



Design it

Structural Solutions Ltd



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Job Ref.

18241

Rev.

0

Date.

June 19

Calculations for:

**35 Intake Lane,
Pogmoor, Barnsley**

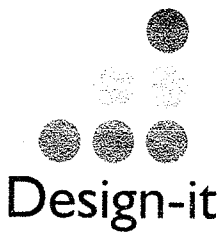
**Structural Design for
Proposed new Build**

Prepared By:

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BEng (Hons) CEng MStructE

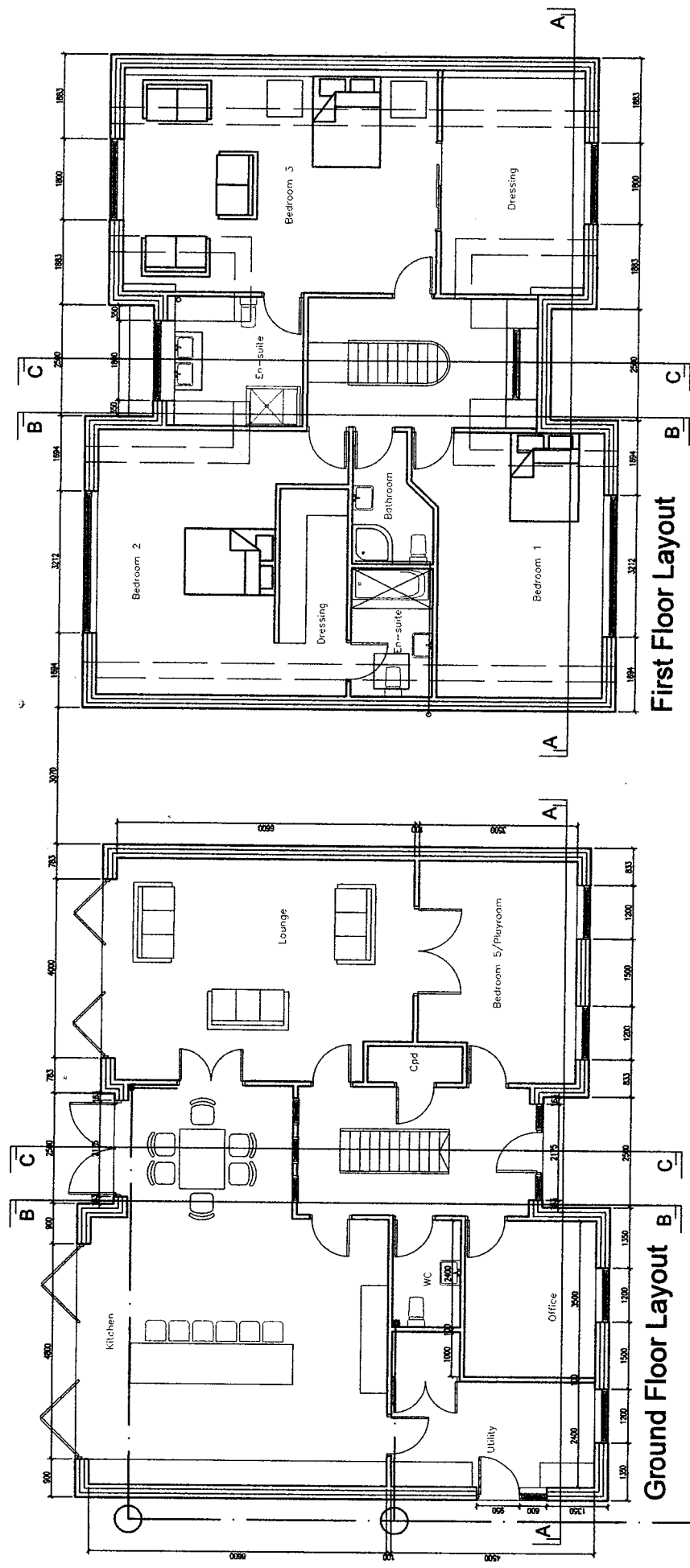
June 2019

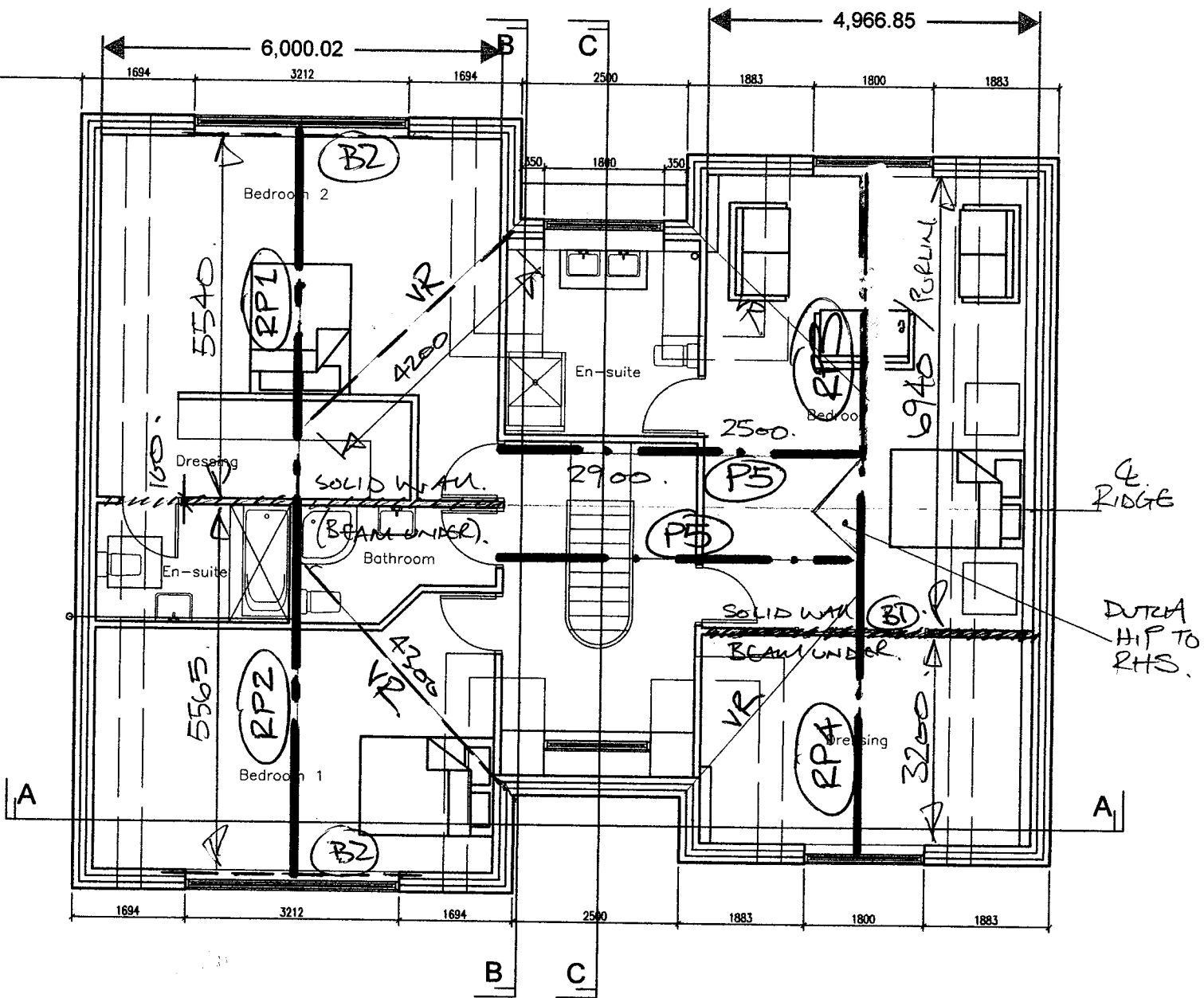
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Ref		Output
	Design Statement It is proposed to build a new detached dormer bungalow on the site at 35 Intake Lane, Pogmoor, Barnsley. The existing property is to be demolished and the new property built over the site of the previous build. Construction is to be masonry external leaf with blockwork inner. First floor is to be traditional timber joists, typically spanning front to back supported on solid G/F walls and steel beams in the depth of the floor. Roof construction is to be steel purlins, off 1st floor walls with single span rafters. The property lies in a known area of mining, as identified by the Coal Mining Risk assessment and site investigation, (produced by others). In this regard it is deemed that a solid raft slab founding in the upper clay layer. The slab will be reinforced top and bottom. These calculations provide details for the roof structure, floor beams, structural lintels and raft slab foundation. Reference should be made to all Architects drawings for proposed layouts <i>Dimensions defined in these calculations are defined here-in for design purposes only.</i> <i>All dimensions defined are indicative and must be confirmed by the builder or steel fabricator on site prior to ordering any steel sections. All building setting out is to be provided by the Architect.</i> <i>Beam lengths must allow for a min of 100m end bearing to each end, unless noted otherwise</i>	

