Structural Calculations

for

Internal Alterations to form Accommodation

at

Former Sunday School Building
Trevarthian Road
St Austell
Cornwall
PL25 4BH

DAS Structures Ltd

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I Eng. A.M.I.Struct.E

01209 861149
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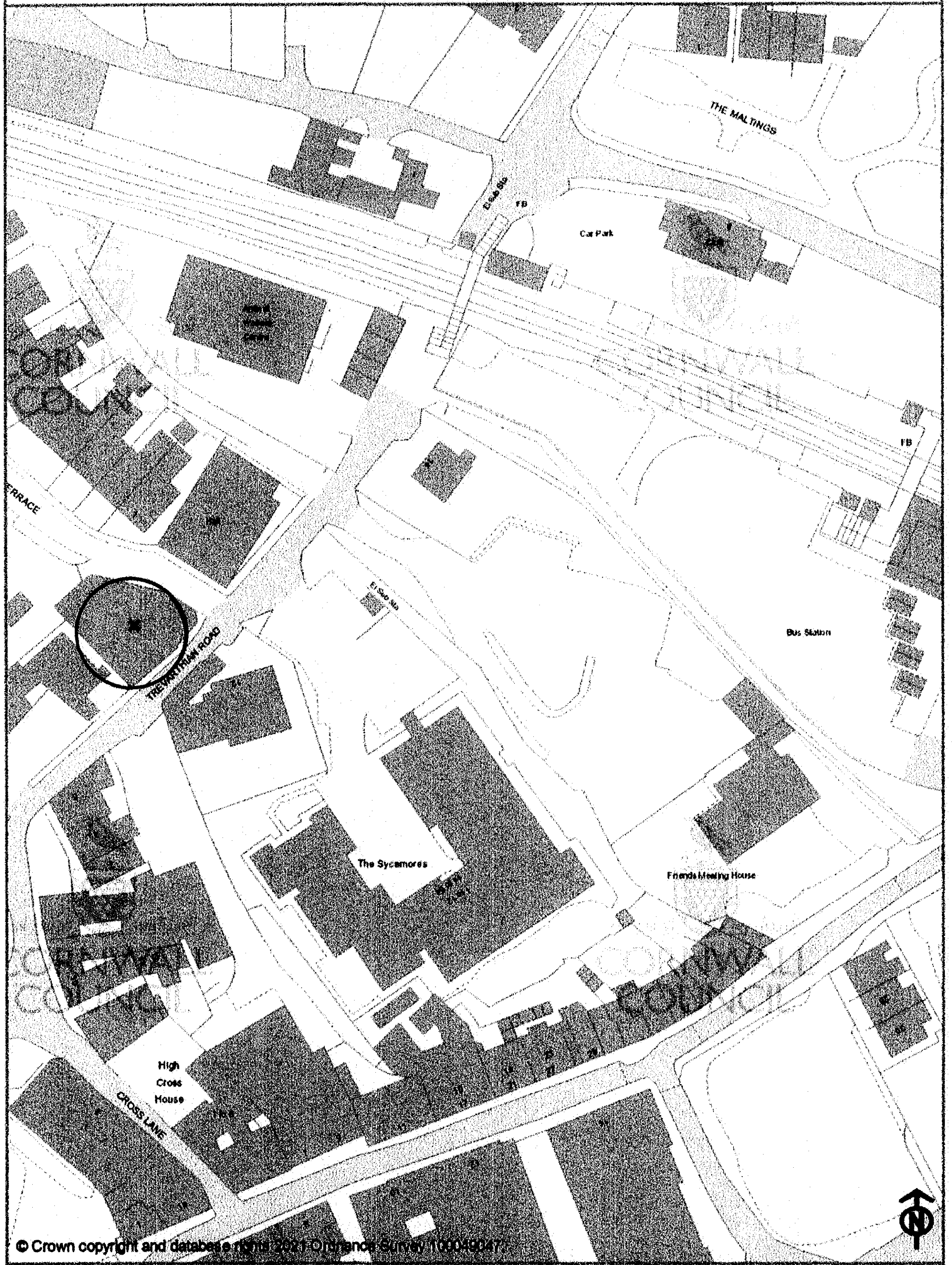
2997-2

May 2021

St Austell



Centre point X: 201522 Y: 52556 2,500 Sheet ref.: SX0152 10,000 Sheet ref.: SX05SW Scale 1:1000

Calculation Preamble

1. It is proposed to convert this former stone-walled two-storey Sunday school building into single storey accommodation above a car parking area.

2. The existing ground floor level is to be raised to allow access for cars, and an opening in the external wall enlarged to allow entry by cars.

3. Steel beams are required to support the proposed new timber first floor over the parking area, and a framework designed to minimise the span of the beam, whilst allowing minimum intrusion from the columns.

4. The two main floor support frames are positioned to avoid openings, leading to a maximum floor span of 4.9m.

5. The accommodation floor needs to be a 'separating floor' and Architects have advised on the construction detail required.

Timber is designed in accordance with BS 5268-2:2002
Masonry is designed in accordance with BS 5628-1:2005
Steelwork is designed in accordance with BS 5950-1:2000

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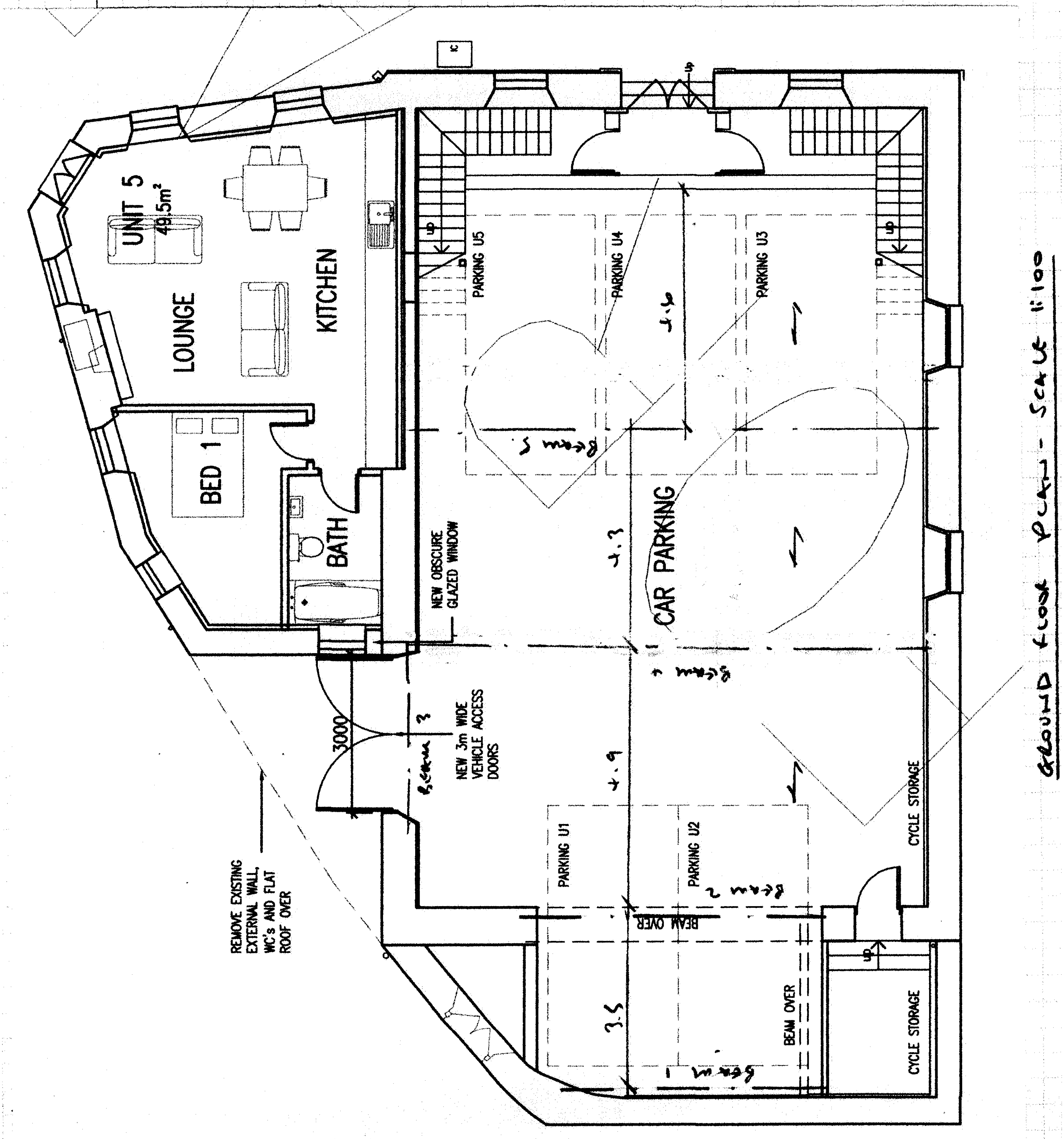
Sheet No:	(i)
Date:	May 21
Calc By:	DAT

