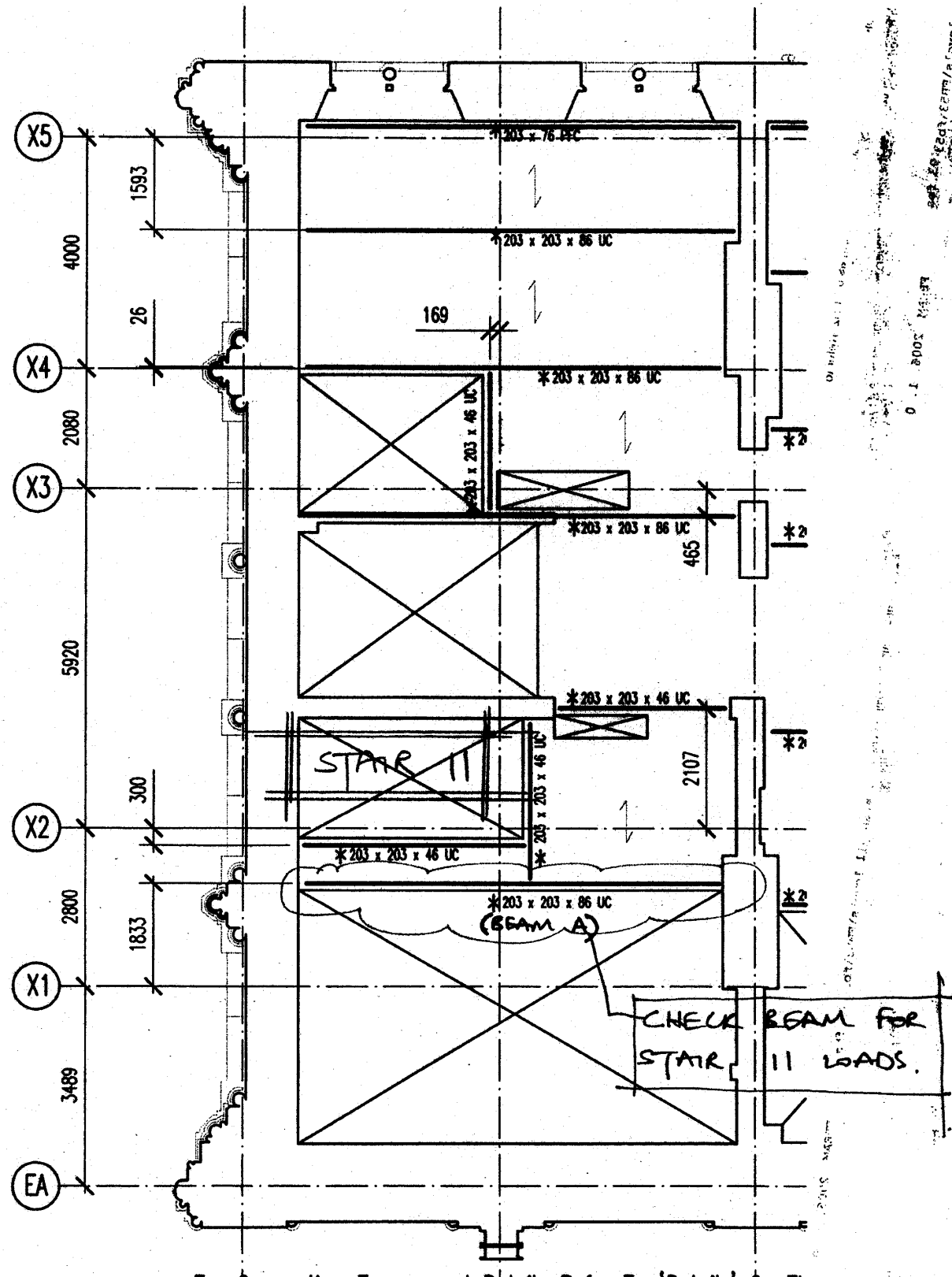
Sxreq = 1.06 × 10^6 mm^3

$I = \frac{75 \times 200^3}{12} = 5.0 \times 10^7\ mm^4$

$S_x = I/y = \frac{5.0 \times 10^7\ mm^4}{100} = 5 \times 10^5\ mm^3$

75 x 225 ?

BARLOW HOUSE - 1ST FLOOR MEZZANINE



For Connection Forces and Details Refer To 'Details' On Th

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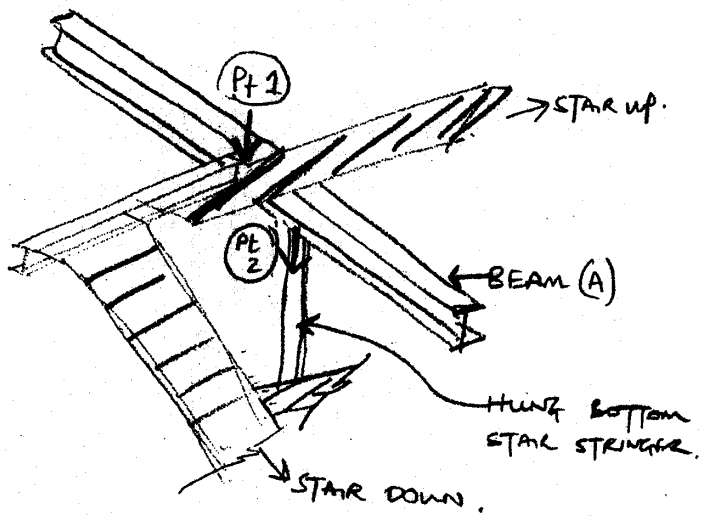
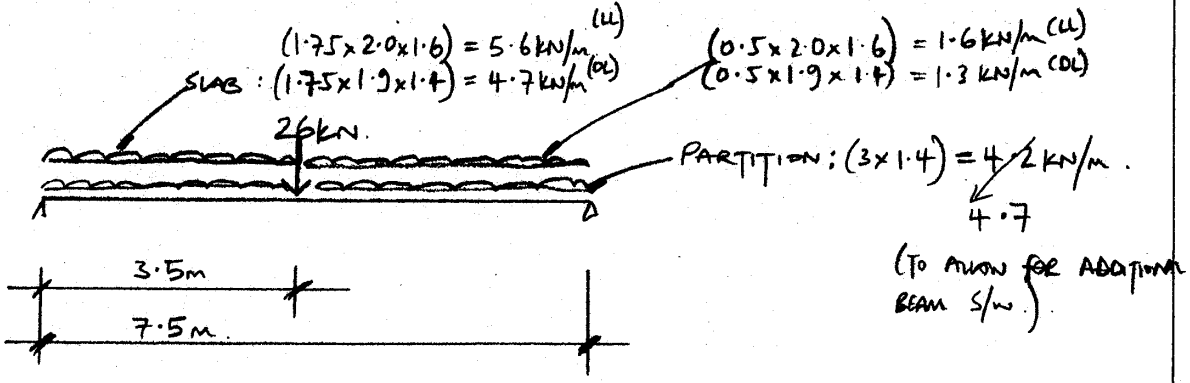
Job Title: ST PANCRAS CHAMBERS.	Drg. Ref.		
	Made by JKL. Date 17/12/07.	Chd.	

BARLOW HOWLE - 1ST FLOOR MEZZ.

A NEW STAIRCASE (STAIR 11) RUNS BETWEEN LEVEL ONE & LEVEL TWO & IS ∴ SUPPORTED OFF THE LEVEL ONE MEZZANINE STEELWORK.



BEAM (A) DESIGN.



Pt 1 STAIR LOAD (2m LOAD WIDTH): (TOTAL) 4m x (1.4 x 2.5 + 1.6 x 4.4) = 42kN.
3m x 3kN/m (PARTITION) = 9kN.
51kN.

Pt 2 = ½ Pt 1. = ½ x 26 = 13kN.

REACTION TO BEAM (A) 51/2 = 26kN + 13 = 39kN.

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BARLOW HOUSE - 1ST FLOOR MEZZ. CONT...

FROM GSA OUTPUT AS FOLLOWS:

MMT : 150 KN-m. (ULS)

SHEAR : 69 KN. (ULS).

DEFLECTION: 42 mm / 1.45 = 29 mm (SLS). TOTAL LOADS.

11 mm (SLS.) LIVE LOADS ONLY.

SPAN/680. ∴ ok

FREQUENCY (RESONANCE) DEADENED BY TOP PARTITION.

203 x 203 x 86 UC. (TRY).

(BASED ON CONSERVATIVE l_e = 3.5m.)

M_b = 292 KN-m.

P_r = 584 KN. ∴ ok.

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St Pancras Chambers

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Made by JKL	Date 17-Dec-2007	Checked

BARLOW HOUSE

LEVEL ONE MEZZ.

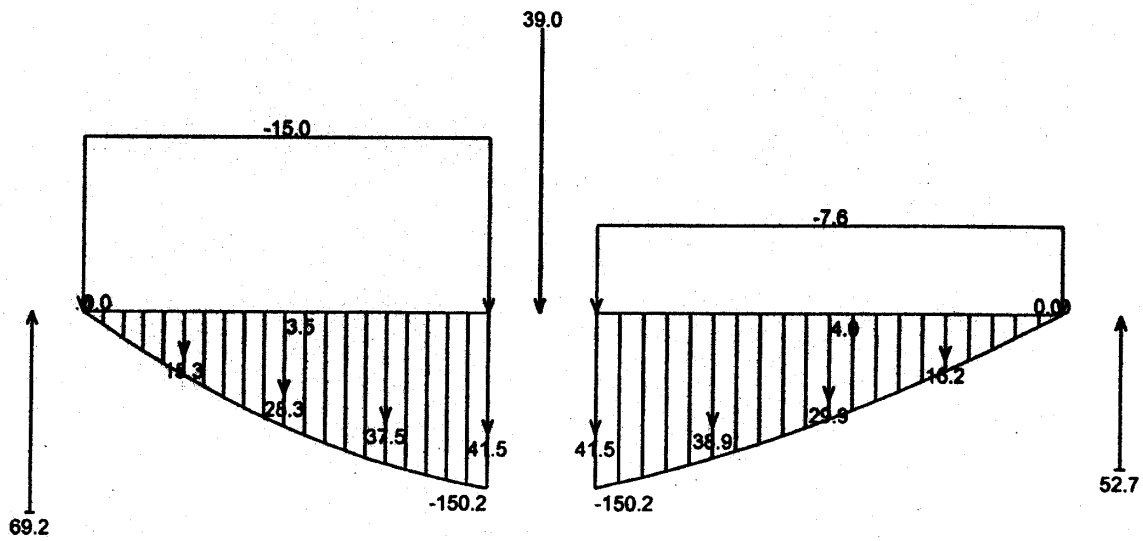
BEAM (A)

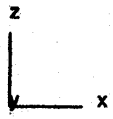
Scale: 1:50.2

Labels:
Element Lengths

Node Loads, Force: 10.0 kN/pic.cm
Beam Loads, Force: 6.3 kN/m/pic.cm
Moment, Myy: 62.5 kNm/pic.cm
Reaction Force, Fx: .0 kN/pic.cm
Reaction Force, Fy: .0 kN/pic.cm
Reaction Force, Fz: 25.0 kN/pic.cm
Resolved Element Translation, |U|: 25.0 mm

Case: L1
Case: A1 "Analysis case 1"





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ARUP | Calculation sheet

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118000	BH/1	

Member/Location

Job Title: ST. PANCRAS CHAMBERS

Drg. Ref.

Made by KCo Date 7/3/07 Chd.

BARLOW HOUSE - ROOF

330 kN/m

- Existing concrete slab will be demolished at roof level & we propose to replace it with steel framing and composite metal deck. This floor supports mechanical equipment including some very heavy chillers.
- As internal walls show signs of cracking, this layout is intended to load internal walls as little as possible and connects steel beams along edge into façade walls.

~~- Typically connections (steel beams into existing masonry) are Type B or C / S 145 and should be a minimum of 450mm away from edge of existing masonry.~~

- In some cases, steel angles are required to support edge of deck (see plan). These will be lightly loaded & can be attached w/ cinter anchors.
- See John Langer's calc. for deck spans & capacities (attached)
- Assume top flange is unrestrained by roofing except in chiller areas where it is restrained by conc. in its final condition.

General Loading

	Beam self weight	0.4 kN/m²
DL	Acoustic ceiling	0.4 kN/m²
	100mm Rigid Insulation	0.3 kN/m²
	Lytag finish	1.6 kN/m²
	80 Deck + 110 NWC fill	3.3 kN/m² + 0.2 kN/m² fill (5.0 kN/m² r)
LL	Mech Equip + Pad	7.5 kN/m²

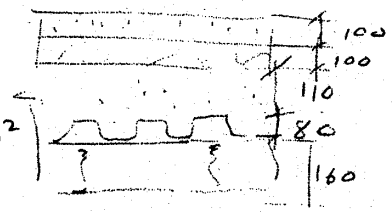
{ DL = 6.2 kN/m²
{ LL = 7.5 kN/m²
5.0

Agreed w/ client deck

~~Steel beam designs use this #~~

See note pg. 2

Roof Buildup



from arch. dwg 20.02.07

~~TYPE 2 SLAB~~

RC SLAB DEPTH = 190mm

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Member/Location

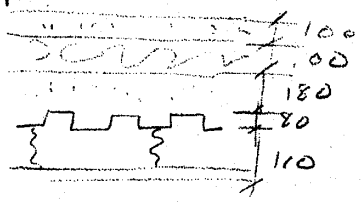
Job Title: ST. PANCRAS CHAMBERS

Drg. Ref.

Made by KCo Date 7/3/07 Chd.

BARLOW HOUSE - ROOF CONT.

TYPE 1 SLAB
RC SLAB DEPTH = 260mm.



Loading at Chillers

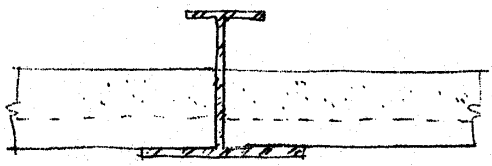
	Beam selfweight	6.4 kN/m²
DL	Acoustic Ceiling	0.4 kN/m²
	100mm Rigid Insul.	0.3 kN/m²
	Lylag Finish	1.6 kN/m²
	80 Deck + 180 NWC slab	5 kN/m²

LL Mech. Equip + Pad 5.0 ~~7.5~~ kN/m²

5 kN/m² Agreed w/ client
Steel Beams will be designed for 7.5 kN/m² but loads taken down structure will use 5 kN/m²

{ DL = 7.7 kN/m²
{ LL = ~~7.5~~ 5.0 kN/m²

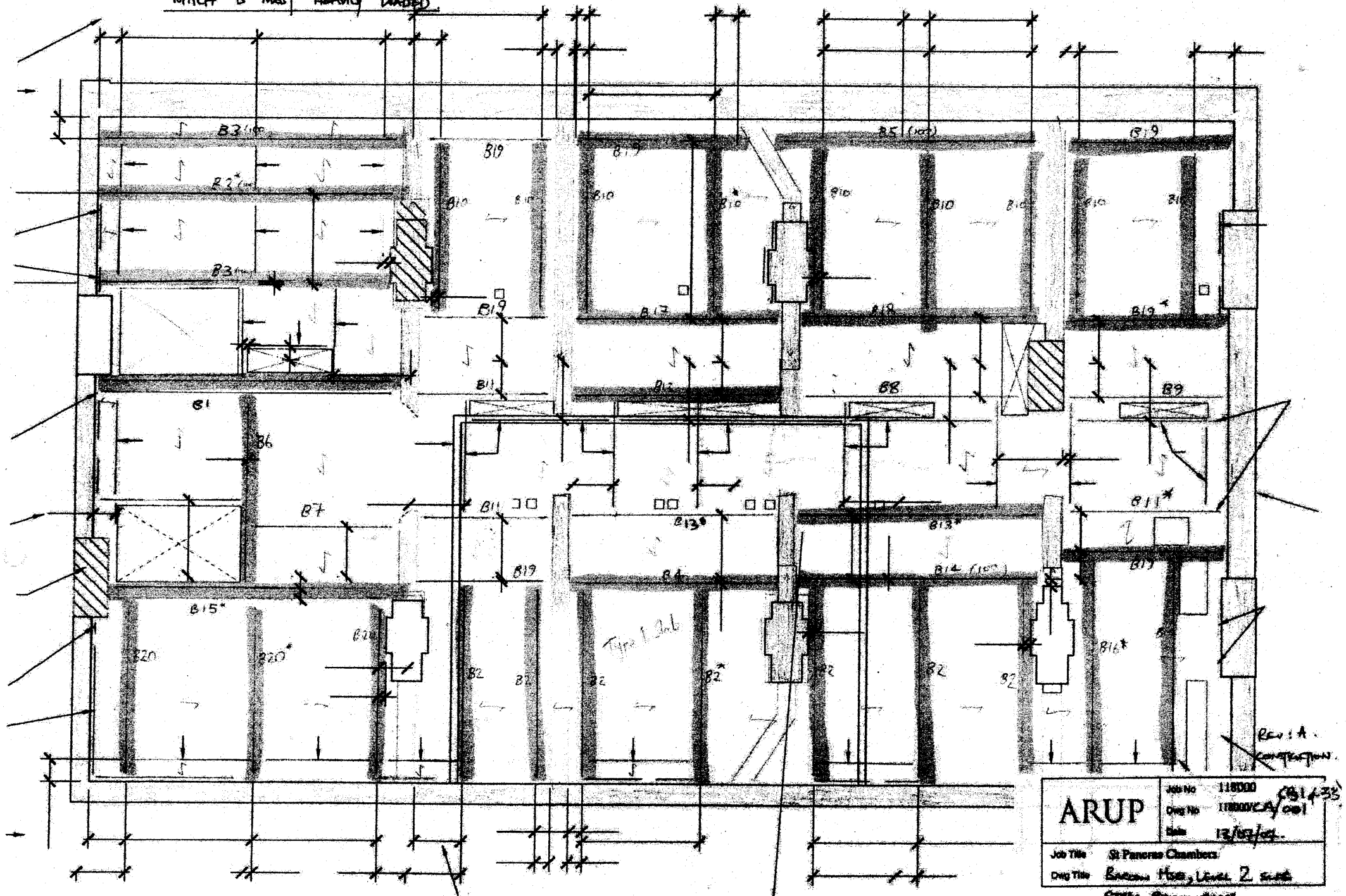
For steel beams, we are using ASB type beams:



See attached plan for locations of beams.

*Assume beams are laterally restrained at connection

NOTE : STARRED * BEAM REFERENCES DENOTE BEAM OF THAT TYPE WHICH IS MOST HEAVILY LOADED.



Rev : A.
CONSTRUCTION.

ARUP	Job No	118000
	Dwg No	118000/CA/001
	Date	13/07/04.
Job Title	St Pancras Chambers	
Dwg Title	Basement, Level 2 Slab Steel Beam Layout.	

CB1433

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ARUP | Calculation sheet

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Member/Location

Job Title: ST PANCRAS CHAMBERS.

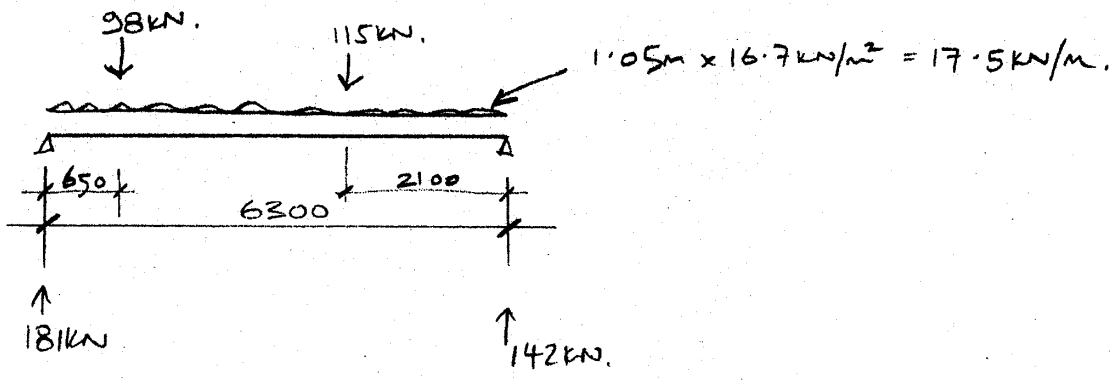
Drg. Ref.

Made by JKL. Date 20/7/07 Chd.

BARLOW HSE (LEVEL 2)

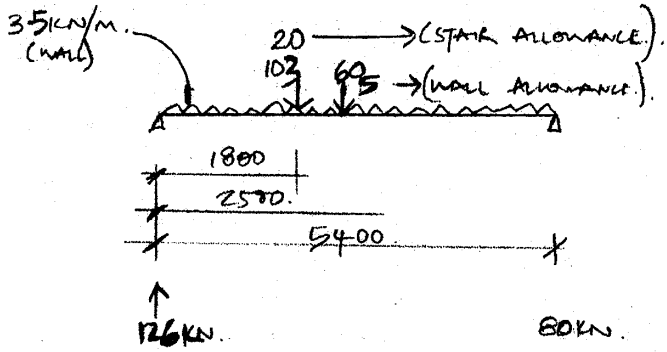
BEAM X3/X4 & E20/E21

B17.



BEAM X2/X3 & E18.

B6



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ARUP | Calculation sheet

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Member/Location

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Made by RHM Date 20 July 07 Chd.

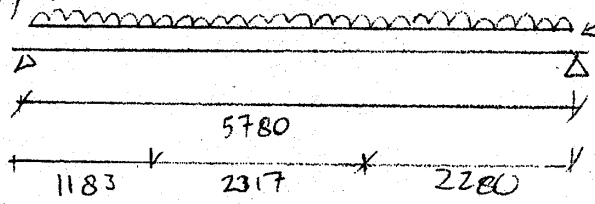
B12

Type 1

(4.1m²) 77kN ↓ (4.1m²) 77kN ↓

17.5kN/m

Type 2



LL Deflection: 5mm (steel only)

Span/1150

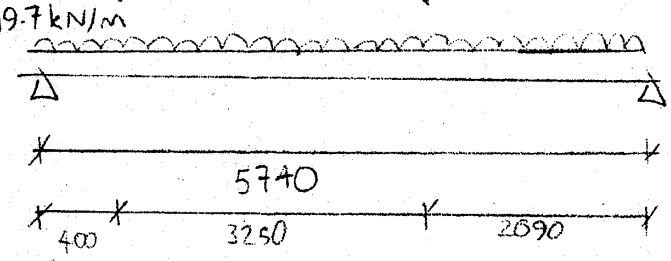
B4

Type 1 slab

(6.2m²) 117kN ↓ (8.73m²) 164kN ↓

Width (line load) from B12

19.7kN/m



LL Defl^n: 6mm (steel only)

Span/957

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ARUP | Calculation sheet

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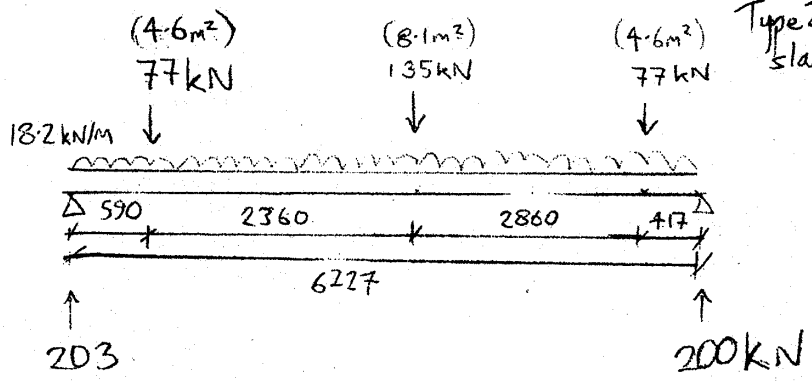
Made by RHS | Date 20 July 07 | Chd.

B1A

Take: Length of B18
Point loads from B14/B2
Line load from B18

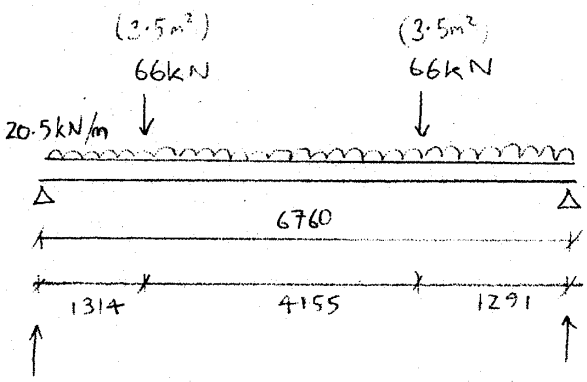


Total Defn under LL: 7mm (steel only)

Span/890

B13

Take: Length of B13
Line load of B8



Total Defn under LL: 6mm (steel only)

Span/1127

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ARUP | Calculation sheet

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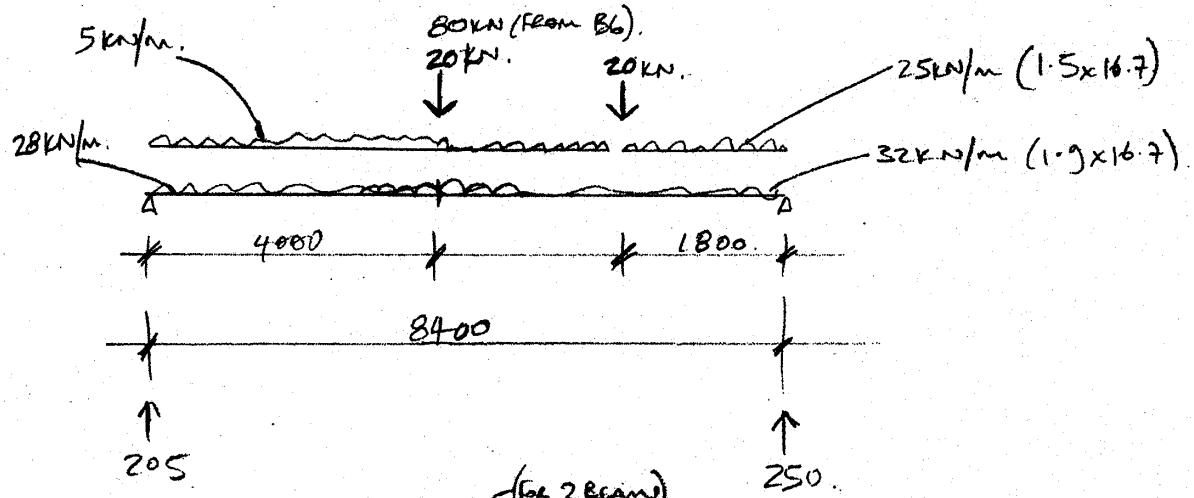
Member/Location

Job Title: ST PANCRAS CHAMBERS.

Drg. Ref.

Made by JW. Date 20/7/07. Chd.

B1. ✓

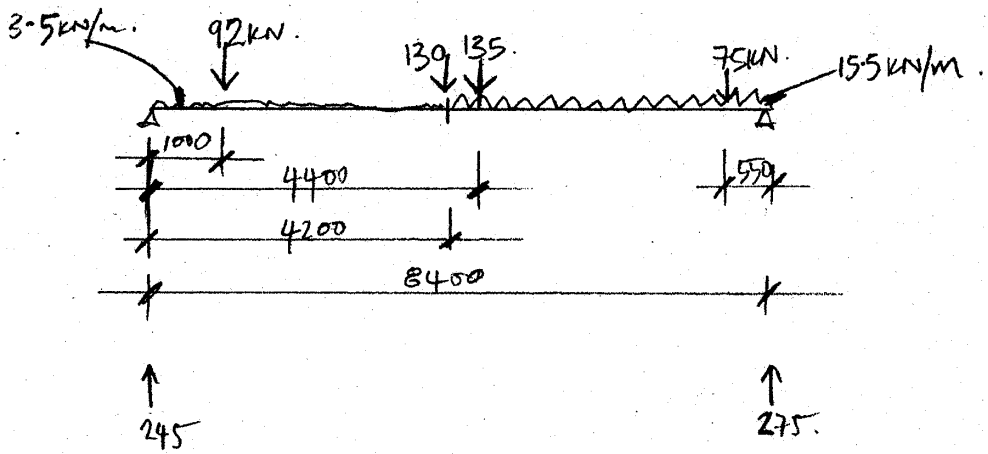


(FOR 2 BEAMS)

TOTAL DEFLECTION 80/1.45 = 55mm (STEEL ONLY) I&I

∴ POUR CONCRETE INFILL PRIOR TO SLABS TO INCREASE I_{yy}. CONCRETE.