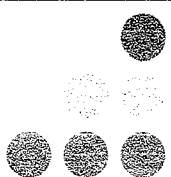
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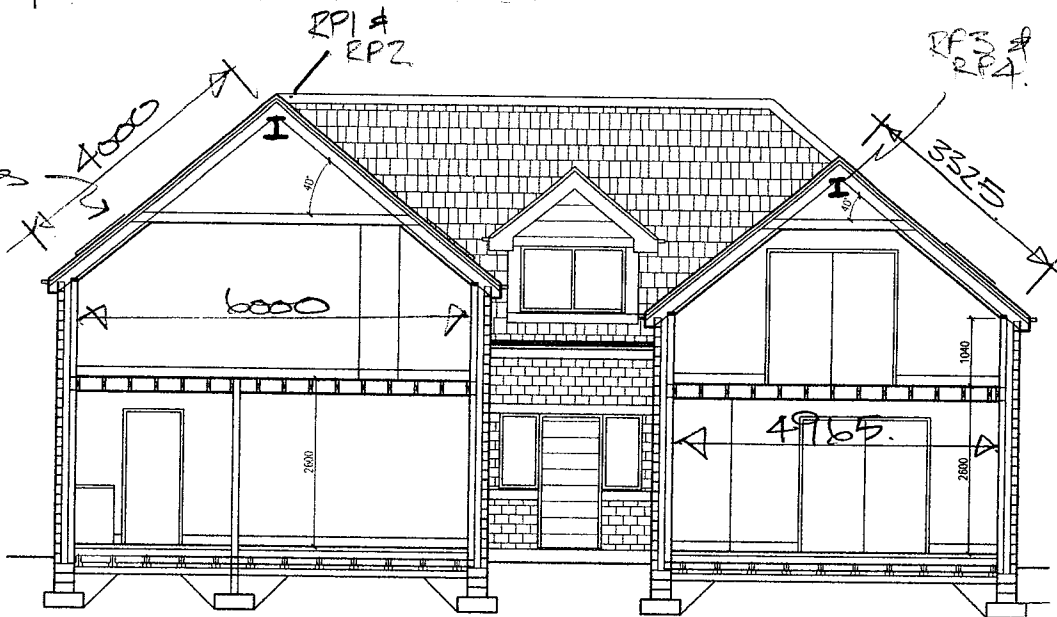
Ref	Loadings (BS 6399 Pt 1)			Output
	Dead		kN/m ²	
	House Roof	Tiles + Rafters (inc felt + battens)	0.65	
		Insulation	0.08	
		Lining / Ceiling PB + skim + services	0.22	
			0.95	
	Floors	Joists + Flooring	0.3	
		Insulation	0.07	
		Ceiling PB + skim	0.19	
			0.56	
	Walls	Masonry outer leaf	2.17	
		Blockwork + plaster	1.95	
		Blockwork + Plaster both sides	2.1	
		Studwork	0.8	
	Imposed			
		Ceiling Tie	0.25	
		Roof < 30°	0.6	
		Roof > 30°	0.75	
		Snow at change in height	1.5	
		Floor (Domestic)	1.5	
	Deflection limits	Beams general	L/250 (long term) (D+S)	
		Beams over external windows & Doors	L/500	
		Purlins (roof)	L/250 (med Load)	
		Floor joists	Lx0.003 (Long term)	

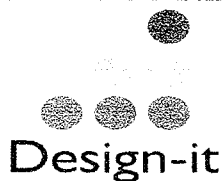




PLAN ON ROOF
SUPPORTS.

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Ref	Output
<u>ROOF STRUCTURE</u>  Section A-A	
<u>RAFTERS - R1 (TYPICAL).</u> ADOPT SLOPE SPAN = 4m. REFER TO SHT (6) FOR RAFTER SECTION DESIGN. ADOPT. <u>200 x 50 GR C16</u> <u>RAFTERS AT 450 C/C.</u> (R1)	



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Timber Design

Design of Rafters to Proposed Rear Extension - R1

Loadings (serviceability)

Dead 0.95 kN/m²
Imposed 0.6 kN/m²

Joist size

Width, b mm 50 Rafter c/c 450 mm
Depth, d mm 200 Rafter span 4 m

Section Modulus , Z = 333.333333 cm³

Applied Moment = 1.40 kNm

Applied bending stress = 4.19 N/mm² (M/Z)

Permissible Bending Stress BS 5268 Tables 8 - 15

Timber grade **C16**
Bending stress = 5.6 N/mm²

Modification factors

K3 = 1.00 Load duration cl 2.8 (1-long, 1.25-med, 1.5-short)
K7 = 1.0456 Depth mod factor 2.10.6
K8 = 1.10 Load Sharing cl 2.9 (4 or more @ 610 max c/c)

Modified Permissible Bending Stress = 6.44 N/mm²
Bending Stress OK

Deflection

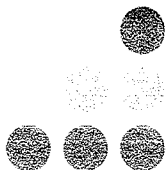
E = 8500
I = 33333333.33 mm⁴
UDL 0.27 kN/m

Imposed Load Deflection = 3.18 mm

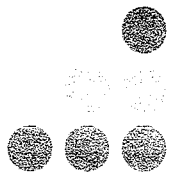
Allowable = 12 mm **Deflection OK**

Provide Min Section - Rafters R1

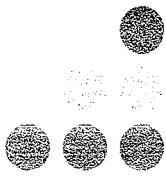
200 x 50 Grade C16 Rafters @ Max 450 c/c

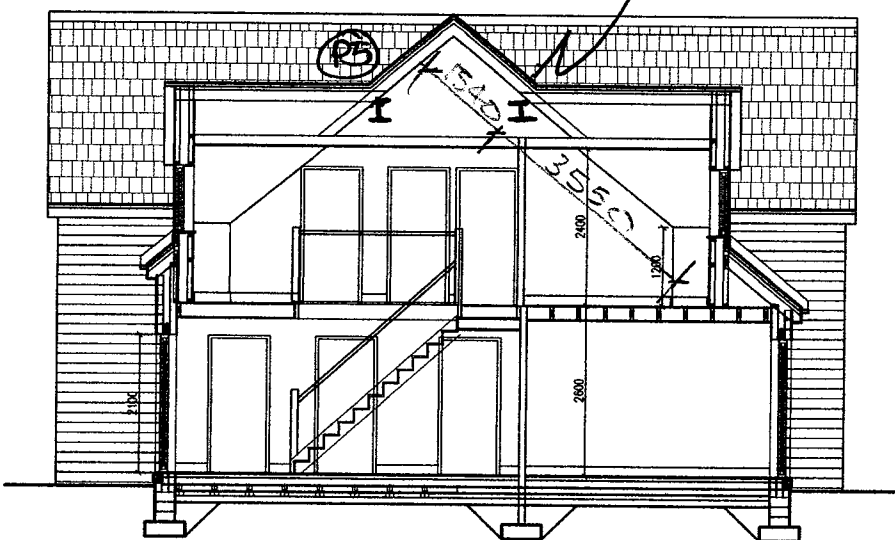
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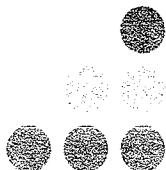
Ref		Output
	<u>DESIGN OF RIDGE PURLINS RP1 & RP2</u> ADOPT MAX SPAN = 5.565m. ∴ ROOF SPAN = 4m. ∴ UDL = $4 \left[(0.95 \times 1.4) + (0.6 \times 1.6) \right] = 9.2 \text{ kN/m.}$ ∴ $M_{\max} = 9.2 \times \frac{5.565^2}{8} = 35.61 \text{ kN.m.}$ $I_{xx \text{ REQD.}} = \frac{1.8557 \times 6.133 \times 5.565^3}{(4/300)} = 1961 \text{ cm}^4.$ $F_v = 26 \text{ kN.}$ ADOPT $L_e = 1.0$ (RAPTERS BEAMS MOUNTED ON TO TOP PLATING). ∴ ADOPT <u>203 x 133 x 25 kg/m UB.</u> $M_{ux} = 71.0 \text{ kN.m} > 35.61 \text{ kN.m.}$ $I_{xx \text{ PROV}} = 2356 \text{ cm}^4 > 1961 \text{ cm}^4 \text{ } \angle \text{OK}$	RP1 & RP2.

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Ref		Output
	<u>DESIGN OF PURLINS RP3 & RP4.</u> SPAN = 6940 mm. Roof Slope = 3325 mm. $L_{UDL} = 3.325 \left[(0.95 \times 1.4) + (0.6 \times 1.6) \right] = 7.61 \text{ kN/m.}$ $\therefore M_{max} = \frac{7.61 \times 6.94^2}{8} = 45.82 \text{ kN.m.}$ $I_{xx \text{ reqd.}} = \frac{1.8557 \times 5.1 \times 6.94^3}{(L/300)} = 3163 \text{ cm}^4.$ $F_v = 26 \text{ kN.}$ $L_b = 1.0$ (BATTERS BIRDS MOUNTED ON TOP FLANGE). $\therefore$ ADOPT <u>254 x 102 x 28 kg/m UB</u> RP3 & RP4. $M_{ex} = 94 \text{ kN.m} > 45.82 \text{ kN.m.}$ $I_{xx \text{ prov}} = 4008 \text{ cm}^4 > 3163 \text{ cm}^4 \text{ } \angle \text{OK.}$	

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Ref		Output
	PURLINS P5.  Section C-C ROOF SLOPE $= \frac{(1.54 + 3.55)}{2} = 2.545 \text{mm.}$ $\therefore \text{UCL} = 2.545 \left[(0.95 \times 1.4) + (0.6 \times 1.6) \right] = 5.83 \text{ kN/m}$ PURLIN SPAN $= 2900 \text{mm.}$ $\therefore M_{\text{max}} = \frac{5.83 \times 2.9^2}{8} = 6.13 \text{ kN.m.}$ $I_{xx} \text{ REQD} = \frac{1.8557 \times 3.89 \times 2.9^3}{(L/300)} = 17 \text{ cm}^4.$ $P_V = 8.5 \text{ kN.}$	

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Ref	Output
	ADOPT LG = 1.0. BY INSPECTION ADOPT (MIN) <u>178 x 102 x 19 kg/m UB</u> (45). <u>VALLEY RAFTERS.</u> ADOPT RAFTER SPAN = 4500mm. Roof SPAN = 3m TO O. — TOTAL LOAD = $0.5 \times 3 \times 4.5 \times \left[(0.95 \times 1.4) + (0.6 \times 1.6) \right]$ = 15.5 kN. — REFER TO SHTS (11) FOR BEAM DESIGN ∴ ADOPT. (MIN). <u>178 x 102 x 19 kg/m UB</u> VALLEY RAFTERS. $FV = (2 \times 15) / 3 = 10 \text{ kN}$

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Job Ref : 35 Intake Lane, Pogmoor

Sheet : /10000

Made by :

Date : 25 June 2019 / Ver. 2015.05

Checked :

Approved :

11

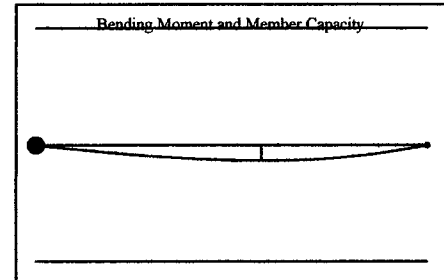
Axial with Moments (Member)

Member 1 (N.1-N.2) @ Level 1 in Load Case 1

Member Loading and Member Forces

Loading Combination : 1 UT + 1.4 D1 + 1.6 L1

UT TY2 -011.000 (kN)

**Member Forces in Load Case 1 and Maximum Deflection from Load Case 2**

Member No.	Node End1 End2	Axial Force (kN)	Shear Force (kN)	Bending Moment (kN.m)	Maximum Moment (kN.m @ m)	Maximum Deflection (mm @ m)
1	1 2	0.000C 0.000C	3.667 -7.333	0.000 0.000	@ 6.351 2.610	@ 4.700 2.340