Project: FORMER SUNDAY SCHOOL BUILDING
TREVARTHIAN ROAD, ST AUSTELL PL25 4BH
INTERNAL ALTERATIONS



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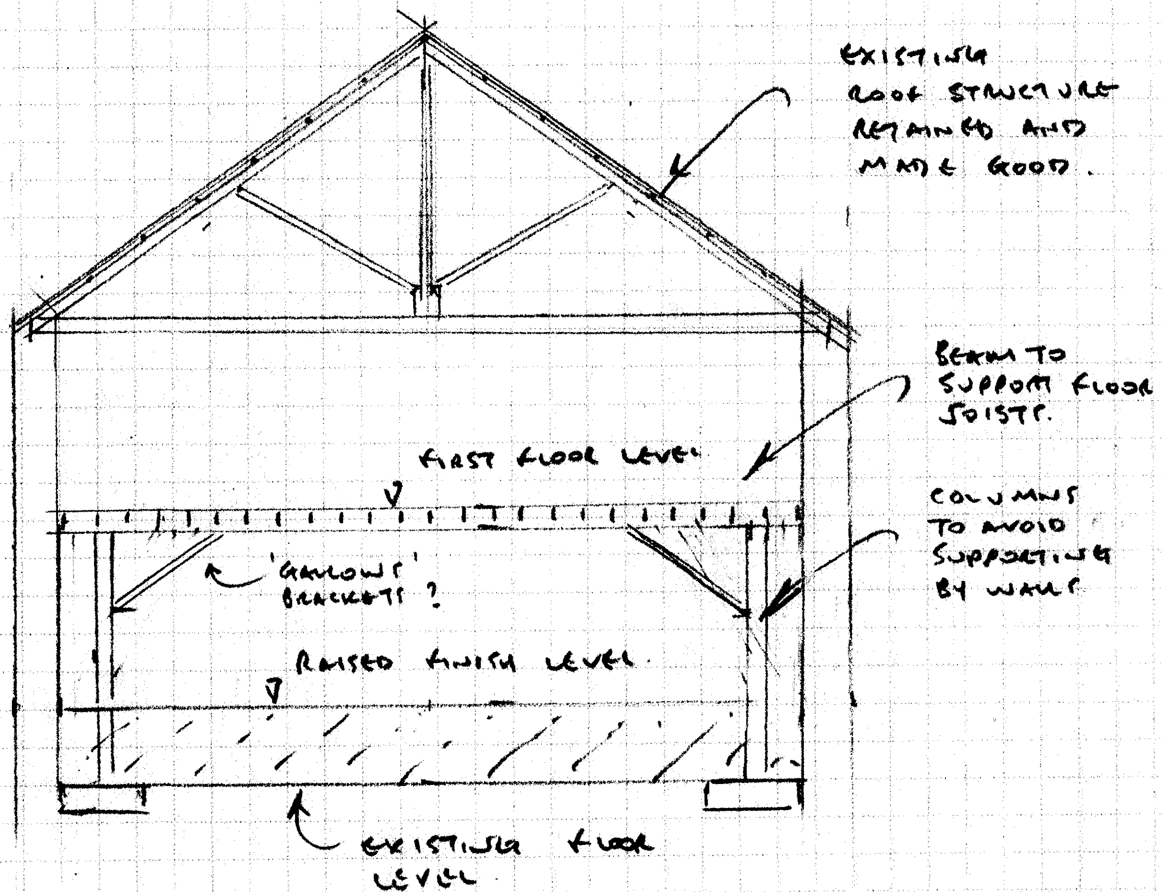
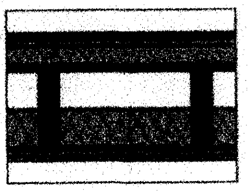
Job No:	2997-2
Sheet No:	(ii)
Date:	May 21
Calc By:	DM

Project: Former Sunday School
Trevarthian Road, St Austell PL25 4BY
Internal Alterations.



SECTION - SCALE 1:100



SEPARATING TIMBER PLATFORM FLOOR CONSTRUCTION (NEW BUILD)

- 18mm T&G flooring grade chipboard spot bonded to 15mm plasterboard (total mass 28kg/m²)
- 50mm (min) Rockwool Rockfloor resilient layer
- Min 15mm OSB floor deck on timber floor joists (200mm x 50mm @ 400 ctrs.)
- 100mm Rockwool Flexi® between joists
- Resilient bars fixed at right angles to joists at 400mm centres
- Ceiling finish 2 layers 15mm plasterboard 26kg/m²

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Project: FORMER SUNDAY SCHOOL BUILDING TREVARTHIAN ROAD, ST AUSTELL PL25 4BH INTERNAL ALTERATIONS.	Date: MAY 21
	Calc By: DAT

- LOADINGS
- ROOF LOADS - PITCH APPROX 35°

DEAD LOAD

NATURAL SLATE	=	50.0 kg/m²
BATTENS	=	2.5
RAFTERS 50x50 @ 300	=	3.5
PURLINS	=	4.2
PRINCIPAL RAFTER	=	4.2
		64.4 kg/m²

DL = 64.4 x 9.81/1000 x 1/cos 35 = 0.77 kN/m²

PRINCIPAL TIE	=	4.2 kg/m²
CEILING JOISTS	=	4.9
INSULATION	=	5.0
P/BOARD + SKIM	=	15.0
		29.1 kg/m²

DL = 29.1 x 9.81/1000 = 0.29 kN/m²

- FLOOR LOADS (SEPARATING FLOOR)

DEAD LOADS

18mm T&G CHIPBOARD	=	incl.
+ 15mm PLASTERBOARD	=	28.0 kg/m²
50mm ROCKWOOL	=	2.0
15mm OSB FLOOR DECK	=	10.5
100mm ROCKWOOL FLEXI	=	5.0
RESILIENT BARS @ 400	=	2.0
2 LAYERS 15 P/BOARD	=	26.0
SKIM FINISH	=	3.0
FLOOR JOISTS @ 400	=	16.8
		93.3 kg/m²

DL = 93.3 x 9.81/1000 = 0.92 kN/m²

IMPOSED LOADS

ALLOWANCE FOR DOMESTIC USE

UDL = 1.50 kN/m²

CONC = 1.4 kN

- PARTITIONS

2 x P/BOARD + SKIM	=	26.0 kg/m²
97 x 47 @ 400 STUDS	=	4.5
97 x 47 CILL/HEAD	=	1.7
ACOUSTIC QUILT	=	2.5
		34.7 kg/m²

DL = 34.7 x 9.81/1000 = 0.34 kN/m²

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	Calc By: KMT

• **FLOOR JOISTS – MAX SPAN = 4.9m.**

$WD = 0.92 \times 0.4 = 0.368\ kN/m$

$WI = 1.50 \times 0.4 = 0.6\ kN/m$ OR $1.4kN$.

$BM_1 = \frac{wl^2}{8} = \frac{0.968 \times 4.9^2}{8} = \underline{2.905\ kNm}$

$BM_2 = \frac{0.368 \times 4.9^2}{8} + \frac{1.4 \times 4.9}{2} = 2.82\ kNm$

$SF = \frac{0.968 \times 4.9}{2} = \underline{2.37\ kN}$

* SEE ALSO COMPUTER CALCS. TRY 222 × 72 @ 400 c/c

$\sigma m = \frac{M}{Z} = \frac{2.905 \times 10^6 \times 6}{72 \times 222^2} = 4.91\ N/mm^2 < 8.5\ \therefore ok$

DEFLECTION $= 10.5\ mm < 0.003 \times 4900 = 14.7mm > 14.0$ ∴ ok.

∴ 222 × 72 (C24) @ 400 c/c ADEQUATE

— USE ALSO FOR 4.6m SPAN. —

• **FLOOR JOISTS – SPAN 4.3m.**

$WD = 0.368\ kN/m$

$WL = 0.60\ kN/m$ OR $1.4kN$

$BM_1 = \frac{0.968 \times 4.3^2}{8} = 2.237\ kNm$

$BM_2 = \frac{0.368 \times 4.3^2}{8} + \frac{1.4 \times 4.3}{4} = \underline{2.35\ kNm}$

$SF = \frac{0.368 \times 4.3}{2} + 1.4 = \underline{2.19\ kN}$.

* SEE ALSO COMPUTER CALCS TRY 222 × 47 C24

$\sigma m = \frac{M}{Z} = \frac{2.35 \times 10^6 \times 6}{47 \times 222^2} = 6.09\ N/mm^2 < 8.5\ \therefore ok.$

DEFLECTION $= 9.7mm < 0.003 \times 4300 = 12.9mm$ ∴ ok

∴ 222 × 47 (C24) @ 400 c/c ADEQUATE FOR SPANS UP TO 4.3m.

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Trevarthian Road, Terrace, St Austell PL254BE

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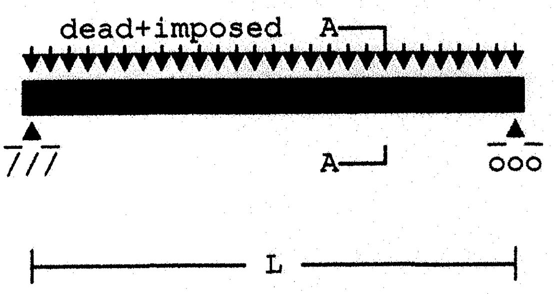
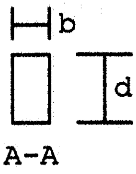
Location: Floor joists - span 4.9m max





Domestic floor joist

These calculations follow the domestic floor joists example by V C Johnson in TRADA Design Aid DA1 and BS5268-2:2002.

The following assumptions are made in these calculations:

- that the timber has a moisture content of service class 1 or 2 (i.e. K2=1)
- the floor can adequately distribute any concentrated point load to at least two joists.
- the centres of joists do not exceed 610 mm
- that load sharing of the joists can occur & K8 = 1.1.

Effective span of joist	L=4.9 m
Centres of joists	crs=400 mm
Dead load including self weight	dead=0.92 kN/m^2
Imposed udl load (on floor)	live=1.5 kN/m^2
Imposed point load (on one joist)	PL=1.4 kN
Depth of section	d=222 mm
Width of section	b=72 mm
Bearing length	lb=50 mm
Strength class C24 to Table 8.	
Duration of loading	K3=1
Depth factor	K7=(300/d)^0.11=1.0337
Load sharing (Clause 2.9)	K8=1.1

From BS5268-2 Table 18, bearing is < 75 mm from joist end.

Bearing Modification factor	K4=1.0

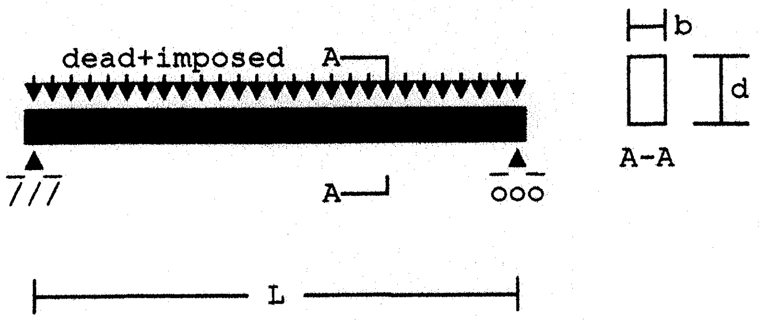
DESIGN SUMMARY

Joists: 222 mm x 72 mm @ 400 mm crs
Strength class C24 to Table 8.

Bending stress	4.9124 N/mm^2
Permissible bending	8.5278 N/mm^2
Deflection	10.572 mm
Limiting deflection	14 mm
Shear stress	0.22256 N/mm^2
Permissible shear	0.781 N/mm^2
Bearing stress	0.65878 N/mm^2
Permissible bearing	2.64 N/mm^2

No252

Location: Floor joists - span 4.3m max



Domestic floor joist

These calculations follow the domestic floor joists example by V C Johnson in TRADA Design Aid DA1 and BS5268-2:2002.