B15



TOTAL DEFLECTION. 96/1.45 = 66mm (STEEL ONLY)

corus

ASB Properties

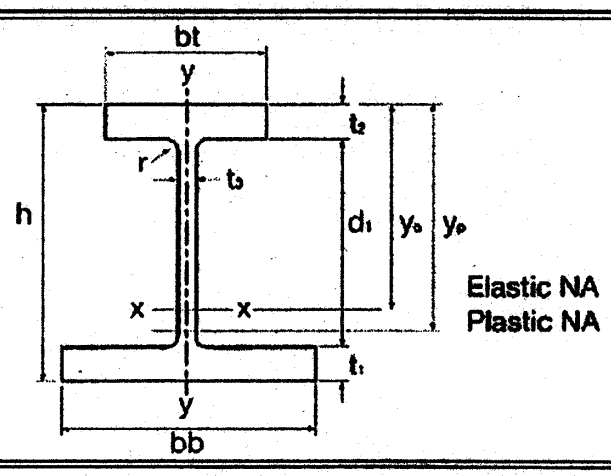


Table 1.4 ASB properties

Designation	Mass	Second moment of area		Radius of gyration		Elastic neutral axis	Elastic modulus			Plastic neutral axis	Plastic modulus		Buckling parameter	Tors in
		I_x	I_y	r_x	r_y	y_e	Z_x Top	Z_x Bottom	Z_y	y_p	S_x	S_y	u	
	kg/m	cm⁴	cm⁴	cm	cm	cm	cm³	cm³	cm³	cm	cm³	cm³	-	
300 ASB(FE)249	249.2	52,920	13194	12.9	6.40	19.2	2757	3526	843	22.6	3760	1513	0.82	6.
300 ASB196	195.5	45,871	10463	13.6	6.48	19.8	2321	3178	714	28.1	3055	1230	0.84	7
300 ASB(FE)185	184.6	35,657	8752	12.3	6.10	18.0	1984	2542	574	21.0	2658	1031	0.82	8
300 ASB155	155.4	34,514	7989	13.2	6.35	18.9	1825	2522	553	27.3	2361	950	0.84	9
300 ASB(FE)153	152.8	28,398	6840	12.1	5.93	17.4	1628	2094	456	20.4	2159	817	0.82	9
300 ASB(FE)136	136.4	22,216	6256	11.3	6.00	16.3	1367	1771	417	19.2	1805	741	0.81	1
300 ASB124	123.9	23,453	6410	12.2	6.37	17.3	1360	1898	445	25.7	1729	761	0.83	1
300 ASB105	104.7	19,249	5298	12.0	6.30	16.8	1145	1606	370	25.3	1440	633	0.83	1
300 ASB(FE)100	100.3	15,506	4245	11.0	5.76	15.6	995	1291	289	18.4	1294	511	0.81	1
280 ASB74	73.6	12,191	3334	11.4	5.96	15.7	776	1062	234	21.3	978	403	0.83	1

Notes: 1. ASB(FE) are fire engineered sections

280 ✓ ✓

PROPERTIES FOR ACB280(74) CHECKED AGAINST BLUE BOOK

Print Close

corus

ASB dimensions for detailing

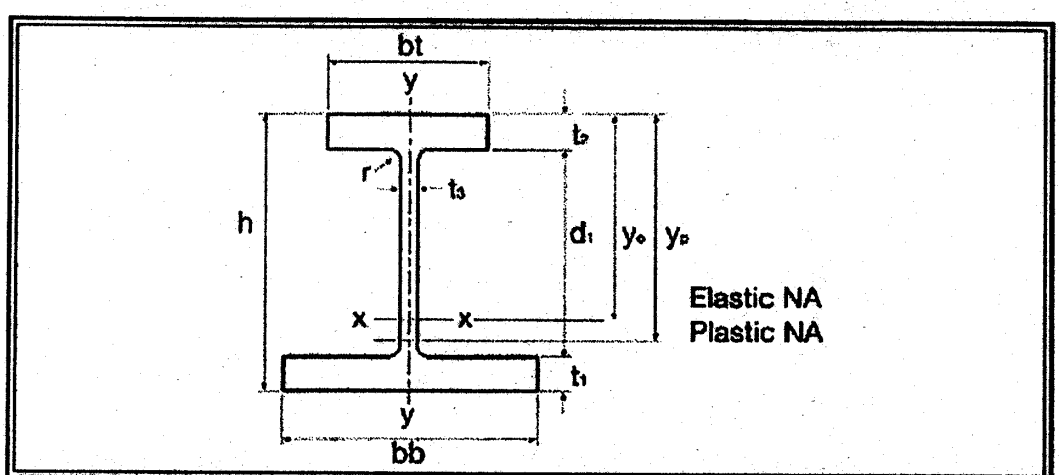


Table 1.3 ASB dimensions for detailing

			Width of Flange		Thickness			Depth between
		Depth	Top	Bottom	Web	Flange	Root radius	Flanges
Designation	Mass	h	bt	bb	t_3	t_1 & t_2	r	d_1
	kg/m	mm	mm	mm	mm	mm	mm	mm
300 ASB(FE)249	249.2	342	203	313	40	40	27	262
300 ASB196	195.5	342	183	293	20	40	27	262
300 ASB(FE)185	184.6	320	195	305	32	29	27	262
300 ASB155	155.4	326	179	289	16	32	27	262
300 ASB(FE)153	152.8	310	190	300	27	24	27	262
280 ASB(FE)136	136.4	288	190	300	25	22	27	244
280 ASB124	123.9	296	178	288	13	26	24	244
280 ASB105	104.7	288	176	286	11	22	24	244
280 ASB(FE)100	100.3	276	184	294	19	16	24	244
280 ASB74	73.6	272	175	285	10	14	24	244

Notes: 1. ASB(FE) are fire engineered sections

Print Close

OUTSTAND ELEMENT OF COMPRESSION FLANGE

$b/T = 88/14 = 6.2 < 7$ ∴ PLASTIC ($\varepsilon = 7$).

$d/t = 244/10 = 24.4 < 35$ ∴ PLASTIC.

B1 = Beam reference

Double 280 ABS 74 = Beam type

Type 2 = Slab type

Input
Properties
Calc
Output

EI =	25010	kNm²
ry =	0.0506	m
x =	157	
Sx =	0.978	x 10³ cm³
u =	0.81	
βw =	1	

Max shear =	248 kN
Temp Max Moment (Mx) =	368 kNm
Final Max Moment (Mx) =	554 kNm

Strength (Final loading)

Mcx =	661 kNm	Pass
Final Mx =	554 kNm	84%
Shear capacity, Pv =	3992 kN	Pass
Max shear, Fv =	248 kN	6%

Lateral Torsional Buckling (temporary loading)

λ = *67.11*

v = *1.11* => mLT = *0.925*

From BS-5950-1 Table 18

λLT = *60.34* => pb = *239* N/mm²

From BS-5950-1 Table 16

Mb / mLT =	505 kNm	Pass
Temporary Mx =	368 kNm	73%

Beam and loading

Span =	8.4 m	Ly =	4 m
Width of loading = N/A	m	DL =	6.2
Final Service Loading =	11.2 kPa	LL (final) =	5
Final Factored Loading =	16.68 kPa	LL (temp) =	1.5
Temporary Factored Loading =	11.08 kPa		
	Load (kN/m)		
Service LL (line) =	N/A		

Deflection =	24 mm	
Span / Δx	350	Pass
Deflection limit =	200	57%

NOTE: SPAN / 200 DEFLECTION CRITERIA SUITABLE FOR THIS CASE DUE TO LACK OF BRITTLE FINISHES/PARTITIONS

CONSTRUCTION.

Job Title St. Pancras Chambers Job number 118000	ARUP
20-Jul-07 RHM	Drawing number: 118000/CA/01

REV : A.

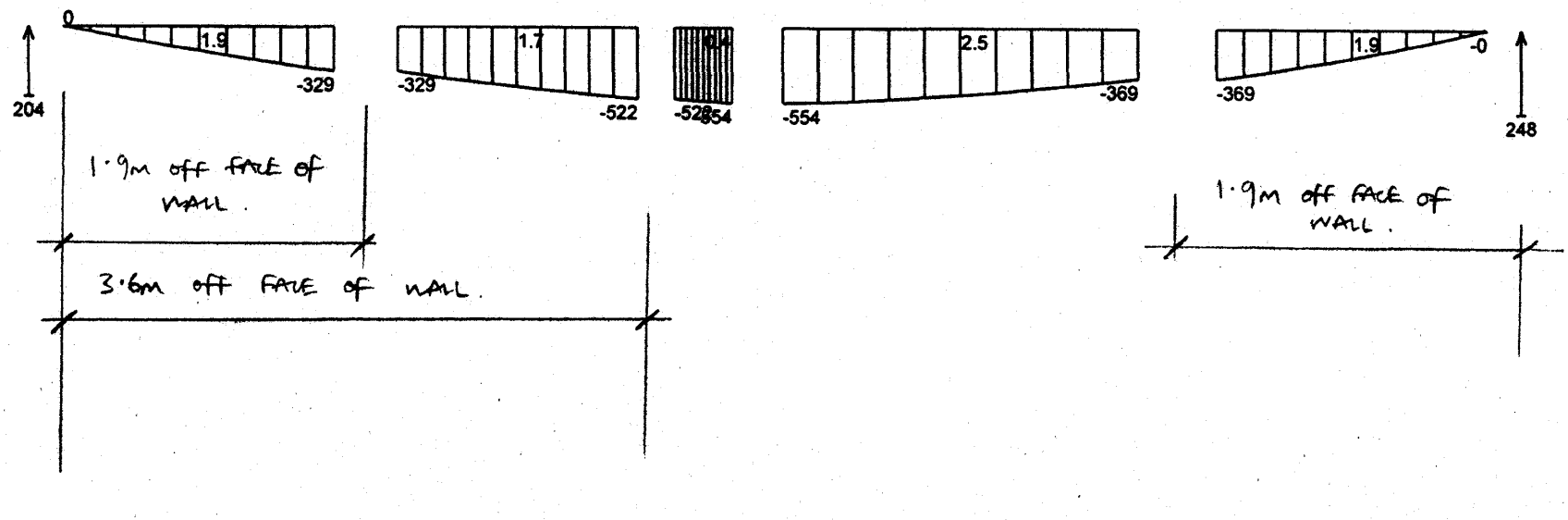
CONSTRUCTION.

ARUP

St Pancras Chambers
Beam B1

Job No.	Sheet No.	Rev. A.
118000	CA/001	
Drg. Ref.	(P3 of 33)	
Made by JKL	Date 17-Jul-2007	Checked

Scale: 1:37.8
Labels:
Element Lengths
Moment, Myy: 500.0 kNm/pic.cm
Reaction Force, Fx: .0 kN/pic.cm
Reaction Force, Fy: .0 kN/pic.cm
Reaction Force, Fz: 200.0 kN/pic.cm
Case: L1
Case: A1 "Analysis case 1"



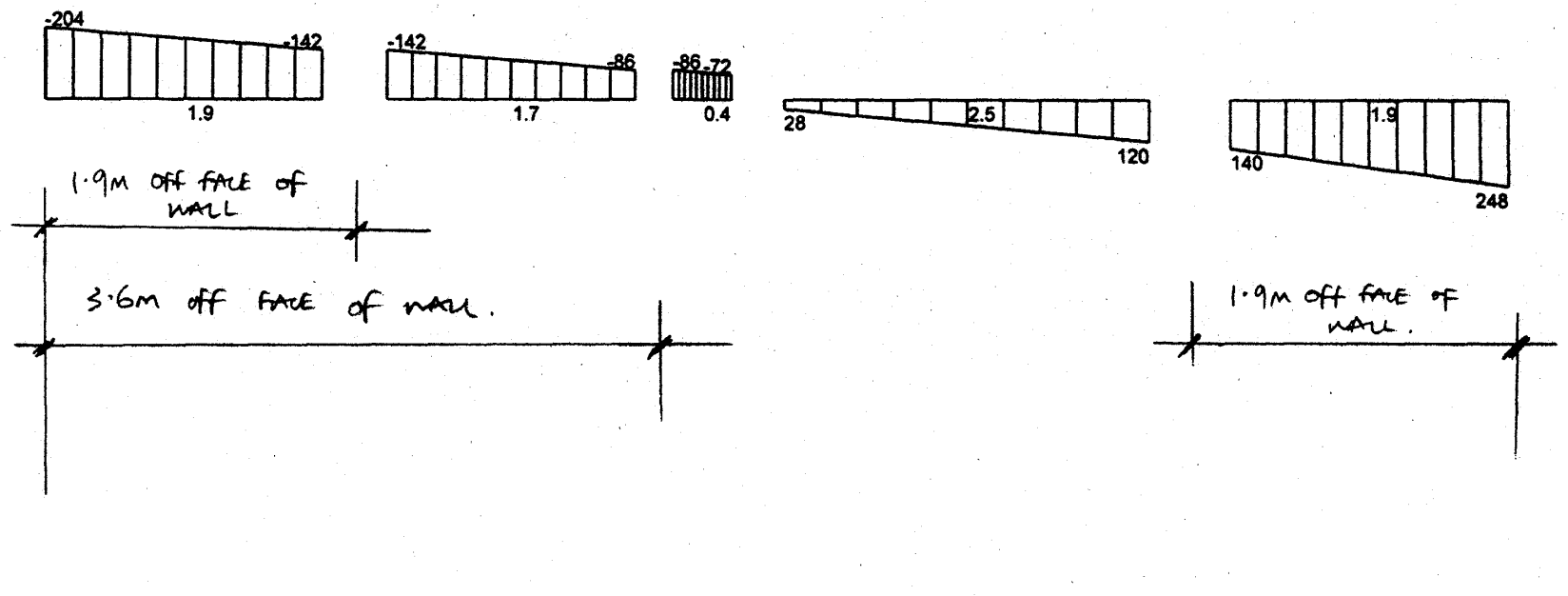
CONSTRUCTION

ARUP

St Pancras Chambers
Beam B1

Job No.	Sheet No..	Rev.
118000	CA/001	A
Drg. Ref.	(Pg 4 of 33)	
Made by JKL	Date 17-Jul-2007	Checked

Scale: 1:37.8
Labels:
Element Lengths
Shear Force, Fz: 200.0 kN/pic.cm
Case: L1
Case: A1 "Analysis case 1"



B2 = Beam reference
280 ABS 74 = Beam type
Type 1 = Slab type

Input
Calc
Output

EI = [redacted] kNm²
ry = [redacted] m
x = [redacted]
Sx = [redacted] x 10³ cm³
u = [redacted]
βw = [redacted]

NOTE: UDL ASSUMED

Max shear =	164 kN
Temp Max Moment (Mx) =	158 kNm
Final Max Moment (Mx) =	225 kNm

Beam and loading

Span =	5.485 m	Ly =	5.485 m
Width of loading =	3.185 m	DL =	7.7
Final Service Loading =	12.7 kPa	LL (final) =	5
Final Factored Loading =	18.78 kPa	LL (temp) =	1.5
Temporary Factored Loading =	13.18 kPa		

Load (kN/m)

Service LL (line) =	15.925

Strength (Final loading)

Mcx =	331 kNm	Pass
Final Mx =	225 kNm	68%
Shear capacity, Pv =	1996 kN	Pass
Max shear, Fv =	164 KN	8%

Deflection =	8 mm	
Span / Δx	731	Pass
Deflection limit =	360	49%

Lateral Torsional Buckling (temporary loading)

λ =	92.03			
v =	0.97	=>	mLT =	0.925
			From BS-5950-1 Table 18	
λLT =	74.09	=>	pb =	205 N/mm²
			From BS-5950-1 Table 16	

Mb / mLT =	217 kNm	Pass
Temporary Mx =	158 kNm	73%

CONSTRUCTION.

Job Title St. Pancras Chambers Job number 118000	ARUP
13-Jul-07 RHM	Drawing number: 118000/CA/01

B3 = Beam reference
280 ABS 74 = Beam type
Type 2 = Slab type

Input
Properties
Calc
Output

EI = [redacted] kNm²
ry = [redacted] m
x = [redacted]
Sx = [redacted] x 10³ cm³
u = [redacted]
βw = [redacted]

Max shear =	136 kN
Temp Max Moment (Mx) =	190 kNm
Final Max Moment (Mx) =	286 kNm

Strength (Final loading)

Mcx =	331 kNm	Pass
Final Mx =	286 kNm	86%
Shear capacity, Pv =	1996 kN	Pass
Max shear, Fv =	136 KN	7%

Lateral Torsional Buckling (temporary loading)

λ = *70.26*
v = *1.11* => mLT = *0.925*
From BS-5950-1 Table 18
λLT = *64.73* => pb = *239* N/mm²
From BS-5950-1 Table 16

Mb / mLT =	253 kNm	Pass
Temporary Mx =	190 kNm	75%

NOTE: UDL ASSUMED

Beam and loading

Span =	8.375 m	Ly =	4.1875 m
Width of loading =	1.954 m	DL =	6.2
Final Service Loading =	11.2 kPa	LL (final) =	5
Final Factored Loading =	16.68 kPa	LL (temp) =	1.5
Temporary Factored Loading =	11.08 kPa		
	Load (kN/m)		
Service LL (line) =	9.77		

Deflection =	25 mm	
Span / Δx	335	Pass
Deflection limit =	200	60%

NOTE: SPAN / 200 DEFLECTION CRITERIA SUITABLE FOR THIS CASE DUE TO LACK OF BRITTLE FINISHES/PARTITIONS

CONSTRUCTION.

Job Title
St. Pancras Chambers
Job number
118000
13-Jul-07
RHM

ARUP

Drawing number:
118000/KA/001
Pg 6 of 33
Rev A

NOTE = FORCES FOR B4 CAN BE ADOPTED CONSERVATIVELY FOR B19.

B4 = Beam reference
280 ABS 74 = Beam type
Type 1 = Slab type

Input
Properties
Calc
Output

EI =	25010	kNm²
ry =	0.0566	m
x =	1.67	
Sx =	0.973	x 10³ cm³
u =	0.83	
βw =	1	

Max shear =	225 kN
Temp Max Moment (Mx) =	218 kNm
Final Max Moment (Mx) =	311 kNm

Strength (Final loading)

Mcx =	331 kNm	Pass
Final Mx =	311 kNm	94%
Shear capacity, Pv =	1996 kN	Pass
Max shear, Fv =	225 KN	11%

Lateral Torsional Buckling (temporary loading)

λ =	*38.93*			
v =	*1.35*	=>	mLT =	*0.925*
			From BS-5950-1 Table 18	
λLT =	*43.62*	=>	pb =	*309* N/mm²
			From BS-5950-1 Table 16	

Mb / mLT =	327 kNm	Pass
Temporary Mx =	218 kNm	67%

Beam and loading

Span =	5.74 m	Ly =	2.32 m
Width of loading =	N/A m	DL =	7.7
Final Service Loading =	12.7 kPa	LL (final) =	5
Final Factored Loading =	18.78 kPa	LL (temp) =	1.5
Temporary Factored Loading =	13.18 kPa		
	Load (kN/m)		
Service LL (line) =	N/A		

Deflection =	11 mm	
Span / Δx	522	Pass
Deflection limit =	360	69%

CONSTRUCTION.

Job Title St. Pancras Chambers Job number 118000	ARUP
23-Jul-07 RHM	Drawing number: 118000/CA/01

Pg 7 of 33
Rev A.

CONSTRUCTION

ARUP

St Pancras Chambers
Barlow Roof steel
B4

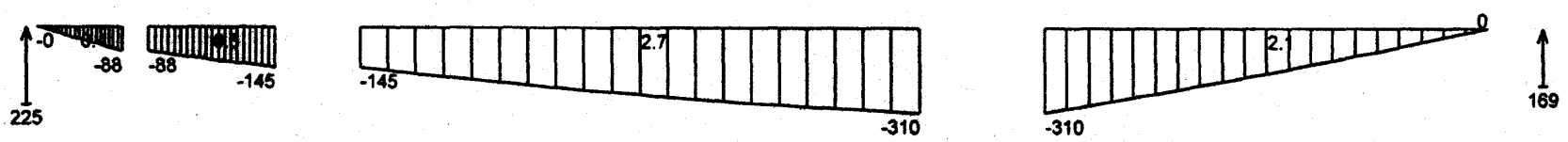
Job No.	Sheet No.	Rev. A
118000-1	CA/001	
Drg. Ref.	(Pg 8 of 33)	
Made by RHM	Date 20-Jul-2007	Checked

Scale: 1:25.9

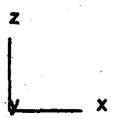
Labels:
Element Lengths

Moment, Myy: 250.0 kNm/pic.cm
Reaction Force, Fx: .0 kN/pic.cm
Reaction Force, Fy: .0 kN/pic.cm
Reaction Force, Fz: 200.0 kN/pic.cm

Case: L1 "Final Factored"
Case: A1 "Final Factored"



1m FROM FACE OF WALL



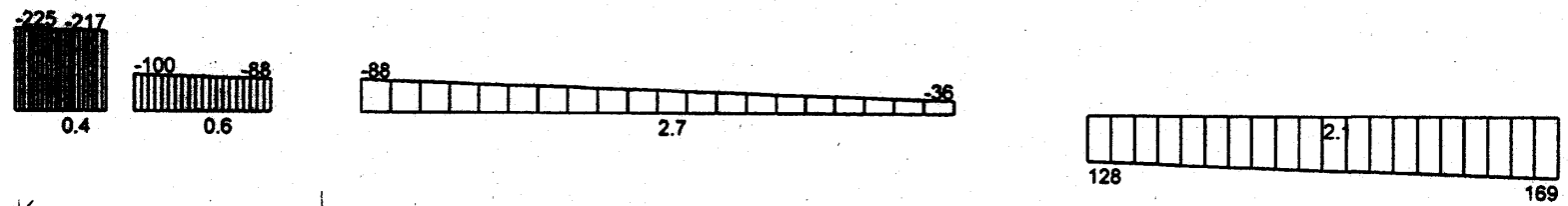
CONSTRUCTION.

ARUP

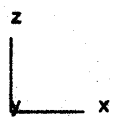
St Pancras Chambers
Barlow Roof steel
B4

Job No.	Sheet No.	Rev.
118000-1	CA/001	A
Drg. Ref.	(Pg 9 of 33)	
Made by RHM	Date 20-Jul-2007	Checked

Scale: 1:25.9
Labels:
Element Lengths
Shear Force, Fz: 200.0 kN/pic.cm
Case: L1 "Final Factored"
Case: A1 "Final Factored"



1m FROM FACE OF WALL



B5 = Beam reference
280 ABS 100 = Beam type
Type 2 = Slab type

Input
Properties
Calc
Output

EI = [redacted] kNm²
ry = [redacted] m
x = [redacted]
Sx = [redacted] x 10³ cm³
u = [redacted]
βw = [redacted]

Max shear =	176 kN
Temp Max Moment (Mx) =	222 kNm
Final Max Moment (Mx) =	335 kNm

Strength (Final loading)

Mcx =	424 kNm	Pass
Final Mx =	335 kNm	79%
Shear capacity, Pv =	2726 kN	Pass
Max shear, Fv =	176 KN	6%

Lateral Torsional Buckling (temporary loading)

λ = *49.65*
v = *1.16* => mLT = *0.925*
From BS-5950-1 Table 18
λLT = *46.65* => pb = 292 N/mm²
From BS-5950-1 Table 16

Mb / mLT =	407 kNm	Pass
Temporary Mx =	222 kNm	55%

Beam and loading

Span =	7.52 m	Ly =	2.86 m
Width of loading = N/A	m	DL =	6.2
Final Service Loading =	11.2 kPa	LL (final) =	5
Final Factored Loading =	16.68 kPa	LL (temp) =	1.5
Temporary Factored Loading =	11.08 kPa		

Load (kN/m)
Service LL (line) = N/A

Deflection =	18 mm	
Span / Δx	418	Pass
Deflection limit =	360	86%

CONSTRUCTION

Job Title St. Pancras Chambers Job number 118000	ARUP
23-Jul-07 RHM	Drawing number: 118000/CA/01

(Pg 10 of 33)
REV A.

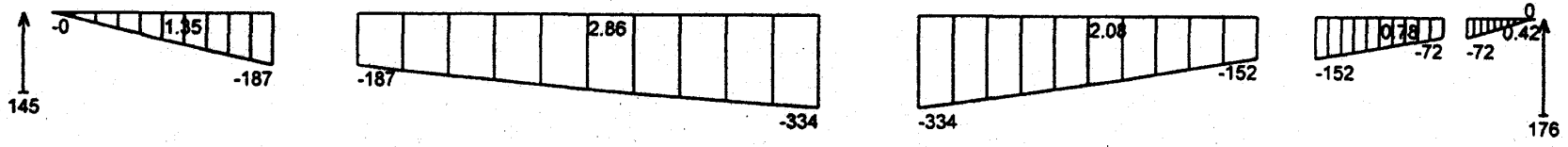
CONSTRUCTION

ARUP

St Pancras Chambers
Barlow Roof steel
B5

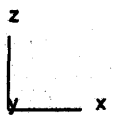
Job No.	Sheet No.	Rev.
118000-1	CA/001	A
Drg. Ref.	(Pg 11 of 33)	
Made by RHM	Date 23-Jul-2007	Checked

Scale: 1:33.73
Labels:
Element Lengths
Moment, Myy: 250.00 kNm/pic.cm
Reaction Force, Fx: .00 kN/pic.cm
Reaction Force, Fy: .00 kN/pic.cm
Reaction Force, Fz: 125.00 kN/pic.cm
Case: L1 "Final Factored"
Case: A1 "Final Factored"



1·35m FROM FACE OF WALL

1·2m FROM FACE OF WALL



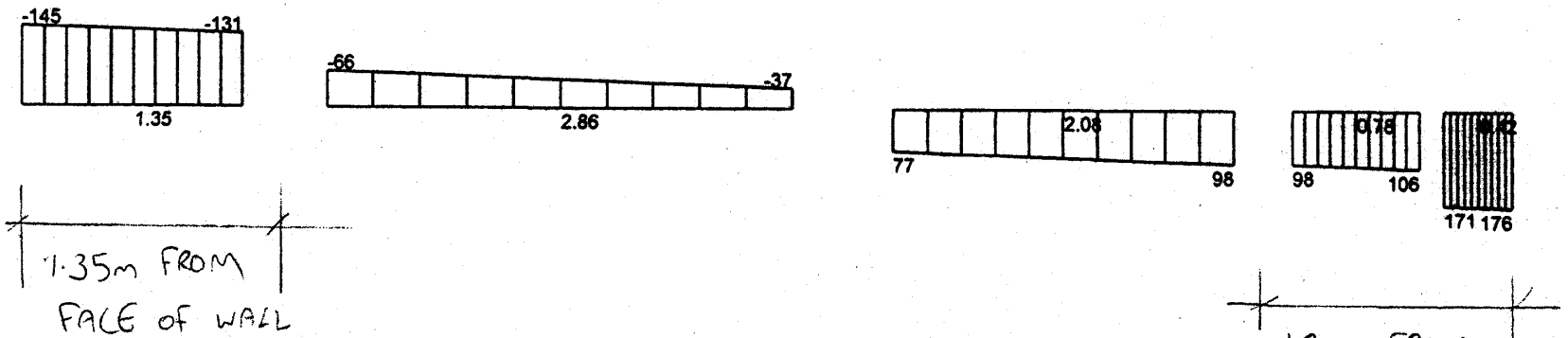
CONSTRUCTION

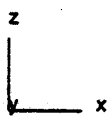
ARUP

St Pancras Chambers
Barlow Roof steel
B5

Job No.	Sheet No.	Rev.
118000-1	CA/001	A
Drg. Ref.	(Pg 12 of 33)	
Made by RHM	Date 23-Jul-2007	Checked

Scale: 1:33.73
Labels:
Element Lengths
Shear Force, Fz: 125.00 kN/pic.cm
Case: L1 "Final Factored"
Case: A1 "Final Factored"





B6 = Beam reference

280 ABS 74 = Beam type

Type 2 = Slab type

Input
Properties
Calc
Output

EI =	[illegible]	kNm²
ry =	[illegible]	m
x =	[illegible]	
Sx =	[illegible]	x 10³ cm³
u =	[illegible]	
βw =	[illegible]	

Max shear =	126 kN
Temp Max Moment (Mx) =	147 kNm
Final Max Moment (Mx) =	222 kNm

Strength (Final loading)

Mcx =	331 kNm	Pass
Final Mx =	222 kNm	67%
Shear capacity, Pv =	1996 kN	Pass
Max shear, Fv =	126 KN	6%

Lateral Torsional Buckling (temporary loading)

λ =	*92.28*			
v =	*0.97*	=>	mLT =	*0.925*
			From BS-5950-1 Table 18	
λLT =	*74.30*	=>	pb =	*205* N/mm²
			From BS-5950-1 Table 16	

Mb / mLT =	217 kNm	Pass
Temporary Mx =	147 kNm	68%

Beam and loading

Span =	5.5 m	Ly =	5.5 m
Width of loading =	N/A m	DL =	6.2
Final Service Loading =	11.2 kPa	LL (final) =	5
Final Factored Loading =	16.68 kPa	LL (temp) =	1.5
Temporary Factored Loading =	11.08 kPa		

Load (kN/m)

Service LL (line) = N/A

Deflection =	8 mm	
Span / Δx	688	Pass
Deflection limit =	360	52%

CONSTRUCTION.