Classification and Properties (BS 5950: 2000)

Section (19.04 kg/m)

178x102 UB 19 [Grade 43]

Class = Fn(b/T,d/t,py,F,Mx,My)

6.41, 30.58, 275, 0, 6.35, 0

(Axial: Non-Slender)

Plastic

Auto Design Load Cases

1

Moment Capacity Check Mc

Fv/Pv

0.006 / 140.818 =

0

Low Shear

Mc = py.Sxx ≤ 1.2 py.Zxx

275 x 171.3 ≤ 1.2 x 275 x 152.63 =

47.108 kN.m

MA/Mc

6.35 / 47.108 =

0.135

OK

Equivalent Uniform Moment Factor mLT $m_{LT} = 0.2 + (1.5M_2 + 1.5M_3 + 1.5M_4) / M_{max}$ $0.2 + (1.5x4 + 1.5x6 + 1.5x5) / 6 \geq 0.44$

0.906

Table 18

Lateral Buckling Check Mb

Le = 1.00 L

1 x 4.5 =

4.5 m

 $\lambda = Le / r_{yy}$

4.5 / 2.38

189.08

OK

v = Fn(x, Le, ryy, λ)

22.562, 4.5, 2.38, 189.08

0.686

Table 19

 $\lambda_{LT} = u.v.\lambda.\sqrt{\beta_w}$

0.889 x 0.686 x 189.08 √ 1

115.4

pb = Fn(py, λ_{LT})

275, 115.4

101.49 N/mm²

Table 16

Mb = Sxx.pb ≤ Mc

171.3 x 101.49 ≤ 47.108 =

17.385 kN.m

MA/(Mb/m_{LT})

6.35 / (17.385 / 0.906)

0.331

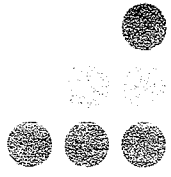
OK

Deflection Check - Load Case 2 $\delta \leq \text{Span} / 360$

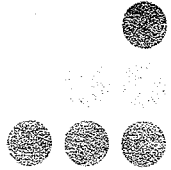
4.7 ≤ 4500 / 360

4.7 mm

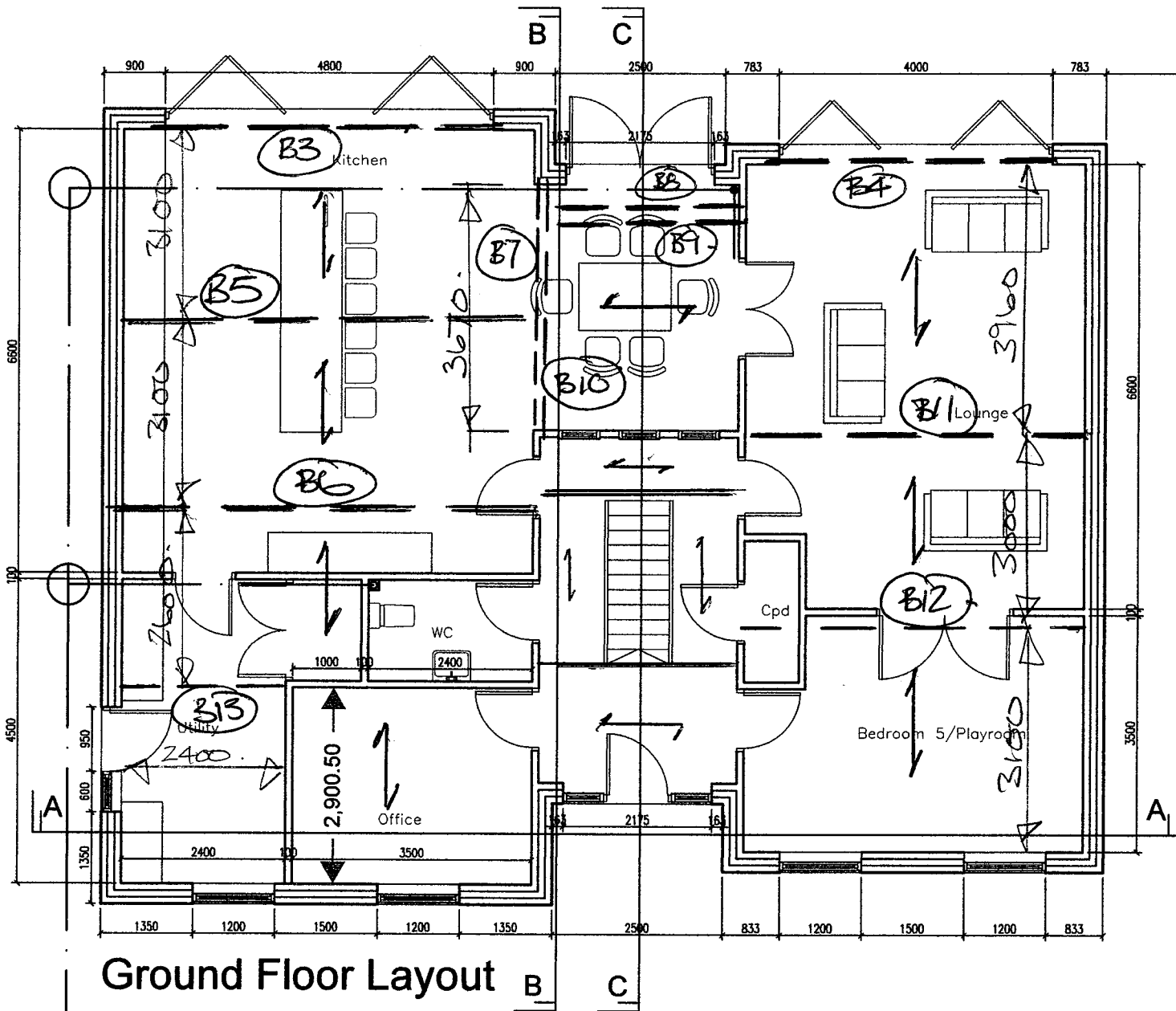
OK

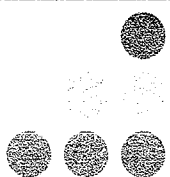
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Ref		Output
	<u>LINTEL OVER BED 3 DOOR.</u> LINTEL TO OPENING IN SOLID WALL BETWEEN BEDROOM AREA & DRESSING ROOM. OPENING WIDTH = 2M. LOADS. WALL = $1.16 \times 2.1 \times 1.4 = 3.4 \text{ kN/m.}$ REACTION FROM RP3 + RP4 = $5.07 \times 7.61 = 38.6 \text{ kN.}$ $\therefore 38.6 / 2.32 = 16.64 \text{ kN/m.}$ $\sum = 20 \text{ kN/m.}$ $\therefore M_{\text{max}} = 20 \times 2^2 / 8 = 10 \text{ kN.m.}$ $I_{\text{xx REQD.}} = 1.8557 \times 13.33 \times 2^3 = 200 \text{ cm}^4$ (L/300) $\therefore$ BY INSPECTION. ADOPT <u>178x102x19 kg/m UB</u> (BI) <u>PADSTONE.</u> PROVIDE <u>200 LG x 100 WIDE x 150DP</u> <u>CONCRETE PADSTONE.</u>	

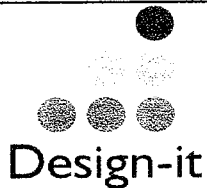
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	<u>LINTELS TO 1ST FLOOR FRONT & REAR</u> <u>WINDOWS TO BEDS 1 & 2.</u> ASSUME STANDARD CATNIC LINTEL WITH ADDITION OF BEAM TO INNER LEAF. LOADS WALLS $= 1.5 \times 2.17 \times 1.4 = 4.56 \text{ kN/m.}$ INNER $= 1.5 \times 1.95 \times 1.4 = 4.1 \text{ kN/m.}$ END REACTION FROM RIGHT $= 26 \text{ kN.}$ $\therefore 26/4 = 6.5 \text{ kN/m.}$ TOTAL $= 15.16 \text{ kN/m.}$ INNER $= 10.6 \text{ kN/m.}$ $\therefore M_{\text{max}} = \frac{10.6 \times 3.2^2}{8} = 13.57 \text{ kN.m.}$ $I_{xx \text{ REQD.}} = \frac{3.1 \times 7.1 \times 3.2^3}{(L/500)} = 721 \text{ cm}^4$ $\therefore L_e = 3.2 \text{ m.}$ $\therefore$ ADOPT <u>178x102x19 kg/m UB</u> B2 $M_b = 25 \text{ kN.m} > 13.57 \text{ kN.m.}$ $I_{xx \text{ PROV}} = 1357 \text{ cm}^4 > 721 \text{ cm}^4 \text{ OK.}$	



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Ref		Output
	<u>DESIGN OF BEAM (B3)</u> BEAM LINTEL OVER LHS BI-FOLD DOOR. PROPOSED OPENING = 4800mm. LOADS. $\text{ROOF} = 3.1/2 [(0.56 \times 1.4) + (1.2 \times 1.6)] = 4.94 \text{ kN/m}$ $\text{WALLS} = \begin{array}{l} \text{INNER} = 3.5 \times 1.95 \times 1.4 = 9.6 \text{ kN/m} \\ \text{OUTER} = 3.5 \times 2.17 \times 1.4 = 10.6 \text{ kN/m} \end{array}$ $\therefore M_{\max} = \frac{26 \times 4.8^2}{8} = 74.88 \text{ kN.m}$ $M_T = 10.6 \times 0.175 = 1.855 \text{ kN.m (ULT.)}$ 1.325 kN.m (SER) $I_{xx \text{ REQD}} = 3.1 \times 17.33 \times 4.8^3 = 5941 \text{ cm}^4$ $(L/500)$ $F_v = 62 \text{ kN}$ $\therefore L_e = 4.8 \text{ m}$ REFER TO SHT (16) FOR BEAM IN TORSION DESIGN. ADOPT. <u>305 x 165 x 54 kg/m UB</u> (B3)	



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Section				Sheet no./rev.	
Beam Lintel B3				1/0	
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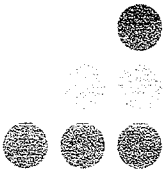
TEDDS calculation version 1.0.03