The following assumptions are made in these calculations:

- that the timber has a moisture content of service class 1 or 2 (i.e. K2=1)
- the floor can adequately distribute any concentrated point load to at least two joists.
- the centres of joists do not exceed 610 mm
- that load sharing of the joists can occur & K8 = 1.1.

Effective span of joist	L=4.3 m
Centres of joists	crs=400 mm
Dead load including self weight	dead=0.92 kN/m^2
Imposed udl load (on floor)	live=1.5 kN/m^2
Imposed point load (on one joist)	PL=1.4 kN
Depth of section	d=222 mm
Width of section	b=47 mm
Bearing length	lb=50 mm
Strength class C24 to Table 8.	
Duration of loading	K3=1
Depth factor	K7=(300/d)^0.11=1.0337
Load sharing (Clause 2.9)	K8=1.1

From BS5268-2 Table 18, bearing is < 75 mm from joist end.

Bearing Modification factor	K4=1.0

DESIGN SUMMARY

Joists: 222 mm x 47 mm @ 400 mm crs
Strength class C24 to Table 8.

Bending stress	6.1015 N/mm^2
Permissible bending	8.5278 N/mm^2
Deflection	9.692 mm
Limiting deflection	12.9 mm
Shear stress	0.31501 N/mm^2
Permissible shear	0.781 N/mm^2
Bearing stress	0.93243 N/mm^2
Permissible bearing	2.64 N/mm^2

No252

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	Calc By: DKT

• BEAM 1 - SPAN 4.5m

CONSIDERED TO SUPPORT HALF OF 3.5m FLOOR JOIST SPAN + STOREY HEIGHT OF LINING STUD WALL.

$W_D = 0.92 \times 3.5/2 = 1.61$ kN/m
$W_I = 1.50 \times 3.5/2 = 2.63$
$W_{ALL} = 0.34 \times 2.4 = 0.82$

DESIGN LOAD $= (1.61 + 0.82) \times 1.4 + (2.63 \times 1.6)$
$= 3.40 + 4.21 = 7.61$ kN/m

$M_x = \frac{wl^2}{8} = \frac{7.61 \times 4.5^2}{8} = \underline{19.26\text{ kNm}}$

$F_v = \frac{wl}{2} = \frac{7.61 \times 4.5}{2} = \underline{17.1\text{ kN}}$

* SEE ALSO COMPUTER CALCS   TRY 203x 133x 25 UB.

ASSUME NO LATERAL RESTRAINT / DEAD BEARING ONLY.

$M_b = 21.9$ kNm > 19.26 S.OK.

DEFLECTION DL = 2.7mm IL = 2.9mm
LIMIT = 4500 / 360 = 12.5mm S.OK.

∴ 203x 133 x 25 UB (S275) ADEQUATE

• BEAM 2 - SPAN 5.5m

SUPPORTS 3.5 + 4.9m SPANS OF FLOOR + PARTITION.

$W_D = 0.92 \times (3.5 + 4.9)/2 = 3.86$ kN/m
$W_I = 1.50 \times (3.5 + 4.9)/2 = 6.30$

PARTITION $= 0.34 \times 2.4 = 0.82$ kN/m

DESIGN LOAD $= (3.86 + 0.82) \times 1.4 + (6.3 \times 1.6)$
$= 6.55 + 10.08 = 16.63$ kN/m

$M_x = \frac{wl^2}{8} = \frac{16.63 \times 5.5^2}{8} = \underline{62.88\text{ kNm}}$

$F_v = \frac{wl}{2} = \frac{16.63 \times 5.5}{2} = \underline{45.7\text{ kN}}$

* SEE ALSO COMPUTER CALCS   TRY 305x 165x 54 UB.

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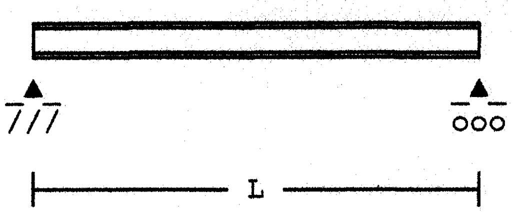
Location: Beam 1



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=4.5 m
203 x 133 x 25 UB.	
Young's Modulus	E=205 kN/mm²
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.5 m
Dead load (unfactored)	Gku(1)=1.61+0.82=2.43 kN/m
Imposed load (unfactored)	Qku(1)=2.63 kN/m
Maximum span bending moment	19.263 kNm
Design shear force	Fv=17.123 kN
Buckling parameter	u=(4*Sx^2*(1-Iy/Ix)/(A^2*((D-T)/10)^2))^0.25 =0.87693
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =84.554 N/mm²

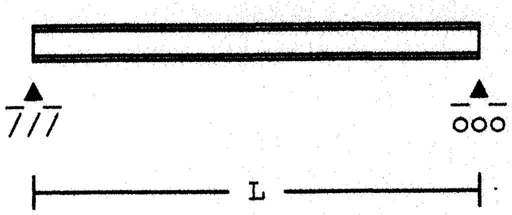
UNIVERSAL BEAM DESIGN SUMMARY

203 x 133 x 25 UB Grade S 275

	Maximum shear force	17.123 kN
	Shear capacity	191.11 kN
	Max. applied moment	19.263 kNm
	Moment capacity	70.95 kNm
	Buckling resistance	21.815 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	21.815 kNm
	Unfactored DL defln	2.7047 mm
	Unfactored LL defln	2.9274 mm
	Limiting deflection	12.5 mm
Unfactored end shears	DL shear at LHE	5.4675 kN
	LL shear at LHE	5.9175 kN
	DL shear at RHE	5.4675 kN
	LL shear at RHE	5.9175 kN

No408

Location: Beam 2



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=5.5 m
305 x 165 x 54 UB.	
Young's Modulus	E=205 kN/mm^2
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=5.5 m
Dead load (unfactored)	Gku(1)=4.68 kN/m
Imposed load (unfactored)	Qku(1)=6.3 kN/m
Maximum span bending moment	62.89 kNm
Design shear force	Fv=45.738 kN
Buckling parameter	u=(4*Sx^2*(1-Iy/Ix)/(A^2*((D-T)/10)^2))^0.25 =0.88907
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =88.745 N/mm^2

UNIVERSAL BEAM DESIGN SUMMARY

305 x 165 x 54 UB Grade S 275

	Maximum shear force	45.738 kN
	Shear capacity	404.61 kN
	Max. applied moment	62.89 kNm
	Moment capacity	232.65 kNm
	Buckling resistance	75.078 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	75.078 kNm
	Unfactored DL defln	2.3249 mm
	Unfactored LL defln	3.1296 mm
	Limiting deflection	15.278 mm
Unfactored end shears	DL shear at LHE	12.87 kN
	LL shear at LHE	17.325 kN
	DL shear at RHE	12.87 kN
	LL shear at RHE	17.325 kN

No408

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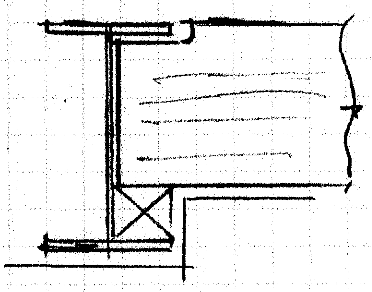
Project: FORMER SUNDAY SCHOOL BUILDING, TREVARTHIAN ROAD, ST AUSTELL PL25 4BH, INTERNAL ALTERATIONS	Date: MAY 21
	Calc By: DAK

ASSUME NO LATERAL RESTRAINT / DEAD BEARING ONLY

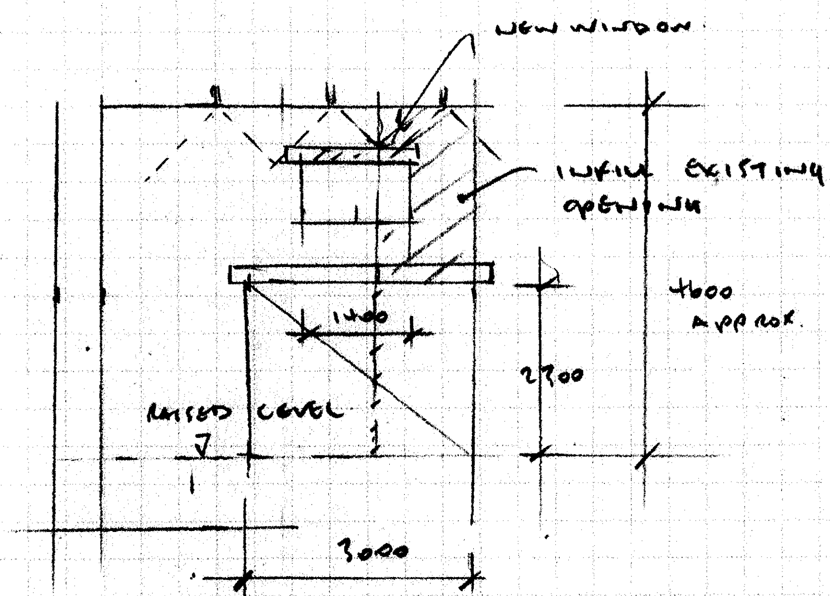
M_b = 75.0 kNm > 62.08 ∴ ok

DEFLECTION DL = 2.3mm SL = 3.1mm
LIMIT = 5500/360 = 15.2 ∴ ok

∴ <u>305 x 165 x 54 UB (S275) ADEQUATE</u>



• <u>GARAGE DOOR OPENING</u> – 3m CLEAR.



– <u>LINTEL OVER WINDOW</u> – SPAN 1.4m.