Job Title St. Pancras Chambers	ARUP
Job number 118000	
23-Jul-07 RHM	Drawing number: 118000/CA/01

(Pg 13 of 33)
Rev A.

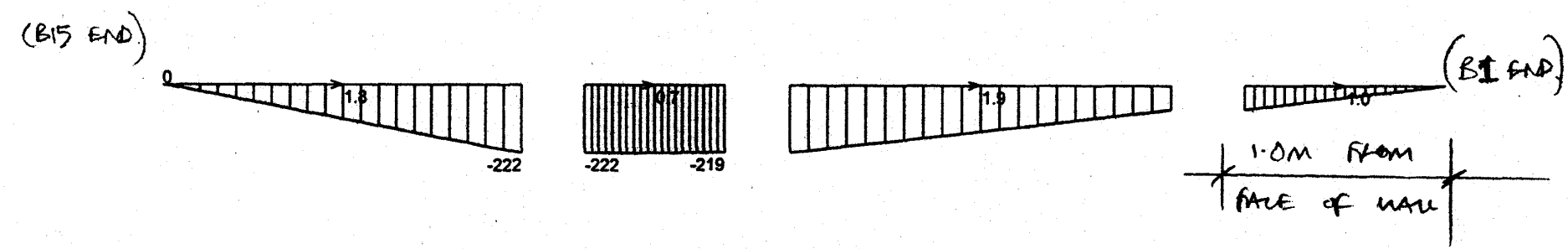
CONSTRUCTION

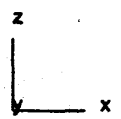
ARUP

St Pancras Chambers
B6

Job No.	Sheet No.	Rev.
118000	CA/001	A
Drg. Ref.	(Pg 14 of 33)	
Made by JKL	Date 17-Jul-2007	Checked

Scale: 1:24.3
Labels:
Element Lengths
Moment, Myy: 200.0 kNm/pic.cm
Case: L1 "Final Factored"
Case: A1 "Final Factored"





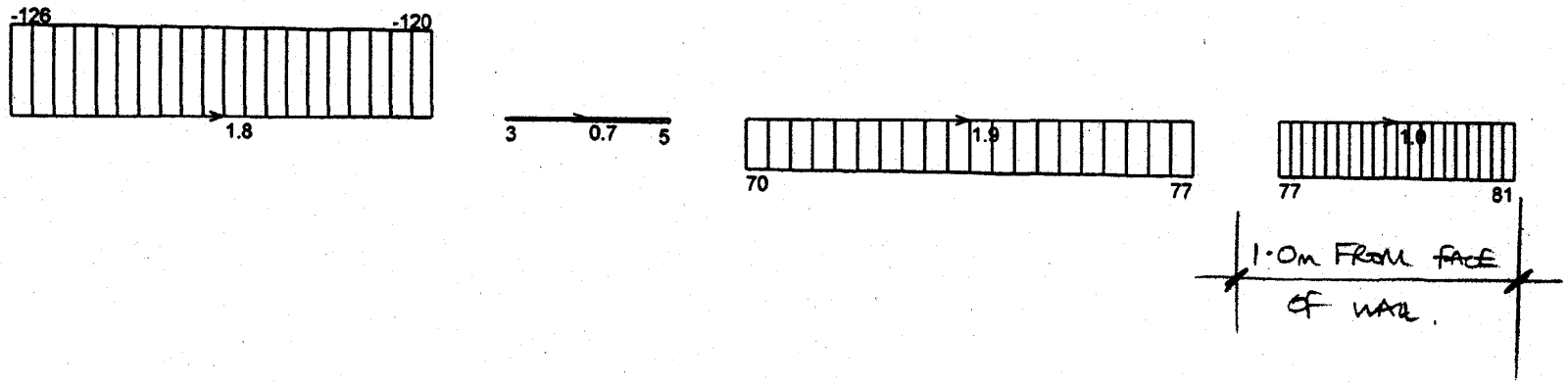
ARUP

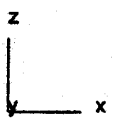
St Pancras Chambers
B6

CONSTRUCTION

Job No.	Sheet No.	Rev. A
118000	CA/001	
Drg. Ref.	(Pg 15 of 33)	
Made by JKL	Date 17-Jul-2007	Checked

Scale: 1:24.3
Labels:
Element Lengths
Shear Force, Fz: 100.0 kN/pic.cm
Case: L1 "Final Factored"
Case: A1 "Final Factored"





B10 = Beam reference
280 ABS 74 = Beam type
Type 2 = Slab type

Input
Properties
Calc
Output

EI = [redacted] kNm²
ry = [redacted] m
x = [redacted]
Sx = [redacted] x 10³ cm³
u = [redacted]
βw = [redacted]

Max shear =	131 kN
Temp Max Moment (Mx) =	106 kNm
Final Max Moment (Mx) =	160 kNm

Strength (Final loading)

Mcx =	331 kNm	Pass
Final Mx =	160 kNm	48%
Shear capacity, Pv =	1996 kN	Pass
Max shear, Fv =	131 kN	7%

Lateral Torsional Buckling (temporary loading)

λ = *81.80*
v = *1.05* => mLT = *0.925*
From BS-5950-1 Table 18
λLT = *71.28* => pb = *205* N/mm²
From BS-5950-1 Table 16

Mb / mLT =	217 kNm	Pass
Temporary Mx =	106 kNm	49%

NOTE: UDL ASSUMED

Beam and loading

Span =	4.875 m	Ly =	4.875 m
Width of loading =	3.225 m	DL =	6.2
Final Service Loading =	11.2 kPa	LL (final) =	5
Final Factored Loading =	16.68 kPa	LL (temp) =	1.5
Temporary Factored Loading =	11.08 kPa		

Load (kN/m)
Service LL (line) = 16.125

Deflection =	5 mm	
Span / Δx	1028	Pass
Deflection limit =	360	35%

CONSTRUCTION

Job Title St. Pancras Chambers Job number 118000 13-Jul-07 RHM	ARUP Drawing number: 118000/CA/01

(Pg 16 of 33)
Rev A.

B12 = Beam reference
280 ABS 74 = Beam type
Type 1 = Slab type

Input
Properties
Calc
Output

EI =	25010 kNm²
ry =	0.0866 m
x =	16.7
Sx =	0.978 x 10³ cm³
u =	0.93
βw =	1

Max shear =	143 kN
Temp Max Moment (Mx) =	146 kNm
Final Max Moment (Mx) =	213 kNm

Strength (Final loading)

Mcx =	331 kNm	Pass
Final Mx =	213 kNm	64%
Shear capacity, Pv =	1996 kN	Pass
Max shear, Fv =	143 KN	7%

Lateral Torsional Buckling (temporary loading)

λ =	38.93			
v =	1.35	=>	mLT =	0.925
			From BS-5950-1 Table 18	
λLT =	43.62	=>	pb =	309 N/mm²
			From BS-5950-1 Table 16	

Mb / mLT =	327 kNm	Pass
Temporary Mx =	146 kNm	45%

Beam and loading

Span =	5.74 m	Ly =	2.32 m
Width of loading = N/A	m	DL =	7.7
Final Service Loading =	12.7 kPa	LL (final) =	5
Final Factored Loading =	18.78 kPa	LL (temp) =	1.5
Temporary Factored Loading =	13.18 kPa		
	Load (kN/m)		
Service LL (line) =	N/A		

Deflection =	10 mm	
Span / Δx	574	Pass
Deflection limit =	360	63%

CONSTRUCTION.

Job Title St. Pancras Chambers Job number 118000 23-Jul-07 RHM	ARUP Drawing number: 118000/CA/01

(Pg 17 of 33)
REV A.

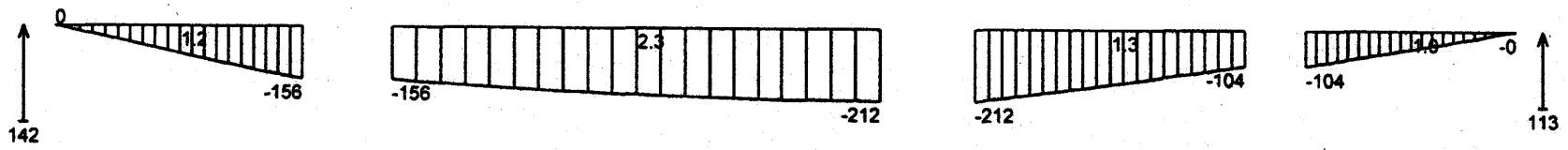
CONSTRUCTION.

ARUP

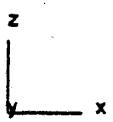
St Pancras Chambers
Barlow Roof steel
B12

Job No.	Sheet No.	Rev.
118000-1	CA/001	A.
Drg. Ref.	(Pg 18 of 33)	
Made by RHM	Date 20-Jul-2007	Checked

Scale: 1:26.0
Labels:
Element Lengths
Moment, Myy: 200.0 kNm/pic.cm
Reaction Force, Fx: .0 kN/pic.cm
Reaction Force, Fy: .0 kN/pic.cm
Reaction Force, Fz: 100.0 kN/pic.cm
Case: L1 "Final Factored"
Case: A1 "Final Factored"



1m FROM FACE OF WALL



ARUP

St Pancras Chambers
Barlow Roof steel
B12

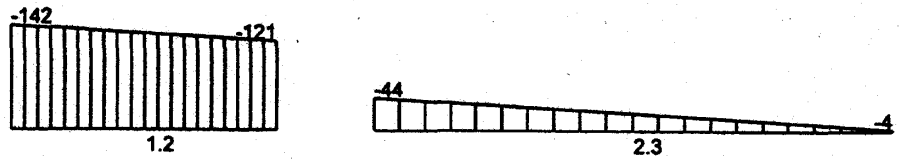
CONSTRUCTION.

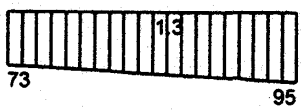
Job No.	Sheet No.	Rev.
118000-1	CA/001	A

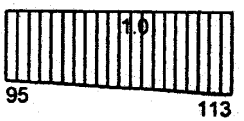
Drg. Ref. (CA 19 of 33)

Made by	Date	Checked
RHM	20-Jul-2007	

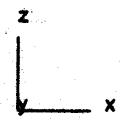
Scale: 1:26.0
Labels:
Element Lengths
Shear Force, Fz: 100.0 kN/pic.cm
Case: L1 "Final Factored"
Case: A1 "Final Factored"







1m FROM FACE OF WALL



NOTE : FORCES FOR B13 CAN BE ADOPTED CONSERVATIVELY FOR B8.

B13 = Beam reference
280 ABS 74 = Beam type
Type 1 = Slab type

Input
Properties
Calc
Output

EI = [redacted] kNm²
ry = [redacted] m
x = [redacted]
Sx = [redacted] x 10³ cm³
u = [redacted]
βw = [redacted]

Max shear =	179 kN
Temp Max Moment (Mx) =	194 kNm
Final Max Moment (Mx) =	277 kNm

Beam and loading

Span =	6.76 m	Ly =	4.155 m
Width of loading = N/A	m	DL =	7.7
Final Service Loading =	12.7 kPa	LL (final) =	5
Final Factored Loading =	18.78 kPa	LL (temp) =	1.5
Temporary Factored Loading =	13.18 kPa		

Load (kN/m)
Service LL (line) = N/A

Strength (Final loading)

Mcx =	331 kNm	Pass
Final Mx =	277 kNm	84%
Shear capacity, Pv =	1996 kN	Pass
Max shear, Fv =	179 KN	9%

Deflection =	12 mm	
Span / Δx	563	Pass
Deflection limit =	360	64%

Lateral Torsional Buckling (temporary loading)

λ = *69.71*
v = *1.11* => mLT = *0.925* From BS-5950-1 Table 18
λLT = *64.23* => pb = *239* N/mm² From BS-5950-1 Table 16

Mb / mLT =	253 kNm	Pass
Temporary Mx =	194 kNm	77%

CONSTRUCTION

Job Title St. Pancras Chambers	ARUP
Job number 118000	
23-Jul-07 RHM	Drawing number: 118000/CA/01

(Pg 20 of 33)
REV A

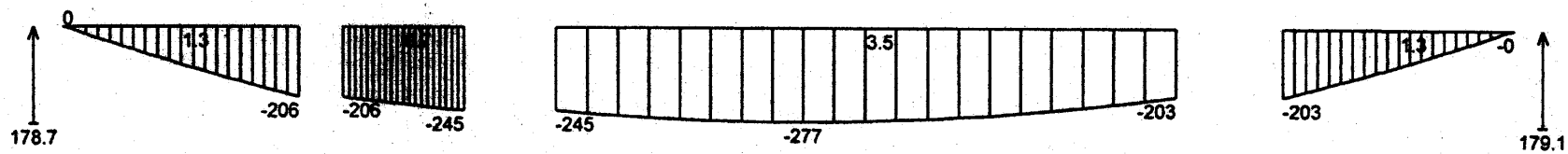
CONSTRUCTION.

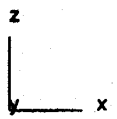
ARUP

St Pancras Chambers
Barlow Roof steel
B13

Job No.	Sheet No.	Rev.
118000-1	CA/007	A
Drg. Ref.	(Pg 21 + 33)	
Made by RHM	Date 23-Jul-2007	Checked

Scale: 1:30.5
Labels:
Element Lengths
Moment, Myy: 200.0 kNm/pic.cm
Reaction Force, Fx: .0 kN/pic.cm
Reaction Force, Fy: .0 kN/pic.cm
Reaction Force, Fz: 125.0 kN/pic.cm
Case: L1 "Final Factored"
Case: A1 "Final Factored"





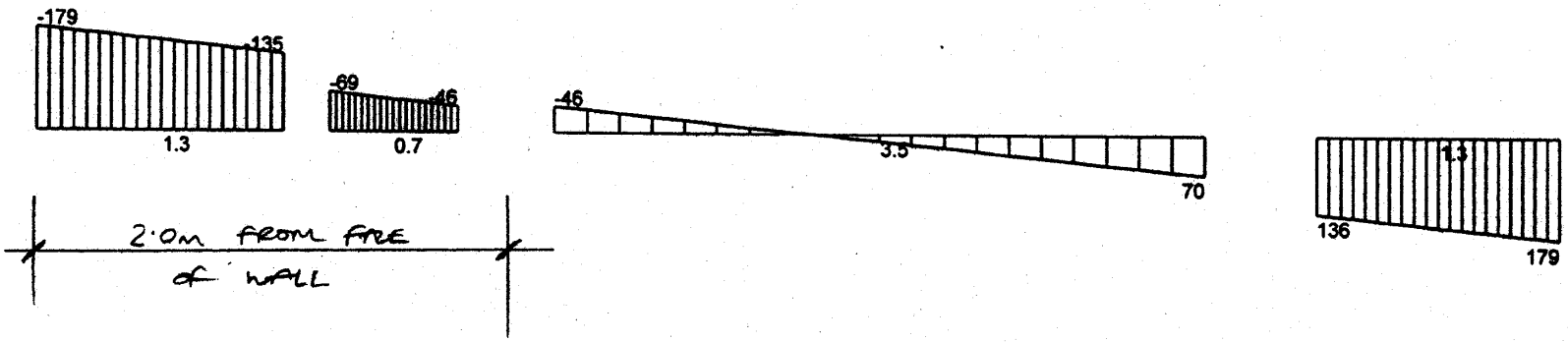
CONSTRUCTION

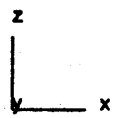
ARUP

St Pancras Chambers
Barlow Roof steel
B13

Job No.	Sheet No.	Rev.
118000-1	CA/001	A
Drg. Ref.	(Pg 22 of 33)	
Made by RHM	Date 23-Jul-2007	Checked

Scale: 1:30.5
Labels:
Element Lengths
Shear Force, Fz: 125.0 kN/pic.cm
Case: L1 "Final Factored"
Case: A1 "Final Factored"





NOTE : FORCES FOR B14 CAN BE ADOPTED CONSERVATIVELY FOR B18

B14 = Beam reference
280 ABS 100 = Beam type
Type 1 = Slab type

Input
Properties
Calc
Output

EI = [redacted] kNm²
ry = [redacted] m
x = [redacted]
Sx = [redacted] x 10³ cm³
u = [redacted]
βw = [redacted]

Max shear =	210 kN
Temp Max Moment (Mx) =	243 kNm
Final Max Moment (Mx) =	369 kNm

Beam and loading

Span =	7.58 m	Ly =	2.86 m
Width of loading =	N/A m	DL =	7.7
Final Service Loading =	12.7 kPa	LL (final) =	5
Final Factored Loading =	18.78 kPa	LL (temp) =	1.5
Temporary Factored Loading =	13.18 kPa		

Load (kN/m)
Service LL (line) = N/A

Strength (Final loading)

Mcx =	424 kNm	Pass
Final Mx =	369 kNm	87%
Shear capacity, Pv =	2726 kN	Pass
Max shear, Fv =	210 KN	8%

Deflection =	16 mm	
Span / Δx	474	Pass
Deflection limit =	360	76%

Lateral Torsional Buckling (temporary loading)

λ = *49.65*
v = *1.16* =>
mLT = *0.925*
From BS-5950-1 Table 18
λLT = *46.65* =>
pb = *292* N/mm²
From BS-5950-1 Table 16

Mb / mLT =	407 kNm	Pass
Temporary Mx =	243 kNm	60%

CONSTRUCTION.

Job Title
St. Pancras Chambers
Job number
118000
23-Jul-07
RHM

ARUP

Drawing number:
118000/CA/01

(Pg 23 of 33)
REV A

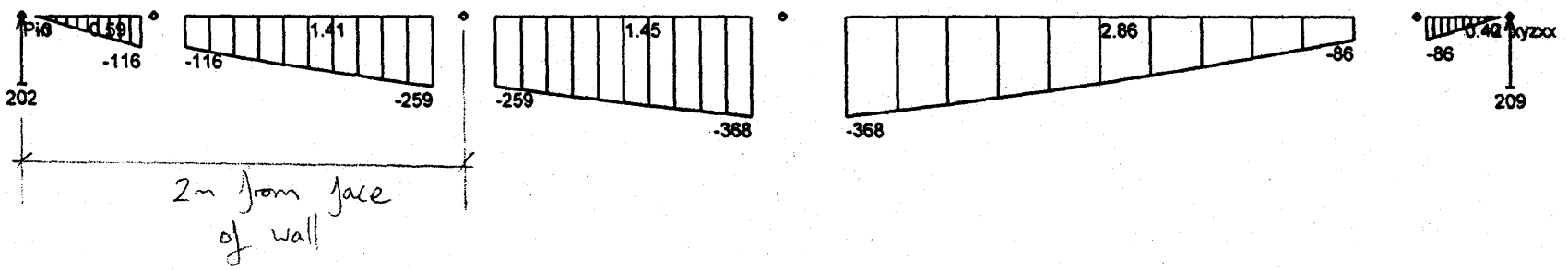
CONSTRUCTION

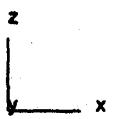
ARUP

St Pancras Chambers
Barlow Roof steel
B14

Job No.	Sheet No.	Rev. A
118000-1	CA/001	
Drg. Ref.	(Pg 24 of 33)	
Made by RHM	Date 20-Jul-2007	Checked

Scale: 1:30.30
Labels:
Element Lengths
Reaction Force, Fz: 200.00 kN/pic.cm
Moment, Myy: 250.00 kNm/pic.cm
Case: L1 "Final Factored"
Case: A1 "Final Factored"





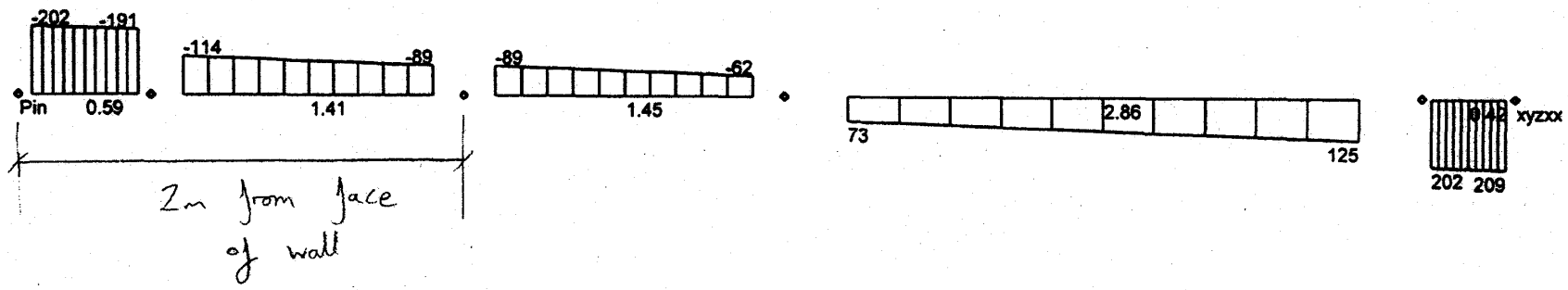
CONSTRUCTION

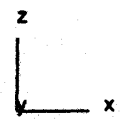
ARUP

St Pancras Chambers
Barlow Roof steel
B14

Job No.	Sheet No.	Rev. A
118000-1	CA/001	
Drg. Ref.	(Pg 25 of 33)	
Made by RHM	Date 20-Jul-2007	Checked

Scale: 1:30.30
Labels:
Element Lengths
Shear Force, Fz: 200.00 kN/pic.cm
Case: L1 "Final Factored"
Case: A1 "Final Factored"





B15 = Beam reference
Double 280 ABS 100 = Beam type
Type 2 = Slab type

Input Properties
Calc
Output

EI = [illegible] kNm²
ry = [illegible] m
x = [illegible]
Sx = [illegible] x 10³ cm³
u = [illegible]
βw = [illegible]

Max shear =	270 kN
Temp Max Moment (Mx) =	455 kNm
Final Max Moment (Mx) =	690 kNm

Strength (Final loading)

Mcx =	848 kNm	Pass
Final Mx =	690 kNm	81%
Shear capacity, Pv =	5453 kN	Pass
Max shear, Fv =	270 KN	5%

Lateral Torsional Buckling (temporary loading)

λ = *72.74*
v = *0.97* => mLT = *0.925*
From BS-5950-1 Table 18
λLT = *57.15* => pb = *257* N/mm²
From BS-5950-1 Table 16

Mb / mLT =	717 kNm	Pass
Temporary Mx =	455 kNm	63%

Beam and loading

Span =	8.38 m	Ly =	4.19 m
Width of loading = N/A	m	DL =	6.2
Final Service Loading =	11.2 kPa	LL (final) =	5
Final Factored Loading =	16.68 kPa	LL (temp) =	1.5
Temporary Factored Loading =	11.08 kPa		

Load (kN/m)
Service LL (line) = N/A

Deflection =	22 mm	
Span / Δx	381	Pass
Deflection limit =	360	95%

CONSTRUCTION

Job Title
St. Pancras Chambers
Job number
118000
23-Jul-07
RHM

ARUP

Drawing number:
118000/CA/01

(Pg 26 of 33)
Rev A.

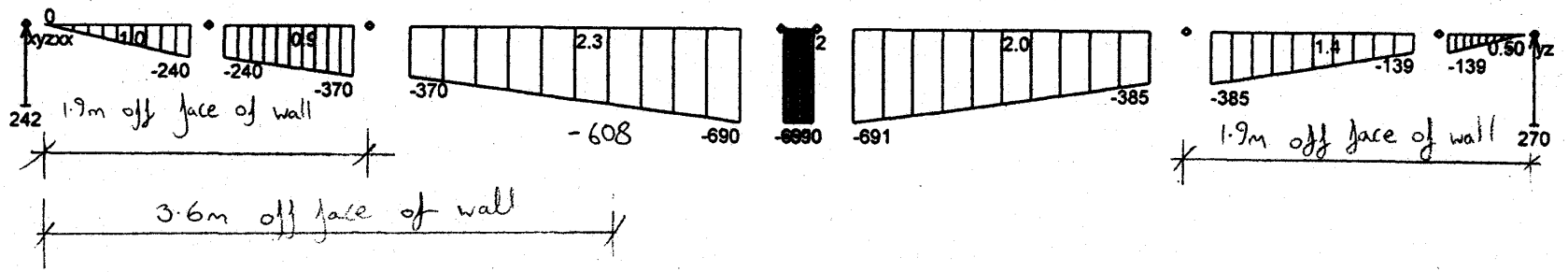
CONSTRUCTION.