Member design checks for simple beam with torsion to BS 5950 & SCI-P-057

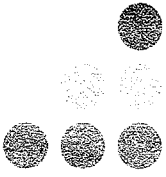
Summary

Span & restraint	L = 4800 mm	Torsion fixed & warping free each end.
Design load effects	$F_{vy} = 60.0 \text{ kN}$ $M_x = 74.9 \text{ kNm}$ $T_q = 1.86 \text{ kNm}$ torsion from uniformly distributed loads.	
Material	Grade = "S275"	$p_y = 275 \text{ N/mm}^2$
Section	"UB 305x165x54"	"Plastic"

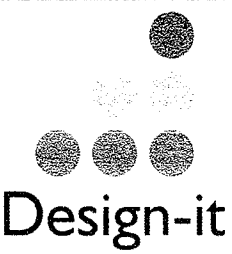
Check	Load	Capacity	Notes	Result
Twist angle	$\phi_{sls} = 0.97 \text{ deg}$	$\phi_{lim} = 2.00 \text{ deg}$	$T_{qu} = 1.33 \text{ kNm}$	Pass
Shear	$F_{vy} = 60.0 \text{ kN}$	$P_{vy} = 404.6 \text{ kN}$	Low shear	Pass
Moment	$M_x = 74.9 \text{ kNm}$	$M_{cx} = 232.7 \text{ kNm}$	Low shear	Pass
LTB	$M_{LT} = 74.9 \text{ kNm}$	M_b / m_{LT} $= 112.3 \text{ kNm}$	$L_{E_LT} = 5.4 \text{ m}$ $m_{LT} = 1.00$	Pass
Overall buckling	Index $i_b = 0.853$	Limit = 1.0	$\sigma_{byt} = 14 \text{ N/mm}^2$ $\sigma_w = 24 \text{ N/mm}^2$	Pass
Local capacity	$\sigma_{bx} + \sigma_{byt} + \sigma_w$ $= 138 \text{ N/mm}^2$	$p_y = 275 \text{ N/mm}^2$	$\sigma_{bx} = 99 \text{ N/mm}^2$ $\sigma_{byt} = 14 \text{ N/mm}^2$ $\sigma_w = 24 \text{ N/mm}^2$	Pass
Combined shear stresses	$\tau_w = 41 \text{ N/mm}^2$ $\tau_f = 31 \text{ N/mm}^2$	$p_v = 165 \text{ N/mm}^2$	Web governs	Pass

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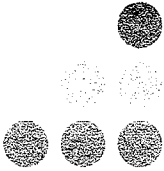
Ref		Output
	DESIGN OF PRT TO U/S B3. $M_T = 1.855 \text{ kNm}$. $\therefore$ PRT TAK, $t = \sqrt{\frac{6 \times 1.855 \times 10^6}{1000 \times 275}} = 6.4 \text{ mm}$. $\therefore$ PROVIDE (MIN) <u>8mm PRT</u> TO U/S B3 PLATE TO EXTEND LENGTH OF OPENING PLUS BEARING LENGTH EACH SIDE. ADOPT (MIN) 200mm END BEARING EACH SIDE. <u>DESIGN OF BEAM B4.</u> PROPOSED OPENING = 4m. LOADS. FLOOR = $3.96 \left[\frac{0.96 \times 1.4}{2} + (1.5 \times 1.6) \right] = 6.3 \text{ kN/m}$. WALLS = $3 \times 1.95 \times 1.4 = 8.2 \text{ kN/m}$. OUTER = $3 \times 2.17 \times 1.4 = 9.11 \text{ kN/m}$. <u>$\Sigma \text{UDL} = 24 \text{ kN/m}$</u>	

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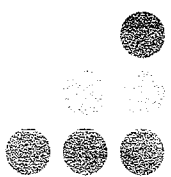
Ref	Output
	$M_{max} = 24 \times \frac{4^2}{8} = 48 \text{ kN.m.}$ $M_T = 9.11 \times 0.175 = 1.59 \text{ kN.m (UCT)}$ 1.14 kN.m (SLD) $I_{xx \text{ REQD}} = 3.1 \times 16 \times 4^3 = 3174 \text{ cm}^4$ $(L/500)$ $F_v = 48 \text{ kN.}$ REFER TO SHT FOR BEAM IN TORSION DESIGN. ADAPT. <u>254 x 146 x 43 kg/m UB (B4)</u> $I_{xx \text{ PROV}} = 6558 \text{ cm}^4 > 3174 \text{ cm}^4 \text{ OK.}$ BY INSPECTION. PROVIDE (MIN) <u>8mm RT TO U/S OF B4</u> AS PER NOTES & REQUIREMENTS FOR BS.

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	IT.	JUNE 19.	IT.	JUNE 19.	19

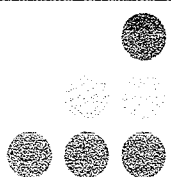
Ref	Output
	DESIGN OF BEAM (B5) FLOOR SUPPORT BEAM. ASSUME BEAM IN DEPTH OF FLOOR. <u>FLOOR JOISTS.</u> ADOPT MAX SPAN = 4m. REFER TO SHT FOR JOIST DESIGN. ADOPT (mine) <u>200x50 JOISTS</u> J1. <u>@ MAX 400 C/C. GR CL.</u> NOGGIN'S @ 1/3 POINTS. ABOVE 3.1m. NOGGIN'S @ 1/2 POINT TO MAX 3.1m. LOADS TO (B5) FLOOR = $3.1 \left[(0.5 \times 1.4) + (1.5 \times 1.6) \right] = 9.87 \text{ kN/m.}$ STUD WALL = $0.8 \times 24 \times 1.4 = 2.7 \text{ kN/m.}$ $\Sigma \text{un} = 12.6 \text{ kN/m.}$ SPAN = 6m. $\therefore M_{\text{max}} = 12.6 \times 6^2 / 8 = 56.7 \text{ kN.m.}$ $I_{\text{xx REQD}} = 1.8557 \times 8.4 \times 6^3 = 3367 \text{ cm}^4$ (L/300)

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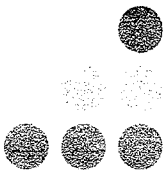
Ref		Output
	$L_e = 6m$. (Beam in depth of roof). $\therefore$ TRY. $203 \times 203 \times 46$ kg/m $M_b = 88.7 \text{ kN.m} > 56.7 \text{ kN.m}$ $I_{xx \text{ prov}} = 4564 \text{ cm}^4 > 3367 \text{ cm}^4 \therefore \text{OK}$ ADOPT <u>$203 \times 203 \times 46$ kg/m UB</u> (B5) $F_v = 38 \text{ kN}$. <u>PADSTONE</u> OUTER WALL. - ASSUME MIN 5N BLOCKWORK $\therefore$ PROVIDE (MIN). <u>$300 \text{ LG} \times 100 \text{ WIDE} \times 200 \text{ DP}$</u> CONCRETE PADSTONES.	

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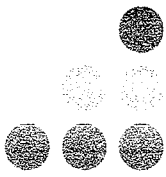
Ref		Output
	<u>DESIGN OF BEAM BL.</u> BEAM SUPPORTS FLOOR PLUS UPPER SOLID LOAD BEARING WALL. LOADS. Floor $= 2.85 \times 3.184 = 9.1 \text{ kN/m}$ WALL $= (1.5 + 1.475) \times 2.1 \times 1.4 = 8.75 \text{ kN/m}$ PURLIN DISTRIBUTED REACTION $= 5.85 \times 9.2 = 54 \text{ kN}$ $\therefore 54 / 5.2 = 10.4 \text{ kN/m}$ $\sum_{\text{wall}} = 28.3 \text{ kN/m}$ $\therefore M_{\text{max}} = 28.3 \times \frac{6^2}{8} = 127.4 \text{ kN.m}$ $I_{xx \text{ reqd}} = 1.8557 \times 18.9 \times 6^3 = 7576 \text{ cm}^4$ (L/300) $R_v = 85 \text{ kN}$ $L_E = 6.0 \text{ m}$ $\therefore$ ADOPT <u>203x203x86 kg/m UC</u> <u>FOR BEAM (BL)</u> $M_b = 200 \text{ kN.m} > 127.4 \text{ kN.m}$ $I_{xx \text{ prov}} = 9462 \text{ cm}^4 > 7576 \text{ cm}^4$	

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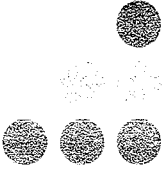
Ref	Output
	DESIGN OF BEAM (B7) BEAM SPANS OVER DINING AREA AND SUPPORTS UPPER WALL AND ROOF AND BEAM B5. LOADS. REACTION B5 = 38 kN. WALL = $1.4 \times 2.1 \times 1.4 = 4.7 \text{ kN/m}$. ROOF = $2 \left[(0.95 \times 1.4) + (0.6 \times 1.6) \right] = 4.58$. $\Sigma \text{UDL} = 9.3 \text{ kN/m}$ ~~SAY 10 kN/m.~~ BEAM SPAN = 3.67 m. $\therefore M_{\max} = 10 \times \frac{3.67^2}{8} + 38 \times \frac{3.67}{4} = 51.71 \text{ kN.m}$. $I_{xx} \text{ REQD.} = \frac{1.8557 \times 6.67 \times 3.67^3}{(L/300)} + \frac{2.9717 \times 25.3 \times 3.67^2}{4}$ $= 612 + 1013 = 1625 \text{ cm}^4$. $L_b = 1.84 \text{ m}$. TRY 203x133x30 kg/m UB. $M_b = 74.1 \text{ kN.m} > 51.71 \text{ kN.m}$. $I_{xx} \text{ PROV} = 2887 \text{ cm}^4 > 1625 \text{ cm}^4 \text{ OK}$. ADOPT <u>203x133x30 kg/m UB</u> (B7)

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Ref		Output
	DESIGN OF BEAMS (B8) + (B9) B9 (INNER) GREATER LOAD. LOADS. WALL = $2.12 \times 1.95 \times 1.4 = 5.8 \text{ kN/m}$ ROOF = $3.55 \left[(0.95 \times 1.4) + (0.6 \times 1.6) \right] = 4.1 \text{ kN/m}$ $\Sigma \text{WL} = 9.9 \text{ kN/m}$ ADOPT 12 kN/m SPAN = 3 m $M_{\text{max}} = 12 \times \frac{3^2}{8} = 13.5 \text{ kN.m}$ $I_{\text{xx REQD}} = 1.8557 \times 8 \times 3^3 = 400 \text{ cm}^4$ $L_e = 3 \text{ m}$ ADOPT <u>$178 \times 102 \times 19 \text{ kg/m UB}$</u> <u>FOR BEAMS B8 + B9</u> $M_D = 25 \text{ kN.m} > 13.5 \text{ kN.m}$ $I_{\text{xx PROV}} = 1357 \text{ cm}^4 > 400 \text{ cm}^4 \quad \text{OK}$	(B8) + (B9)