LOAD FROM TRUSSES @ 1.5m CENTRES

PD = 1.06 x 11/2 x 1.5 = 8.75 kN.
PI = 0.5 x 11/2 x 1.5 = 4.13 kN

0.6 STONE WALL = 26 x 0.6 x 0.6 = 9.4 kN/m

TEXTURED RANGE OF LINTELS LOAD/SPAN TABLE – Uniformly distributed service loads (kN/m)

The self weight of the lintel must be subtracted from the load given

Manufacture size to order	Section	P100	P150	P220	P255	S10	R15	R15A	R22	R22A	S15	R21	R21A
	Profile	65 x 100	65 x 150	65 x 220	65 x 255	100 x 100	100 x 140	140 x 100	100 x 215	215 x 100	150 x 140	140 x 215	215 x 140
	CLEAR SPAN	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
600	300	37.75	37.75	70.68	91.44	50.67	71.24	117.11	152.89	210.67	110.56	180.00	280.00
750	450	21.33	23.56	53.00	68.58	35.56	53.43	87.83	114.67	158.00	82.92	135.00	210.00
900	600	13.65	15.08	37.12	43.95	22.76	39.68	68.55	83.63	126.40	66.33	108.00	168.00
1050	750	9.48	10.47	25.78	30.52	15.80	27.56	47.60	58.07	105.33	55.28	79.80	131.65
1200	900	6.97	7.69	18.94	22.42	11.61	20.24	34.98	42.67	90.29	47.38	58.63	96.73
1350	1050	5.33	5.89	14.50	17.17	8.89	15.50	26.78	32.67	69.28	41.46	44.89	74.06
1500	1200	4.21	4.65	11.46	13.56	7.02	12.25	21.16	25.81	54.74	36.85	35.47	58.51
1650	1350	3.41	3.77	9.28	10.99	5.69	9.92	17.14	20.91	44.34	30.61	28.73	47.40
1800	1500	2.82	3.11	7.67	9.08	4.70	8.20	14.16	17.28	36.64	25.30	23.74	39.17
2100	1650	2.37	2.62	6.44	7.63	3.95	6.89	11.90	14.52	30.79	21.26	19.95	32.91
2100	1800	2.02	2.23	5.49	6.50	3.37	5.87	10.14	12.37	26.24	18.11	17.00	28.04
2400	1950	1.74	1.92	4.73	5.61	2.90	5.06	8.74	10.67	22.62	15.62	14.66	24.18
2400	2100	1.52	1.68	4.12	4.83	2.53	4.41	7.62	9.29	19.71	13.61	12.77	21.06
2700	2250	1.33	1.47	3.44	3.98	2.22	3.88	6.69	8.17	17.32	11.96	11.22	18.51
2700	2400	1.18	1.30	2.87	3.32	1.97	3.43	5.93	7.23	15.34	10.59	9.94	16.40
3000	2550	1.05	1.16	2.42	2.80	1.76	3.06	5.29	6.45	13.68	9.45	8.87	14.63
3000	2700	0.95	1.04	2.05	2.38	1.58	2.75	4.75	5.79	12.28	8.48	7.96	13.13
3300	2850	0.82	0.94	1.76	2.04	1.42	2.48	4.28	5.23	11.08	7.65	7.18	11.85
3300	3000	0.70	0.85	1.52	1.76	1.29	2.25	3.89	4.74	10.05	6.94	6.51	10.75
3600	3150	0.61	0.78	1.32	1.53	1.18	2.05	3.54	4.32	9.16	6.33	5.94	9.79
3600	3300	0.53	0.71	1.15	1.34	1.08	1.88	3.24	3.95	8.38	5.79	5.43	8.96
*4200	3450	-	-	-	-	-	1.72	2.98	3.63	7.70	5.31	4.99	8.23
*4200	3600	-	-	-	-	-	1.59	2.74	3.35	7.09	4.90	4.60	7.58
*4200	3750	-	-	-	-	-	1.47	2.54	3.09	6.56	4.53	4.25	7.01
*4200	3900	-	-	-	-	-	1.36	2.35	2.87	6.08	4.20	3.94	6.50

Note 1: No assistance from composite brickwork is assumed or included in these figures.

Note 2: Bearings: Minimum bearing length to be 150mm each. Where required provide suitable padstones. Ensure, however, that all bearings are sound.

Note 3: Where information on self weights of individual units is given this is intended for guidance purposes only. They are subject to change depending upon changes to the raw materials used so it is important to check the actual weights before use if this is a critical factor.

Please Note! Clear spans shown above are in increments of 150mm and the safe working Loads indicated are based on the clear span plus 150mm of bearing each end.

Our lintels are supplied in a standard range of lengths and can be cut to length as required.

The safe working load per metre is the least of the flexural strength, the shear strength or the load that gives a deflection limit of L/325.

Deflection information is available for our full range of lintels, please see Supreme DoP or contact our technical department.

Lengths indicated * are not available in all section sizes

The safe working loads indicated in the grey band on the shorter spans are governed by shear. The safe working loads indicated in the grey band on the longer spans are not great relative to the span, these lintels should be used with caution.

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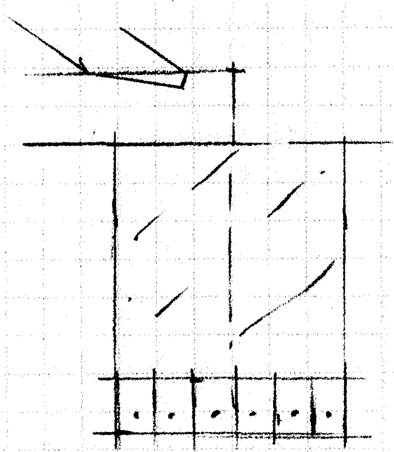
ASSUME LOAD FROM TRUSS IS DISTRIBUTED AS TRIANGLE

$\therefore$ EQUIV UDL $= \dfrac{(8.25 + 4.31) \times 8}{6 \times 1.4} = 12.0$ kN/m

ASSUME 3 / 100 x 140 DEEP LINTELS SUPPORT LOAD.



LOAD / LINTEL

$= \dfrac{12.0 + (9.4/2)}{3} = 5.57$ kN/m

SAFE LOAD FOR 100 x 140 DP

$= 14.16 - (24 \times 0.1 \times 0.14)$

$= 13.8$ kN/m > 5.57

$\therefore$ R15 & LINTELS ADEQUATE.

PROVIDE 150mm LONG BEARING

BEAM OVER GARAGE DOOR OPENING (BEAM 3).

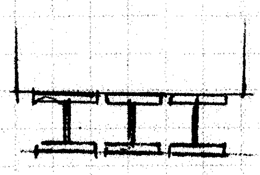
IGNORE WINDOW OPENING ABOVE – TAKE ALL LOADS ABOVE.
ASSUME LOAD FROM TRUSSES IS DISTRIBUTED.

ROOF DL $= 1.061 \times 11/2 = 5.83$ kN/m
IL $= 0.5 \times 11/2 = 2.75$ kN/m

WALL DL $= 26 \times 0.6 \times 2.3 = 35.88$ kN/m.

DESIGN LOAD $= (5.83 + 35.88) \times 1.4 + (2.75 \times 1.6)$
$= 62.8$ kN/m.

ASSUME A MINIMUM OF 3 / 152 x 152 UC'S OVER OPENING



$\therefore$ LOAD / BEAM $= 62.8/3$
$= 21$ kN/m.

$M_x = \dfrac{wl^2}{8} = \dfrac{21 \times 3^2}{8} = 23.6$ kNm.

* SEE ALSO COMPUTER CALCS.

TRY 152 x 152 x 30 UC.

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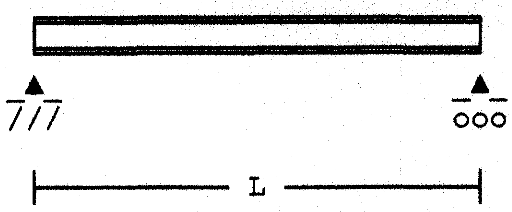
Location: Beam 3



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=3.0 m
152 x 152 x 30 UC.	
Young's Modulus	E=205 kN/mm²
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=3.0 m
Dead load (unfactored)	Gku(1)=13.90 kN/m
Imposed load (unfactored)	Qku(1)=0.92 kN/m
Maximum span bending moment	23.549 kNm
Design shear force	Fv=31.398 kN
Buckling parameter	u=(4*Sx^2*(1-Iy/Ix)/(A^2*((D-T)/10)^2))^0.25 =0.84888
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =183.1 N/mm²

UNIVERSAL COLUMN DESIGN SUMMARY

152 x 152 x 30 UC Grade S 275

	Maximum shear force	31.398 kN
	Shear capacity	169.03 kN
	Max. applied moment	23.549 kNm
	Moment capacity	68.2 kNm
	Buckling resistance	45.409 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	45.409 kNm
	Unfactored DL defln	4.0865 mm
	Unfactored LL defln	0.27047 mm
	Limiting deflection	8.3333 mm
Unfactored end shears	DL shear at LHE	20.85 kN
	LL shear at LHE	1.38 kN
	DL shear at RHE	20.85 kN
	LL shear at RHE	1.38 kN

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	Calc By: DAT

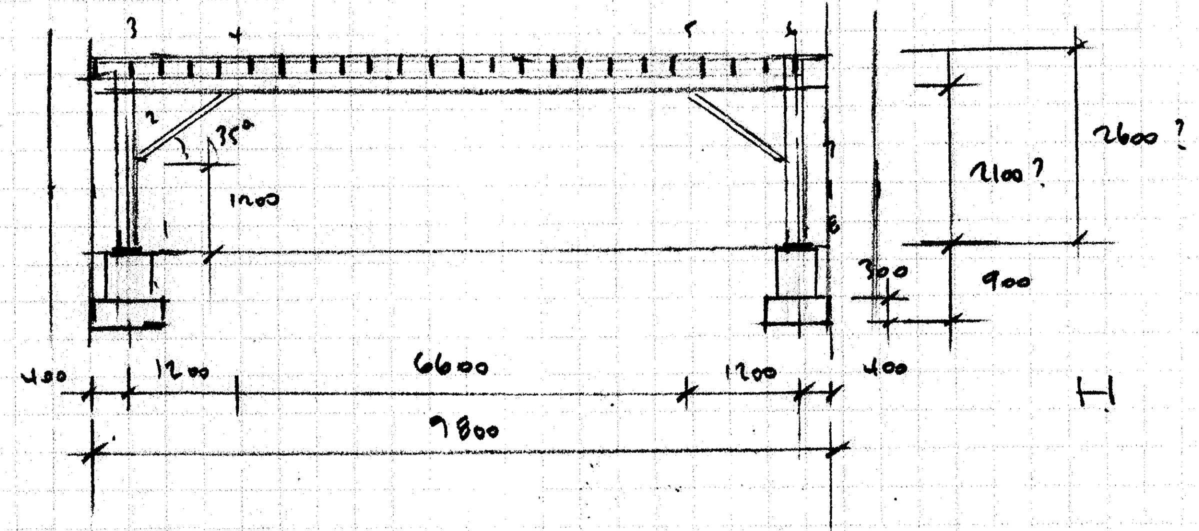
ASSUME NO LATERAL RESTRAINT / DEAD BEARING ONLY

Mb = 45.4 kNm > 27.6 ∴ OK

DEFLECTION DL = 4mm < 3000/360 = 8.3 ∴ OK

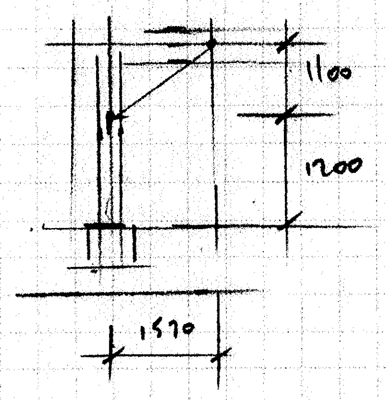
∴ 3 / 152 x 152 x 23 UC's (S275) ADEQUATE

- BEAM 4 - BEAM 5 SIMILAR BUT SLIGHTLY LESS LOADED.