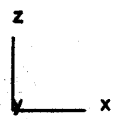
ARUP

St Pancras Chambers
Beam B15

Job No.	Sheet No.	Rev.
118000	CA/001	A
Drg. Ref.	(Pg 27 of 33)	
Made by JKL	Date 20-Jul-2007	Checked

Scale: 1:37.7
Labels:
Element Lengths
Reaction Force, Fx: .0 kN/pic.cm
Reaction Force, Fy: .0 kN/pic.cm
Reaction Force, Fz: 200.0 kN/pic.cm
Moment, Myy: 500.0 kNm/pic.cm
Case: L1
Case: A1 "Analysis case 1"





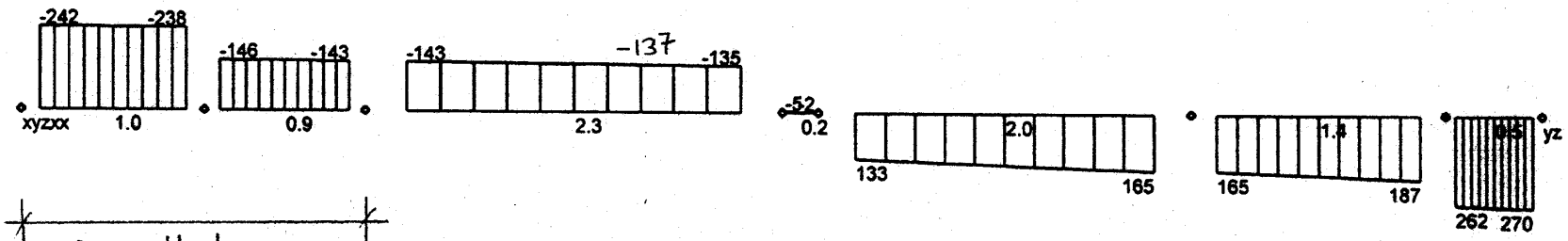
CONSTRUCTION

ARUP

St Pancras Chambers
Beam B15

Job No.	Sheet No.	Rev.
118000	CA/001	A
Drg. Ref.	(Pg 28 of 33)	
Made by JKL	Date 20-Jul-2007	Checked

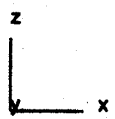
Scale: 1:37.7
Labels:
Element Lengths
Shear Force, Fz: 200.0 kN/pic.cm
Case: L1
Case: A1 "Analysis case 1"



1·9m off face of wall

3·6m off face of wall

1·9m off face of wall



B16 = Beam reference
280 ABS 74 = Beam type
Type 2 = Slab type

Input
Properties
Calc
Output

EI =	25010	kNm²
ry =	0.0596	m
x =	16.7	
Sx =	0.978	x 10³ cm³
u =	0.83	
βw =	1	

NOTE: UDL ASSUMED

Beam and loading

Span =	6.413 m	Ly =	6.413 m
Width of loading =	2.02 m	DL =	6.2
Final Service Loading =	11.2 kPa	LL (final) =	5
Final Factored Loading =	16.68 kPa	LL (temp) =	1.5
Temporary Factored Loading =	11.08 kPa		

Load (kN/m)
Service LL (line) = 10.1

Max shear =	108 kN
Temp Max Moment (Mx) =	115 kNm
Final Max Moment (Mx) =	173 kNm

Deflection =	9 mm	
Span / Δx	721	Pass
Deflection limit =	360	50%

Strength (Final loading)

Mcx =	331 kNm	Pass
Final Mx =	173 kNm	52%
Shear capacity, Pv =	1996 kN	Pass
Max shear, Fv =	108 KN	5%

Lateral Torsional Buckling (temporary loading)

λ = *107.60*
v = *0.93* => mLT = *0.925*
From BS-5950-1 Table 18
λLT = *83.06* => pb = *175* N/mm²
From BS-5950-1 Table 16

Mb / mLT =	185 kNm	Pass
Temporary Mx =	115 kNm	62%

CONSTRUCTION

Job Title	ARUP
St. Pancras Chambers	
Job number	
118000	
13-Jul-07	Drawing number:
RHM	118000/CA/01

(Pg 29 of 33)
Rev A

B17 = Beam reference
280 ABS 74 = Beam type
Type 2 = Slab type

Input
Properties
Calc
Output

EI =	25010	kNm²
ry =	0.0598	m
x =	167	
Sx =	0.078	x 10³ cm³
u =	0.83	
βw =	1	

Max shear =	**181 kN**
Temp Max Moment (Mx) =	**172 kNm**
Final Max Moment (Mx) =	**259 kNm**

Strength (Final loading)

Mcx =	**331 kNm**	**Pass**
Final Mx =	**259 kNm**	**78%**
Shear capacity, Pv =	**1996 kN**	**Pass**
Max shear, Fv =	**181 kN**	**9%**

Lateral Torsional Buckling (temporary loading)

λ =	*105.70*			
v =	*0.93*	=>	mLT =	*0.925*
			From BS-5950-1 Table 18	
λLT =	*81.59*	=>	pb =	*175* N/mm²
			From BS-5950-1 Table 16	

Mb / mLT =	**185 kNm**	**Pass**
Temporary Mx =	**172 kNm**	**93%**

Beam and loading

Span =	6.3 m	**Ly =**	6.3 m
Width of loading =	N/A m	**DL =**	6.2
Final Service Loading =	11.2 kPa	**LL (final) =**	5
Final Factored Loading =	16.68 kPa	**LL (temp) =**	1.5
Temporary Factored Loading =	11.08 kPa		

Load (kN/m)

Service LL (line) =	N/A

Deflection =	**29 mm**	
Span / Δx	**217**	**Pass**
Deflection limit =	**200**	**92%**

NOTE: SPAN / 200 DEFLECTION CRITERIA SUITABLE FOR THIS CASE DUE TO LACK OF BRITTLE FINISHES/PARTITIONS

CONSTRUCTION.

Job Title	ARUP
St. Pancras Chambers	
Job number	
118000	
23-Jul-07	**Drawing number:**
RHM	118000/CA/01

(Pg 30 of 33)
R&A.

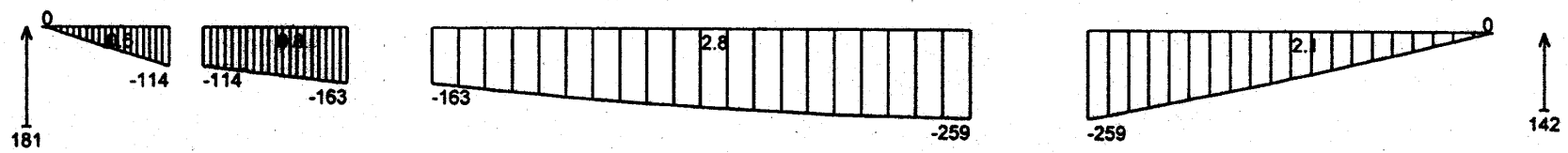
CONSTRUCTION.

ARUP

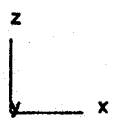
St Pancras Chambers
Barlow House roof steel
B17

Job No.	Sheet No.	Rev.
118000	CA/001	A
Drg. Ref.	CPg 31 of 33	
Made by JKL	Date 16-Jul-2007	Checked

Scale: 1:28.4
Labels:
Element Lengths
Moment, Myy: 200.0 kNm/pic.cm
Reaction Force, Fx: .0 kN/pic.cm
Reaction Force, Fy: .0 kN/pic.cm
Reaction Force, Fz: 125.0 kN/pic.cm
Case: L1
Case: A1 "Analysis case 1"



1·4m FROM FACE OF WALL



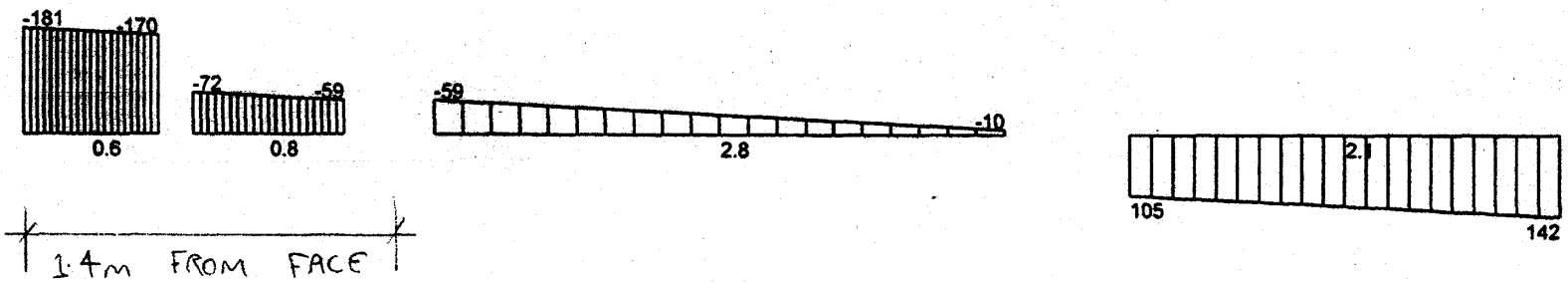
CONSTRUCTION.

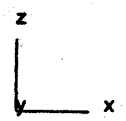
ARUP

St Pancras Chambers
Barlow House roof steel
B17

Job No.	Sheet No.	Rev.
118000	CA/001	A
Drg. Ref.	(Pg 32 of 33)	
Made by JKL	Date 16-Jul-2007	Checked

Scale: 1:28.4
Labels:
Element Lengths
Shear Force, Fz: 125.0 kN/pic.cm
Case: L1
Case: A1 "Analysis case 1"





B18 = Beam reference

280 ABS 74 = Beam type

Type 2 = Slab type

Input
Properties
Calc
Output

EI =	[redacted]	kNm²
ry =	[redacted]	m
x =	[redacted]	
Sx =	[redacted]	x 10³ cm³
u =	[redacted]	
βw =	[redacted]	

Max shear =	187 kN
Temp Max Moment (Mx) =	217 kNm
Final Max Moment (Mx) =	328 kNm

Strength (Final loading)

Mcx =	331 kNm	Pass
Final Mx =	328 kNm	99%
Shear capacity, Pv =	1996 kN	Pass
Max shear, Fv =	187 KN	9%

Lateral Torsional Buckling (temporary loading)

λ =	*47.99*			
v =	*1.29*	=>	mLT =	*0.925*
			From BS-5950-1 Table 18	
λLT =	*51.38*	=>	pb =	*274* N/mm²
			From BS-5950-1 Table 16	

Mb / mLT =	290 kNm	Pass
Temporary Mx =	217 kNm	75%

Beam and loading

Span =	7.58 m	Ly =	2.86 m
Width of loading = N/A	m	DL =	6.2
Final Service Loading =	11.2 kPa	LL (final) =	5
Final Factored Loading =	16.68 kPa	LL (temp) =	1.5
Temporary Factored Loading =	11.08 kPa		
	Load (kN/m)		
Service LL (line) =	N/A		

Deflection =	8 mm	
Span / Δx	948	Pass
Deflection limit =	360	38%

Job Title
St. Pancras Chambers
Job number
118000
13-Jul-07
RHM

ARUP

Drawing number:
118000/CA/01

NOTE: FORCES FOR B20 CAN BE + OPTED CONSERVATIVELY FOR .7.

B20 = Beam reference
280 ABS 74 = Beam type
Type 2 = Slab type

Input
Properties
Calc
Output

NOTE: UDL ASSUMED

EI =	25010	kNm²
ry =	0.0596	m
x =	18.7	
Sx =	0.978	x 10³ cm³
u =	0.83	
βw =	1	

Beam and loading

Span =	4.98 m	Ly =	4.98 m
Width of loading =	3.42 m	DL =	6.2
Final Service Loading =	11.2 kPa	LL (final) =	5
Final Factored Loading =	16.68 kPa	LL (temp) =	1.5
Temporary Factored Loading =	11.08 kPa		
	Load (kN/m)		
Service LL (line) =	17.1		

Max shear =	142 kN	
Temp Max Moment (Mx) =	117 kNm	
Final Max Moment (Mx) =	177 kNm	

Deflection =	5 mm	
Span / Δx	909	Pass
Deflection limit =	360	40%

Strength (Final loading)

Mcx =	331 kNm	Pass
Final Mx =	177 kNm	53%
Shear capacity, Pv =	1996 kN	Pass
Max shear, Fv =	142 kN	7%

Lateral Torsional Buckling (temporary loading)

λ =	*83.56*			
v =	*1.01*	=>	mLT =	*0.925*
			From BS-5950-1 Table 18	
λLT =	*70.05*	=>	pb =	*205* N/mm²
			From BS-5950-1 Table 16	

Mb / mLT =	217 kNm	Pass
Temporary Mx =	117 kNm	54%

CONSTRUCTION

Job Title St. Pancras Chambers Job number 118000 13-Jul-07 RHM	ARUP Drawing number: 118000/CA/01

(Pg 33 of 33)
Rev A.

OAP20

ARUP | Calculation sheet

Job No.	Sheet No.	Rev.
118000.		

Member/Location

Job Title: ST PANCRAS CHAMBERS

Drg. Ref.

Made by JKL. Date 9.07.07. Chd.

BARLOW HOUSE

LEVEL TWO (SLAB)

TRIMMING STEEL

MAX SPAN = 4.0m.

LOADED WIDTH = 1.75m.

ULS (BASED ON DEEPER SLAB.

$= 19 kN/m^2$

$M_{MT} = (1.75 \times 19) \times 4.0^2/8.$

$= 67 kN\text{-}m.$

ADOPT 152UB37.

$M_b = 77.$ BEARING = 64mm ∴ ok

SHEAR $= 2 \times 19$

$= 38 kN.$

DEFLECTION $= 5 \times (5 \times 4 \times 1.75) \times 4000^3/(384 \times 205 \times 2210 \times 10^4)$

$= 6mm$ (LIVE LOAD).

SPAN/360 = 11mm > 6mm ∴ ok

OAP20

ARUP | Calculation sheet

Job No.	Sheet No.	Rev.
118000	2	

Member/Location

Job Title: ST PANCRAS CHAMBERS.

Drg. Ref.

Made by JML. Date 07.03.07. Chd.

BARLOW HOUSE - ROOF

OVER CHILLERS (SLAB REDESIGN.)

		kN/m²	
DL	ceiling	0.5	
	slab.	(from program).	260mm (on 80 DECK)
	finish.	1.6	
LL.		7.5 (PLANT).	

Over Chillers 3.4m span works with no propping.

Limit span to 3.25m. over chillers

Typical - Not over Chillers

LOADS AS ABOVE BUT S/W REDUCED FOR 190mm SLAB on (80 DECK).

Max span with no propping 3.9m

Limit span to 3.5m typical.

RICHARD LEES STEEL DECKING LTD. ASHBOURNE, DERBYSHIRE DE6 1HD
STRUCTURAL & FIRE RATING DESIGN FOR COMPOSITE METAL DECK FLOOR SLABS

Deck Profile	Deck Grade kN/mm²	Slab Depth mm	Conc. Type	Concrete Density kg/m³	Conc. Grade N/mm²	Struc. Span type	Suppt. Width mm	Fire Rtg. mins	Fire Reinf type	Fire Span type
Ribdeck 80 80/1.2	S350	260	NW	2400 Wet 2350 Dry	30	S	100	60	STEEL	S

(0.14 kN/m²) (Approx.Conc.Vol. = 0.233 m³/m² level suppts)

Support crs /Cons.Span /Comp.Span	Span Description	<----Unfactored Loads kN/m²----> Impd.	Parts.	Fins.	Servs.	Cons. Allow.	No.Concentrated Loads Point Loads	Transv. Lin.Lds	Parall. Lin.Lds
3.400m 3.380m 3.400m	(single span)	7.50	0.00	1.60	0.50	1.50	0	0	0

Hard.Flr.Fins.= 0 mm

Fire Rating - Fire Engineering Analysis

Deck Ref.	Construction Type	1 No Top Mesh heavier of Crk.-Cont.	Fire-Reinf	Fire Reinf. Bars bottom	Fact.Load Prop Runner
80/1.2	Single - unpropped	A252(0.1%)	A252	BAR REINFS	0.00 kN/m

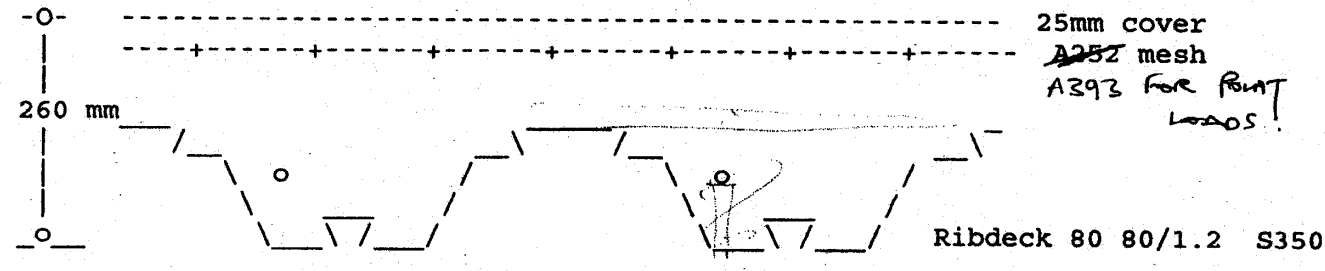
MAX SPANNING CASE (WITH NO PROPPING)

<-------Deck Properties /m width----------> Area mm²	Ixx cm^4	yb mm	Ult.Moms. kNm +ve	-ve	Resid. kN	Pw kN	Vc kN	<--Composite Props. /m width--> Ixx(av) cm^4	Ult.M. kNm	Shr.Bd. kN	Vrt.Shr. kN
1848	237.6	42.5	18.66	15.94	6.89	81.5	114.1	8911	117.43	104.50	83.24

(Span/Depth Limit = 35 Construction Span/Deflect Limit = 130)

Design Calculations -- Applied / (Resistance) Values (per Metre width) Construction Stage			Composite Stage		
Deck Deflection	23.67	(26.00)mm	Super Load Defl.	0.71	(9.71)mm
Int.Reac.Web Bklg	0.00	(81.50)kN	Tot.Imp.Ld.Defl.	0.91	(13.60)mm
Neg.Support Mom.	0.00	(15.94)kNm	Comp.Bding Mom.	32.74	(117.43)kNm
Pos.Bending Mom.	14.68	(18.66)kNm	Comp.Shear Bond.	25.40	(104.50)kN
Web Bklg.End.Sppt	17.37	(32.60)kN	Comp.Slab Shear	38.52	(83.24)kN
Web Shear at Reac.	17.37	(114.10)kN	Fire Bdg Moment	21.84	(24.03)kNm

Es=205 kN/mm² Mod.R.= 10 mr=204.4 kr= 0.1562



|<-------300mm crs------->| T10 Bars @ 300.0 eff crs
(50.0mm side 50.0mm bottom cover bar centre)

Suggested mesh - A393(0.2%) for effective distribution of high applied loads.

General Notes
(1) Design of Composite Metal Deck Floors in accordance with BS 5950 Pts 4 & 6
(2) Fire Design based on S.C.I. Public.056 Fire Eng.Method
(3) Refer to full printout for detailed calculations
(4) Refer to RLSD Technical Dept. for designs beyond the scope of Deckspan

Arup Group Ltd 13 Fitzroy St W1T 4BQ	Job No : 118000 Location: Roof Barlow House Project : St Pancras Calcs By: JKL

BARLOW HOUSE ROOF

ST. PANCRAS 7/03/07

RICHARD LEES STEEL DECKING LTD. ASHBOURNE, DERBYSHIRE DE6 1HD
STRUCTURAL & FIRE RATING DESIGN FOR COMPOSITE METAL DECK FLOOR SLABS

Deck Profile	Deck Grade kN/mm²	Slab Depth mm	Conc. Type	Concrete Density kg/m³	Conc. Grade N/mm²	Struc. Span type	Suppt. Width mm	Fire Rtg. mins	Fire Reinf type	Fire Span type
Ribdeck 80 80/1.2 (0.14 kN/m²)	S350	190	NW	2400 Wet 2350 Dry	30	S	100	60	STEEL	S

(Approx.Conc.Vol. = 0.167 m³/m² level suppts)

Support crs /Cons.Span /Comp.Span	Span Description	<----Unfactored Loads kN/m²----> Impd.	Parts.	Fins.	Servs.	Cons. Allow.	No.Concentrated Loads Point Loads	Transv. Lin.Lds	Parall. Lin.Lds
3.900m	(single span)	7.50	0.00	1.60	0.50	1.50	0	0	0
3.880m							Hard.Flr.Fins.=		0 mm
3.900m									

MAX SPANNING CAPACITY (WITH NO PROPPING)

Fire Rating - Fire Engineering Analysis

Deck Ref.	Construction Type	1 No Top Mesh Crk.-Cont.	heavier of Fire-Reinf	Fire Reinf. Bars bottom	Fact.Load Prop Runner
80/1.2	Single - unpropped	A193(0.1%)	A193	BAR REINFS	0.00 kN/m

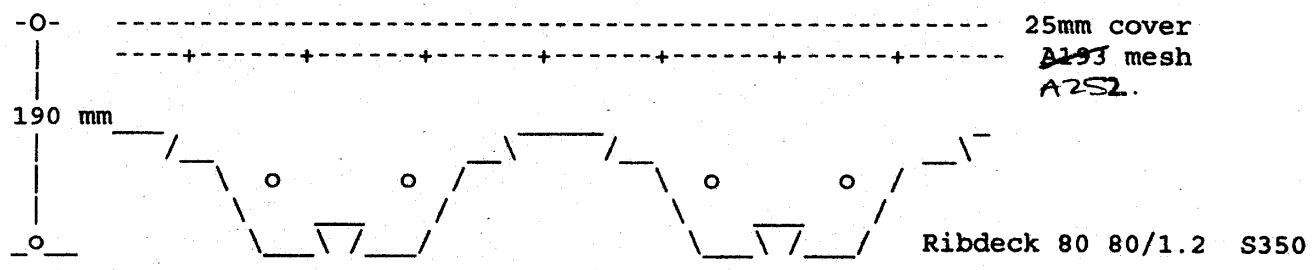
<-------Deck Properties /m width----------> Area mm²	Ixx cm^4	yb mm	Ult.Moms. kNm +ve	-ve	Resid. kN	Pw kN	Vc kN	<--Composite Props. /m width--> Ixx(av) cm^4	Ult.M. kNm	Shr.Bd. kN	Vrt.Shr. kN
1848	237.6	42.5	18.66	15.94	6.89	81.5	114.1	3586	75.32	64.15	70.81

(Span/Depth Limit = 35 Construction Span/Deflect Limit = 130)

Design Calculations -- Applied / (Resistance) Values (per Metre width)

Construction Stage			Composite Stage		
Deck Deflection	29.72	(29.85)mm	Super Load Defl.	3.07	(11.14)mm
Int.Reac.Web Bklg	0.00	(81.50)kN	Tot.Imp.Ld.Defl.	3.93	(15.60)mm
Neg.Support Mom.	0.00	(15.94)kNm	Comp.Bding Mom.	39.01	(75.32)kNm
Pos.Bending Mom.	15.23	(18.66)kNm	Comp.Shear Bond.	29.13	(64.15)kN
Web Bklg.End.Sppt	15.70	(32.60)kN	Comp.Slab Shear	40.01	(70.81)kN
Web Shear at Reac.	15.70	(114.10)kN	Fire Bdg Moment	25.83	(31.84)kNm

Es=205 kN/mm² Mod.R.= 10 mr=204.4 kr= 0.1562



Suggested mesh - A252(0.2%) for effective distribution of high applied loads.

General Notes
(1) Design of Composite Metal Deck Floors in accordance with BS 5950 Pts 4 & 6
(2) Fire Design based on S.C.I. Public.056 Fire Eng.Method
(3) Refer to full printout for detailed calculations
(4) Refer to RLSD Technical Dept. for designs beyond the scope of Deckspan

Arup Group Ltd 13 Fitzroy St W1T 4BQ	Job No : 118000 Location: Roof Barlow House Project : St Pancras Calcs By: JKL

RICHARD LEES STEEL DECKING LTD. ASHBOURNE, DERBYSHIRE DE6 1HD
STRUCTURAL & FIRE RATING DESIGN FOR COMPOSITE METAL DECK FLOOR SLABS

Deck Profile	Deck Grade kN/mm²	Slab Depth mm	Conc. Type	Concrete Density kg/m³	Conc. Grade N/mm²	Struc. Span type	Suppt. Width mm	Fire Rtg. mins	Fire Reinf type	Fire Span type
Ribdeck 80 80/1.2 (0.14 kN/m²)	S350	190	NW	2400 Wet 2350 Dry	30	S	100	60	STEEL	S

(Approx.Conc.Vol. = 0.160 m³/m² level suppts)

Support crs /Cons.Span /Comp.Span	Span Description	<----Unfactored Loads kN/m²----> Impd.	Parts.	Fins.	Servs.	Cons.	No.Concentrated Loads Point Allow.	Transv. Loads	Parall. Lin.Lds
3.500m 3.480m 3.500m	(single span)	7.50	0.00	2.00	0.50	1.50	0	0	0

Hard.Flr.Fins.= 0 mm

Fire Rating - Fire Engineering Analysis

Deck Ref.	Construction Type	1 No Top Mesh heavier of Crk.-Cont.	Fire-Reinf	Fire Reinf. Bars bottom	Fact.Load Prop Runner
80/1.2	Single - unpropped	A193(0.1%)	A193	BAR REINFS	0.00 kN/m

<-------Deck Properties /m width---------->

Area mm²	Ixx cm^4	yb mm	Ult.Moms. kNm +ve	-ve	Resid. kN
1848	237.6	42.5	18.66	15.94	6.89

<--Composite Props. /m width-->

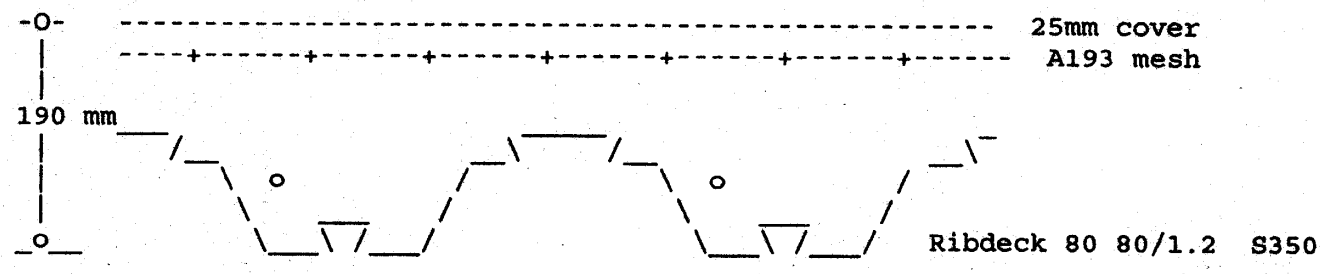
Pw kN	Vc kN	Ixx(av) cm^4	Ult.M. kNm	Shr.Bd. kN	Vrt.Shr. kN
81.5	114.1	3586	75.32	69.37	70.81

(Span/Depth Limit = 35 Construction Span/Deflect Limit = 130)

Design Calculations -- Applied / (Resistance) Values (per Metre width)

Construction Stage			Composite Stage		
Deck Deflection	18.45	(26.77)mm	Super Load Defl.	1.99	(10.00)mm
Int.Reac.Web Bklg	0.00	(81.50)kN	Tot.Imp.Ld.Defl.	2.66	(14.00)mm
Neg.Support Mom.	0.00	(15.94)kNm	Comp.Bding Mom.	31.93	(75.32)kNm
Pos.Bending Mom.	11.90	(18.66)kNm	Comp.Shear Bond.	27.12	(69.37)kN
Web Bklg.End.Sppt	13.68	(32.60)kN	Comp.Slab Shear	36.49	(70.81)kN
Web Shear at Reac.	13.68	(114.10)kN	Fire Bdg Moment	21.17	(23.06)kNm

Es=205 kN/mm² Mod.R.= 10 mr=204.4 kr= 0.1562



Suggested mesh - A252(0.2%) for effective distribution of high applied loads.

General Notes
(1) Design of Composite Metal Deck Floors in accordance with BS 5950 Pts 4 & 6
(2) Fire Design based on S.C.I. Public.056 Fire Eng.Method
(3) Refer to full printout for detailed calculations
(4) Refer to RLSD Technical Dept. for designs beyond the scope of Deckspan

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Arup Group Ltd 13 Fitzroy St W1T 4BQ	Job No : Not here? Location: And here... Project : Put your default stub here... Calcs By: & your default here too?

ACTUAL SPAN IN DESIGN. (E-BAR SHOWN TO BE ADOPTED)

RICHARD LEES STEEL DECKING LTD. ASHBOURNE, DERBYSHIRE DE6 1HD 17/04/08
STRUCTURAL & FIRE RATING DESIGN FOR COMPOSITE METAL DECK FLOOR SLABS

Deck Profile	Deck Grade kN/mm²	Slab Depth mm	Conc. Type	Concrete Density kg/m³	Conc. Grade N/mm²	Struc. Span type	Suppt. Width mm	Fire Rtg. mins	Fire Reinf type	Fire Span type
Holorib 51/0.9 (0.13 kN/m²)	S280	190	NW	2400 Wet 2350 Dry	30	S	100	60	STEEL	S

(Approx.Conc.Vol. = 0.184 m³/m² level suppts)

Support crs /Cons.Span /Comp.Span	Span Description	<----Unfactored Loads kN/m²----> Impd.	Parts.	Fins.	Servs.	Cons. Allow.	No.Concentrated Loads Point Loads	Transv. Lin.Lds	Parall. Lin.Lds
3.500m 3.451m 3.500m	(single span)	7.50	0.00	2.00	0.50	1.50	0	0	0

Hard.Flr.Fins.= 0 mm

Deck Ref.	Construction Type	Fire Rating - Fire Engineering Analysis 1 No Top Mesh heavier of Crk.-Cont. Fire-Reinf	Fire Reinf. Bars bottom	Fact.Load Prop Runner
51/0.9	PROP AT CENTRE SPAN	A193(0.1%)	A193	BAR REINFS 18.64 kN/m

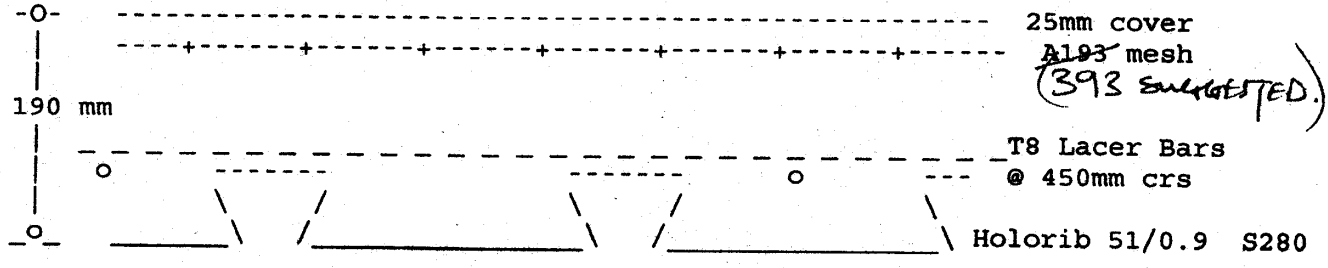
Deck Properties /m width: Area mm²	Ixx cm^4	yb mm	Ult.Moms. +ve	-ve	kNm Resid.	Pw kN	Composite Props. /m width: Vc kN	Ixx(av) cm^4	Ult.M. kNm	Shr.Bd. kN	Vrt.Shr. kN
1597	64.4	16.7	6.05	6.23	3.65	76.2	95.8	4600	65.66	42.85	104.24

(Span/Depth Limit = 35 Construction Span/Deflect Limit = 130)

Design Calculations -- Applied / (Resistance) Values (per Metre width)

Construction Stage			Composite Stage		
Deck Deflection	3.16	(13.27)mm	Super Load Defl.	1.55	(10.00)mm
Int.Reac.Web Bklg	18.64	(76.16)kN	Tot.Imp.Ld.Defl.	2.64	(14.00)mm
Neg.Support Mom.	3.22	(5.80)kNm	Comp.Bding Mom.	33.10	(65.66)kNm
Pos.Bending Mom.	2.47	(6.05)kNm	Comp.Shear Bond.	33.81	(42.85)kN
Web Bklg.End.Sppt	6.53	(30.46)kN	Comp.Slab Shear	37.82	(104.24)kN
Web Shear at Reac.	9.32	(95.80)kN	Fire Bdg Moment	22.00	(23.89)kNm

Es=205 kN/mm² Mod.R.= 10 mr=166.6 kr= 0.0050



T12 Bars @ 300.0 eff crs
(45.0 mm bottom cover to bar centre)

Suggested mesh - A393(0.2%) to control crack widths for propped construction.
Suggested mesh - A393(0.2%) for effective distribution of high applied loads.

