Sand Engineering Consultants

Job No:	u141
Sheet No	L1
Prepared By	DCB
Date	Aug-19
Checked	

Project: CHEDDAR PARISH PAVILION
STRUCTURAL CALCULATIONS

LOADS

Entrance Core Area Roof

KS1000 RW Cladding	0.12	kN/m²	Min 0.06
50x50 Batten	0.04	"	
Rafters	0.15	"	
	0.31	kN/m² MAX	MIN 0.25

Roof Pitch : 32.5°

Loads on Plan DL = 0.37 kN/m²

IL = 0.55 kN/m²

Front Porch Roof

KS 1000 RW	0.12
Battens	0.04
Rafters	0.15
Insulation Between	0.04
" Below	0.02
Soffit Boarding	0.18
	0.55 kN/m²

On Plan

DL = 0.65 kN/m²

LOADING

TRUSSED RAFTER ROOF - DEAD LOADS

Sloping Part of Roof

Sheet Cladding	=	0.12	kN/m^2		
Felt & Battens	=	0.00	kN/m^2		
Rafters	=	0.07	kN/m^2		
SUB-TOTAL	=	0.19	kN/m^2		
Roof Pitch	=	32.50	Degrees	=	0.57 Radians
LOAD ON PLAN	=	0.225	kN/m^2		

Internal Truss Members = 0.07 kN/m^2

Bottom Chord Level

Bottom Chord	=	0.10	kN/m^2
Insulation	=	0.00	kN/m^2
Plasterboard	=	0.16	kN/m^2
TOTAL DL (ON PLAN)	**=**	**0.555**	**kN/m^2**

TRUSSED RAFTER ROOF - IMPOSED LOADS

Sloping Part of Roof

Basic Imposed load	IL =	0.600	kN/m^2		
Roof reduction factor		(60-a) / 30		=	0.917
Adjusted Imposed Load		0.550	kN/m^2		

Attic IL = 0.250 kN/m^2

TOTAL IL (ON PLAN) = 0.800 kN/m^2

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Project: CHEDDAR PARISH PAVILION
STRUCTURAL CALCULATIONS

ROOF STRUCTURE

Entrance Hall Ridge Beam

UDL's

Roof DL = $\frac{7.5m}{2}$ x 0.37 kN/m² = 1.39 kN/m — Ref L1

IL = " x 0.55 " = 2.06 "

Beam S/W DL = say = 0.3 "

Span = 4.4m

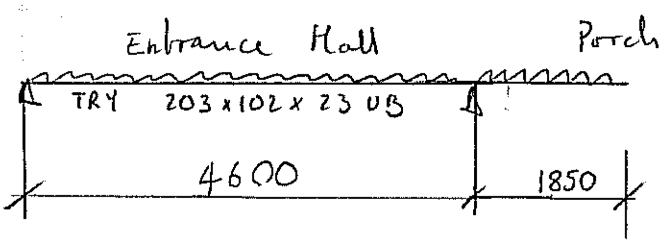
USE 203 x 102 x 23 UB — Ref R2

Front Porch Cantilever Ridge

UDL's Roof DL = $\frac{7.5m}{2}$ x 0.65 kN/m² = 2.44 kN/m

IL = " x 0.55 " = 2.06 kN/m

} ULS 6.71 kN/m



Deflection Limit = $\frac{1850}{180}$ = 10.3mm

Imposed load on tip → δ = 3.1mm OK.

$M = \frac{1.85^2}{2} \times 6.71 = 11.49$ kNm

For Cantilever $L_E = 2.0L = 3.7m$.

USE 203 x 102 x 23 UB — Ref R3-5

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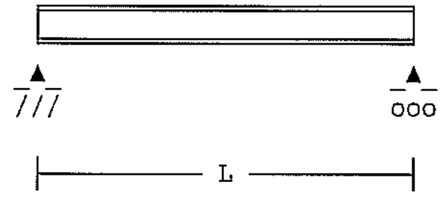
Location: ENTRANCE HALL RIDGE BEAM