FLOOR DL = 0.92 x (4.9 + 4.3) / 2 = 4.23 kN/m
LL = 1.50 x (4.9 + 4.3) / 2 = 6.90 kN/m

* SEE COMPUTER ANALYSIS



TRY 2 / 380 x 100 PFC BACK TO BACK

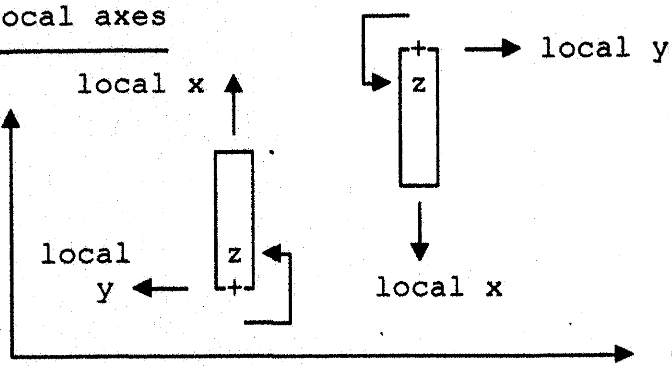
I_x = 15030 x 2 = 30060 cm^4

EQUIV I BEAM = 457 x 191 x 67 (29337 cm^4)

BEAMS
MAX MID SPAN DEFLECTION = 6.5mm SERV LOAD.
MAX ULT MOMENT = 76.5 kNm
MAX ULT SHEAR = 49.7 kN.

COLUMNS
MAX MID HT DEFLECTION = 2.6mm
MAX ULT MOMENT = 48 kNm
MAX ULT SHEAR = 43.7 kN
MAX AXIAL FORCE = 76.3 + (0.4 x 16.96)
= 83.1 kN ULT.

```
* Location: Floor beam with gallows brackets
*
* Data prepared with SCALE proforma 850
*
PRINT DATA, RESULTS FROM 1
TYPE PLANE FRAME
METHOD ELASTIC NODES
NUMBER OF JOINTS 8
NUMBER OF MEMBERS 9
NUMBER OF SUPPORTS 2
NUMBER OF LOADINGS 1
NUMBER OF SEGMENTS 5 TRACE
*
* Global and local axes
* _______________________
*
```

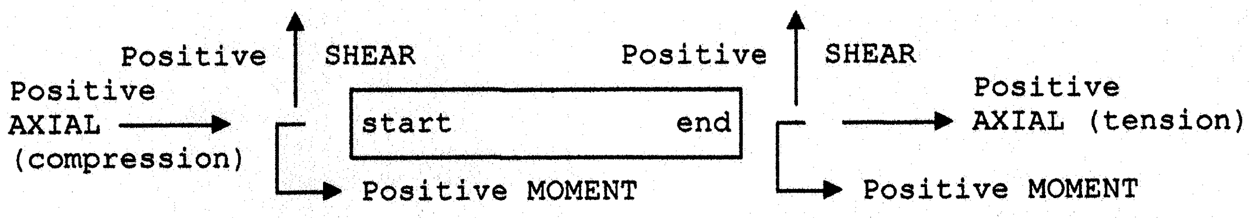



```
* Global Z out
* of paper
```

Local z points out of the paper, positive moment is anti-clockwise.

```
*
* Except in the special case of a continuous beam, the members of a
* structure do not all run along the global axis X; they may be inclin-
* ed to the the global axis and each may run in a different direction.
* In the diagram above, the left member is pointing upwards, the right
* downwards.  To fix the orientation of each member relative to axes X,
* Y and Z, every member is assumed to carry a local set of axes denoted
* x,y,z with axis x running from the START of the member (at the local
* origin) to the END of the member.  Movements along x,y - and
* anticlockwise about z - are considered positive, as depicted above.

*
*
*
*
*
*
```




```
*
* When dealing with the joints of a structure (their coordinates,
* conditions of support, loads etc.) we refer to global axes X, Y and
* Z.  When dealing with a member (its section properties, member loads
* etc.) we refer to its local axes x, y and z.  For quick reference
* the effects of positive results for forces acting on the ends of
* members are depicted above.  THINK OF THE JOINTS AS APPLYING FORCES
* TO THE MEMBER ENDS.
*
* All units are  and combinations thereof.
JOINT COORDINATES
1 0 0 SUPPORT
2 0  1.2
3 0  2.3
4 1.57  2.3
```

```
5 7.43  2.3
6 9  2.3
7 9  1.2
8 9 0 SUPPORT
JOINT RELEASES
1 MOMENT Z
8 MOMENT Z
MEMBER INCIDENCES
1 1  2
2 2  3
3 3  4
4 4  5
5 5  6
6 6  7
7 7  8
8 2  4
9 5  7
MEMBER RELEASES
2 END MOMENT Z
8 START MOMENT Z END MOMENT Z
9 START MOMENT Z END MOMENT Z
6 START MOMENT Z
CONSTANTS E 210E6 ALL G 79E6 ALL
MEMBER PROPERTIES
1 ISECTION DY 0.2032 DZ 0.2036 TZ 0.0072 TY 0.011 R 0.0102
2 ISECTION DY 0.2032 DZ 0.2036 TZ 0.0072 TY 0.011 R 0.0102
3 ISECTION DY 0.4534 DZ 0.1899 TZ 0.0085 TY 0.0127 R 0.0102
4 ISECTION DY 0.4534 DZ 0.1899 TZ 0.0085 TY 0.0127 R 0.0102
5 ISECTION DY 0.4534 DZ 0.1899 TZ 0.0085 TY 0.0127 R 0.0102
6 ISECTION DY 0.2032 DZ 0.2036 TZ 0.0072 TY 0.011 R 0.0102
7 ISECTION DY 0.2032 DZ 0.2036 TZ 0.0072 TY 0.011 R 0.0102
8 ISECTION DY 0.1576 DZ 0.1529 TZ 0.0065 TY 0.0094 R 0.0076
9 ISECTION DY 0.1576 DZ 0.1529 TZ 0.0065 TY 0.0094 R 0.0076
LOADING CASE 1
JOINT LOADS
MEMBER LOADS
3 THRU 5 FORCE Y PROJECTED UNIFORM W -16.96
SOLVE
```

LOADING CASE 1
JOINT DISPLACEMENTS

JOINT	X DISPLACEMENT	Y DISPLACEMENT	Z ROTATION
1	0.000000000	0.000000000	0.002831309
2	-0.002660021	-0.000074256	-0.000179064
3	0.000027188	-0.000089921	-0.002795353
4	0.000065442	-0.004389960	-0.002621779
5	-0.000065442	-0.004389960	0.002621779
6	-0.000027188	-0.000089921	0.002795353
7	0.002660021	-0.000074256	0.000179064
8	0.000000000	0.000000000	-0.002831309
9	-0.000763201	-0.000014851	0.002710894
10	-0.001468603	-0.000029702	0.002349649
11	-0.002058406	-0.000044553	0.001747575
12	-0.002474812	-0.000059404	0.000904670

JOINT	X DISPLACEMENT	Y DISPLACEMENT	Z ROTATION
13	-0.002413983	-0.000077389	-0.001172487
14	-0.001973676	-0.000080522	-0.001945149
15	-0.001387667	-0.000083655	-0.002497051
16	-0.000704523	-0.000086788	-0.002828192
17	0.000034839	-0.000984088	-0.002782736
18	0.000042490	-0.001864645	-0.002750561
19	0.000050141	-0.002726786	-0.002707336
20	0.000057792	-0.003568378	-0.002661572
21	0.000039265	-0.007370964	-0.002015588
22	0.000013088	-0.009120784	-0.000745616
23	-0.000013088	-0.009120784	0.000745616
24	-0.000039265	-0.007370964	0.002015588
25	-0.000057792	-0.003568378	0.002661572
26	-0.000050141	-0.002726786	0.002707336
27	-0.000042490	-0.001864645	0.002750561
28	-0.000034839	-0.000984088	0.002782736
29	0.000704523	-0.000086788	0.002828192
30	0.001387667	-0.000083655	0.002497051
31	0.001973676	-0.000080522	0.001945149
32	0.002413983	-0.000077389	0.001172487
33	0.002474812	-0.000059404	-0.000904670
34	0.002058406	-0.000044553	-0.001747575
35	0.001468603	-0.000029702	-0.002349649
36	0.000763201	-0.000014851	-0.002710894
37	-0.002114928	-0.000937396	-0.002659573
38	-0.001569836	-0.001800537	-0.002659573
39	-0.001024743	-0.002663678	-0.002659573
40	-0.000479650	-0.003526819	-0.002659573
41	0.000479650	-0.003526819	0.002659573
42	0.001024743	-0.002663678	0.002659573
43	0.001569836	-0.001800537	0.002659573
44	0.002114928	-0.000937396	0.002659573

LOADING CASE 1
MEMBER FORCES AT START OF FIRST SEGMENT AND ENDS OF ALL SEGMENTS

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	BENDING MOMENT
1	1	76.3200	-40.1066	0.0000
	9	-76.3200	40.1066	-9.6256
	10	-76.3200	40.1066	-19.2512
	11	-76.3200	40.1066	-28.8767
	12	-76.3200	40.1066	-38.5023
	2	-76.3200	40.1066	-48.1279
2	2	17.5651	43.7526	48.1279
	13	-17.5651	-43.7526	-38.5023
	14	-17.5651	-43.7526	-28.8767
	15	-17.5651	-43.7526	-19.2512
	16	-17.5651	-43.7526	-9.6256
	3	-17.5651	-43.7526	0.0000
3	3	-43.7526	17.5651	0.0000
	17	43.7526	-12.2397	4.6794
	18	43.7526	-6.9142	7.6865
	19	43.7526	-1.5888	9.0215
	20	43.7526	3.7366	8.6843
	4	43.7526	9.0621	6.6749
4	4	40.1066	49.6928	-6.6749
	21	-40.1066	-29.8157	53.2669