General Notes
(1) Design of Composite Metal Deck Floors in accordance with BS 5950 Pts 4 & 6
(2) Fire Design based on S.C.I. Public.056 Fire Eng.Method
(3) Refer to full printout for detailed calculations
(4) Refer to RLSD Technical Dept. for designs beyond the scope of Deckspan

Arup	Job No : Not here?
Address line 1	Location: And here...
Address line 2	Project : Put your default stub here...
	Calcs By: & your default here too?

(HOLORIB CAN BE ADOPTED BUT REQUIRES PROPPING.)

RICHARD LEES STEEL DECKING LTD. ASHBOURNE, DERBYSHIRE DE6 1HD
STRUCTURAL & FIRE RATING DESIGN FOR COMPOSITE METAL DECK FLOOR SLABS 17/04/08.

Deck Profile	Deck Grade kN/mm²	Slab Depth mm	Conc. Type	Concrete Density kg/m³	Conc. Grade N/mm²	Struc. Span type	Suppt. Width mm	Fire Rtg. mins	Fire Reinf type	Fire Span type
Holorib 51/0.9 (0.13 kN/m²)	S280	260	NW	2400 Wet 2350 Dry	30	S	100	60	STEEL	S

(Approx.Conc.Vol. = 0.254 m³/m² level suppts)

Support crs /Cons.Span /Comp.Span	Span Description	<----Unfactored Loads kN/m²----> Impd.	Parts.	Fins.	Servs.	Cons. Allow.	No.Concentrated Loads Point Loads	Transv. Lin.Lds	Parall. Lin.Lds
3.400m 3.351m 3.400m	(single span)	5.00	0.00	3.00	0.50	1.50	0	0 Hard.Flr.Fins.=	0 0 mm

Fire Rating - Fire Engineering Analysis

Deck Ref.	Construction Type	1 No Top Mesh Crk.-Cont.	heavier of Fire-Reinf	Fire Reinf. Bars bottom	Fact.Load Prop Runner
51/0.9	PROP AT CENTRE SPAN	A252(0.1%)	A252	BAR REINFS	22.96 kN/m

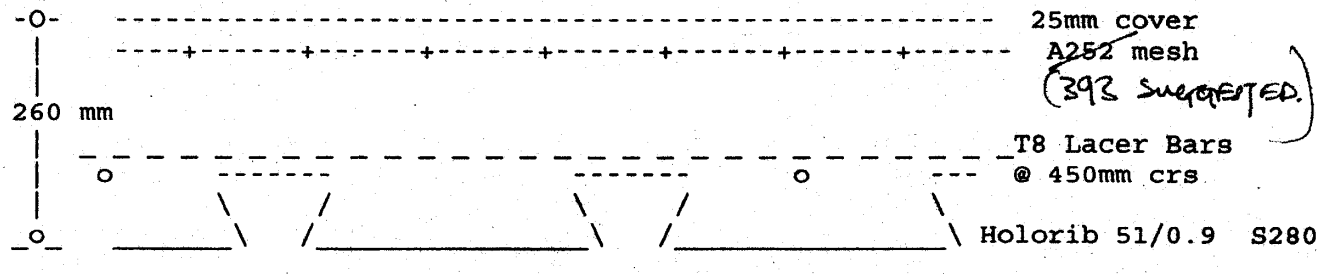
<-------Deck Properties /m width----------> Area mm²	Ixx cm^4	yb mm	Ult.Moms. kNm +ve	-ve	Resid. kN	Pw kN	Vc kN	<--Composite Props. /m width--> Ixx(av) cm^4	Ult.M. kNm	Shr.Bd. kN	Vrt.Shr. kN
1597	64.4	16.7	6.05	6.23	3.65	76.2	95.8	10934	94.77	61.90	120.06

(Span/Depth Limit = 35 Construction Span/Deflect Limit = 130)

Design Calculations -- Applied / (Resistance) Values (per Metre width)

Construction Stage			Composite Stage		
Deck Deflection	3.85	(12.89)mm	Super Load Defl.	0.39	(9.71)mm
Int.Reac.Web Bklg	22.96	(76.16)kN	Tot.Imp.Ld.Defl.	0.95	(13.60)mm
Neg.Support Mom.	3.85	(5.48)kNm	Comp.Bding Mom.	30.76	(94.77)kNm
Pos.Bending Mom.	2.95	(6.05)kNm	Comp.Shear Bond.	30.84	(61.90)kN
Web Bklg.End.Sppt	8.04	(30.46)kN	Comp.Slab Shear	36.19	(120.06)kN
Web Shear at Reac.	11.48	(95.80)kN	Fire Bdg Moment	20.94	(24.48)kNm

Es=205 kN/mm² Mod.R.= 10 mr=166.6 kr= 0.0050



Suggested mesh - A393(0.2%) to control crack widths for propped construction.

General Notes
(1) Design of Composite Metal Deck Floors in accordance with BS 5950 Pts 4 & 6
(2) Fire Design based on S.C.I. Public.056 Fire Eng.Method
(3) Refer to full printout for detailed calculations
(4) Refer to RLSD Technical Dept. for designs beyond the scope of Deckspan

Arup	Job No : 118000
Address line 1	Location: Roof Barlow House
Address line 2	Project : St Pancras
	Calcs By: JKL

(HOLORIB CAN BE ADOPTED BUT REQUIRES PROPPING.)

BARLOW HOUSE ROOF – ALTERNATIVE

RICHARD LEES STEEL DECKING LTD. ASHBOURNE, DERBYSHIRE DE6 1HD
STRUCTURAL & FIRE RATING DESIGN FOR COMPOSITE METAL DECK FLOOR SLABS

Deck Profile	Deck Grade kN/mm²	Slab Depth mm	Conc. Type	Concrete Density kg/m³	Conc. Grade N/mm²	Struc. Span type	Suppt. Width mm	Fire Rtg. mins	Fire Reinf type	Fire Span type
Ribdeck 80 80/1.2	S350	260	NW	2400 Wet 2350 Dry	30	S	100	60	STEEL	S

(0.14 kN/m²) (Approx.Conc.Vol. = 0.233 m³/m² level suppts)

Support crs /Cons.Span /Comp.Span	Span Description	<----Unfactored Loads kN/m²----> Impd.	Parts.	Fins.	Servs.	Cons. Allow.	No.Concentrated Loads Point Loads	Transv. Lin.Lds	Parall. Lin.Lds
3.400m	(single span)	5.00	0.00	3.00	0.50	1.50	0	0	0
3.380m							Hard.Flr.Fins.=		0 mm
3.400m									

Deck Ref.	Construction Type	Fire Rating - Fire Engineering Analysis 1 No Top Mesh heavier of Crk.-Cont.	Fire-Reinf	Fire Reinf. Bars bottom	Fact.Load Prop Runner
80/1.2	Single - unpropped	A252(0.1%)	A252	BAR REINFS	0.00 kN/m

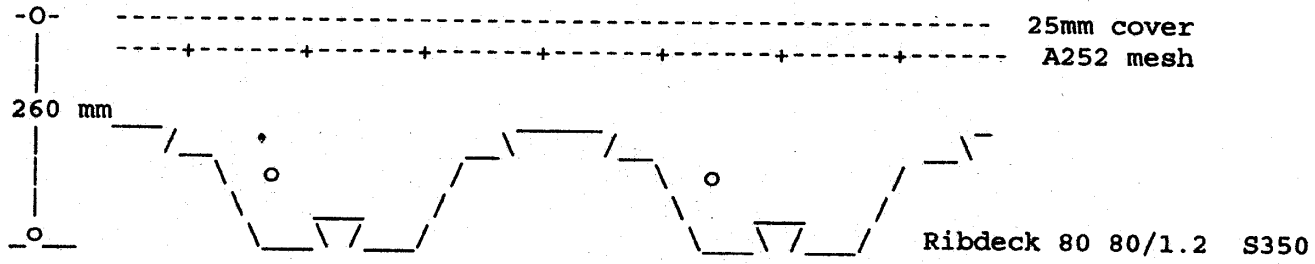
<-------Deck Properties /m width----------> Area mm²	Ixx cm^4	yb mm	Ult.Moms. kNm +ve	-ve	Resid. kN	Pw kN	Vc kN	<--Composite Props. /m width--> Ixx(av) cm^4	Ult.M. kNm	Shr.Bd. kN	Vrt.Shr. kN
1848	237.6	42.5	18.66	15.94	6.89	81.5	114.1	8911	117.43	104.50	83.24

(Span/Depth Limit = 35 Construction Span/Deflect Limit = 130)

Design Calculations -- Applied / (Resistance) Values (per Metre width)

Construction Stage			Composite Stage		
Deck Deflection	23.67	(26.00)mm	Super Load Defl.	0.48	(9.71)mm
Int.Reac.Web Bklg	0.00	(81.50)kN	Tot.Imp.Ld.Defl.	0.81	(13.60)mm
Neg.Support Mom.	0.00	(15.94)kNm	Comp.Bding Mom.	29.79	(117.43)kNm
Pos.Bending Mom.	14.68	(18.66)kNm	Comp.Shear Bond.	21.93	(104.50)kN
Web Bklg.End.Sppt	17.37	(32.60)kN	Comp.Slab Shear	35.05	(83.24)kN
Web Shear at Reac.	17.37	(114.10)kN	Fire Bdg Moment	20.25	(24.03)kNm

Es=205 kN/mm² Mod.R.= 10 mr=204.4 kr= 0.1562



General Notes
(1) Design of Composite Metal Deck Floors in accordance with BS 5950 Pts 4 & 6
(2) Fire Design based on S.C.I. Public.056 Fire Eng.Method
(3) Refer to full printout for detailed calculations
(4) Refer to RLSD Technical Dept. for designs beyond the scope of Deckspan

Arup Group Ltd 13 Fitzroy St W1T 4BQ	Job No : 118000 Location: Roof Barlow House Project : St Pancras Calcs By: JKL

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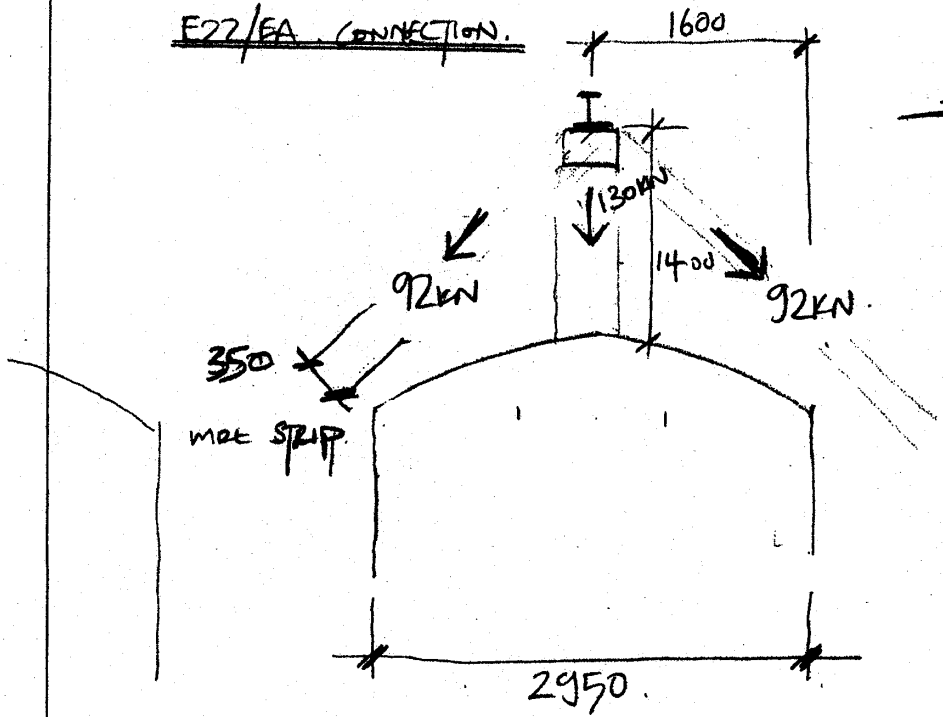
BARLOW HOUSE (REVIEW OF CONNECTION DETAIL OVER OPENING)

E22/EA CONNECTION.

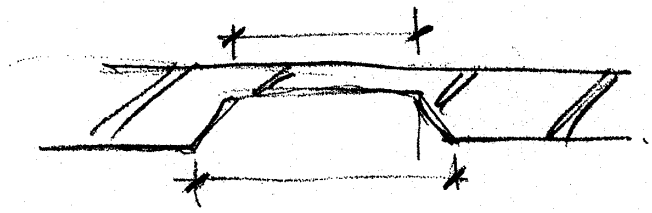


REACTION 130 KN.

LOAD PASSES ALONG TWO COMPRESSION STRUTS.

$92\text{KN} / (230 \times 1.3\text{N/mm}^2)$

$\approx 359\text{mm}$.



LOADS (LATERAL) TEND TO BALANCE OUT BUT CAN BE TAKEN THROUGH COURSE FRICTION. (DETAIL ACCEPTABLE)

1.2 Existing Roof Alterations

To allow air extract from high level plant, roof alterations to the existing timber structure were necessary. The existing timbers were locally cut and trimmed out with a new timber frame.

The calculations are as follows which should be read in conjunction with drawings:

S_6010, S_6321, S_6322, S_6325

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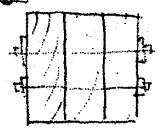
Barlow House Roof Alterations



$E = 10.8\ KN/mm^3$

$\delta = \frac{5\times(0.54\times7.5)\,7500^3}{384\times10.8\times(210\times295^3/12)} = 4.6\ mm$ ✓ ok.

< Span/1000

Under self weight, span 7.5m $\gamma = 6\ KN/m^3$

$w = 0.3^2\times6 = 0.54\ KN/m$; $M = \frac{0.54\times7.5^2}{8} = 3.8\ KNm$

$z = \frac{3\times97\times295^2}{6} = 4.22\times10^6\ mm^3$

$f_{bc} = \frac{3.8\times10^6}{4.22\times10^6} = 0.9\ N/mm^2$ ✓ ok.

Grade bending stress ≥ $5.3\ N/mm^2$

Need to check LT buckling & deflection

Axial stress under applied load 19.7 KN

$A = 3\times97\times295 = 85.8\times10^3\ mm^2$

$f_c = \frac{19.7\times10^3}{85.8\times10^3} = 0.23\ N/mm^2$.

Grade compressive stress ≥ $7.9\ N/mm^2$

Need to check buckling.

Base on 291 x 295 solid section

$r_y = \frac{291}{\sqrt{12}} = 84.0\ mm$

Bow or spring limited to L/300

$\frac{L_e}{r_y} = \frac{7500}{84} = 89$

Note k_2 for service class 3
bending // E 0.6
compression 0.6

$\frac{E_c}{p_{c\parallel}} = \frac{10.8\times10^3\times0.6}{7.9\times0.6} = 1820$

k_3 load duration use LT 1.0

$k_{12} = 0.61$

$\frac{f_c}{p_c} = \frac{0.23}{0.61\times(0.6\times7.9)} = \frac{0.23}{2.9} = 0.08$ ✓ ok.

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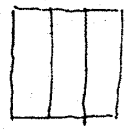
Barlow House
Roof Alterations



0.9 + 0.23 = 1.13 N/mm².

30.5 kN

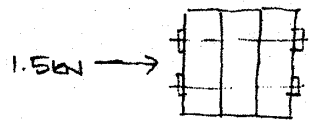
0.9 − 0.23 = 0.67 N/mm²

Total compression

$$291 \times \frac{1.13}{2} \times 295 \times \frac{1.13}{(1.13+0.67)} = 30.5 \text{ kN}$$

Resist 2½% of this as a lateral load applied to upper ⅛ of the timber section = 0.76 kN

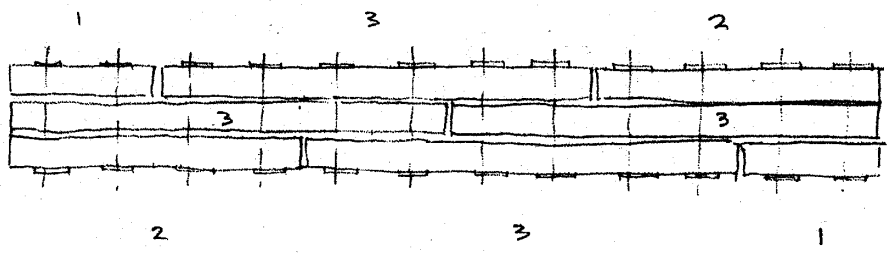
design whole section to resist = 1.5 kN



Provide bolts to carry shear

By inspection o.k

Proposed timber arrangement



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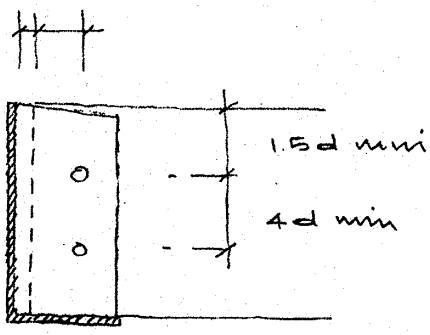
7d = 7x16 = 112 + 15 tolerance on installation = 130 s.

Nom 15 gap



Barlow House
Roof Alterations.

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Timber to timber bolted connections

4.

M16 bolts in 97 thick timbers, load parallel to grain

Basic single shear capacity = 5.05 kN long term load

Maximum axial load per timber = $\frac{19.7}{3}$ = 6.6 kN

2 no. bolts transfer load ✓ ok. 2 x 5.05 = 10.1 kN > 6.6 kN ok.

End bolted detail capacity = 1.25 x 4 x 5.05 = 25.2 kN
4 bolts. > 19.7 kN
✓ ok.

Barlow House
Roof Alterations.

1.3 Connecting Barlow House and West Wing

To allow the East Wing and Barlow House to be connected at high level a link bridge was introduced which required an existing window to be opened up to form a doorway. One of the roof rafters is located over the existing window, therefore a series of steel lintels were introduced to re-support the roof loads and allows the opening to be extended as required.

The calculations are as follows which should be read in conjunction with drawings:

S_6005, S_6006

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Member/Location

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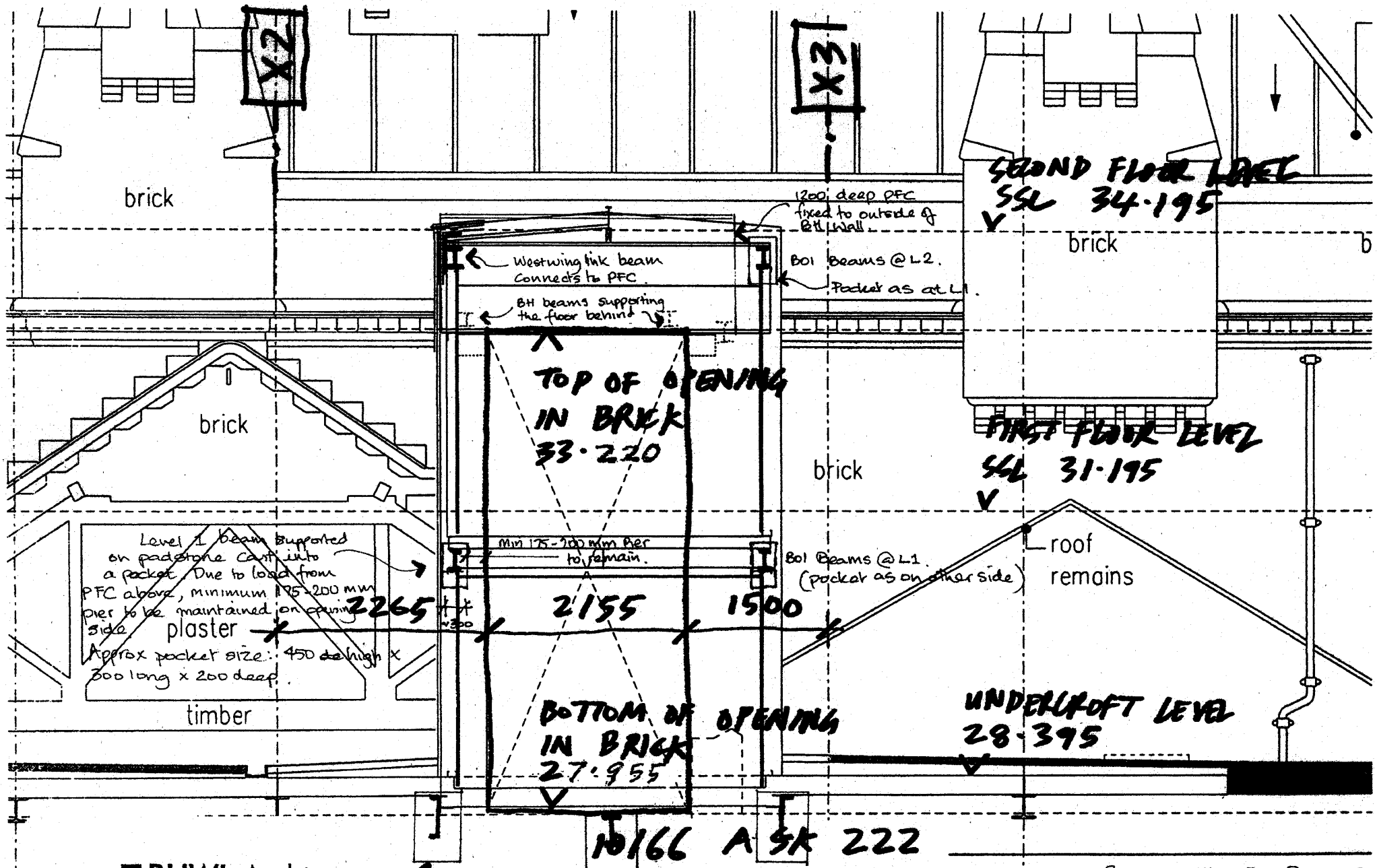
BARLOW HOWE - TIMBER ALTERATION ADJACENT TO LINK BRIDGE.

- BETWEEN THE WEST WING AND BARLOW HOWE, THERE IS A LINK BRIDGE.
- THE LINK BRIDGE IS APPROXIMATELY CENTRALLY LOCATED ON ONE OF THE MAIN TIMBER TRUSSES ON PLAN.
- DUE TO HEAD HEIGHTS IT IS NECESSARY TO NOTCH THE TIMBER TRUSS BOTTOM CHORD.
- THE FOLLOWING CALCS DEAL WITH THIS DETAIL:



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London WC2H 9DZ

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Job Title: ST PANCRAS CHAMBERS.	Drg. Ref.		
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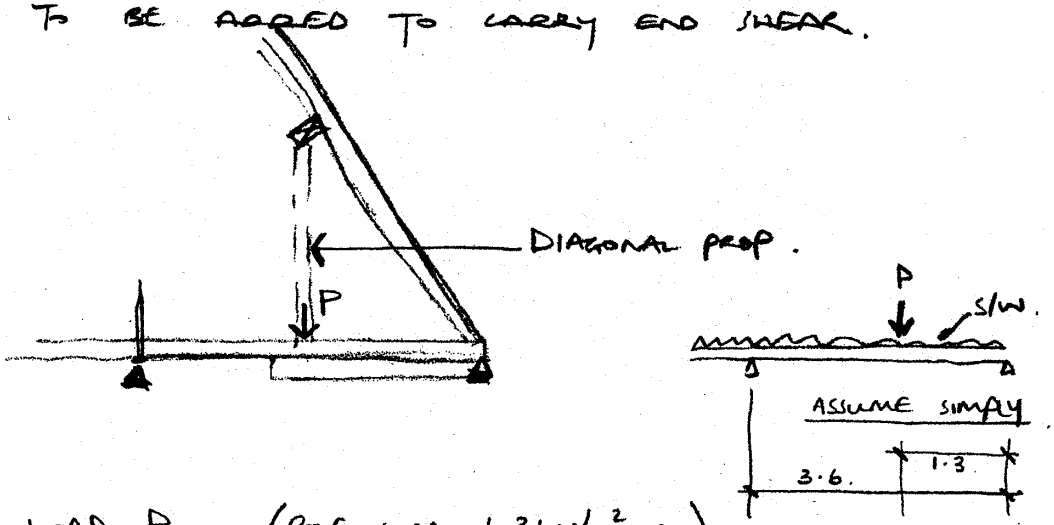
BARLOW HOUSE — TRUSS ALTERATIONS ADJACENT TO LINK BRIDGE.

BOTTOM CHORD OF THE THE ∧ TRUSS OVER THE LINK BRIDGE ENTRANCE CARRIES ITS OWN SELF WEIGHT & A POINT LOAD FROM A DIAGONAL PROP SUPPORTING A PURLIN. TIMBER IS TO BE NOTCHED, ∴ NEW STEEL END SUPPORT TO BE ADDED TO CARRY END SHEAR.



LOAD P (Roof load 1.3 kN/m² SLS)

$$(5.8 \times 0.5) \times 2.0 \times \left(\underset{(DL)}{1.3\ kN/m^2} + \underset{(LL)}{0.75\ kN/m^2}\right) = 12\ kN.$$

S/W.