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	IT	JUNE 19	IT	JUNE 19	24

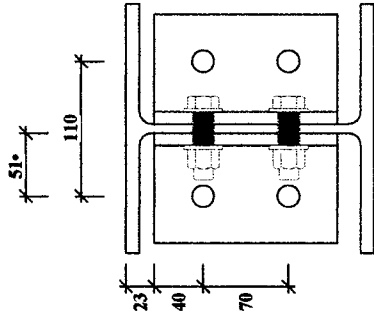
Ref	Output
	<u>DESIGN OF BEAM B10.</u> LOADS. FLOOR = $1.5 \left[(0.56 \times 1.4) + (1.5 \times 1.6) \right] = 4.78 \text{ kN/m.}$ REACTIONS FROM B8 + B9 = 36 kN. BY INSPECTION ADOPT <u>203 x 133 x 30 kg/m UB</u> (B10) <u>DESIGN OF BEAM B11.</u> BY INSPECTION. ADOPT SAME AS (B5) — <u>203 x 203 x 46 kg/m UC</u> (B11) <u>DESIGN OF BEAM B12.</u> BY INSPECTION. ADOPT SAME AS (B6). — ADOPT <u>203 x 203 x 86 kg/m UC</u> (B12)

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	IT	JUNE 19	IT	JUNE 19	25

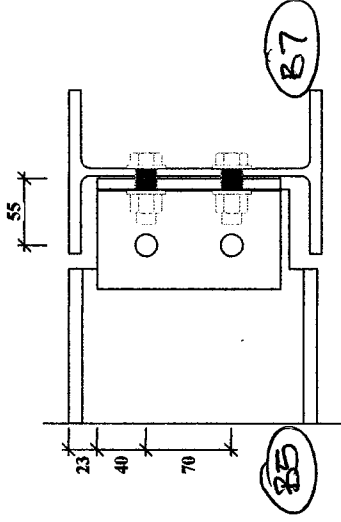
Ref		Output
	DESIGN OF <u>B/B</u> FLOOR = $\frac{(2.9 + 2.6)}{2} \times 3.184 = 8.76 \text{ kN/m}$ STUDWALL = $0.8 \times 2.8 \times 1.4 = 3.14 \text{ kN/m}$ $\Sigma \text{UDL} = 12 \text{ kN/m}$ $M_{\text{max}} = \frac{12 \times 2.4^2}{8} = 8.64 \text{ kN.m}$ $I_{\text{required}} = 1.8557 \times 8 \times 2.4^3 = 206 \text{ cm}^4$ $\therefore L_e = 2.4 \text{ m}$ ADOPT. (min) <u>178 x 102 x 19 kg/m UB</u> <u>B/B</u>	

Beam to Beam Angle Cleats

CONNECTION BB to B7



Left: 203x203 UC 46 (S275)
Top Notch: 23 x 56 mm Lg. net
Bot Notch: 23 x 56 mm Lg. net
2 No. 90 x 10 L x 150 mm (2x2 kg)
With 2 No. 18 mm holes in webs
and 2 No. 18 mm holes in top
and 2 No. 18 mm holes in bottom
For 6 No. 16 mm Ø Grade 8.8 Bolts.



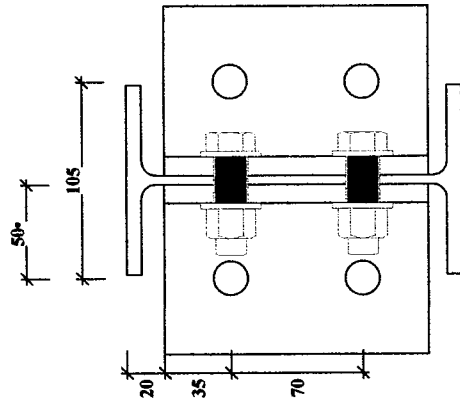
Beam: 203x133 UB 30 (S275)
Beam to web gap 10 mm
Web to cleat gap 2 mm
Cleats grade: Grade 43

Beam to Beam Angle Cleats

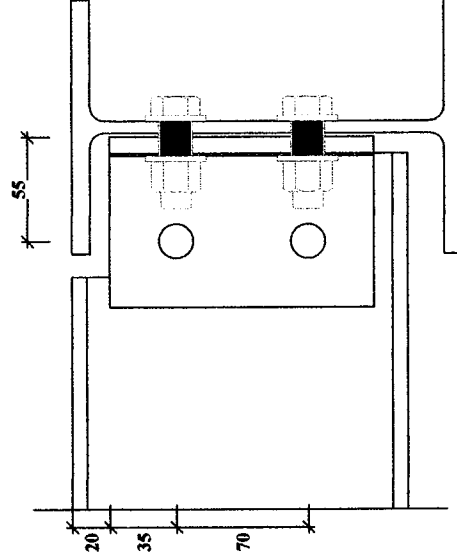
12302

Beam to Beam Angle Cleats

Connections BO & B7 to B70.

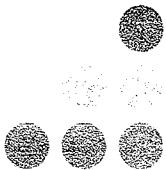


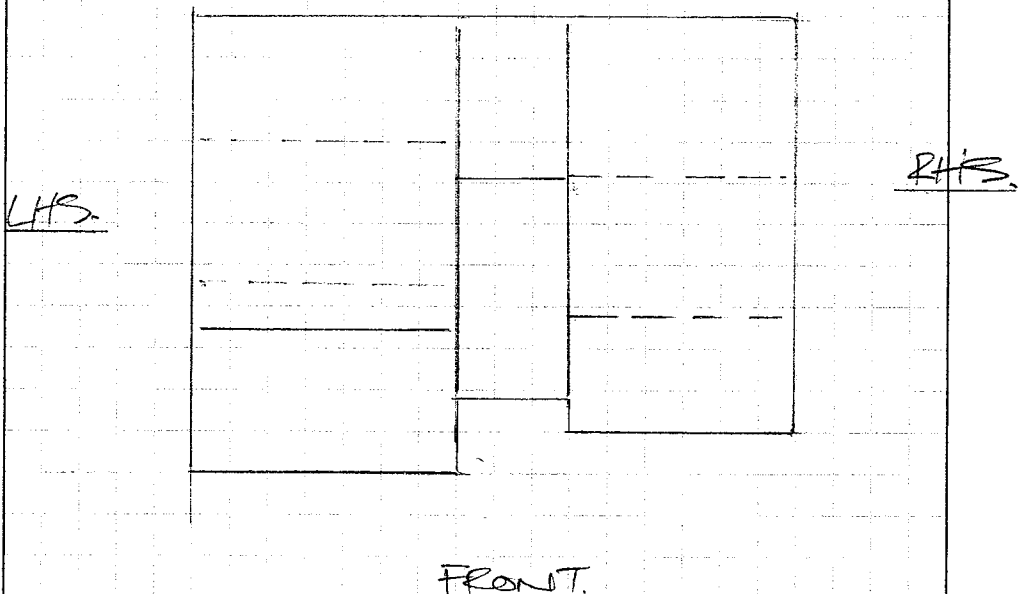
Left: 178x102 UB 19 (S275)
Top Notch: 20 x 56 mm Lx net
2 No. 90 x 90 x 10 L x 140 mm (2x2 kg)
With 2 No. 18 mm holes in webs
and 2 No. 18 mm holes in toes
For 6 No. 16 mm Ø Grade 8.8 Bolts.

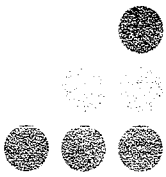


Beam: 203x133 UB 30 (S275)
Beam to web gap 10 mm
Web to cleat gap 2 mm
Cleats grade: Grade 43

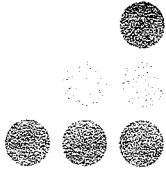
Beam to Beam Angle Cleats

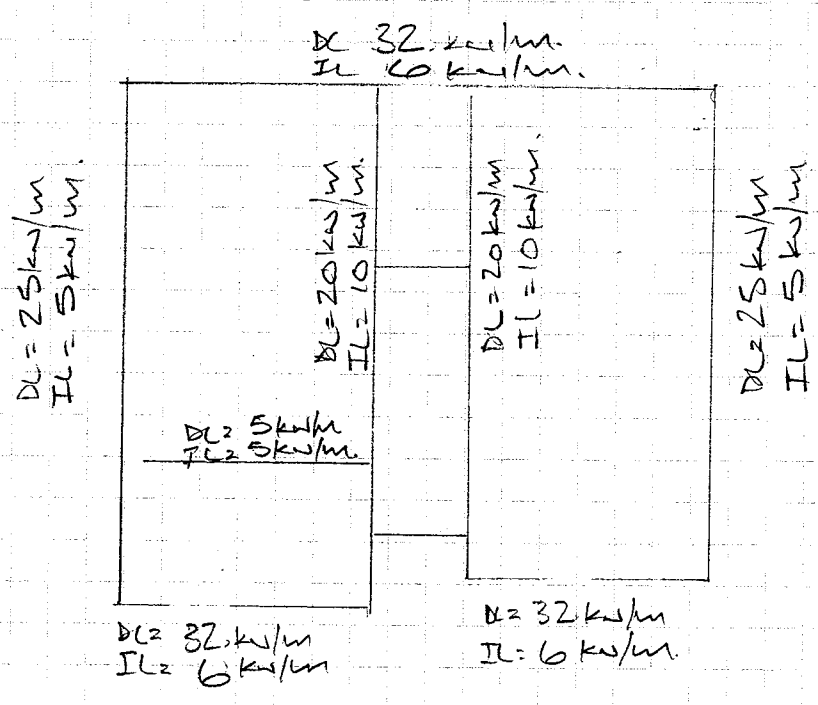
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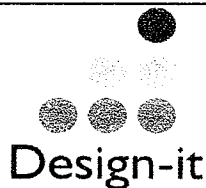
Ref		Output
	<u>PROPOSED PART SLAB.</u> <u>REAR.</u>  <u>FRONT.</u> <u>LOADS TO LHS</u> ROOF - DL = $0.95 \times 4.1 / 2 = 2 \text{ kN/m}$ IL = $0.6 \times 4.1 / 2 = 1.25 \text{ kN/m}$ WALLS - DL $(2.17 + 1.95) \times 4.57 = 18.8 \text{ kN/m}$ REACTION FROM FLOOR BEAMS. DL = 5 kN. IL = 12.4 kN < DIST LOADS = $5 / 4.8 = 1 \text{ kN/m}$ $12.4 / 4.8 = 2.6 \text{ kN/m}$ <u>Σ</u> DL = 22 kN/m <u>IL</u> = 4 kN/m	