	22	-40.1066	-9.9386	76.5628
	23	-40.1066	9.9386	76.5628
	24	-40.1066	29.8157	53.2669
	5	-40.1066	49.6928	6.6749
5	5	-43.7526	9.0621	-6.6749
	25	43.7526	-3.7366	8.6843
	26	43.7526	1.5888	9.0215
	27	43.7526	6.9142	7.6865
	28	43.7526	12.2397	4.6794
	6	43.7526	17.5651	0.0000
6	6	17.5651	-43.7526	0.0000
	29	-17.5651	43.7526	-9.6256
	30	-17.5651	43.7526	-19.2512
	31	-17.5651	43.7526	-28.8767
	32	-17.5651	43.7526	-38.5023
	7	-17.5651	43.7526	-48.1279
7	7	76.3200	40.1066	48.1279
	33	-76.3200	-40.1066	-38.5023
	34	-76.3200	-40.1066	-28.8767
	35	-76.3200	-40.1066	-19.2512
	36	-76.3200	-40.1066	-9.6256
	8	-76.3200	-40.1066	0.0000
8	2	102.3939	0.0000	0.0000
	37	-102.3939	0.0000	0.0000
	38	-102.3939	0.0000	0.0000
	39	-102.3939	0.0000	0.0000
	40	-102.3939	0.0000	0.0000
	4	-102.3939	0.0000	0.0000
9	5	102.3939	0.0000	0.0000
	41	-102.3939	0.0000	0.0000
	42	-102.3939	0.0000	0.0000
	43	-102.3939	0.0000	0.0000
	44	-102.3939	0.0000	0.0000
	7	-102.3939	0.0000	0.0000

LOADING CASE 1
SUPPORT REACTIONS

JOINT	X FORCE	Y FORCE	Z MOMENT
1	40.1066	76.3200	0.0000
8	-40.1066	76.3200	0.0000

EQUILIBRIUM CHECK	SUM OF FORCES	REACTION
FORCES IN DIRECTION X	0.0000	0.0000
FORCES IN DIRECTION Y	-152.6400	152.6400
MOMENTS ABOUT AXIS Z	-686.8800	686.8800

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Internal Alterations

Page:
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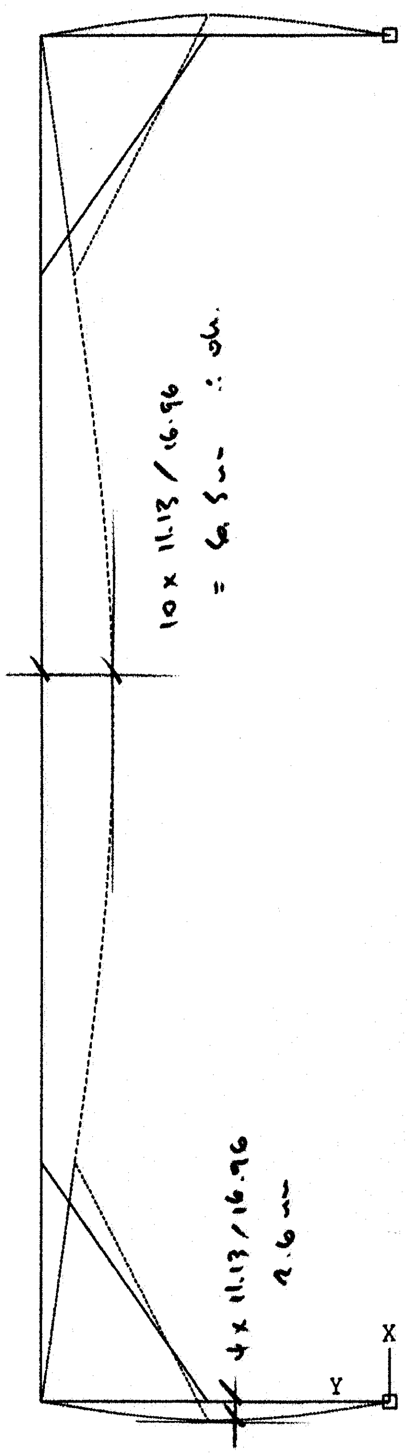
Structure scale 1 cm = 0.500   □ = supports

Deflectn scale 1 cm = 0.01000   1

DEFLECTION.
REDUCED FROM ULT LOADS
TO SERVICE LOADS.



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Page:
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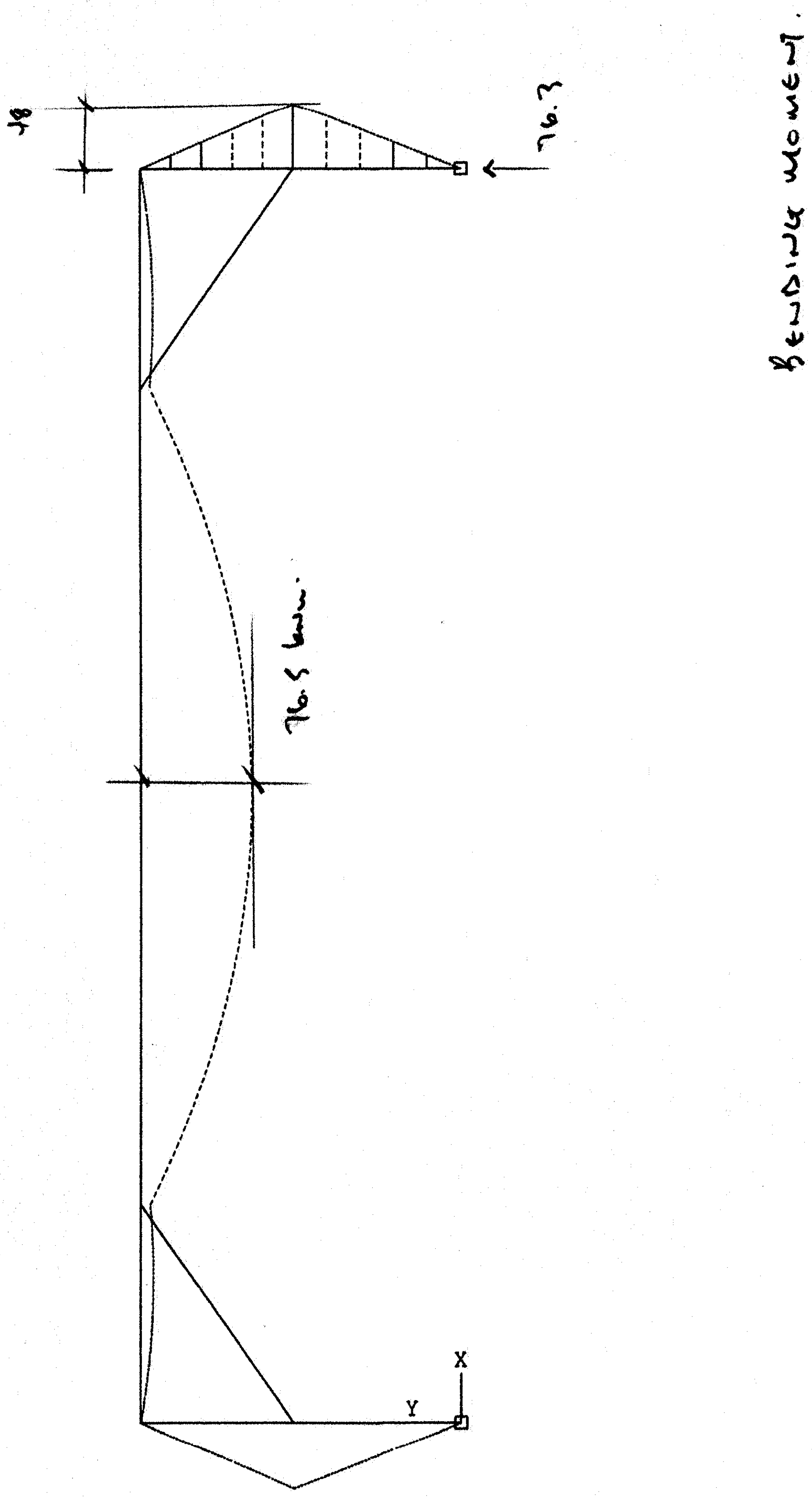
Internal Alterations

Structure scale 1 cm = 0.500   □ = supports

Moment Z scale 1 cm = 50.00   1



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Internal Alterations

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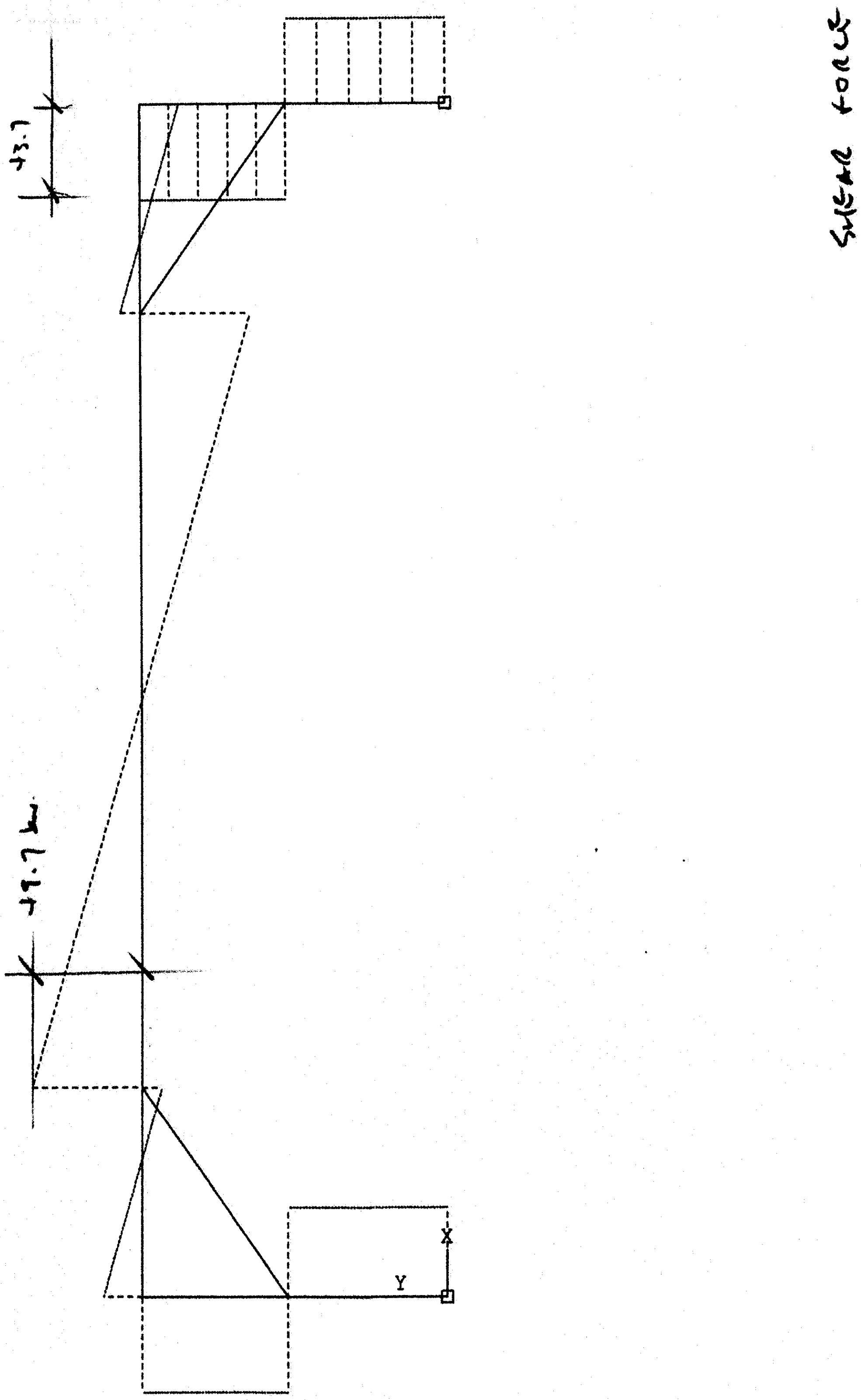
Structure scale 1 cm = 0.500 □ = supports