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

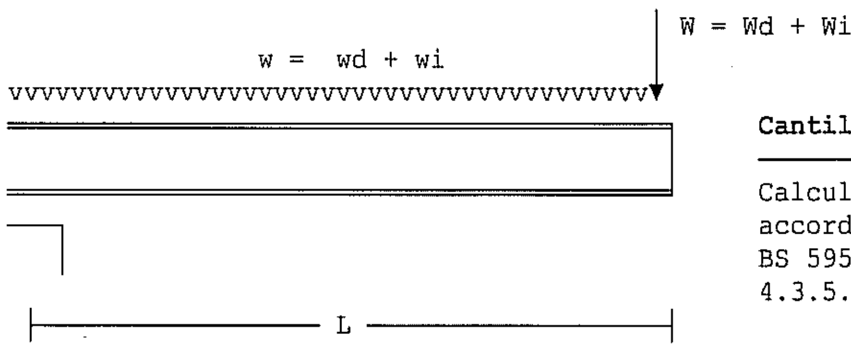
Beam span	L=4.4 m
203 x 102 x 23 UB.	
Young's Modulus	E=205 kN/mm²
Dead load factor	gamd=1.4
Imposed load factor	gami=1.6
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.4 m
Dead load (unfactored)	Gku(1)=1.69 kN/m
Imposed load (unfactored)	Qku(1)=2.06 kN/m
Maximum span bending moment	13.702 kNm
Design shear force	Fv=12.456 kN

UNIVERSAL BEAM DESIGN SUMMARY

203 x 102 x 23 UB Grade S 275

	Maximum shear force	**12.456 kN**
	Shear capacity	**181.05 kN**
	Max. applied moment	**13.702 kNm**
	Moment capacity	**64.35 kNm**
	Buckling resistance	**19.093 kNm**
	Moment factor (mLT)	**0.925**
	Resistance (Mb/mLT)	**20.641 kNm**
	Unfactored DL defln	**1.9068 mm**
	Unfactored LL defln	**2.3242 mm**
	Limiting deflection	**12.222 mm**
Unfactored end shears	**DL shear at LHE**	**3.718 kN**
	LL shear at LHE	**4.532 kN**
	DL shear at RHE	**3.718 kN**
	LL shear at RHE	**4.532 kN**

Location: PORCH CANTILEVER



Cantilevered beam

Calculations are in accordance with BS5950 BS 5950-1:2000 clause 4.3.5.4 or 4.3.5.5.

Cantilever span L=1.85 m

Section properties

203 x 102 x 23 UB.
Dimensions (mm): D=203.2 B=101.8 t=5.4 T=9.3 r=7.6
Properties (cm): Ix=2110 Iy=164 Sx=234 Sy=49.8 J=7.02
A=29.4 ry=2.3618 rx=8.4716

Strength of steel - Clause 3.1.1

The material thickness is 9.3 mm and the steel grade is S 275.
Design strength py=275 N/mm^2
Young's Modulus E=205 kN/mm^2

Classification - Clause 3.5.2

Classify outstand element of compression flange:
Parameter (Table 11 Note b) e=(275/py)^0.5=(275/275)^0.5
=1
Outstand b=B/2=101.8/2=50.9 mm
Ratio b'T=b/T=50.9/9.3=5.4731
As b/T ≤ 9e (9), outstand element of compression flange is classified as Class 1 plastic.
Classify web of section:
Depth between fillet radii d=D-2*(T+r)=203.2-2*(9.3+7.6)
=169.4 mm
Ratio d't=d/t=169.4/5.4=31.37
Neutral axis is assumed to be at mid-depth of section.
As d/t ≤ 80e (80), web is classified as Class 1 plastic.

Shear buckling

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

Loading on cantilever

Distributed loads:
Dead load (including self weight) wd=2.44 kN/m
Imposed load wi=2.06 kN/m

Factored distributed load	w=1.4*wd+1.6*wi=1.4*2.44+1.6*2.06 =6.712 kN/m
Moment at support	Mu=w*L^2/2=6.712*1.85^2/2 =11.486 kNm
Accompanying shear force	Fu=w*L=6.712*1.85=12.417 kN

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.
Restraint conditions:
1. Support is continuous, with partial torsional restraint.
2. Cantilever is free at tip.
3. Normal loading conditions will be considered.