$0.38 \times 0.19 \times 5\ kN/m^3 = 0.6\ kN/m.$

SHEAR = 9 kN (SLS)
9 x 1.45 = 13 kN (ULS)

GIVEN THE DIFFICULTY OF ESTIMATING LOADS, THE ALTERNATIVE IS TO LOOK AT THE BENDING CAPACITY OF THE TIMBER (TO ENSURE LOADS ARE SAFE)

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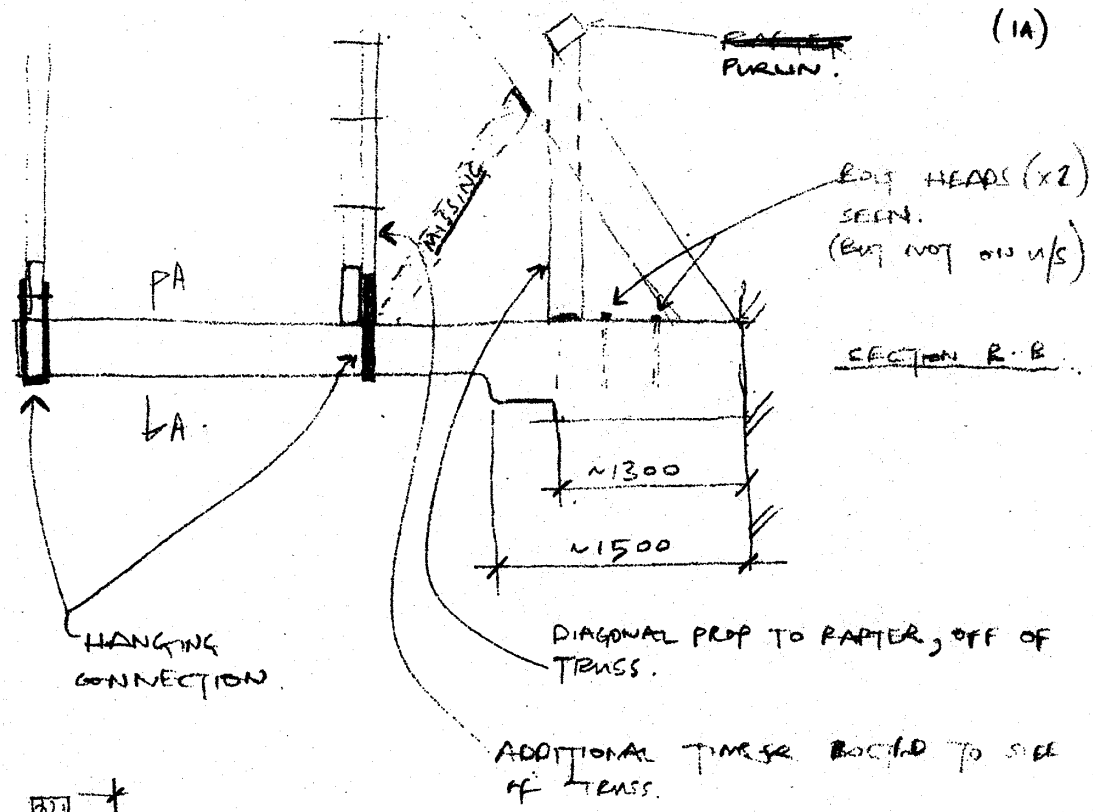
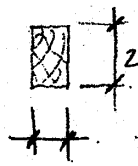
(1A)

EARLSDON HOUSE TRUSS

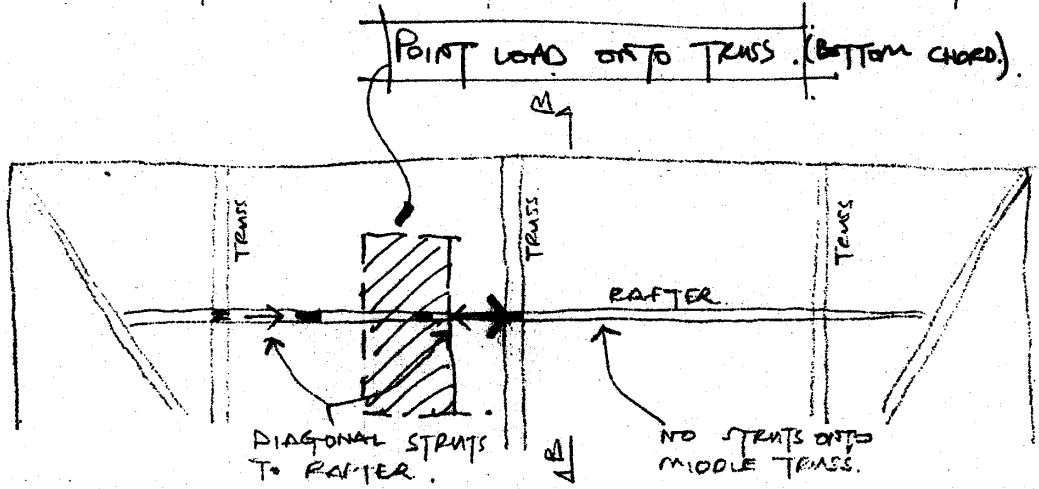


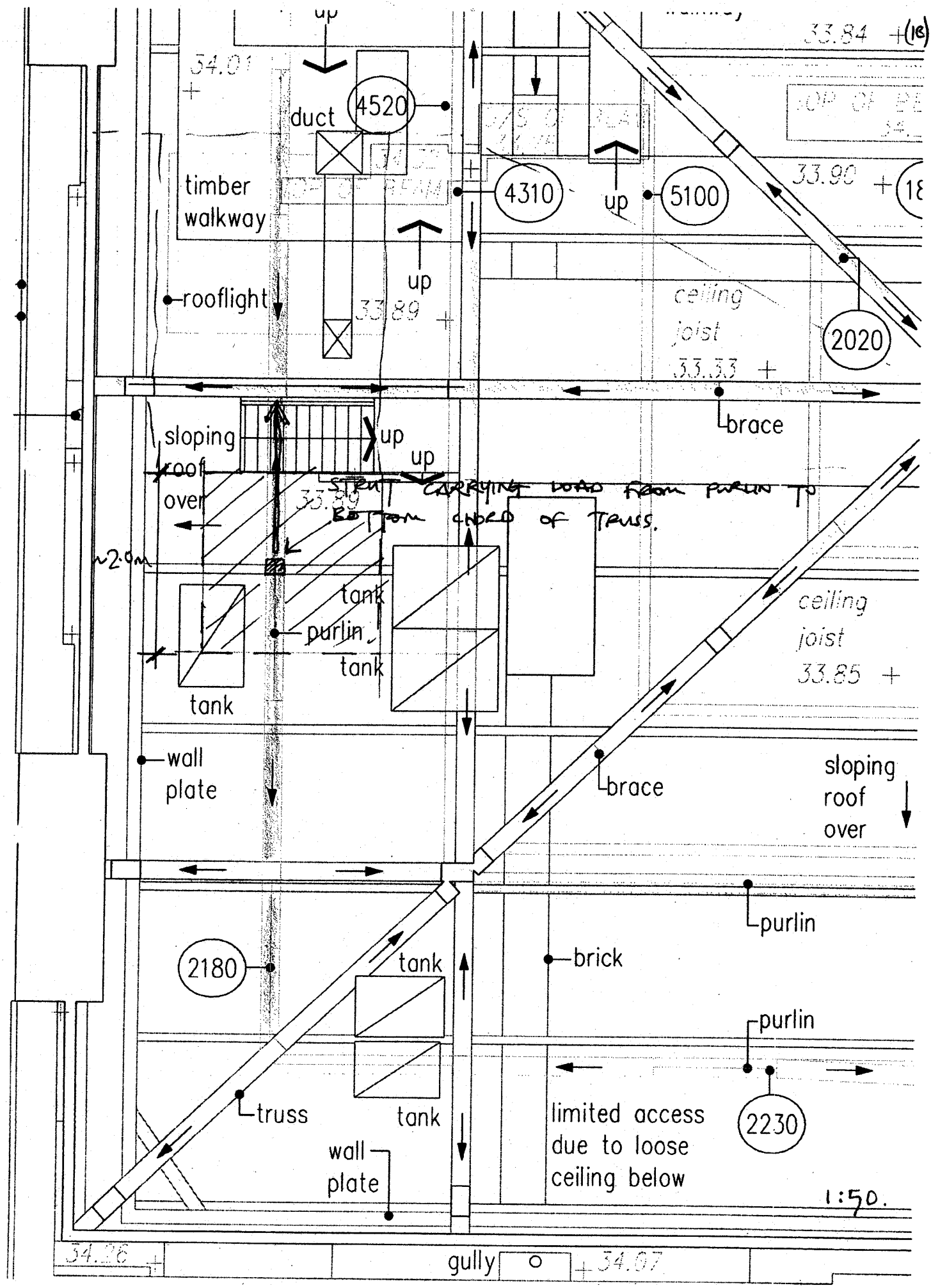


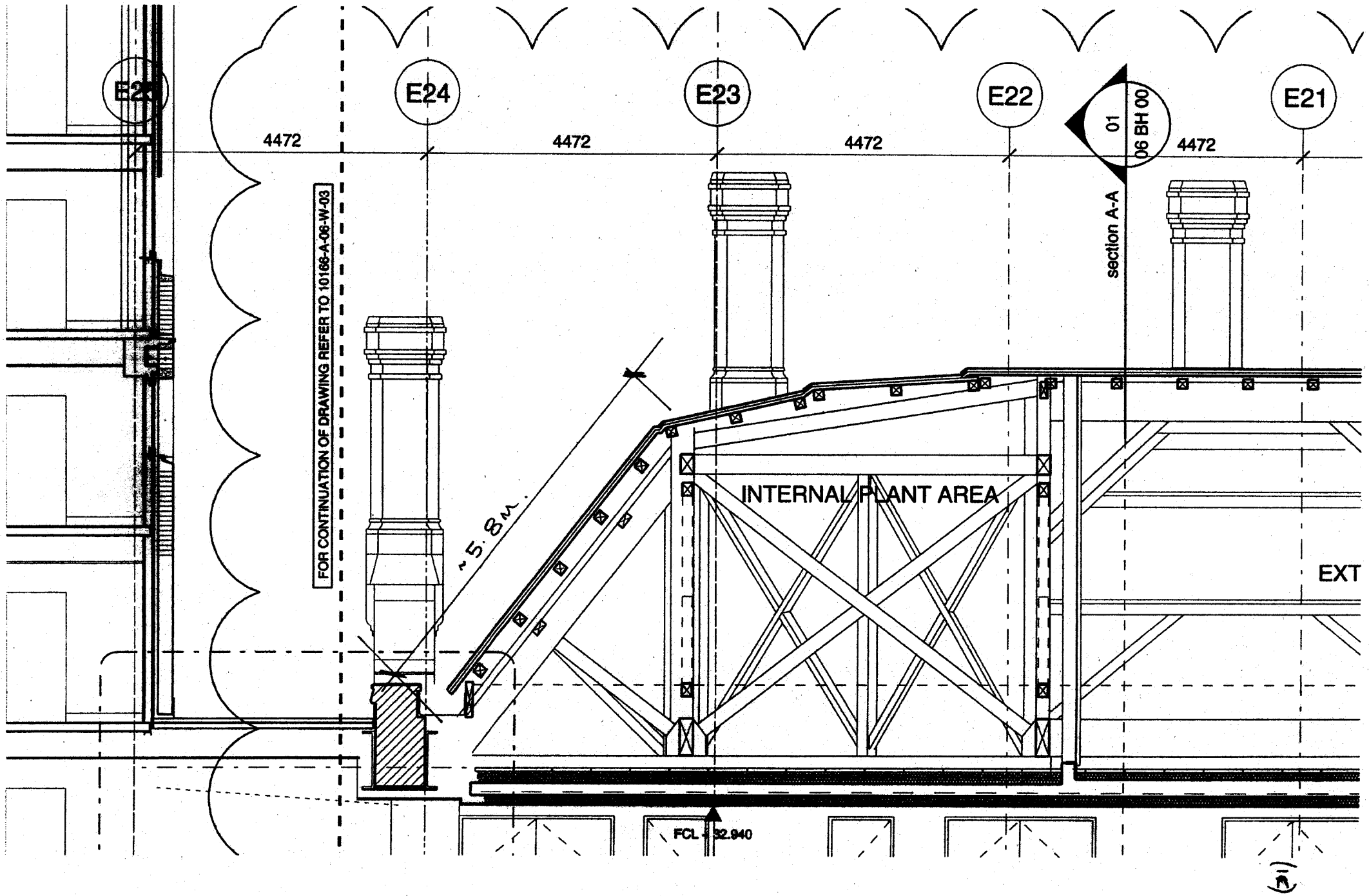
SECTION A-A.

- HATCHED MEMBER MISSING FROM TRUSS. (MORTICES SEEN IN DIAGONAL TRUSS MEMBER)

- ALL BOTTOM TIMBER TRUSS ELEMENTS SAME WIDTH.




up
34.01
duct
4520
timber
walkway
4310
up
5100
33.84
33.90
rooflight
33.89
up
ceiling
joist
33.33
2020
brace
sloping
roof
over
up
up
STRUT CARRYING LOAD FROM PURLIN TO BOTTOM CHORD OF TRUSS.
33.89
~2.0m
tank
purlin
tank
tank
ceiling
joist
33.85
wall
plate
brace
sloping
roof
over
purlin
2180
tank
brick
purlin
truss
tank
limited access
due to loose
ceiling below
2230
wall
plate
1:50.
34.26
gully
34.07


E25
E24
E23
E22
E21
4472
4472
4472
4472
section A-A
01
06 BH 00
FOR CONTINUATION OF DRAWING REFER TO 10168-A-06-W-03
~5.8M.
INTERNAL PLANT AREA
EXT
FCL - 32.940

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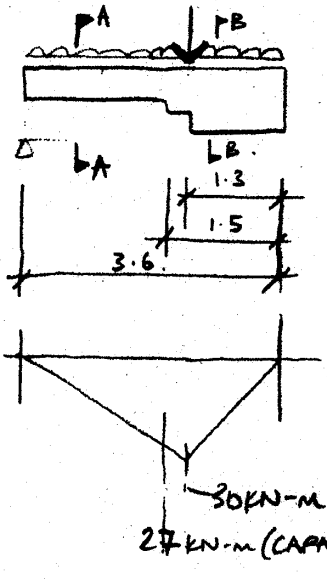
(2)

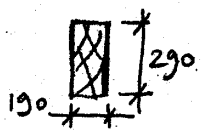


BARLOW HSE TRUSS. (PG2).

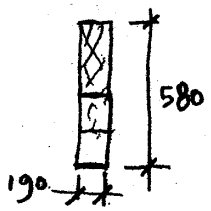
BASED ON C27.

BENDING σ : $10 N/mm^2$

SECTION A-A.



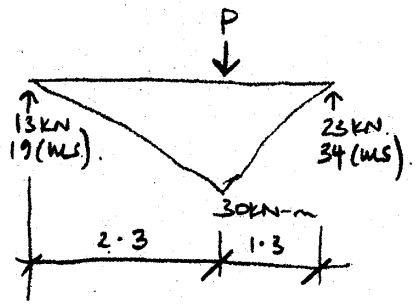
SECTION B-B



ALLOW BM AT SECTION A-A.

$M/Z = \sigma$ $\quad M/(190 \times 290^2/6) = 10 N/mm^2$

$M = 27 kN\text{-}m$

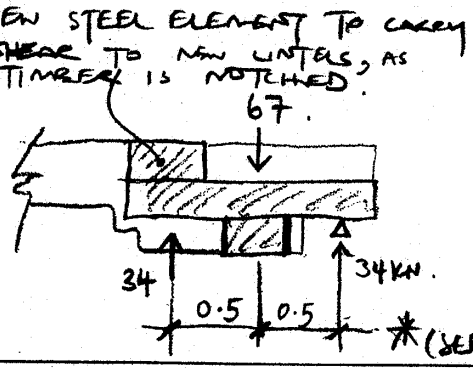


$P \times 1.3 \times 2.3 / 3.6 = 30.$

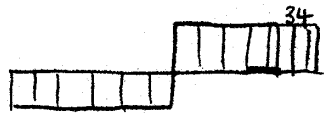
$P = 36 kN.$ (SLS)

AS ULS $36 \times 1.45 = 54 kN$ (ULS)

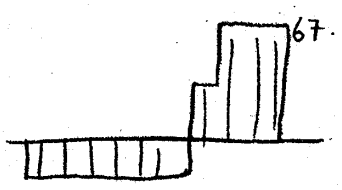
NEW STEEL ELEMENT TO CARRY END SHEAR TO NEW LINTELS, AS TIMBER IS NOTCHED.



* (SEE PG 3).



ORIGINAL SHEAR DIAGRAM.



NEW SHEAR DIAGRAM.

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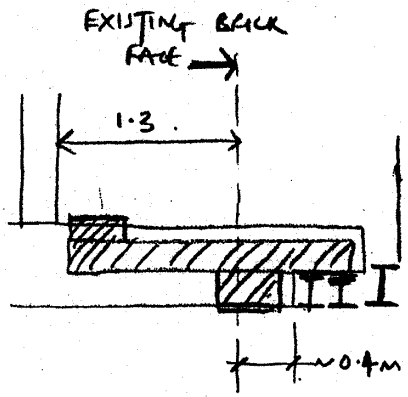
BARLOW HOWE TRUSS (PG 3)



LOADS BASED ON BENDING CAPACITY ARE FOR MORE CONSERVATIVE & ARE ∴ ADOPTED.

STEEL MEMBER AS HATCHED TO BE LOCATED ADJACENT TO NEW LINTEL & ADJACENT TO VERTICAL (DIAGONAL) TIMBER TO TOP OF HORIZONTAL TIMBER

BASED ON THIS REACTIONS ON PG 2 ARE VERY CONSERVATIVE.

CHECK INCREASE SHEAR IN TIMBER

$67 \times 10^3 / (580 \times 190) = 0.61 \text{ N/mm}^2 < 1.1 \text{ N/mm}^2$ ∴ OK.

CHECK BEARING (BASED ON 300WD STRAPS UNDER/OVER)

$67 \times 10^3 / (300 \times 190) = 1.2 \text{ N/mm}^2 < 2.0 \text{ N/mm}^2$ ∴ OK.

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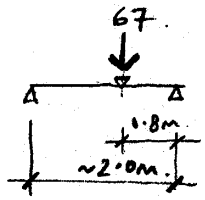
(4).

BARLOW HOUSE TRUSS (PG 4)

EXISTING TIMBER ELEMENTS OK.

DESIGN OF STEEL SECTIONS.



$M_{MT} = 67 \times 0.8 \times 1.2 / 2.0$

$= 32\ KN\text{-}m$.

DESIGN PFC's (x2) FOR STRENGTH BUT ALSO FOR SAME STIFFNESS AS TIMBER SECTION.

I_{yy} TIMBER $= 190 \times 580^3/12$.

$= 308{,}000\ cm^4$.

$EI = (308\,000 \times 10^4) \times 7.7$.

$= 2.37 \times 10^{10}$.

I_{yy} 300x 100x 46 PFC

$= 8230\ cm^4$ (x2)

$EI = ((8230 \times 10^4) \times 2) \times 205$

$= 3.37 \times 10^{10}$

$M_b = 149\ KN\text{-}m$ ∴ ok

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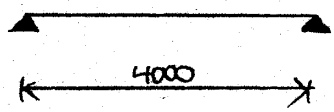
STEEL BEAM ADJACENT TO TIMBER TRUSS. (PFC's) (5)

SKETCH



LW = 0.75m max

DL = 0.75 x 6.0 = 4.5 kN/m

LL = 0.75 x 7.5 = 5.6 kN/m.

TRY 150x90x24 PFC

STRENGTH. $M_u = (1.4 \times 4.5 + 1.6 \times 5.6) \times 4^2/8$

= ~~30.5 kN-m.~~ → 35 kN-m WITH ACOUSTIC WALLING.

M_{Bx} (Le = 4m) = 37.1 kN.m

SERVICE $\Delta = \frac{5}{384} \times \frac{(4.5+5.6) \times 4000^4}{205 \times 1162E4}$

= ~~14.3mm~~ → 17mm WITH ACOUSTIC WALLING. ≡ SPAN/~~280~~ → 230 (DL+LL).

=> ADOPT 150x90x24 PFC

SHEAR FOR CONNECTIONS $(1.4 \times 4.5) + (1.6 \times 5.6) \times 2 = 34$ kN.

FOLLOW UP NOTE: ACOUSTIC WALLING REQD OVER 1.5m FROM FACE OF WALL $2 \times 1.0 \times 2.62 \times 1.4 = 7.3$ kN/m

SEE NOTES ABOVE ∴ ok

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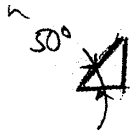
LINTELS UNDER BARLOW HOUSE TRUSS ADJACENT TO LINK BRIDGE.

(6)

PARAPET $17 \times 0.225 \times 20 = 7.7\text{kN/m}$

ROOF 2.4kN/m^2 & 0.6kN/m^2 LL (IN PLAN).

PT LOAD ON TRUSS. (PLACE ALL LOAD ON TRUSS).



$DL = 11 \times \frac{1}{\cos 50^\circ} \times 2.4 = 41\text{kN}$ ($\times 1.4 = 57\text{kN}$ ULS)

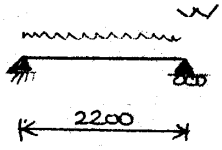
$LL = 11 \times 0.6 = 7\text{kN}$ ($\times 1.6 = 11\text{kN}$ ULS)

↑ SEE SKETCH DIAGRAM (1A).

LINTEL DESIGN

PROPOSE 1 LINTEL FOR PARAPET & LINTELS AS REQD ACROSS WALL WIDTH.

① PARAPET LINTEL. (MOST OUTER LINTEL).



STRENGTH. $M_u = 1.4 \times 7.7 \times 2.2^2/8 = 6.5\text{kN.m}$

$M_{B152UC23LE=2} = 55\text{kN.m}$ (GRADE S355) => OK.

SERVICE $\Delta = \frac{5}{384} \times \frac{7.7 \times 2200^4}{205 \times 1250E4} = 0.93\text{mm}$

$= \text{SPAN}/2365$ => OK

=> 152UC23 OK TO SUPPORT PARAPET

SHEAR FOR CONNECTIONS: $7.7 \times 1.1\text{m} \times 1.4 = 12\text{kN}$.

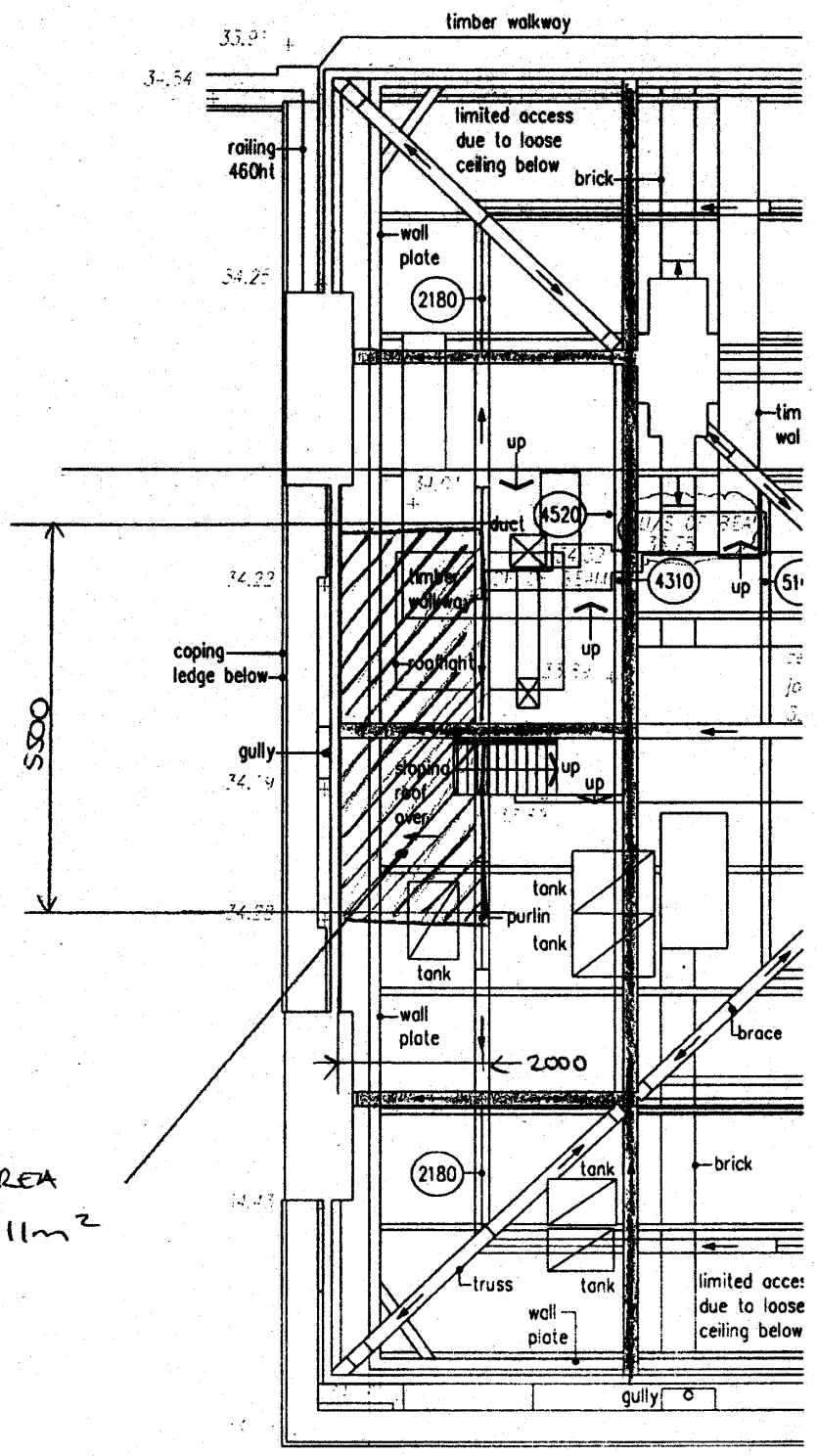
ST PANCRAS.

EXTRACT

SS94-03B

SURVEY DRG.

(6A)



TRIBUTARY AREA

$= 5.5 \times 2 = 11 m^2$

PART BARLOW HOUSE ROOF SURVEY

1:100

SKETCH DIAGRAM (1A)

OAP20

ARUP | Calculation sheet

Job No.	Sheet No.	Rev.
118000		

Member/Location:

Job Title: ST PANCRAS.

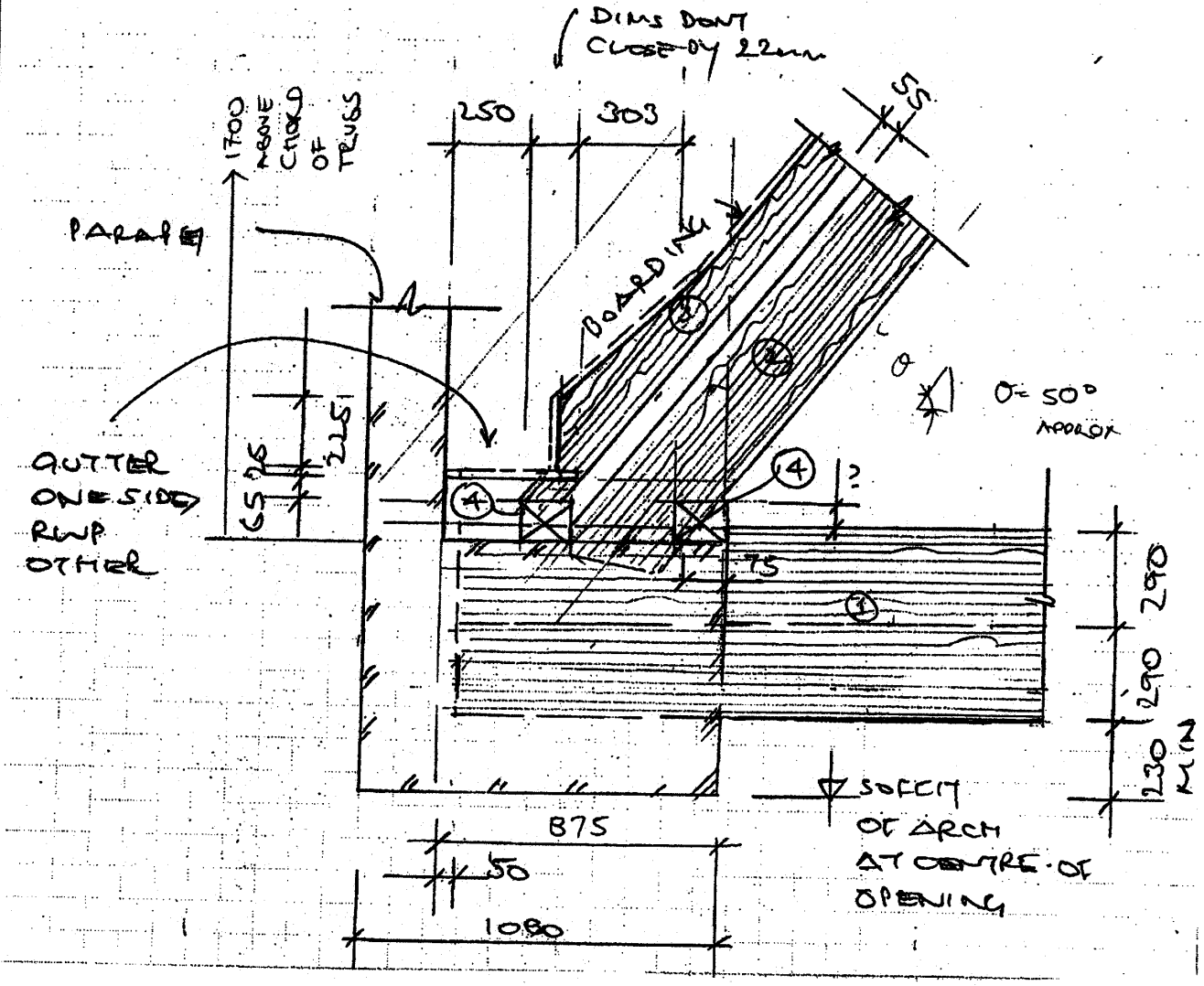
Drg. Ref.:

Made by MZ Date 3/9/07 Chd.