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Ref		Output
	<u>FRONT & REAR</u> 1ST FLOOR - $DL = 3.9/2 \times 0.56 = 1.1 \text{ kN/m}$ $IL = 3.9/2 \times 1.5 = 2.93 \text{ kN/m}$ WALL - $DL = 5.95 \times (2.17 + 1.95) = 24.5 \text{ kN/m}$ RIDGE - $DL = \frac{(2 \times 5.56) \times 4}{2} / 4 = 5.6 \text{ kN/m}$ $IL = \frac{(2 \times 5.56/2) \times (2.25 \times 2)}{4} = 3.5 \text{ kN/m}$ $\sum DL = 32 \text{ kN/m}$ $IL = 6 \text{ kN/m}$ <u>INTERNAL WALLS</u> WALLS = $DL = 2.1 \times 6 = 12.6 \text{ kN/m}$ FLOOR BEAM DISTRIBUTED = $DL = 3 \text{ kN/m}$ $IL = 5 \text{ kN/m}$ ROOF = $DL = 4 \text{ kN/m}$ $IL = 3 \text{ kN/m}$ $\sum DL = 20 \text{ kN/m}$ $IL = 10 \text{ kN/m}$	

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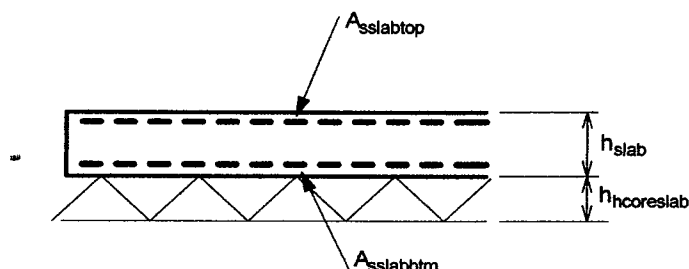
Ref		Output
	<u>SLAB LOADS.</u> SCREEN + INS = 4 kN/m^2 IMPOSED = 1.5 kN/m^2 <u>LINE LOADS</u>  DL = 32 kN/m IL = 6 kN/m DL = 25 kN/m IL = 5 kN/m DL = 20 kN/m IL = 10 kN/m DL = 20 kN/m IL = 10 kN/m DL = 32 kN/m IL = 6 kN/m DL = 25 kN/m IL = 5 kN/m DL = 32 kN/m IL = 6 kN/m DL = 20 kN/m IL = 10 kN/m DL = 20 kN/m IL = 10 kN/m DL = 25 kN/m IL = 5 kN/m DL = 25 kN/m IL = 5 kN/m	



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RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)

TEDDS calculation version 1.0.05



Raft and soil definition

Soil definition

Allowable bearing pressure $q_{allow} = 100.0 \text{ kN/m}^2$

No of types of soil **One type only**

Design depres'n dia under slab

Soil density

Firm

$\phi_{depslab} = 1350 \text{ mm}$

Raft slab definition

Max dim between joints $l_{max} = 15.000 \text{ m}$

Depth of h'core beneath slab $h_{hcoreslab} = 150 \text{ mm}$

Concrete strength $f_{cu} = 35 \text{ N/mm}^2$

Slab mesh reinf't strength $f_{yslab} = 500 \text{ N/mm}^2$

Min mesh for shrinkage (top) **A193**

Mesh adopted in bottom **A393**

Cover to top reinforcement $C_{top} = 25 \text{ mm}$

Slab thickness

$h_{slab} = 250 \text{ mm}$

(Dispersal allowed for bearing pressure check)

Poissons ratio of concrete $\nu = 0.2$

Partial safety factor for reinf't $\gamma_s = 1.15$

Actual mesh adopted in top **A393**

Cover to btm reinforcement

$C_{btm} = 35 \text{ mm}$

Slab edge reinforcement

Mesh provided in top **2 x A393**

Min width of edge reinf't **1062 mm**

Mesh provided in bottom

A393

Slab design checks

Basic loading

Slab self weight $W_{slab} = 6.0 \text{ kN/m}^2$

Hardcore

$W_{hcoreslab} = 3.0 \text{ kN/m}^2$

Applied loading

Dead udl $W_{Dudl} = 4.0 \text{ kN/m}^2$

Live udl

$W_{Ludl} = 1.5 \text{ kN/m}^2$

Slab load number 1

Dead load $W_{D1} = 20.0 \text{ kN/m}$

Load type

Line load

Live load

$W_{L1} = 10.0 \text{ kN/m}$

Slab bearing pressure check

Max area of steel req'd to distribute loads

$A_{slabbbreq} = 2 \text{ mm}^2/\text{m}$

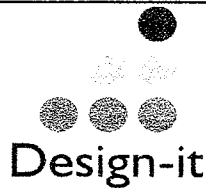
Pass - $A_{slabbbreq} \leq A_{slabbbtm}$ - Area of reinforcement provided to distribute the load is adequate

The allowable bearing pressure will not be exceeded

Slab bending check

Area of steel req'd in top $A_{slabtopreq} = 325 \text{ mm}^2/\text{m}$

Pass - $A_{slabtopreq} \leq A_{slabtop}$ - Area of reinforcement provided in top to span local depressions is adequate



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Area of steel reqd in btm

$A_{slab btm req} = 325 \text{ mm}^2/\text{m}$

Pass - $A_{slab btm req} \leq A_{slab btm}$ - Area of reinforcement provided in bottom to span local depressions is adequate

Applied shear stress

$v = 0.118 \text{ N/mm}^2$

Shear capacity

$v_c = 0.475 \text{ N/mm}^2$

Pass - $v \leq v_c$ - Shear capacity of slab is adequate

Internal slab deflection check

Allowable span to depth ratio $\text{Ratio}_{allow} = 52.000$

Actual span to depth ratio

$\text{Ratio}_{actual} = 7.634$

Pass - $\text{Ratio}_{actual} \leq \text{Ratio}_{allow}$ - Slab span to depth ratio is adequate

Slab edge design checks

Basic loading

Hardcore

$W_{coreslab} = 3.0 \text{ kN/m}^2$

Slab self weight

$W_{slab} = 6.0 \text{ kN/m}^2$

Edge load number 1

Load type

Longitudinal line load

Dead load

$W_{Dedge1} = 25.0 \text{ kN/m}$

Live load

$W_{Ledge1} = 5.0 \text{ kN/m}$

Longitudinal line load width

$b_{edge1} = 215 \text{ mm}$

Centroid of load

$X_{edge1} = 200 \text{ mm}$

Slab edge bearing pressure check

Applied bearing pressure

$q_{edge} = 66.8 \text{ kN/m}^2$

Pass - $q_{edge} \leq q_{allow}$ - Applied bearing pressure is less than allowable

Slab edge bending and shear check

Area of steel reqd in top

$A_{sedgetop req} = 325 \text{ mm}^2/\text{m}$

Pass - $A_{sedgetop req} \leq A_{sedgetop}$ - Area of reinforcement provided in top of slab is adequate

Area of steel reqd in bottom

$A_{sedgebtm req} = 325 \text{ mm}^2/\text{m}$

Pass - $A_{sedgebtm req} \leq A_{sedgebtm}$ - Area of reinforcement provided in bottom of slab is adequate

Applied shear stress

$V_{edge} = 0.267 \text{ N/mm}^2$

Design conc shear strength

$V_{cedge} = 0.599 \text{ N/mm}^2$

Pass - $V_{edge} \leq V_{cedge}$ - Shear capacity of the slab is not exceeded

Slab edge deflection check

Allowable span to depth ratio $\text{Ratio}_{allowedge} = 52.000$

Actual span to depth ratio

$\text{Ratio}_{actualedge} = 7.429$

Pass - $\text{Ratio}_{actualedge} \leq \text{Ratio}_{allowedge}$ - Slab span to depth ratio is adequate

Corner design checks

Applied loading

Corner load number 1

Load type

Line load in x direction

Dead load

$W_{Dcorner1} = 32.0 \text{ kN/m}$

Live load

$W_{Lcorner1} = 6.0 \text{ kN/m}$

Centroid of load

$y_{corner1} = 200 \text{ mm}$

Corner load number 2

Load type

Line load in y direction

Dead load

$W_{Dcorner2} = 25.0 \text{ kN/m}$

Live load

$W_{Lcorner2} = 5.0 \text{ kN/m}$

Centroid of load

$x_{corner2} = 200 \text{ mm}$

Corner bearing pressure check

Area of top steel reqd in slab to distribute load

$A_{scornerbp} = 326 \text{ mm}^2/\text{m}$

Pass - $A_{scornerbp} \leq A_{sedgetop}$ - Area of reinforcement provided to resist eccentric moment is adequate

The allowable bearing pressure at the corner will not be exceeded

Slab corner bending and shear check

Area of top steel reqd in slab $A_{scorner} = 699 \text{ mm}^2/\text{m}$

Pass - $A_{scorner} \leq A_{sedgetop}$ - Area of reinforcement provided in top of slab at corners is adequate

Applied shear stress

$V_{corner} = 0.465 \text{ N/mm}^2$

Design conc shear strength

$V_{ccorner} = 0.599 \text{ N/mm}^2$

Pass - $V_{corner} \leq V_{ccorner}$ - Shear capacity of the slab is not exceeded

Slab corner deflection check

Allowable span to depth ratio $\text{Ratio}_{allowcorner} = 8.480$

Actual span to depth ratio

$\text{Ratio}_{actualcorner} = 5.058$



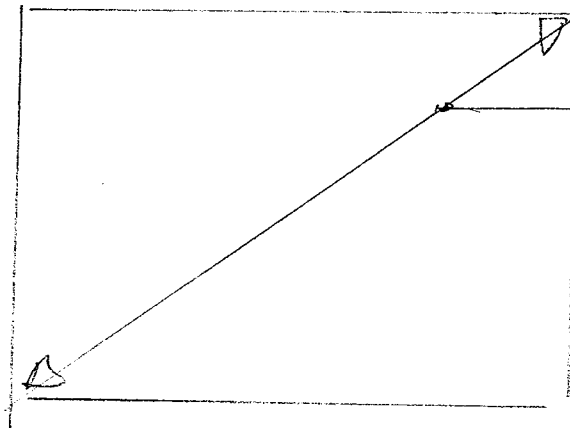
Project				Job Ref.	
35 Intake Lane, Pogmoor, Barnsley				18241	
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Pass - $\text{Ratio}_{\text{actual corner}} \leq \text{Ratio}_{\text{allow corner}}$ - Slab span to depth ratio is adequate

NOTE :-

PROVIDE AN ADDITIONAL 1200mm STRIP IN
BOTTOM LAYER OF B503 MESH TO PERIMETER
AND UNDER INTERNAL LOAD BEARING WALLS.