Effective length (Table 14) Le=2.0*L*1000=2.0*1.85*1000
=3700 mm

Resistance to lateral-torsional buckling - Clause 4.3.6

Radius of gyration (minor axis)	ry=(Iy/A)^0.5=(164/29.4)^0.5 =2.3618 cm
Slenderness of section	lambda=Le/(ry*10)=3700/(2.3618*10) =156.66
Buckling parameter	u=(4*Sx^2*(1-Iy/Ix)/(A^2*((D-T) /10)^2))^0.25 =(4*234^2*(1-164/2110)/(29.4^2 *((203.2-9.3)/10)^2))^0.25 =0.88792
Torsional index	x=0.566*((D-T)/10)*(A/J)^0.5 =0.566*((203.2-9.3)/10)*(29.4 /7.02)^0.5 =22.459
Ratio	ratio=lambda/x=156.66/22.459 =6.9752
Slenderness factor	v=1/((1+0.05*ratio^2)^0.25) =1/((1+0.05*6.9752^2)^0.25) =0.73467
Ratio βw	betaw=1.0
Equivalent slenderness	lamLT=u*v*lambda*(betaw)^0.5 =0.88792*0.73467*156.66*(1)^0.5 =102.19
Limiting slenderness	lamlo=0.4*((PI^2*E*10^3)/py)^0.5 =0.4*((3.1416^2*205*10^3) /275)^0.5 =34.31
Perry coefficient	etaLT=0.007*(lamLT-lamlo) =0.007*(102.19-34.31) =0.47518
Elastic strength	pe=PI^2*E*10^3/(lamLT^2) =3.1416^2*205*10^3/(102.19^2) =193.74 N/mm²
Factor	phiLT=(py+(etaLT+1)*pe)/2 =(275+(0.47518+1)*193.74)/2 =280.4 N/mm²

Factor pey=pe*py=193.74*275=53277

Bending strength pb=(pey)/(phiLT+((phiLT^2-pey)^0.5))
=(53277)/(280.4+((280.4^2
-53277)^0.5))
=121.19 N/mm²

Buckling resistance moment Mb=Sx*pb/10^3=234*121.19/10^3
=28.36 kNm

Equivalent uniform moment factor mLT=1.0

Since M ≤ Mb/mLT (11.486 kNm ≤ 28.36 kNm) section O.K. for lateral torsional buckling resistance.

Check section for combined moment and shear

Maximum shear

Shear area Av=t*D=5.4*203.2=1097.3 mm²

Shear capacity Pv=0.6*py*Av/10^3=0.6*275*1097.3/10^3
=181.05 kN

Design shear force Fv=12.417 kN

Since Fv ≤ Pv (12.417 kN ≤ 181.05 kN) shear force within shear capacity.

Maximum moment

Elastic modulus Z=Ix/(D/20)=2110/(203.2/20)
=207.68 cm³

Since Fv < 0.6 Pv

Moment capacity for compact sec Mc=py*Sx/10^3=275*234/10^3
=64.35 kNm

Since M ≤ Mc (11.486 kNm ≤ 64.35 kNm) applied moment within moment capacity.

N.B. Buckling resistance is less than moment capacity and therefore buckling resistance controls the design.

Check for deflection

Imposed load only considered in deflection calculation

UDL for deflection calculation wu=wi=2.06 kN/m

Deflection for udl yu=(3*wu*L^4/24)*10^5/(E*Ix)
=(3*2.06*1.85^4/24)*10^5/(205*2110)
=0.69731 mm

Total deflection yt=0.69731 mm

Limiting deflection ylim=L*10^3/180=1.85*10^3/180
=10.278 mm

Since yt ≤ ylim (0.69731 mm ≤ 10.278 mm), O.K. for deflection.

UNIVERSAL BEAM DESIGN SUMMARY

203 x 102 x 23 UKB Grade S 275	
Design shear force	12.417 kN
Shear capacity	181.05 kN
Design moment	11.486 kNm
Moment capacity	64.35 kNm
Buckling resistance	28.36 kNm
Deflection	0.69731 mm
Limiting deflection	10.278 mm

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Project: CHEDDAR PARISH PAVILION
STRUCTURAL CALCULATIONS

Common Rafters

Front Covered Porch:

Sloping span length = 4·45 m

Loads Normal to Rafters

UDL's DL = 0·4m x 0·65 kN/m² x $\cos^2 32{\cdot}5°$ = 0·19 kN/m

IL = 0·4m x 55 " " = 0·16 kN/m

Point load = 0·9 Cos 32·5° = 0·76 kN

USE 170 x 47 C24 @ 400 — Ref R7–10
NOGGINS MID LENGTH

Entrance Hall / WC's

Loads Normal to Rafter

UDL's DL = 0·6 m x 0·37 kN/m² x $\cos^2 32{\cdot}5$ = 0·16 kN/m

IL = " 0·55 x " = 0·23

USE 170 x 47 C24 @ 600 — Ref R11 – 12

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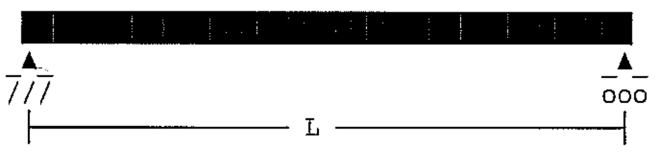
Location: FRONT COVERED PORCH - COMMON RAFTER - UDL'S

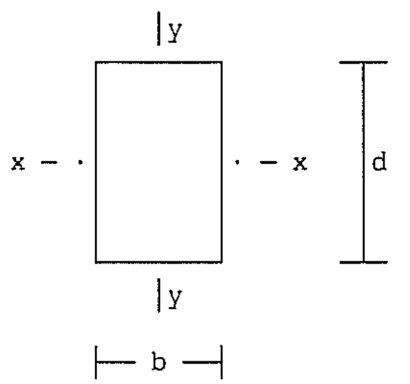
Timber beam to BS5268-2:2002



Simply supported beam subjected to vertical loads.

Beam span	L=4.45 m
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.45 m
Dead load (unfactored)	Gku(1)=0.19 kN/m
Imposed load (unfactored)	Qku(1)=0.16 kN/m
Maximum span bending moment	0.86636 kNm
Design shear force	Fve'=0.77875

Section design parameters



Design axial load (+ve comp)	Fa=0.496 kN
Depth of section	d=170 mm
Width of section	b=45 mm
Eff length for bending about xx	Lex=4450 mm
Eff length for bending about yy	Ley=2225 mm
Length of bearing	lb=50 mm

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

Bearing Modification factor K4=1.0

Strength class C24 to Table 8.

Timber service class adopted tmclass=2

Timber service class modification factor K2=1 as Table 16.

Duration of loading K3=1.25

Depth factor K7=(300/d)^0.11=1.0645

Load-sharing modification factor K8=1.1

Modification factor K12=(0.5+(1+eta)*C/2)-((0.5+(1+eta)*C/2)^2 -C)^0.5=0.14061

No notches exist at the support K5=1.0

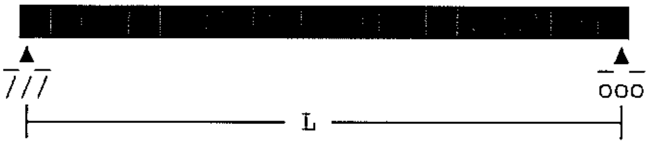
DESIGN SUMMARY

Member: 170 mm x 45 mm
Strength class C24 to Table 8.
Moisture service class 2

Bending stress	3.997 N/mm^2
Permissible bending	10.977 N/mm^2
Comp Stress @ Mids.	0.064837 N/mm^2
Permiss compression	1.5274 N/mm^2
Interaction factor	0.40863
Deflection	9.1828 mm
Limiting deflection	13.35 mm
Shear stress	0.1527 N/mm^2
Permissible shear	0.97625 N/mm^2
Bearing stress	0.34611 N/mm^2
Permissible bearing	3.3 N/mm^2

Location: FRONT COVERED PORCH - COMMON RAFTER - POINT LOAD CHECK

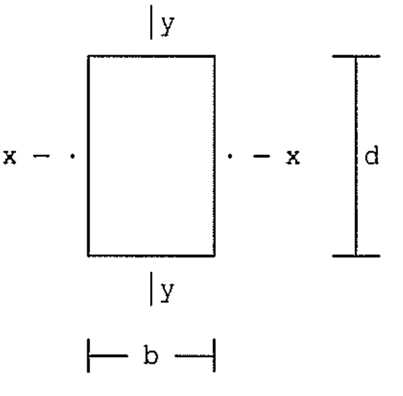
Timber beam to BS5268-2:2002



Simply supported beam subjected to vertical loads.