DESIGN LOAD ON BARLOW HOUSE NORTH WINDOW OPENING.

- OPENING IS 2.155m WIDE

- SKETCH.



ARUP | Calculation sheet

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Member/Location

Job Title: ST PANCRAS CHAMBERS.

Drg. Ref.

Made by JKL. Date 12/12/07. Chd.

LINTELS UNDER BACKOW HOUSE TRUSS ADJACENT TO LINK BRIDGE.

(7).

(2) LINTELS UNDER TRUSS CHORD.

1ST LINTEL TAKES LOADS FROM PFC'S RUNNING PARALLEL TO TIMBER TRUSS, CARRYING SLAB,

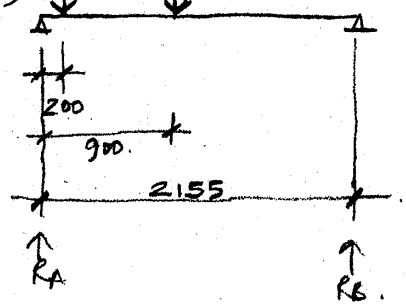
PLUS... REACTION FROM TRIMMING STEEL (WEST END OF LINTEL),

PLUS... ½ REACTION FROM BOTTOM CHORD OF TRUSS. (BASED ON LOAD SPREAD OVER MINIMUM OF 2 LINTELS.

PLUS... ½ REACTION FROM TIMBER TRUSS (DIAGONAL ELEMENT) SPLIT OVER 2 LINTELS.

(41 + 11 + 13 KN/2) FROM TRUSS.

(FROM TRIMMER) 25KN. 34KN (x2) FROM PFC.



LINTEL (2) WORST CASE

R_A : $(34 \times 2 + 33) \times 1.26/2.16 + 25 = 84$ KN.

R_B : $(34 \times 2 + 33) \times 0.9/2.16 + 25 \times 0.2/2.16 = 44$ KN.

OTHER LINTELS MAY CARRY A COMBINATION OF TRUSS LOAD, ROOF LOADS, 300mm BRICK BUILD-UP TO REPLACE EXISTING.

SAY $((41 + 11 + 13)/2)/2 + 1.4 \times 0.3 \times 0.23 \times 22 \times 1.1\text{m} = 19$ KN EACH END.

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LINTELS UNDER BARLOW HOUSE TRUS ADJACENT TO LINK BRIDGE.

(8).

DESIGN OF LINTEL ② WORST CASE

$M_{MT} = 100 \times 0.9 \times 1.26/2.16 + 25 \times 0.2 \times 1.96/2.16$

(SEE DIAGRAM ON PREVIOUS PAGE) $= 57$ KN-m.

TRY 152 x 152 x 37 M_b (2m) = 102 KN-M ∴ OK.

DEFLECTION $= (100/1.45) \times 2155^3/48 \times 205 \times 2210 \times 10^4$

$= 3$ mm.

SPAN/720.

FOR RC SLAB THIS IS ACCEPTABLE. FOR TRUSS THIS IS MORE MOVEMENT THAN WOULD BE ACCEPTED, HOWEVER, AS TRUSS IS SUPPORTED BY ADDITIONAL LINTELS THE ACTUAL DEFLECTION IT SEES WILL BE LESS.

IF THE 2ND LINTEL IN WAS TO CARRY ALL THE TRUSS LOAD, DEFLECTION WOULD BE ~ 1/2. SPAN/1500. ∴ OK.

FOLLOW UP NOTE: ACOUSTIC WALLING REQD

TOTAL ADDITIONAL LOAD.

$2 \times 1.5m \times 1.0 \times 2.62 \times 1.4$

$= 11$ KN (LOAD FROM PFC's).

ADDITIONAL 0.35mm DEFLECTION.

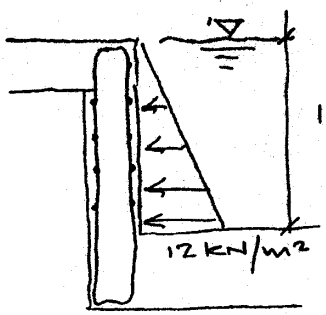
2 St Pancras Chambers

2.1 Swimming Pool Structure

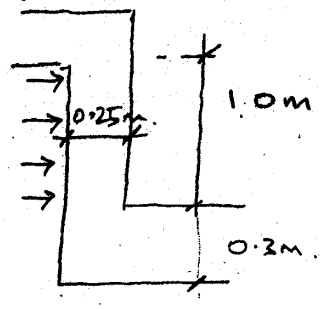
The spa are is formed by a reinforced concrete structure which is capable of resisting both lateral forces from water pressure when the spa is full and also from soil pressure when the spa is empty. The calculations are as follows which should be read in conjunction with drawings:

S_4001 – GA, S_1800 - Details

water pressure



compacted fill pressure 20 kPa



Design pool sides as cantilevers from base

250mm wall $d = 250 - 30$ cover $- 10$ bar $- 10\ \phi/2 - 5$ tol $= 195$ mm

$F_{ult} = 1.4 \times \frac{12 \times 1.2}{2} = 10.1$ KN/m

$M_{ult} = 10.1 \times \frac{1.2}{3} = 4.0$ KNm/m

$\frac{M_{ult}}{bd^2} = \frac{4.0 \times 10^6}{10^3 \times 195^2} = 0.11$ nominal rebar sufficient

Use 0.4% to control cracking in section & provide both faces

$A_s = 0.4 \times 1950 = 780\ mm^2/m$ T12.125p 904 mm²/m

Distribution bars 0.13% bth

$A_{sd} = 0.13 \times 2500 = 325\ mm^2/m$ T10.150p 523 mm²/m.

In base use T12.125##p full mat T&B

Check earth pressure on empty pool wall

$F_{ult} = 1.4 \times 20 \times 1.0 = 28$ KN/m

$M_{ult} = 28 \times 0.5 = 14.$ KNm/m by inspection rebar above or

ARUP	118000	SWIMMING POOL WALLS DESIGN.	
ST PANCRAS CHAMBERS	BY JB.	06.06.06.	

2.2 Taxi Rank Formation

The former taxi rank has been re-constructed with a reinforced concrete slab which has been shaped to incorporate a number of pits, services channels and a central corridor. The slab itself has been designed to carry crowd loading and the vertical elements which form the walls of service routes have been designed to carry lateral loads including those generated by soil pressure.

The calculations are as follows which should be read in conjunction with drawings:

S_4010, S_4011, S_4012, S_4013, S_4014, S_4015, S_4016, S_4021, S_4022, S_4041, S_4042, S_4043, S_4044

S_4001_SKS_01, S_4010_SKS_01, _4010_SKS_02

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118000-00
Design Statement

The original taxi rank structure was a ground bearing roadway supported on a combination of original ground and made ground. It was considered that as no signs of settlement was visible, that the made ground was full consolidated after many years of use as roadway. New slabs were considered as ground bearing, but an allowance was soft spots was included with in the sizing of the reinforcement. A 200mm thk slab with T12 @150 EW top and bottom are provided and this is deem to be more than sufficient for a ground bearing slab.

The service corridor that runs through the area design is included in chapter 1 and is designed as a cantilevering retaining wall which is propped at its head by the ground slab.

There are a series of glazed screens that are supported by structural steelwork. This steelwork has discreet footings, the design of which is included in Chapter 2.

In chapter 3 a summery of the reinforced details is provided.

St Pancras Chambers
118000-00
Chapter 1- Underground Service Corridor

OAP20

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Drg. Ref.:

Made by JB. Date 08.03.07. Chd.

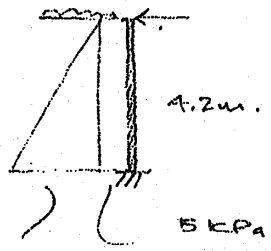
UNDERGROUND CORRIDOR (TAXI RANK) - RETAINING WALL.

Designed as propped cantilever, with earth pressure loads transfred onto existing structure by roof slab.

10 kPa surcharge load

$K_o = 0.5$

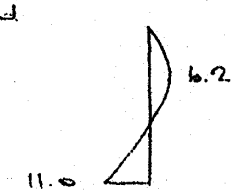
$\delta = 18$



$9 \times 4.2 = 37.8$ kPa.

5kPa load (Unfactored)

BMD

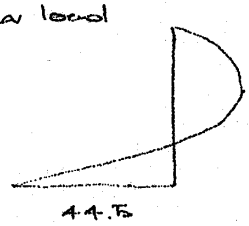


$M_A = \frac{wL}{8} = \frac{5 \times 4.2^2}{8} = 11.0$ kNm/m

$M_C = \frac{9wL}{128} = 6.2$ kNm/m @ 3/8 span = 1.58m from top

Triangular load (Unfactored)

BMD



$M_A = -\frac{2wL}{15} = 2 \times \frac{37.8 \times 4.2^2}{2 \times 15} = 44.5$ kNm/m

$M_C = 0.0596wL = 19.9$ kNm/m @ $0.447L = 1.88$m from top.

Ultimate design envelope.

$M_A = 1.4(11 + 44.5) = 77.7$ kNm/m.

$M_{span} = 1.4(6.2 + 19.9) = 36.5$ kNm/m.

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ARUP | Calculation sheet

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UNDERGROUND CORRIDOR CONT...

By inspection of services trench cantilever moments, can adopt same details for earth side vertical bars.

Check span moment bars.

$\frac{M_{span}}{bd^2} = \frac{36.9 \times 10^6}{10^3 \times 235^2} = 0.66$; $z = 0.95d = 223$ mm

$A_s = 375 mm^2/m$ T16@190p 1340 mm^2/m ok.

250 slab

Finishes 100 mm	2.4	KN/m²
Slab o/w	6.0	"
ceiling services	0.4	"
	8.8	"

ult. load 12.3 KN/m²

LL 5.0 8.0

20.3 KN/m².

$M = \frac{20.3 \times 4^2}{8} = 40.6$ KN/m

$d = 250 - 30 - 15$ say $= 205$ mm

$\frac{M}{bd^2} = 0.97$ $z = 0.93d = 195$ mm $A_s = 476 mm^2/m$

T16@150p

Base slab

Nominally cast onto ground provide minimum steel

$0.15 \times \frac{400 \times 1000^3}{10^2} = 600 mm^2/m$

T16@150p.

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	Made by JB	Date 08-03-07	Chd.

200 wall UNDERGROUND CORRIDOR CONST.

check slenderness: $\frac{4200}{200} = 21$

Tie in to existing brickwork to provide lateral restraint at 2 levels.

Min steel sufficient
$0.4 \times 2000 = 800\ mm^2/m$

T12@150 EF $= 2 \times 753 = 1506\ mm^2/m$ ✓ ok.

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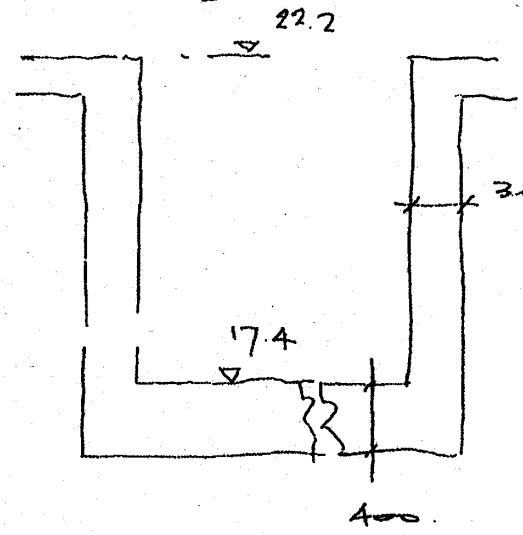
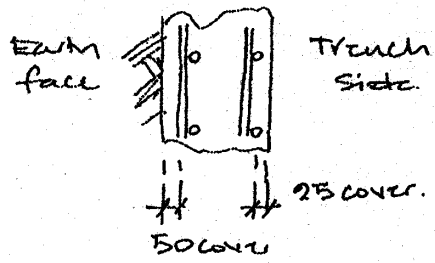
Made by JD Date 02.07 Chd.

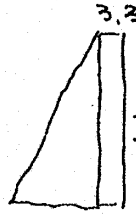
Platform level services trenches - TAXI RANK.



Rebar arrangement.



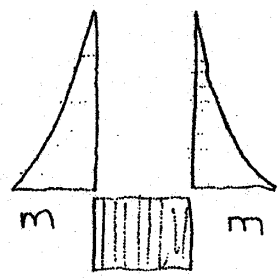
Retained soil height = 4.8 m
Design for cantilever condition with 10kPa surcharge
Granular back fill $\gamma = 18$ $K_a = 0.333$



$F_1 = 3.3 \times 4.8 = 15.8$ kN/m

$F_2 = \frac{32.1 \times 4.8}{2} = 77.1$ kN/m

$4.8 \times 0.33 \times 18 + 3.3 = 32.1$ kPa



$m = 15.8 \times \frac{4.8}{2} + 77.1 \times \frac{4.8}{3} = 161$ kNm/m

$d = 200 - 50 - 15$ say $= 235$ mm

$\frac{m}{bd^2} = 2.92$

C40 concrete $z = 0.91d = 214$ mm

$A_s = 1720$ mm^2/m

T25 @ 150p 3273 mm^2/m

Use T20 @ 150 in upper section and elsewhere use T16 @ 150 p.

Use T12 horizontal bars. 150p.

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ARUP | Calculation sheet

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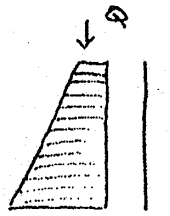
Job Title: ST PANCRAS CHAMBERS

Drg. Ref.

Made by JB Date 30.04.07. Chd.

DRAINAGE RC DETAILS

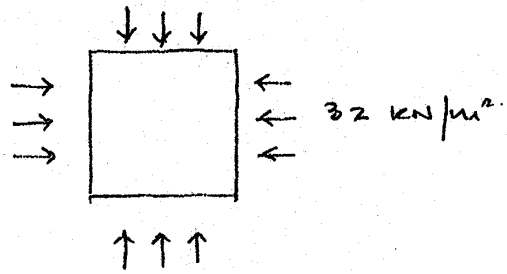
Design walls to span horizontally.
Soil $\gamma = 18$, $K_o = 0.5$ say and



Q = 200 slab plus LL 5.0 kN/m² = 10 kN/m². unfactored.

σ_{max} = 10 x 0.5 + 2 x 18 x 0.5 = 23 kN/m² unfactor
= 23 x 1.4 = 32 kN/m².

Manholes are square in plan max. clear open = 1.5m.



Design as simply supported, span 1.75 m and put rebar in both faces.

W = 32 x 1.75 = 56 kN/m
M = 56 x 1.75 / 8 = 12.3 kNm/m

h = 250 ; 40 cover to 10mm bar say; d = 205
M/bd² = 0.29 z = 0.95d = 195mm
C35 Concrete
A_s = 12.3 x 10⁶ / 195 x 437 = 145 mm²/m

$A_{s\,min}$ = 0.15 x 2050 = 308 mm²/m

Provide 0.2% in each face vertically & horizontally as min

T10@150p.

Provide T12@150p in base.

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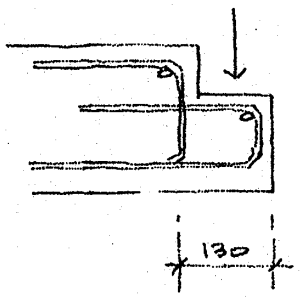
DRAINAGE RC DETAILS CONST...

4.5 KN point load from manhole cover on 600 mm length



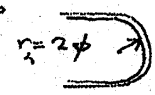
25 top cover

35 bottom cover

60

max. link dimension
= 125 − 60 = 65

max. bar size
= 65/6 = 11 mm.



Use 8mm bars.

P = 1.6 x 4.5 = 7.2 KN

M/125 = 7.2 x 0.13 = 0.94 KNm

h = 125
- 25 cover
- 8 ϕ
- 4 ϕ/2
= 85 say.

$\frac{M}{bd^2} = \frac{0.94 \times 10^6}{600 \times 85^2} = 0.21$ min rebar sufficient

0.15% = 128 mm²/m

T8@150c = 335 mm²/m

St Pancras Chambers
118000-00
Chapter 2- Glazed Screen Footings

OAP20

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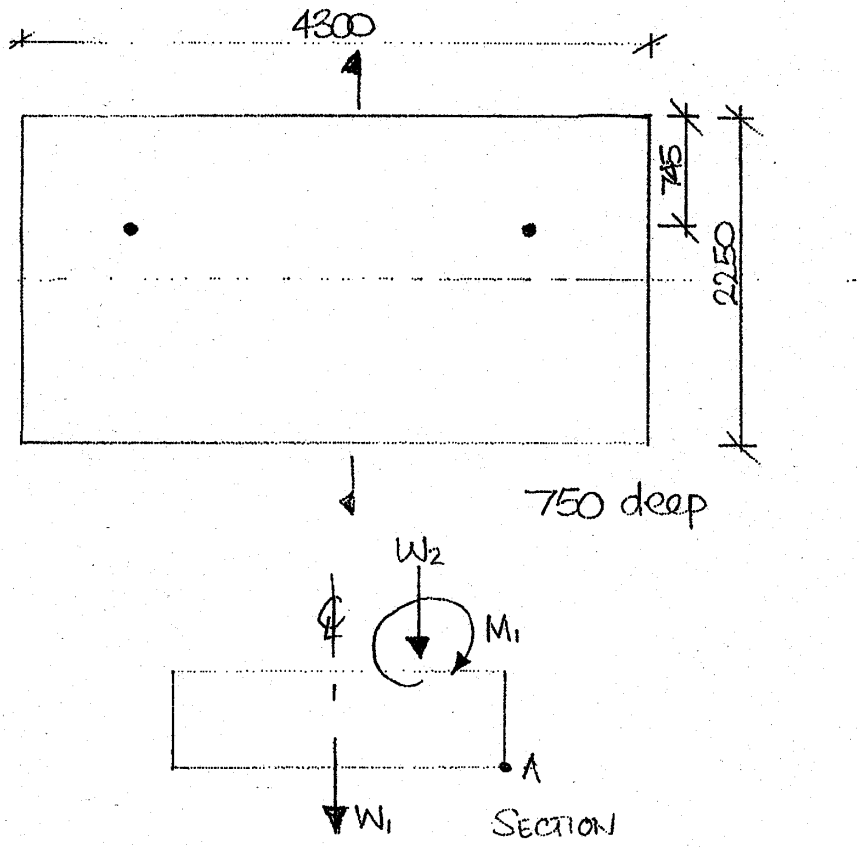
	Member/Location INTERNAL GLAZED SCREEN		
Job Title ST. PANCRAS CHAMBERS	Drg. Ref.		
	Made by KDB	Date 08/06/07	Chd.

REINFORCEMENT FOR INTERNAL GLAZED SCREEN PAD FOOTING



W_1 = Pad SW = 175 kN

W_2 = Screen SW = 48 kN

M_1 = Wind overturning = 120 kNm

Bending Reinforcement

Moments about ℄ $= (W_2)\left(\frac{2.25}{2} - 0.745\right) + M_1$

$= 18.2 + 120$

≈ 138 kNm

Total vatical load = $W_1 + W_2 = 223$ kN

$\frac{M}{N} = \frac{138}{223} = 0.62 > \frac{D}{6} = \frac{2.25}{6} = 0.375$ ⇒ Outside middle third

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ARUP | Calculation sheet

Job No.	Sheet No.	Rev.
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Member/Location: INTERNAL GLAZED SCREEN

Job Title: ST. PANCRAS CHAMBERS

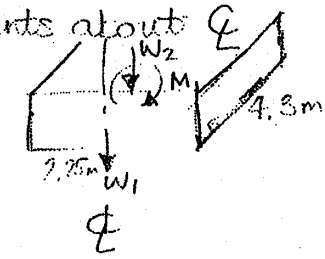
Drg. Ref.

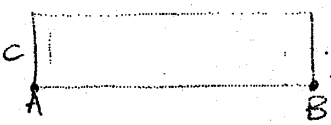
Made by: KDB Date: 11/06/07 Chd.

Pressure beneath footing:

$\sigma_b = \frac{P}{A} \pm \frac{M}{Z}$

Moments about ℄

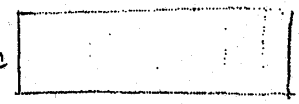




$\frac{P}{A} = \frac{W_1}{(4.3)(2.25)} = 18 \ \frac{kN}{m^2}$

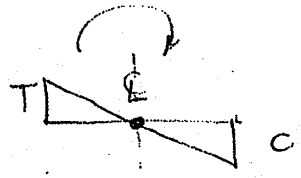
℄

+



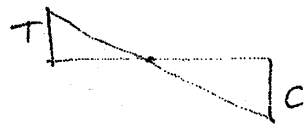
$\frac{W_2}{A} = \frac{48}{(4.3)(2.25)} + \frac{M}{Z}$

$= 5 \ kN/m^2$

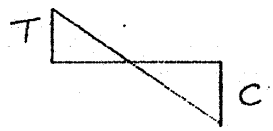


$\frac{M}{Z} = \frac{(W_2)(0.38)(6)}{(4.3)(2.25)^2} = 5.0 \ kN/m^2$

$+ \frac{M_1}{Z} = \frac{(120)(6)}{(4.3)(2.25)^2} = \pm 38 \ kN/m^2$



$\Rightarrow$



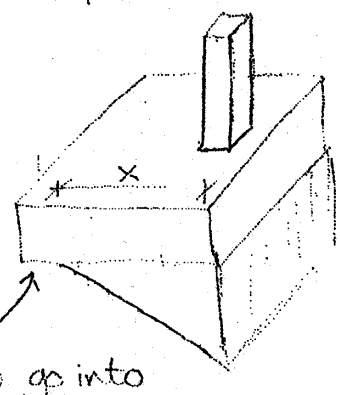
@ A = 18 + 5 − 5 − 38 = −15 kN/m²

@ B = 18 + 5 + 5 + 38 = +61 kN/m²

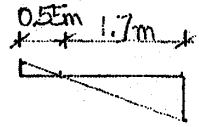
$\sigma_{max\ tension} = -15 \ kN/m^2$

$\sigma_{max\ comp} = +61 \ kN/m^2$

$\Rightarrow$



61 kN/m² = P



Soil cannot go into tension ⇒ 0 pressure here, but ⇒ comp. is carried over a smaller area

e.g. 213 / (1.7 × 1.3) = 31 kN/m² < 100 kN/m²

See previous calcs

- bearing pressure is low

⇒ ok even if bearing area is reduced

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Member/Location

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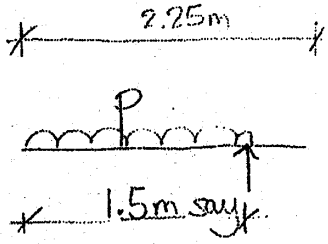
Drg. Ref.

Made by KJB Date 11/06/07 Chd.

Bending Rebar: At face of column



Assume pressure on base bends pad footing — inverted footing

Say $p = (1.4)(61) = 85.4\ kN/m^2$ $[1.4DL + 1.4WL]$

$\Rightarrow M = \frac{(85.4)(1.5)^2}{2} = 96\ kNm$ per m width

$d = 750 - 75 - \frac{20}{2} = 665\ mm$

$k = \frac{96 \times 10^6}{(40)(1000)(665)^2} = 0.005 < 0.156$

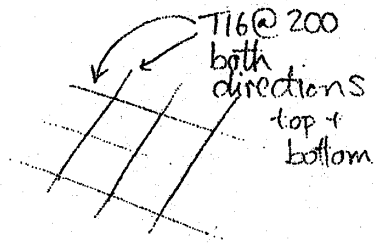
$z = 0.95d = 632\ mm$

$A_s = \frac{96 \times 10^6}{(0.95)(460)(632)} = 348\ mm^2/m$ $\Leftrightarrow$ T16@300 $A_s = 670\ mm^2/m$

Minimum req'd $= \frac{0.13}{100} \times 750 \times 1000 = 975\ mm^2/m$

Say T16 @ 200, $A_s = 1006\ mm^2/m$ bottom bars

Say the same for top, and in opposite directions.



Conservatively could assume $100\ kN/m^2$ (allowable bearing pressure)

$\Rightarrow M = \frac{(100)(1.4)(1.5)^2}{2} = 158\ kNm$

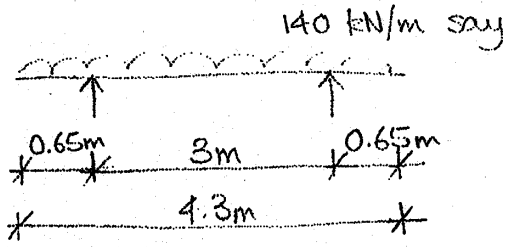
$k = 0.009 \Rightarrow z = 632\ mm$

$A_s = 572\ mm^2/m \Rightarrow$ Minimum steel applies

OAP20

ARUP | Calculation sheet	Job No. 118000	Sheet No.	Rev.
	Member/Location		
Job Title: ST. PANCRAS CHAMBERS	Drg. Ref.		
	Made by KDB	Date 11/06/07	Chd.

Check other direction:



$M = \frac{(140)(3)^2}{8} = 158$ kNm governs but similar to other direction ⇒ Say min steel T16@200.

or

$M = \frac{(140)(0.65)^2}{2} = 30$ kNm

OAP20

ARUP | Calculation sheet

Job No.	Sheet No.	Rev.

Member/Location

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Shear – Maximum on face

V = 175 + 48 = 223 kN

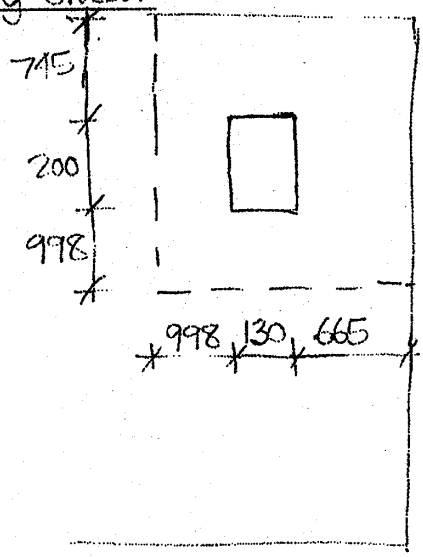
Face of column: 203 × 102 × 43 UB.

Dimensions approx.: 200 × 130 mm

At face of column:

$$\frac{223 \times 10^3}{(665)[(2)(200)+(2)(130)]} = 0.5 \text{ N/mm}^2 < 5 \text{ N/mm}^2 \Rightarrow \text{OK}$$

Punching Shear



1.5d = (1.5)(665) = 998

Length of perimeter = 3736 mm

Previously = 2640 mm

$$v = \frac{223 \times 10^3}{(2640)(665)} = 0.13 \text{ N/mm}^2$$

$$\frac{100A_s}{bd} = \frac{(100)(1006)}{(1000)(665)} = 0.15$$

From Table 3.8 of BS8110: Part 1

$$v_c = (0.34)\left(\frac{40}{25}\right)^{1/3} = 0.40 \text{ N/mm}^2 > v \Rightarrow \text{OK.}$$

⇒ No links req'd for punching shear

By inspection, works even better where perimeter = 3736mm

OAP20

ARUP | Calculation sheet

Job No.	Sheet No.	Rev.

Member/Location

Job Title

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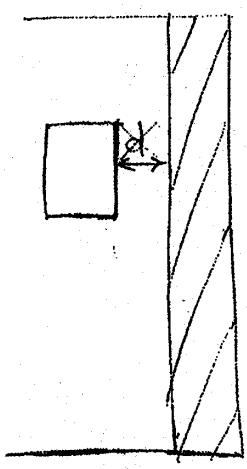
Made by | Date 11/06/07 | Chd.