Force Y scale 1 cm = 30.00 1



DAS Structures Ltd	Job No: 2997-2
	Sheet No: 7
Project: FORMER SUNDAY SCHOOL BUILDING TREVARTHIAN ROAD, ST AUSTELL PL25 4BH INTERNAL ALTERATIONS	Date: MAY 21
	Calc By: DAS

- BEAM DESIGN * SEE ALSO COMPUTER CALCS

USING 2 / 380 x 100 PFC'S BACK TO BACK.

ASSUMING NO LATERAL RESTRAINT

Mb = 60.42 x 2 = 120.8 kNm > 76.5 ∴ ok.

Pv = 573.99 x 2 = 1147.98 kN > 49.7 ∴ ok

- COLUMN DESIGN * SEE ALSO COMPUTER CALCS

USING 203 x 203 x 46 UC

LENGTH BETWEEN RESTRAINTS = 2.2m.

Pc = 1369.2 kN > 83.1 kN ∴ ok.

Mbx = 136.68 kNm > 48.0 kNm ∴ ok

Mby = 275 x 152.26 / 1000 = 41.87 kNm > 10 Nom

COMBINED = 83.1 / 1369.2 + 48 / 136.68 + 10 / 41.87

= 0.06 + 0.35 + 0.24 = 0.65 < 1.0 ∴ ok

∴ 2 / 380 x 100 PFC'S BACK TO BACK SUPPORTED BY 203 x 203 x 46 UC'S ADEQUATE.

- FOUNDATIONS

SERVICE LOAD REACTION = (4.23 + 6.9) x 9.8 / 2
= 54.6

SW BEAMS	=	2 x 54 x 9.8 x 9.81 / 1000 x 2	= 5.2 kN
SW COLUMNS	=	46 x 2.2 x 9.81 / 1000	= 1.0 kN.
SW BASE	=	24 x 1 x 1 x 0.3	= 7.20
SW PLINTH	=	24 x 0.6 x 0.6 x 0.6	= 5.2
			73.3 kN.

ASSUME 100 kN/m² GROUND BEARING CAPACITY

AREA REQD = 73.3 / 100 = 0.733m² (1.0m x 0.8m)

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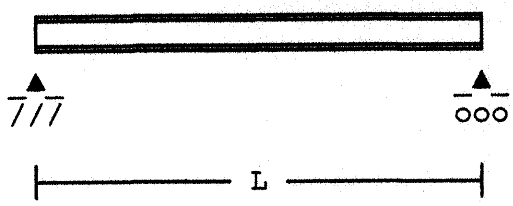
Location: Check on channels back to back



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

(Half load on single channel.)

Beam span L=5.86 m

Section properties

380 x 100 Parallel Flange Channel.
Dimensions (mm): D=380 B=100 t=9.5 T=17.5 r=15
Properties (cm): Ix=15000 Iy=643 Sx=933 Sy=161 J=45.7
A=68.7 Cy=2.7915 H=0.15 ry=3.0593 rx=14.776

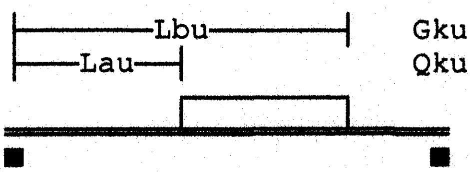
Strength of steel - Clause 3.1.1

The material thickness is 17.5 mm and the steel grade is S 275.
Design strength py=265 N/mm²
Young's Modulus E=205 kN/mm²

Loading

Dead load factor gamd=1.4
Imposed load factor gami=1.6

All loads are positive downwards, reactions are positive upwards, sagging moments are positive.



Gku Dead UDL
Qku Imposed UDL

Distances are measured from left hand support.

Uniformly distributed load 1 of 1
Dist. from left support to start Lau(1)=0 m
Distance from left support to end Lbu(1)=5.86 m
Dead load (unfactored) Gku(1)=4.23/2=2.115 kN/m
Imposed load (unfactored) Qku(1)=6.90/2=3.45 kN/m

BMs at 40th points, from left to right (sagging is positive)

0	3.5494	6.9168	10.102	13.106	15.927
	18.566	21.023	23.299	25.392	27.303
	29.032	30.58	31.945	33.128	34.129
	34.948	35.585	36.04	36.313	36.404
	36.313	36.04	35.585	34.948	34.129
	33.128	31.945	30.58	29.032	27.303
	25.392	23.299	21.023	18.566	15.927
	13.106	10.102	6.9168	3.5494	0

Maximum span bending moment 36.404 kNm

End shears

Shear force at left hand end	24.849 kN
Shear force at right hand end	24.849 kN
Design shear force	Fv=24.849 kN
Unfactored dead shear at LHE	6.197 kN
Unfactored imposed shear at LHE	10.109 kN
Unfactored dead shear at RHE	6.197 kN
Unfactored imposed shear at RHE	10.109 kN
Unfactored dead load deflection	1.0561 mm
Unfactored imposed load deflectn	1.7227 mm
Total DL & imposed deflection	2.7787 mm
Span:defln ratio for dead load	5548.9
Span:defln ratio for imposed load	3401.7
Span:defln ratio for total load	2108.9

From Table 8 of BS5950-1:2000,
Limiting deflection (brittle)   DELlim=L*1000/360=5.86*1000/360
=16.278 mm

Since imposed load deflection ≤ DELlim (1.7227 mm ≤ 16.278 mm)
deflection within limiting value.

Classification - Clause 3.5.2

Classify outstand element of compression flange:
Parameter (Table 11 Note b)   e=(275/py)^0.5=(275/265)^0.5
=1.0187
Outstand   b=B=100 mm
Ratio   b'T=b/T=100/17.5=5.7143
As b/T ≤ 9e (9.1682), outstand element of compression flange is classified as Class 1 plastic.
Classify web of section:
Depth between fillets   d=D-2*(T+r)=380-2*(17.5+15)
=315 mm
Ratio   d't=d/t=315/9.5=33.158
For sections with neutral axis at mid-depth
As d/t ≤ 80e (81.495), web is classified as Class 1 plastic.

Shear buckling

Since d/t ≤ 70e no check for shear buckling is required.

Buckling resistance

Since the beam is subject to possible lateral torsional buckling, the buckling resistance moment Mb is first considered rather than the moment capacity Mc as a guide to selection. A conservative analysis is adopted, taking the worst moment with the greatest effective length.

Effective length - Clause 4.3.5.2

As no point loads are applied to the beam the effective length, Le is found by considering the restraint conditions at the supports from Table 13 based on the length between the restraints equal to the beam span LT=L=5.86 m
Compression flange laterally unrestrained.
Both flanges free to rotate on plan.
Partial restraint against rotation provided only by pressure of bottom flange onto supports
Destabilizing loading conditions therefore

Effective length (Table 13)	Le=1.4*LT+2*D/1000 =1.4*5.86+2*380/1000 =8.964 m

Clause 4.3.6.6 and Table 18

For the destabilizing loading condition
Equivalent uniform moment factor mLT=1.0

Resistance to lateral-torsional buckling - Clause 4.3.6

Radius gyration about minor axis	ry=SQR(Iy/A)=SQR(643/68.7) =3.0593 cm
Slenderness of section	lambda=Le/ry*100=8.964/3.0593*100 =293
Buckling parameter	u=(Iy*Sx^2*(1-Iy/Ix)/(A^2*H*10^6))^0.25 =(643*933^2*(1-643/15000)/(68.7^2*0.15*10^6))^0.25 =0.93269
Torsional index	x=1.132*(A*H*10^6/(Iy*J))^0.5 =1.132*(68.7*0.15*10^6/(643*45.7))^0.5 =21.199
Ratio	ratio=lambda/x=293/21.199 =13.822
Slenderness factor	v=1/((1+0.05*ratio^2)^0.25) =1/((1+0.05*13.822^2)^0.25) =0.55483
Ratio βw	betaw=1.0
Equivalent slenderness	lamLT=u*v*lambda*(betaw)^0.5 =0.93269*0.55483*293*(1)^0.5 =151.63
Limiting slenderness	lamlo=0.4*((PI^2*E*10^3)/py)^0.5 =0.4*((3.1416^2*205*10^3)/265)^0.5 =34.951
Perry coefficient	etaLT=0.007*(lamLT-lamlo) =0.007*(151.63-34.951) =0.81672
Elastic strength	pe=PI^2*E*10^3/(lamLT^2) =3.1416^2*205*10^3/(151.63^2) =88.005 N/mm^2

Factor	phiLT=(py+(etaLT+1)*pe)/2 =(265+(0.81672+1)*88.005)/2 =212.44 N/mm²
Factor	pey=pe*py=88.005*265=23321
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =(23321)/(212.44+((212.44^2 -23321)^0.5)) =64.76 N/mm²
Buckling resistance moment	Mb=Sx*pb/10^3=933*64.76/10^3 =60.421 kNm

Equivalent uniform moment factor mLT=1
Since M ≤ Mb/mLT (36.404 kNm ≤ 60.421 kNm), section OK for lateral torsional buckling resistance.