Beam span	L=4.45 m
Distance from left support	Lc(1)=2.225 m
Dead load (unfactored)	Gkc(1)=0 kN
Imposed load (unfactored)	Qkc(1)=0.76 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.45 m
Dead load (unfactored)	Gku(1)=0.19 kN/m
Imposed load (unfactored)	Qku(1)=0 kN/m
Maximum span bending moment	1.3158 kNm
Design shear force	Fve'=0.80275

Section design parameters



Design axial load (+ve comp) Fa=0.496 kN
Depth of section d=170 mm
Width of section b=45 mm
Eff length for bending about xx Lex=4450 mm
Eff length for bending about yy Ley=2225 mm
Length of bearing lb=50 mm
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor K4=1.0
Strength class C24 to Table 8.
Timber service class adopted tmclass=2
Timber service class modification factor K2=1 as Table 16.
Duration of loading K3=1.25
Depth factor K7=(300/d)^0.11=1.0645
Load-sharing modification factor K8=1.1
Modification factor K12=(0.5+(1+eta)*C/2)-((0.5+(1+eta)*C/2)^2-C)^0.5=0.14061
No notches exist at the support K5=1.0

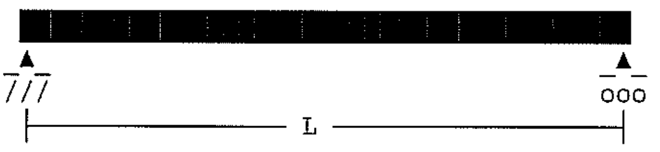
Member: 170 mm x 45 mm
Strength class C24 to Table 8.
Moisture service class 2

DESIGN SUMMARY

Bending stress	6.0706 N/mm²
Permissible bending	10.977 N/mm²
Comp Stress @ Mids.	0.064837 N/mm²
Permiss compression	1.5274 N/mm²
Interaction factor	0.5986
Deflection	12.194 mm
Limiting deflection	13.35 mm
Shear stress	0.1574 N/mm²
Permissible shear	0.97625 N/mm²
Bearing stress	0.35678 N/mm²
Permissible bearing	3.3 N/mm²

Location: ENTRANCE HALL/WC'S - COMMON RAFTER - UDL'S

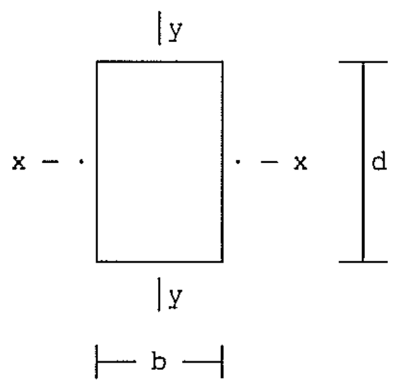
Timber beam to BS5268-2:2002



Simply supported beam subjected to vertical loads.

Beam span	L=4.45 m
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.45 m
Dead load (unfactored)	Gku(1)=0.16 kN/m
Imposed load (unfactored)	Qku(1)=0.23 kN/m
Maximum span bending moment	0.96537 kNm
Design shear force	Fve'=0.86775

Section design parameters



Design axial load (+ve comp)	Fa=0.553 kN
Depth of section	d=170 mm
Width of section	b=45 mm
Eff length for bending about xx	Lex=4450 mm
Eff length for bending about yy	Ley=2225 mm
Length of bearing	lb=50 mm

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

Bearing Modification factor K4=1.0

Strength class C24 to Table 8.

Timber service class adopted tmclass=2

Timber service class modification factor K2=1 as Table 16.