Transverse Shear

500 − d = 165 mm, say 500 wide strip



d = 665 mm

(offset allows for shear enhancement — ignore in this case)

$(85.4)(2.25)(0.5) = 96$ kN (ULS)

[or if $(100)(1.4)(2.25)(0.5) = 157.5$ kN]

$$v = \frac{157.5\times10^3}{(2250)(665)} = 0.11 \text{ N/mm}^2$$

$v_c = 0.40 \text{ N/mm}^2 > v \Rightarrow$ OK without links.

By inspection OK in opposite direction

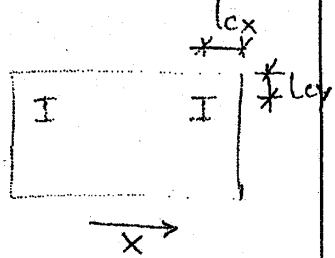
Reinforcement Arrangement

$l_{cx} = 500$ mm say

$$3c/4 + 9d/4 = \frac{(3)(130)}{4} + \frac{(9)(665)}{4}$$

$$= 97.5 + 1496$$

$$= 1594 \text{ mm} > l_c$$



$\Rightarrow$ Uniformly distribute rebar over l_c

$\Rightarrow$ This concerns rebar running in direction X.

$l_{cy} = 745$mm $< 1594 \Rightarrow$ Evenly distributed.

OAP20

ARUP | Calculation sheet

Job No.	Sheet No.	Rev.
11800		

Member/Location

Job Title: ST. PANCRAS CHAMBERS

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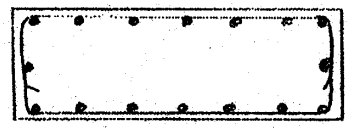
Made by KDB Date 11/06/07 Chd.

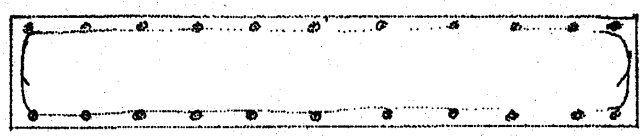
Add to S_4042



← 2.25m →

T16@200 top (for cracking)
2T16 side bars
T16@200 bottom

Say all T16@150 to suit wall rebar (Wall rebar S_4041)



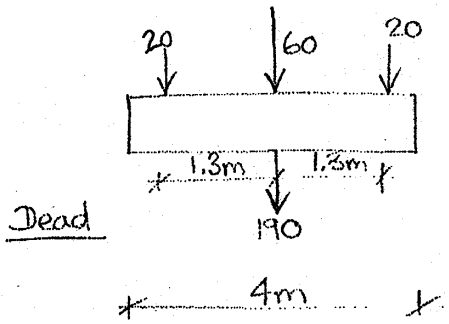
← 4.8m →

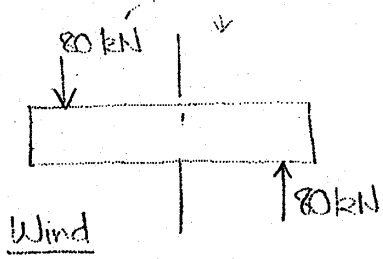
T16@200 top (for cracking)
2T16 side bars
T16@200 bottom

OAP20

ARUP | Calculation sheet	Job No. 118000	Sheet No.	Rev.
	Member/Location SOUTH SCREEN FOOTING		
Job Title ST. PANCRAS CHAMBERS	Drg. Ref.		
	Made by KJB	Date 11/07/06	Chd.

REINFORCEMENT FOR SOUTH GLAZED SCREEN PAD FOOTING





SLS Loads.

Footing is 6600 x 4000 x 300 deep raft

Total vertical DL = $20 + 20 + 60 + 190 = 290$ kN

Moment about ℄ due to dead load = $(20)(1.3) - (20)(1.3)$
$+ (80)(1.3) + (80)(1.3)$
$= 208$ kNm

Stress beneath raft:

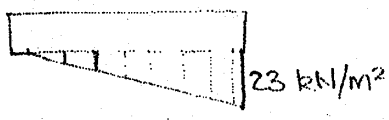
$$\frac{P}{A} + \frac{M}{Z} = \frac{290}{(6.6)(4)} \pm \frac{(208)(6)}{(6.6)(4)^2} = 11 \pm 11.8$$

⇒ Compression = $22.8 \approx 23$ kN/m²

Tension = $-0.8 \approx -1$ kN/m²

$\frac{290}{(6.6)(4)} = 11$ kN/m² << 100 kN/m² allowable bearing pressure

⇒ By inspection, ok.



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ARUP | Calculation sheet

Job No.	Sheet No.	Rev.
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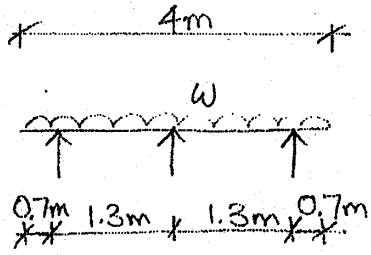
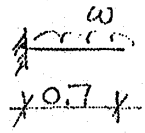
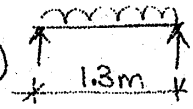
Bending Reinforcement



Invert raft

Check a)  and b) 

$w = 100\ kN/m^2$ say (conservative - max allowable bearing pressure)

$\Rightarrow 100 \times 1.4 = 140\ kN/m$ ULS per m width

a) $M = \frac{(0.7)^2(140)}{2} = 34\ kNm$ $\Rightarrow$ Cantilever governs

b) $M = \frac{(140)(1.3)^2}{8} = 30\ kNm$

$d = 300 - 75 - \frac{20}{2} = 215mm$

$k = \frac{34 \times 10^6}{(40)(1000)(215)^2} = 0.018$

$z = 0.95d = 204mm$

$A_s = \frac{34 \times 10^6}{(0.95)(460)(204)} = 381\ mm^2/m$, say T16@200 top + bottom $A_s = 1006\ mm^2/m$

Min. req'd $= \frac{0.13}{100} \times 215 \times 1000 = 280\ mm^2/m$

Clause 3.11.3.2

$\frac{3c}{4} + \frac{9d}{4} = \frac{(3)(125)}{4} + \frac{(9)(215)}{4} = 94 + 484 = 578mm$

$L_c = 1250/2$ or $250mm \Rightarrow 625 > 578$ but rebar is >> than req'd $\Rightarrow$ will space evenly.

OAP20

ARUP | Calculation sheet

Job No.	Sheet No.	Rev.

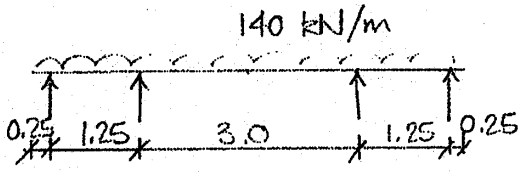
Member/Location

Job Title

Drg. Ref.

Made by | Date 11/06/07 | Chd.

Other direction:



$M = \frac{(140)(3)^2}{8} = 158$ kNm governs

$M = \frac{(140)(0.25)^2}{2} = 4.4$ kNm

Main rebar – top face

$d = 300 - 75 - \frac{20}{2} = 215$ mm

$k = 0.085$

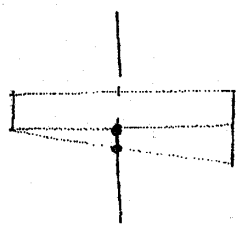
$z = 0.89d = 191$ mm

$A_s = 1893$ mm²/m Say T20@150

$A_s = 2095$ mm²/m

but take pressure at say ℄

$\Rightarrow \omega = \frac{140}{2} = 70$ kN/m



$M = 79$ kNm

$k = 0.043$

$z = 0.95d = 204$ mm

$A_s = 886$ mm²/m Say T16@200, $A_s = 1006$ mm²/m.

Say T16@150 to match slab rebar

Will say this as it is more realistic at max bending location.

Face Shear

$\frac{60}{2} = 30\ kN = V$

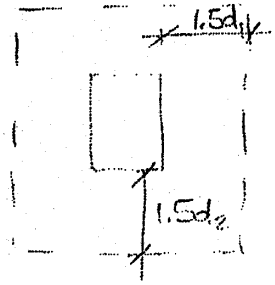
305 x 127 x 48 UB steel column

Say approx. 310 x 125 area.

$$v = \frac{30 \times 10^3}{(215)[(2)(310)+(2)(125)]} = 0.16\ N/mm^2 << 5\ kN/m^2$$

$\Rightarrow$ OK

Punching Shear – Will check a central column (Edge columns carry very small loads)



$1.5d = (1.5)(215) = 323mm$, say $320mm$

Perimeter $= [310 + (2)(320)](2) + [125 + (2)(320)](2)$

$= 1900 + 1530$

$= 3430mm$

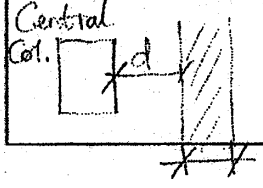
$$v = \frac{30 \times 10^3}{(3430)(215)} = 0.041\ N/mm^2$$

$$\frac{100As}{bvd} = \frac{(100)(1006)}{(1000)(215)} = 0.5 \quad , \quad 200 < d < 225$$

Say $v_c = (0.59)\left(\frac{40}{25}\right)^{1/3} = 0.69\ N/mm^2$

$v < v_c \Rightarrow$ No links required.

Transverse Shear – Check Edge



Say $(100)(1.4)(4)(1.04) = 582\ kN$ (conservative)

$v = 582 \times 10^3 / (4000)(215) = 0.68\ N/mm^2 < 0.69 N/mm^2 \Rightarrow$ OK

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ARUP | Calculation sheet

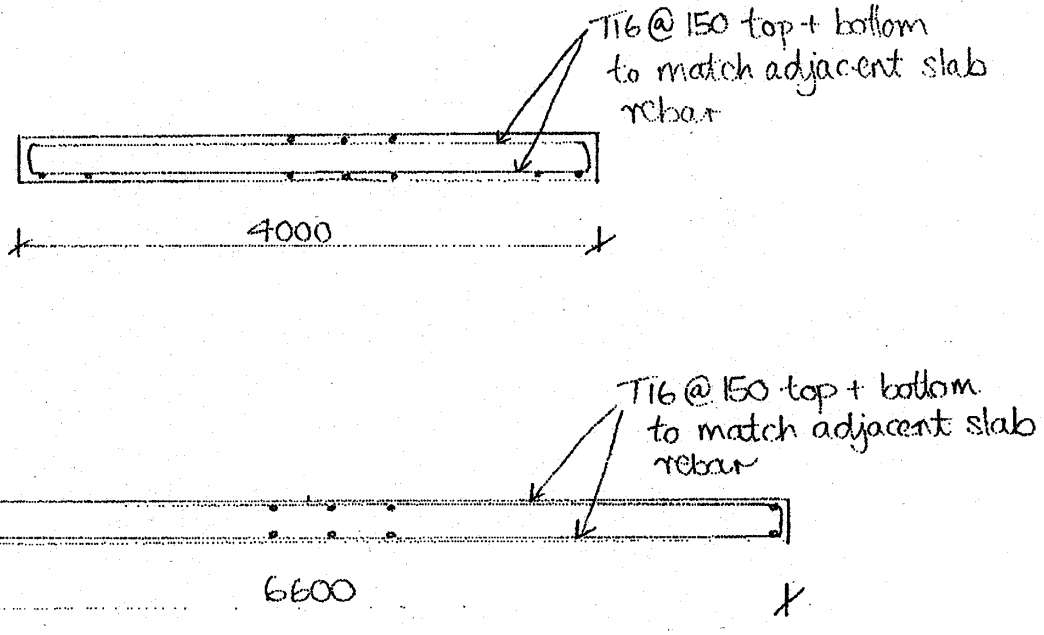
Job No.	Sheet No.	Rev.
118000		

Member/Location: SOUTH SCREEN RAFT

Job Title: ST. PANCRAS CHAMBERS

Drg. Ref.

Made by KJB Date 12/06/07 Chd.

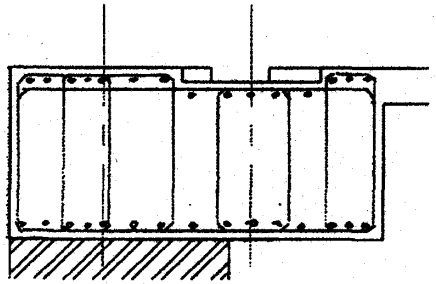


By inspection, small footing for South Screen will be reinforced similarly

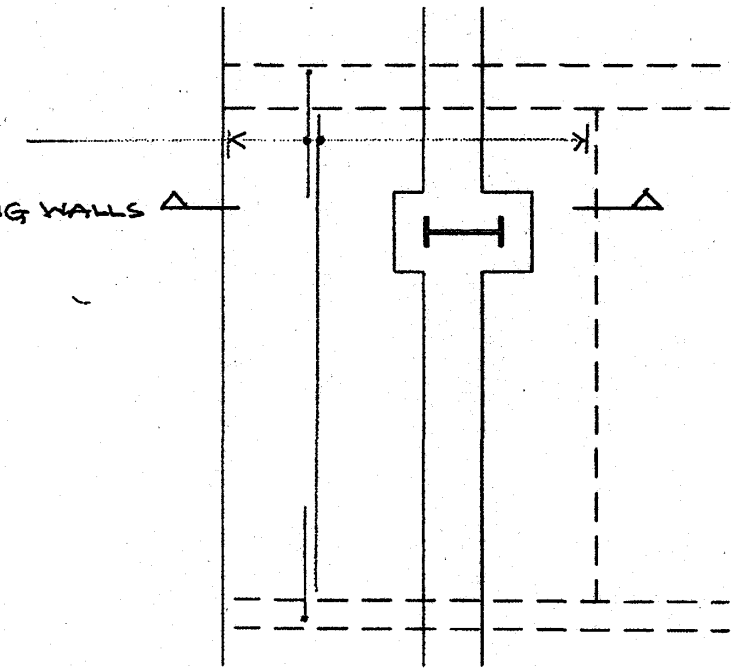
i.e. T16 @ 150 top + bottom, both directions

St Pancras Chambers
118000-00
Chapter 3- Reinforcement details

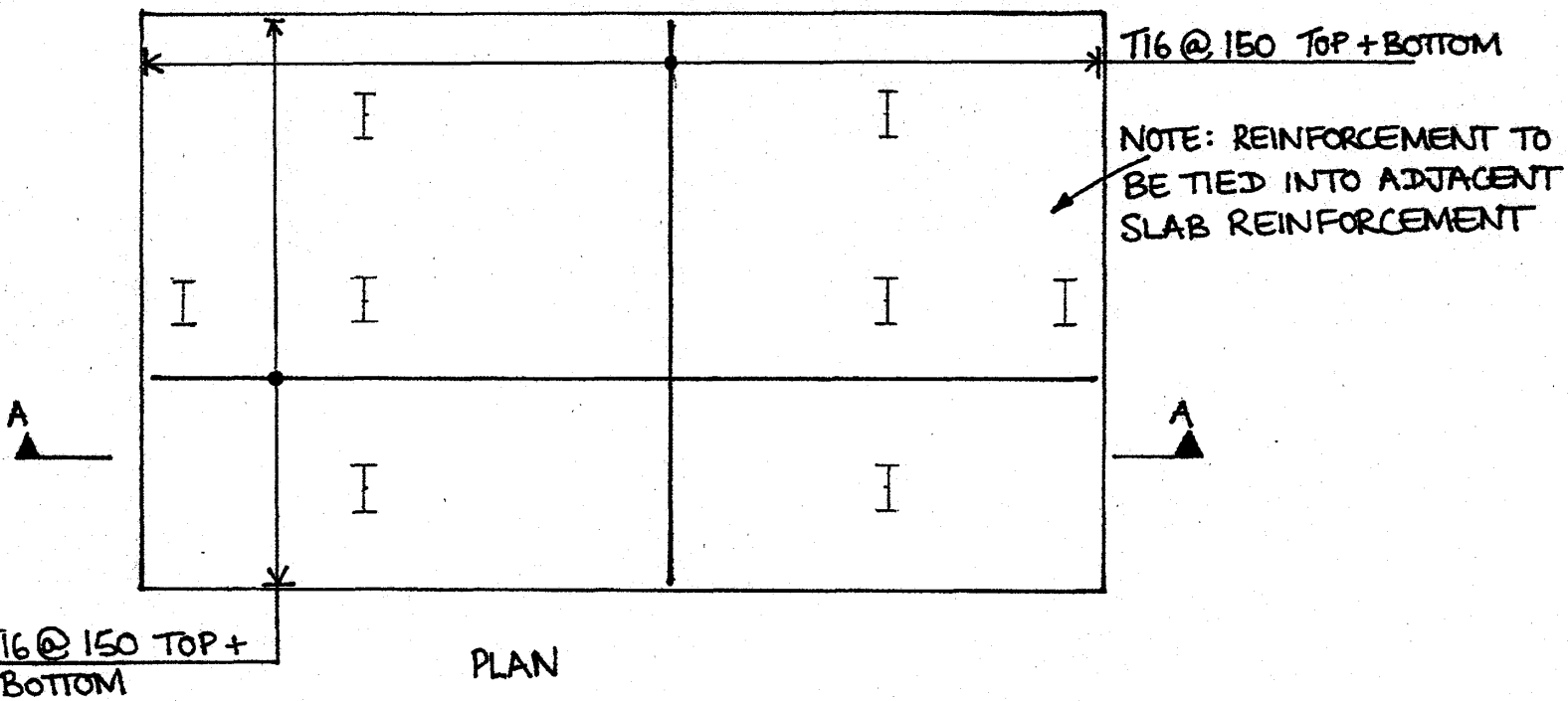
8 LEGS T12 LINKS AT 300 c/c
ALONG BEAM.



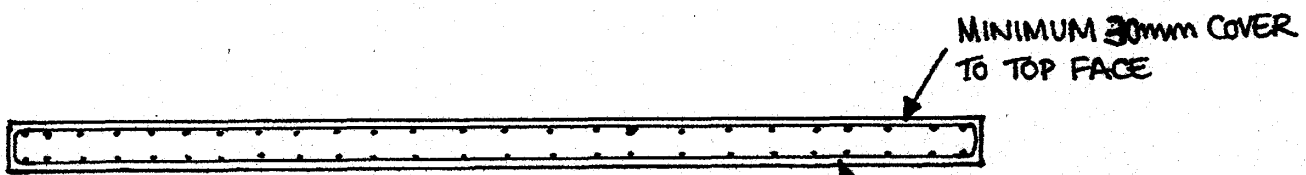
T20@ 150 BOTTOM
T16@ 150 TOP
T16 UB@ 150 AT SUPPORTING WALLS



NORTH.

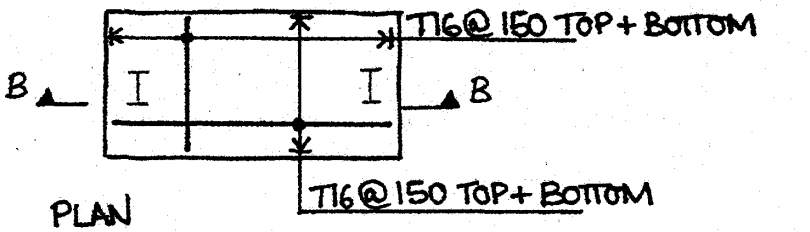


PLAN

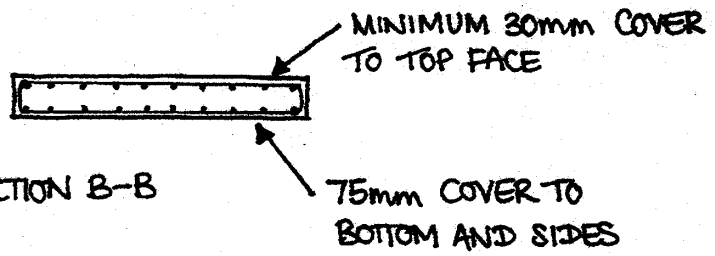


SECTION A-A

SOUTH.



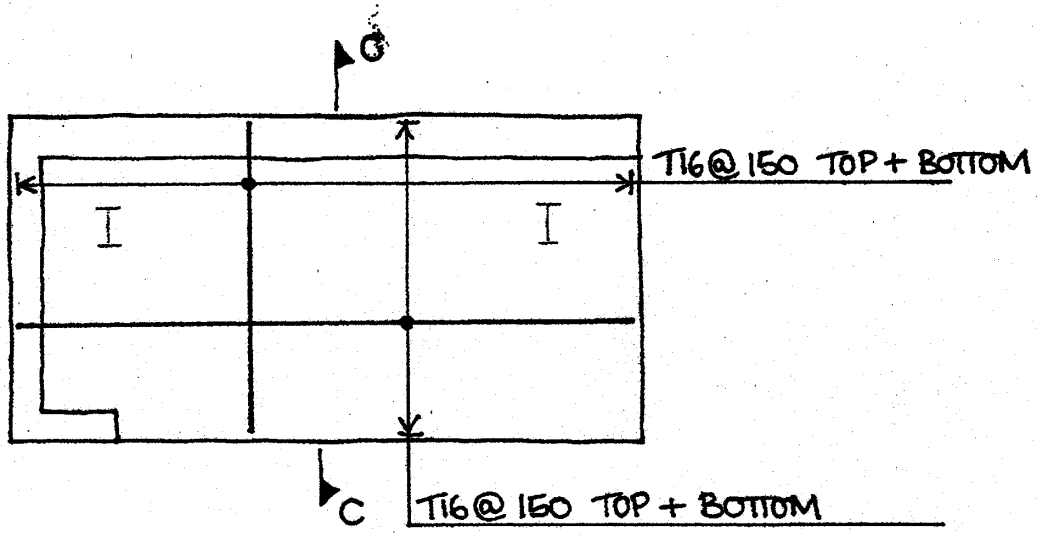
PLAN



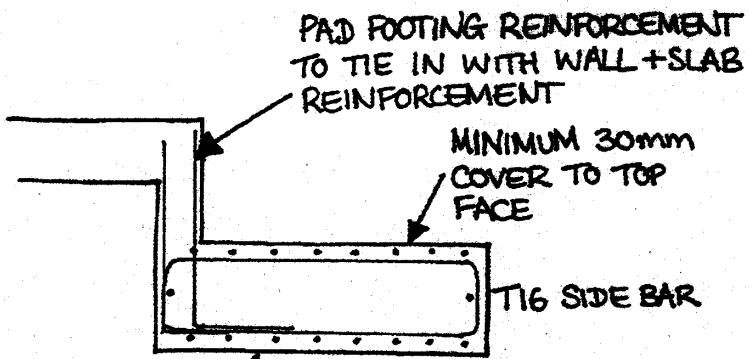
SECTION B-B

NOTE: REINFORCEMENT TO BE TIED INTO ADJACENT SLAB REINFORCEMENT

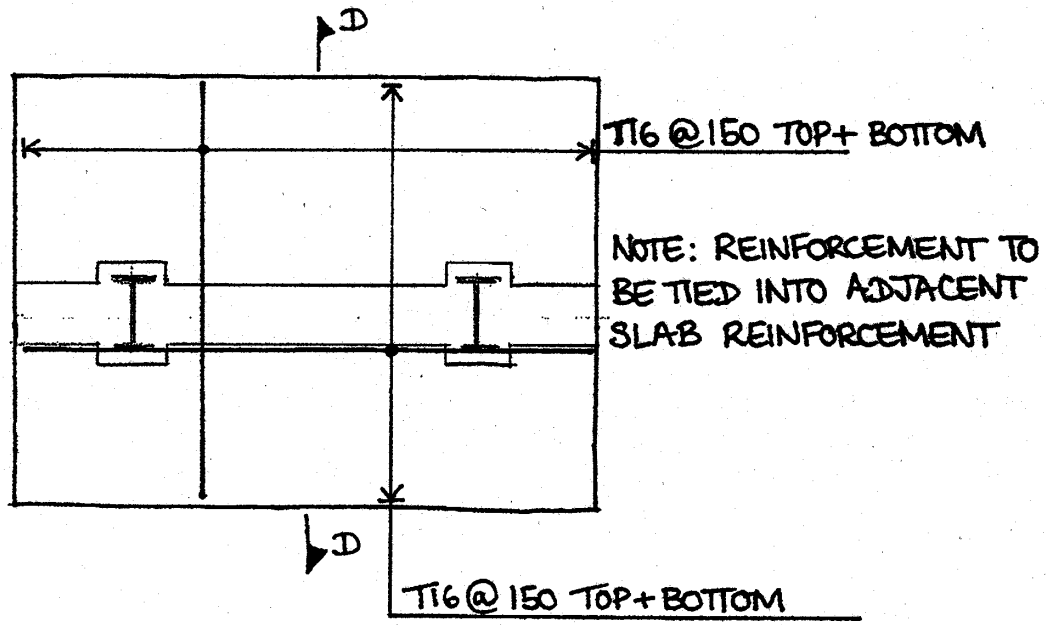
SOUTH.



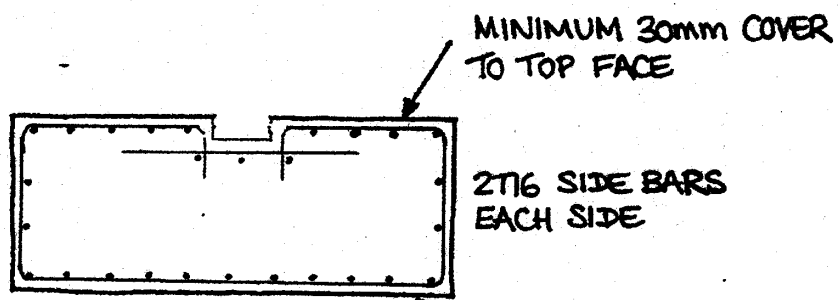
PLAN



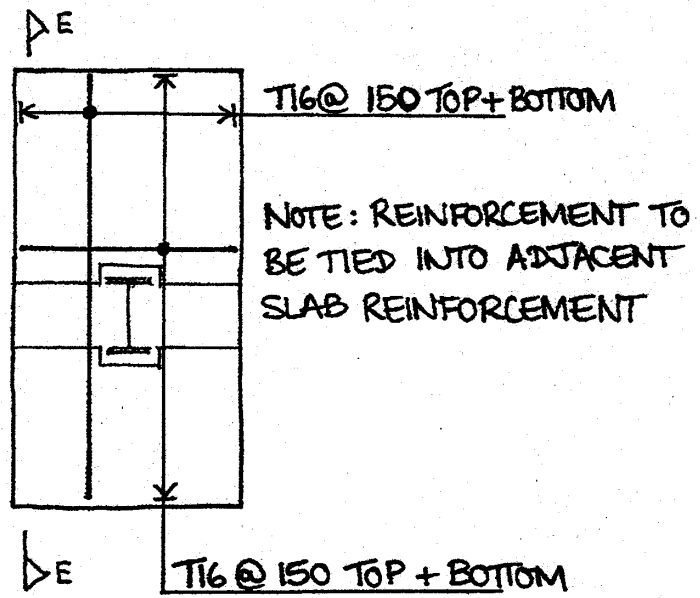
SECTION C-C



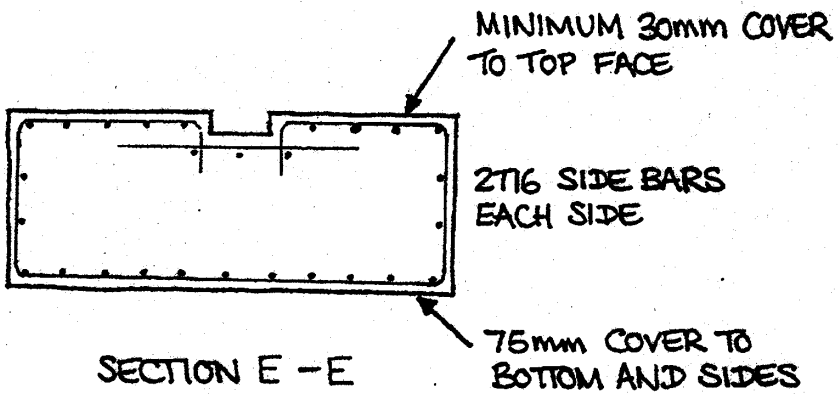
PLAN



SECTION D-D



PLAN



SECTION E – E

NOT TO