Check section for combined moment and shear

Conservatively the maximum shear is checked with the maximum moment.

Shear area	Av=t*D=9.5*380=3610 mm²
Shear capacity	Pv=0.6*py*Av/10^3=0.6*265*3610/10^3 =573.99 kN
Design shear force	Fv=24.849 kN
Elastic modulus	Zx=Ix/(D/20)=15000/(380/20) =789.47 cm³

Since Fv < 0.6 Pv

Moment capacity for compact sec	Mc=py*Sx/10^3=265*933/10^3 =247.25 kNm
Limiting moment capacity	Mclim=1.2*py*Zx/10^3 =1.2*265*789.47/10^3 =251.05 kNm

Since M ≤ Mc (36.404 kNm ≤ 247.25 kNm), applied moment within moment capacity.
N.B. Buckling resistance is less than moment capacity and therefore buckling resistance controls the design.

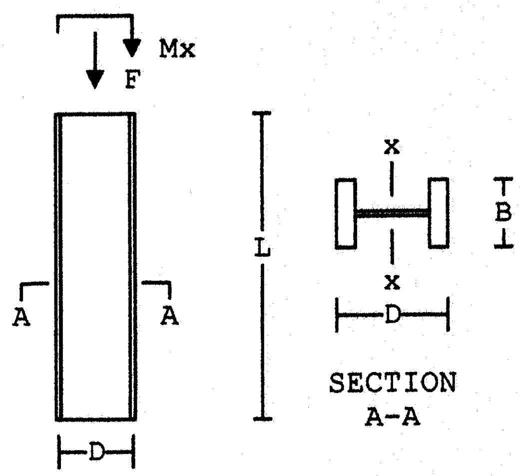
PARALLEL FLANGE CHANNEL DESIGN SUMMARY

380 x 100 Grade S 275

	Maximum shear force	24.849 kN
	Shear capacity	573.99 kN
	Max. applied moment	36.404 kNm
	Moment capacity	247.25 kNm
	Buckling resistance	60.421 kNm
	Moment factor (mLT)	1
	Resistance (Mb/mLT)	60.421 kNm
	Unfactored DL defln	1.0561 mm
	Unfactored LL defln	1.7227 mm
	Limiting deflection	16.278 mm
Unfactored end shears	DL shear at LHE	6.197 kN
	LL shear at LHE	10.109 kN
	DL shear at RHE	6.197 kN
	LL shear at RHE	10.109 kN

No408

Location: Columns



I column in 'simple' construction

Calculations are in accordance with BS5950-1:2000 and 'Steelwork Design Guide to BS5950' published by SCI.

All beams supported by the column are assumed to be fully loaded. Based on Clause 4.7.7 it is not necessary to consider the effect of pattern loading.

Factored axial compressive load	$F=83.1$ kN
Factored BM about major axis x-x	$Mx=48.0$ kNm
Factored BM about minor axis y-y	$My=10$ kNm
Length between restraints	$L=2200$ mm

Section properties

203 x 203 x 46 UC.
Dimensions (mm): D=203.2 B=203.6 t=7.2 T=11 r=10.2
Properties (cm): Ix=4570 Iy=1550 Sx=497 Sy=231 J=22.2
A=58.7 ry=5.1386 rx=8.8235 Zx=449.8 Zy=152.26

Strength of steel - Clause 3.1.1

The material thickness is 11 mm and the steel grade is S 275.

Design strength	$py=275$ N/mm²
Young's Modulus	$E=205$ kN/mm²

Classification - Clause 3.5.2

Classify outstand element of compression flange:

Parameter (Table 11 Note b)	e=(275/py)^0.5=1
Outstand	b=B/2=101.8 mm
Ratio	b'T=b/T=9.2545

As $9e < b/T \leq 10e$ (10), outstand element of compression flange is classified as Class 2 compact.

Classify web of section:

Depth between fillets	d=D-2*(T+r)=160.8 mm
Ratio	d't=d/t=22.333
Ratio	r1=F*10^3/(d*t*py)=0.26101
Limiting value for Class 1	d'tl1=80*e/(1+r1)=63.441

As $d/t \leq$ d'tl1, web is classified as Class 1 plastic

Slenderness - Clause 4.7.3

Effective length factor	ef=1
Effective length	Le=ef*L=2200 mm
Slenderness (minor axis)	lambda=Le/(ry*10)=42.813

Compressive strength: Perry strut formula from Annex C

Limiting slenderness	lamb0=0.2*(PI^2*E*1000/py)^0.5=17.155
Robertson constant for H section	a=5.5 for table 24(c)
Perry factor	eta=0.001*a*(lambda-lamb0)=0.14112
Euler strength	pe=PI^2*E*1000/lambda^2=1103.8 N/mm²
Factor	phi=(py+(eta+1)*pe)/2=767.3 N/mm²
Compressive strength	pc=pe*py/(phi+(phi^2-pe*py)^0.5) =233.26 N/mm²
Compression resistance	Pc=A*pc/10=1369.2 kN

Since F ≤ Pc (83.1 kN ≤ 1369.2 kN), design axial compressive load does not exceed compression resistance, section OK.

Resistance to lateral-torsional buckling - Clauses 4.3.6 & 4.7.7

Equivalent slenderness	lamLT=0.5*L/(ry*10)=21.407
Limiting slenderness	lamlo=0.4*((PI^2*E*10^3)/py)^0.5=34.31

The equivalent slenderness is less than the limiting slenderness.

Bending strength	pb=py=275 N/mm²
Buckling resistance moment	Mbs=Sx*pb/10^3=136.68 kNm

Overall buckling check - Clause 4.7.7

$$\frac{F}{Pc} + \frac{Mx}{Mbs} + \frac{My}{py.Zy} \leq 1$$

Unity factor	uf=F/Pc+Mx/Mbs+My*10^3/(py*Zy)=0.65072

The unity relationship is satisfied.

SECTION — 203 x 203 x 46 UKC Grade S 275

DESIGN SUMMARY

Design strength	275 N/mm²
Compressive strength	233.26 N/mm²
Buckling strength	275 N/mm²
Buckling check	0.65072 ≤ 1

Section is satisfactory for bending, axial load, and overall buckling.

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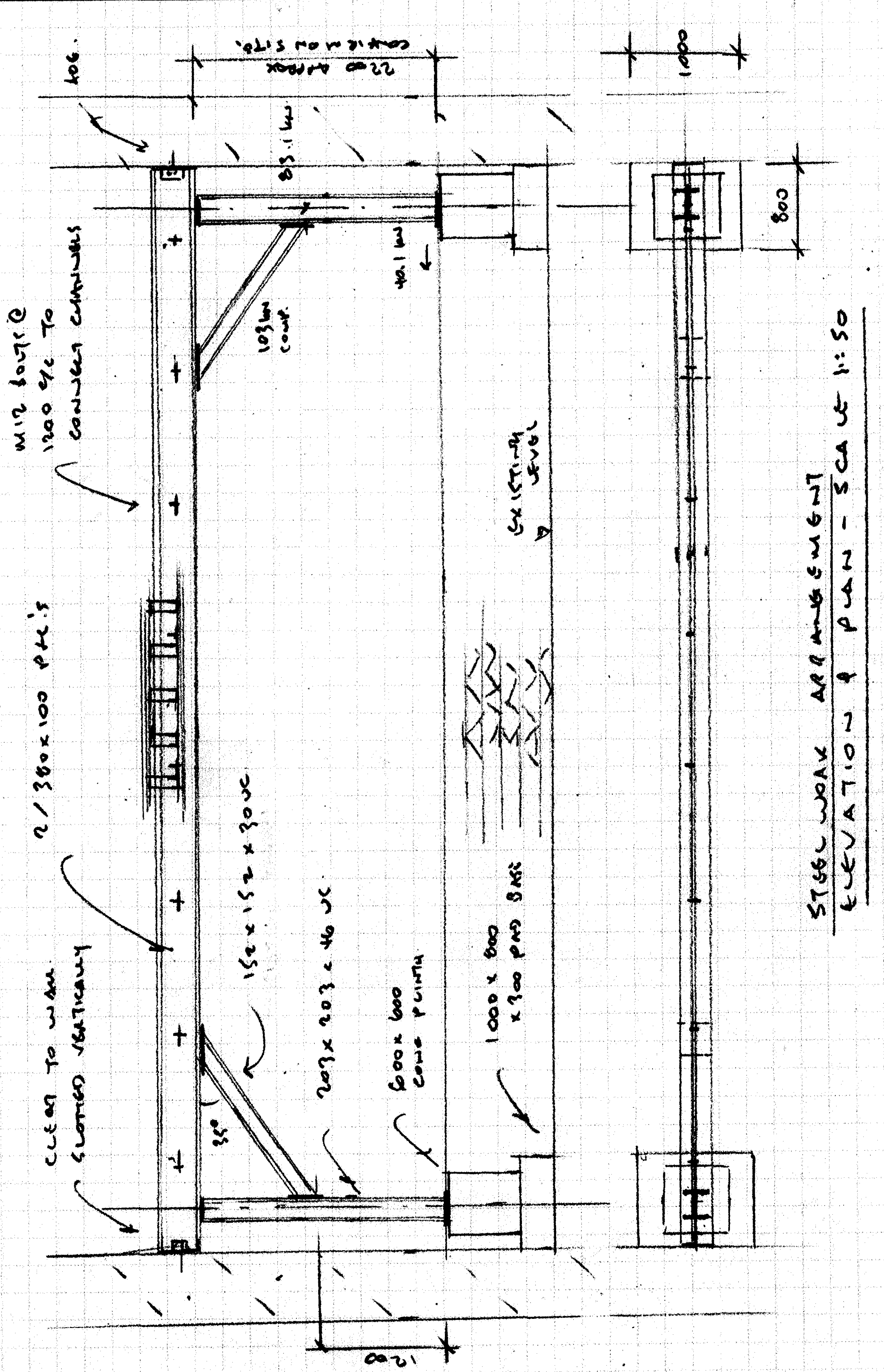

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Job No: 2997-2

Sheet No: 9

Project: FORMER SUNDAY SCHOOL BUILDING
TREVARTHIAN ROAD, ST AUSTELL PL25 4BH
INTERNAL ALTERATIONS

Date: MAY 21

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- STEELWORK DETAILS