Duration of loading K3=1.25

Depth factor K7=(300/d)^0.11=1.0645

Load-sharing modification factor K8=1.1

Modification factor K12=(0.5+(1+eta)*C/2)-((0.5+(1+eta)*C/2)^2
-C)^0.5=0.14061

No notches exist at the support K5=1.0

DESIGN SUMMARY

Member: 170 mm x 45 mm
Strength class C24 to Table 8.
Moisture service class 2

Bending stress	4.4538 N/mm^2
Permissible bending	10.977 N/mm^2
Comp Stress @ Mids.	0.072288 N/mm^2
Permiss compression	1.5274 N/mm^2
Interaction factor	0.45563
Deflection	10.232 mm
Limiting deflection	13.35 mm
Shear stress	0.17015 N/mm^2
Permissible shear	0.97625 N/mm^2
Bearing stress	0.38567 N/mm^2
Permissible bearing	3.3 N/mm^2

Ridge Beam Central Section

UDL's

Average loaded width $= \frac{5.2m}{2} = 2.6m$

DL $= 2.6m \times 0.37 kN/m^2 = 0.96 kN/m$

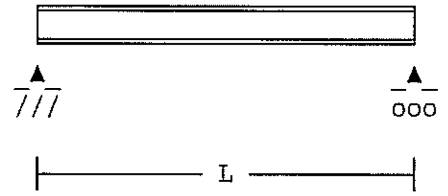
IL $=$ " $\times 0.55$ " $= 1.42 kN/m$

S/W DL = say 0.3 kN/m

Span = 7.1m

USE 254 x 146 x 31 UB — Ref R14

Location: CENTRAL SECTION RIDGE BEAM



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=7.1 m
254 x 146 x 31 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)=7.1 m
Dead load (unfactored)	Gku(1)=1.26 kN/m
Imposed load (unfactored)	Qku(1)=1.42 kN/m
Maximum span bending moment	25.432 kNm
Design shear force	Fv=14.328 kN

UNIVERSAL BEAM DESIGN SUMMARY

254 x 146 x 31 UB Grade S 275

	Maximum shear force	**14.328 kN**
	Shear capacity	**248.89 kN**
	Max. applied moment	**25.432 kNm**
	Moment capacity	**108.07 kNm**
	Buckling resistance	**24.238 kNm**
	Moment factor (mLT)	**0.925**
	Resistance (Mb/mLT)	**26.203 kNm**
	Unfactored DL defln	**4.6116 mm**
	Unfactored LL defln	**5.1972 mm**
	Limiting deflection	**19.722 mm**
Unfactored end shears	**DL shear at LHE**	**4.473 kN**
	LL shear at LHE	**5.041 kN**
	DL shear at RHE	**4.473 kN**
	LL shear at RHE	**5.041 kN**

Valley Beam Arrangement.

Load from New Roof

Average loaded width $= \frac{3.4m}{2} = 1.70\,m$

DL = $1.70m \times 0.31$ kN/m² $= 0.53$ kN/m

IL = " $\times 0.55$ " $= 0.94$ "

Point loads from existing Purlins

DL = $1.5m \times \frac{3.3m}{2} \times 0.22$ kN/m² $= 0.54$ kN

IL = - " - $\times 0.55$ " $= 1.36$ kN.

Geometry

Plan Length = 6.4m

Rise = 2.29m

Slope length = 6.8 m

Lower Valley Beam:

Plan length = 3.6m

Sloping length = 3.83m

USE 240 x 75 C24 — Ref R16, 17

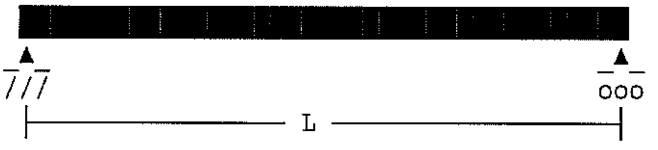
Upper Valley Beam:

Plan Length = 3.8m

Slope length = 4.04 m

Location: VALLEY BEAM - LOWER SECTION

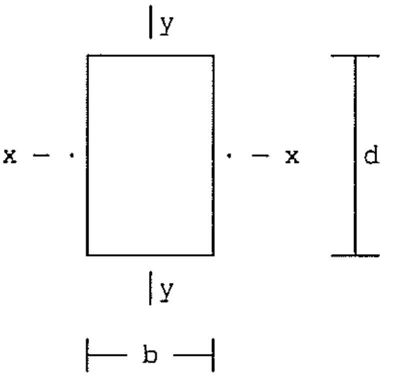
Timber beam to BS5268-2:2002



Simply supported beam subjected to vertical loads.