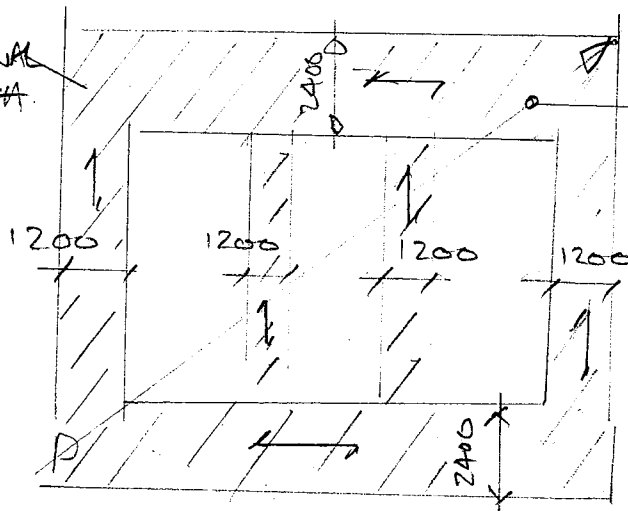
TOP LAYER



2 LAYERS OF
A393 MESH IN
TOP THROUGHOUT.

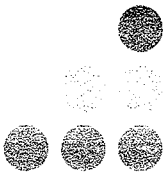
HATCHED AREA
REPRESENTS ADDITIONAL
STRIP OF B503 MESH.

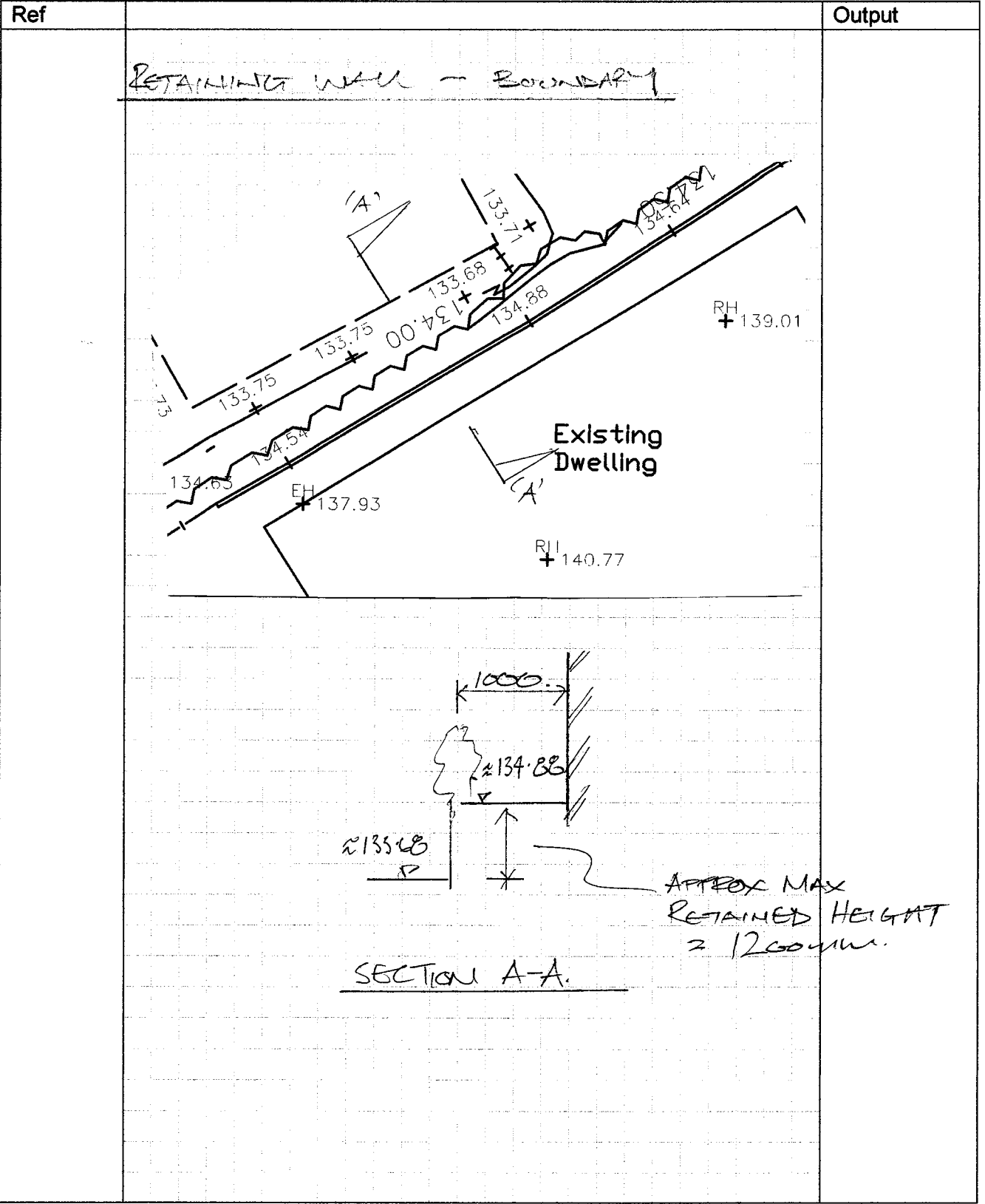
← SPAN
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1 LAYER A393
MESH THROUGHOUT.

BOTTOM LAYER

 Design-it	Project	35 INTAKE LANE, BARNLEY.			Job Ref
	Section	PROPOSED NEW BUILD.			18241.
	Calc By	Date	Checked	Date	Sht No.
	IT	JUNE 19,	IT	JUNE 19,	34



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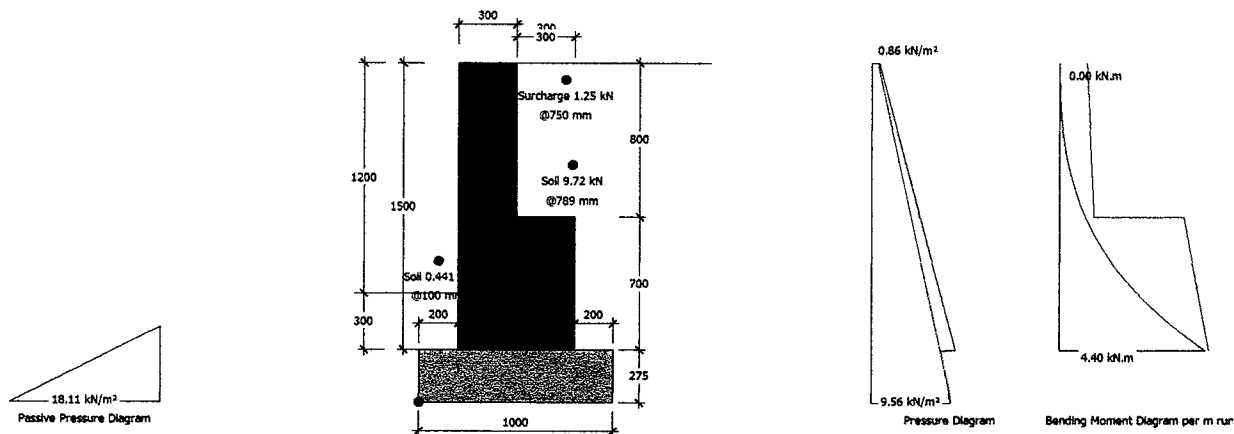
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Approved :

35

MasterKey : Retaining Wall Design to BS 8002, BS EN 1996-1-1:2005 and BS 8110 : 1997**External Wall Cavity Fill****Gravity Masonry Retaining Wall with Un-Reinforced Concrete Base****Summary of Design Data**

Notes

Material Densities (kN/m³)

Concrete grade

Surcharge and Water Table

All dimensions are in mm and all forces are per metre run

Soil 18.00, Concrete 24.00, Masonry 20.00

fcu 30 N/mm², Permissible tensile stress 0.250 N/mm²

Surcharge 2.50 kN/m², Fully drained

Soil Properties

Soil bearing pressure

Back Soil Friction and Cohesion

Base Friction and Cohesion

Front Soil Friction and Cohesion

Allowable pressure @ front 100.00 kN/m², @ back 100.00 kN/m²

 $\phi = \text{Atn}(\tan(34)/1.2) = 29.34^\circ$ $\delta = \text{Atn}(0.75 \times \tan(\text{Atn}(\tan(32)/1.2))) = 21.33^\circ$ $\phi = \text{Atn}(\tan(30)/1.2) = 25.69^\circ$ **Loading Cases** G_{Soil} - Soil Self Weight, G_{Wall} - Wall & Base Self Weight, F_{Heel} - Vertical Loads over Heel, P_a - Active Earth Pressure, $P_{\text{surcharge}}$ - Earth pressure from surcharge, P_p - Passive Earth Pressure

Case 1: Geotechnical Design

 $1.00 G_{\text{Soil}} + 1.00 G_{\text{Wall}} + 1.00 F_{\text{Heel}} + 1.00 P_a + 1.00 P_{\text{surcharge}} + 1.00 P_p$

Case 2: Structural Ultimate Design

 $1.40 G_{\text{Soil}} + 1.40 G_{\text{Wall}} + 1.60 F_{\text{Heel}} + 1.00 P_a + 1.00 P_{\text{surcharge}} + 1.00 P_p$ **Geotechnical Design****Wall Stability - Virtual Back Pressure**