Beam span	L=3.83 m
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=3.83 m
Dead load (unfactored)	Gku(1)=0.73 kN/m
Imposed load (unfactored)	Qku(1)=0.94 kN/m
Maximum span bending moment	3.0621 kNm
Design shear force	Fve'=3.198

Section design parameters



Design axial load (+ve comp) Fa=0 kN
Depth of section d=240 mm
Width of section b=75 mm
Eff length for bending about xx Lex=3830 mm
Eff length for bending about yy Ley=3830 mm
Length of bearing lb=100 mm
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor K4=1.0
Strength class C24 to Table 8.
Timber service class adopted tmclass=2
Timber service class modification factor K2=1 as Table 16.
Duration of loading K3=1.25
Depth factor K7=(300/d)^0.11=1.0248
Load-sharing modification factor K8=1.0
No notches exist at the support K5=1.0

DESIGN SUMMARY

Member: 240 mm x 75 mm
Strength class C24 to Table 8.
Moisture service class 2

Bending stress	4.253 N/mm²
Permissible bending	9.608 N/mm²
Deflection	7.9751 mm
Limiting deflection	11.49 mm
Shear stress	0.2665 N/mm²
Permissible shear	0.8875 N/mm²
Bearing stress	0.4264 N/mm²
Permissible bearing	3 N/mm²

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Job No:	u141
Sheet No	R18
Prepared By	DCB
Date	Aug-19
Checked	

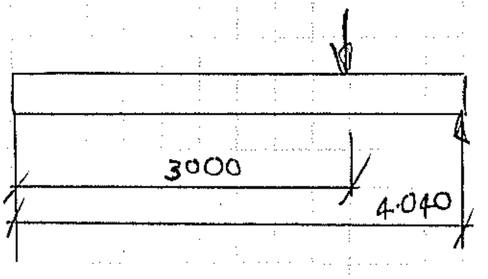
Project: CHEDDAR PARISH PAVILION
STRUCTURAL CALCULATIONS

Load from New Roof.

Average loaded width $= \frac{2.3}{2}m = 1.15\,m$

DL $= 1.15 \times 0.31\ kN/m^2 = 0.36\ kN/m$

IL $=$ " $\times 0.55 = 0.63$ "

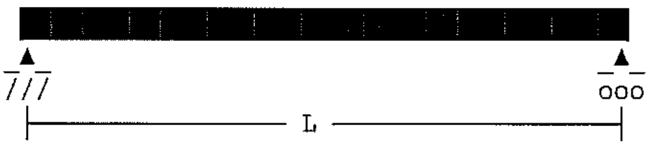


USE 240 x 75 C24

Ref R 19,20

Location: VALLEY BEAM - UPPER SECTION

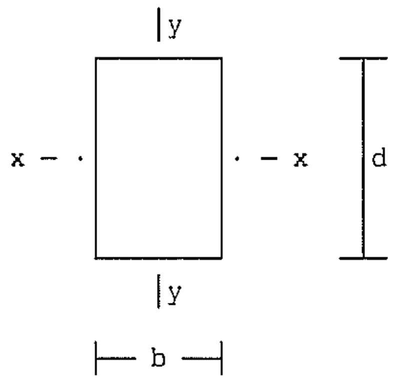
Timber beam to BS5268-2:2002



Simply supported beam subjected to vertical loads.

Beam span	L=4.04 m
Distance from left support	Lc(1)=3 m
Dead load (unfactored)	Gkc(1)=0.54 kN
Imposed load (unfactored)	Qkc(1)=1.36 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=4.04 m
Dead load (unfactored)	Gku(1)=0.56 kN/m
Imposed load (unfactored)	Qku(1)=0.63 kN/m
Maximum span bending moment	3.5163 kNm
Design shear force	Fve'=3.8147 kN

Section design parameters



Design axial load (+ve comp)	Fa=0 kN
Depth of section	d=240 mm
Width of section	b=75 mm
Eff length for bending about xx	Lex=4040 mm
Eff length for bending about yy	Ley=4040 mm
Length of bearing	lb=100 mm

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

Bearing Modification factor	K4=1.0

Strength class C24 to Table 8.

Timber service class adopted	tmclass=2

Timber service class modification factor K2=1 as Table 16.