Case 1 Overturning/Stabilising

5.799/17.341

0.334

OK

Wall Sliding - Virtual Back Pressure $F_x / (R_x \text{Friction} + R_x \text{Passive})$

9.134/(11.896+3.608)

0.589

OK

Soil Pressure

Virtual Back (No uplift)

Max(52.593/100, 8.329/100) kN/m²

0.526

OK

Wall Back (No uplift)

Max(56.938/100, 3.984/100) kN/m²

0.569

OK

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36

Structural Design**Masonry Wall Details**Partial Safety Factor (γ_{mc}/γ_{mf})

Material

Units and Mortar Strength

Blocks Ratio

Compressive Strength (f_k)

Units Category II. Execution Control Class 2

Concrete blocks, $\gamma=20$ kN/m³5.2 N/mm², Mortar designation M12/(i)

Unit height=215, Least horizontal dimensions=102.5

Group 2

3/2.7 Table NA.1

2.10

3.52 N/mm² Table NA.4**Wall Design (Inner Face Tension)**

Critical Section

Section Properties @ Base

Flexural Strength f_{xk1} $M_r = f_{xk1} \cdot Z_b / \gamma_{mf}$ Moment Capacity Check (M/M_r)

Shear Capacity Check

Critical @ 0 mm from base, Case 2

t=600 mm, Area=6000 cm², Z_b=60000 cm³ $f_{xk1}=0.15$, $g_d=0.022$ N/mm², $f_{xk1}=f_{xk1}+0.9 g_d \cdot \gamma_{mf}$

0.203x60000/2.7

M 4.4 kN.m, M_r 4.5 kN.mF 7.8 kN, $f_{vk}0.209$ N/mm², F_{vr} 50.1 kN0.203 N/mm² Table NA.6

4.52 kN.m

0.974 OK

0.16 OK

Base Top Face Tension

Maximum Tensile Stress

Shear Capacity Check

M 0.533, h 275, f_{cu} 30, Permissible 0.25 N/mm²F 5.0 kN, v_c 0.350 N/mm², F_{vr} 48.1 kN0.04 N/mm² OK

0.10 OK

Base Bottom Face Tension

Maximum Tensile Stress

Shear Capacity Check

M 0.975, h 275, f_{cu} 30, Permissible 0.25 N/mm²F 9.5 kN, v_c 0.350 N/mm², F_{vr} 48.1 kN0.08 N/mm² OK

0.20 OK

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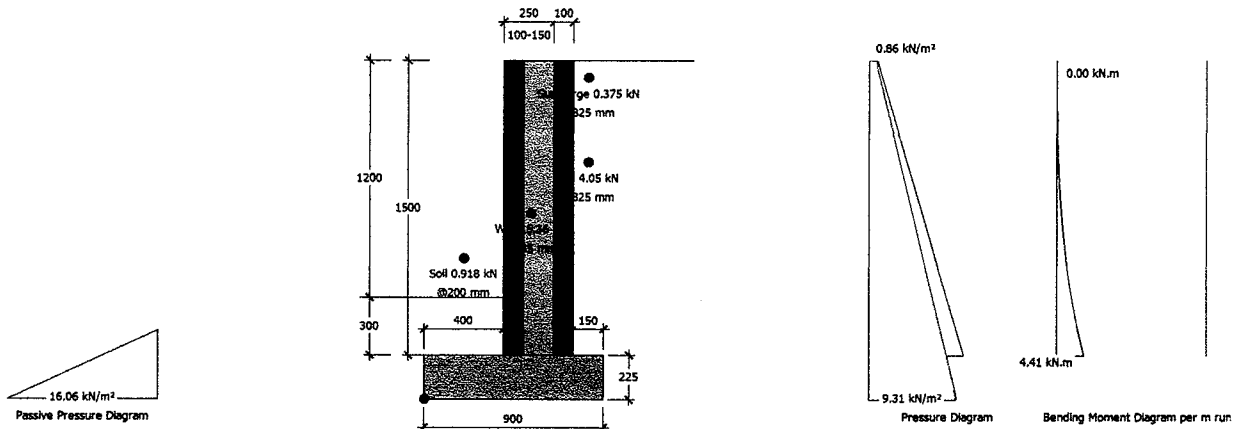
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37

MasterKey : Retaining Wall Design to BS 8002, BS EN 1996-1-1:2005 and BS 8110 : 1997**Cavity Wall 1200 ret****Reinforced Masonry Cavity Filled Retaining Wall with Reinforced Concrete Base****Summary of Design Data****Notes**

Material Densities (kN/m³)

Concrete grade

Concrete covers (mm)

Reinforcement design

Surcharge and Water Table

† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice

All dimensions are in mm and all forces are per metre run

Soil 18.00, Concrete 24.00, Masonry 20.00

fcu 30 N/mm², Permissible tensile stress 0.250 N/mm²

Cavity inner cover 30 mm, Cavity outer cover 110 mm, Base cover 50 mm

fy 460 N/mm², fyt 460 N/mm² designed to BS 8110: 1997

Surcharge 2.50 kN/m², Fully drained

Soil Properties

Soil bearing pressure

Back Soil Friction and Cohesion

Base Friction and Cohesion

Front Soil Friction and Cohesion

Allowable pressure @ front 110.00 kN/m², @ back 110.00 kN/m²

 $\alpha = \text{Atn}(\tan(34)/1.2) = 29.34^\circ$ $\delta = \text{Atn}(0.75 \times \tan(\text{Atn}(\tan(33)/1.2))) = 22.09^\circ$ $\phi = \text{Atn}(\tan(30)/1.2) = 25.69^\circ$ **Loading Cases** G_{Soil} - Soil Self Weight, G_{Wall} - Wall & Base Self Weight, F_{Heel} - Vertical Loads over Heel, P_a - Active Earth Pressure, $P_{\text{surcharge}}$ - Earth pressure from surcharge, P_p - Passive Earth Pressure

Case 1: Geotechnical Design

 $1.00 G_{\text{Soil}} + 1.00 G_{\text{Wall}} + 1.00 F_{\text{Heel}} + 1.00 P_a + 1.00 P_{\text{surcharge}} + 1.00 P_p$

Case 2: Structural Ultimate Design

 $1.40 G_{\text{Soil}} + 1.40 G_{\text{Wall}} + 1.60 F_{\text{Heel}} + 1.00 P_a + 1.00 P_{\text{surcharge}} + 1.00 P_p$ **Geotechnical Design****Wall Stability - Virtual Back Pressure**

Case 1 Overturning/Stabilising

5.362/12.058

0.445

OK

Wall Sliding - Virtual Back Pressure $F_x / (R_{x\text{Friction}} + R_{x\text{Passive}})$

8.704/(8.403+2.837)

0.774

OK

Soil Pressure

Virtual Back (No uplift)

Max(42.417/110, 3.590/110) kN/m²

0.386

OK

Wall Back

48.415/110 kN/m², Length under pressure 0.855 m

0.440

OK

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38

Structural Design**Outer Leaf Details**Partial Safety Factor (γ_{mc}/γ_{mf})

Units Category II. Execution Control Class 2

3/2.7 Table NA.1

Material

Concrete blocks, $\gamma=20$ kN/m³

Units and Mortar Strength

5.2 N/mm², Mortar designation M12/(i)

Blocks Ratio

Unit height=215, Least horizontal dimensions=102.5

2.10

Compressive Strength (f_k)

Group 2

3.52 N/mm² Table NA.6**Inner Leaf Details**Partial Safety Factor (γ_{mc}/γ_{mf})

Units Category II. Execution Control Class 2

3/2.7 Table NA.1

Material

Concrete blocks, $\gamma=20$ kN/m³

Units and Mortar Strength

5.2 N/mm², Mortar designation M12/(i)

Blocks Ratio

Unit height=215, Least horizontal dimensions=102.5

2.10

Compressive Strength (f_k)

Group 2

3.52 N/mm² Table NA.6**Wall Design (Inner Steel)**

Critical Section

Critical @ 0 mm from base, Case 2

Steel Provided (Cover)

Dist. B10@200 (30 mm in cavity)

Main B10@200 (30 mm in cavity)

393 mm²

OK

Leverarm $z=fn(d,b,As,fy,f_k)$ 215 mm, 1000 mm, 393 mm², 460 N/mm², 3.5 N/mm²

155 mm

 $Mr=fn(above,\gamma_{mf},\gamma_{ms})$

2.70, 1.15

24.1 kN.m

Moment Capacity Check (M/Mr)

M 4.4 kN.m, Mr 24.1 kN.m

0.183

OK

Shear Capacity Check

F 7.8 kN, f_{ck} 0.705 N/mm², Fvr 75.8 kN

0.10

OK

Base Top Steel Design

Steel Provided (Cover)

Main B10@200 (50 mm) Dist. B10@200 (60 mm)

393 mm²

OK

Compression Steel Provided (Cover)

Main B10@200 (50 mm) Dist. B10@200 (60 mm)

393 mm²Leverarm $z=fn(d,b,As,fy,F_{cu})$ 170 mm, 1000 mm, 393 mm², 460 N/mm², 30 N/mm²

162 mm

 $Mr=fn(above,As',d',x,x/d)$ 393 mm², 55 mm, 14 mm, 0.08

27.7 kN.m

Moment Capacity Check (M/Mr)

M 1.5 kN.m, Mr 27.7 kN.m

0.054

OK

Shear Capacity Check

F 8.9 kN, vc 0.510 N/mm², Fvr 86.8 kN

0.10

OK

Base Bottom Steel Design

Steel Provided (Cover)

Main B10@200 (50 mm) Dist. B10@200 (60 mm)

393 mm²

OK

Compression Steel Provided (Cover)

Main B10@200 (50 mm) Dist. B10@200 (60 mm)

393 mm²Leverarm $z=fn(d,b,As,fy,F_{cu})$ 170 mm, 1000 mm, 393 mm², 460 N/mm², 30 N/mm²

162 mm

 $Mr=fn(above,As',d',x,x/d)$ 393 mm², 55 mm, 14 mm, 0.08

27.7 kN.m

Moment Capacity Check (M/Mr)

M 4.4 kN.m, Mr 27.7 kN.m

0.157

OK

Shear Capacity Check

F 12.1 kN, vc 0.510 N/mm², Fvr 86.8 kN

0.14

OK