Duration of loading	K3=1.25
Depth factor	K7=(300/d)^0.11=1.0248
Load-sharing modification factor	K8=1.0
No notches exist at the support	K5=1.0

DESIGN SUMMARY

Member: 240 mm x 75 mm
Strength class C24 to Table 8.
Moisture service class 2

Bending stress	4.8838 N/mm²
Permissible bending	9.608 N/mm²
Deflection	10.11 mm
Limiting deflection	12.12 mm
Shear stress	0.31789 N/mm²
Permissible shear	0.8875 N/mm²
Bearing stress	0.50863 N/mm²
Permissible bearing	3 N/mm²

CEILING BEAMS

Entrance Hall Beam

Point load – from Ridge Beam

$DL = \frac{11.5m}{2} \times (1.39 + 0.3)\ kN/m = 9.72\ kN$ — Ref R1

$IL = \text{"} \times 2.06 = 11.85\ kN$

UDL

Ceiling Joists

$DL = 0.35\ kN/m^2 \times \frac{2.5m}{2} = 0.44\ kN/m$

$IL = 0.25\ \text{"} \times \text{"} = 0.31\ kN/m$

Wall

$DL = 1.7m \times (0.1 \times 16\ kN/m^3 + \overset{\text{plaster}}{0.36}) = 3.37\ kN/m$

Beam s/w say $DL = 0.25$

$\Sigma DL = 4.06\ kN/m \quad \Sigma IL = 0.31\ kN/m$

TDL

Wall $DL = 1m \times (0.1 \times 16 + 0.36) = 1.98\ kN/m$

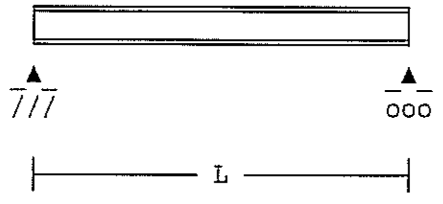
Span = 3.1m

USE 203 x 133 x 25 UB — Ref R22

Location: CEILING BEAM - ENTRANCE HALL

Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.



Beam span	L=3.1 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
Distance from left support	Lc(1)=1.55 m
Dead load (unfactored)	Gkc(1)=9.72 kN
Imposed load (unfactored)	Qkc(1)=11.85 kN
Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=3.1 m
Dead load (unfactored)	Gku(1)=4.06 kN/m
Imposed load (unfactored)	Qku(1)=0.31 kN/m
DL at centre (unfactored)	Gkt(1)=1.98 kN/m
Imposed at centre (unfactored)	Qkt(1)=0 kN/m
Maximum span bending moment	34.884 kNm
Design shear force	Fv=28.011 kN
Quarter point	m2=19.714 kNm
Mid-span	m3=34.884 kNm
Three quarter point	m4=19.714 kNm

UNIVERSAL BEAM DESIGN SUMMARY

203 x 133 x 25 UB Grade S 275

	Maximum shear force	**28.011 kN**
	Shear capacity	**191.11 kN**
	Max. applied moment	**34.884 kNm**
	Moment capacity	**70.95 kNm**
	Buckling resistance	**34.905 kNm**
	Moment factor (mLT)	**0.86954**
	Resistance (Mb/mLT)	**40.142 kNm**
	Unfactored DL defln	**2.593 mm**
	Unfactored LL defln	**1.6109 mm**
	Limiting deflection	**8.6111 mm**
Unfactored end shears	**DL shear at LHE**	**12.688 kN**
	LL shear at LHE	**6.4055 kN**
	DL shear at RHE	**12.688 kN**
	LL shear at RHE	**6.4055 kN**

Central Ceiling Beam

UDLS

Ceiling Joists $DL = \left(\frac{4.2 + 2.5}{2}\right) \times 0.35\,kN/m^2 = 1.17$

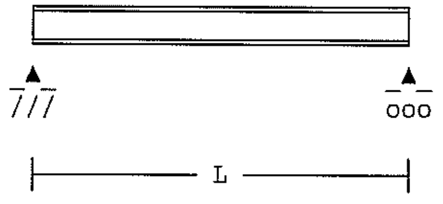
$IL = \quad '' \quad \times 0.25 \quad = 0.84$

Beam S/W $DL =$ say 0.25

Span = 3.5

USE 203 x 102 x 23 UB — Ref R24

Location: CEILING BEAM - CENTRAL ENTRANCE AREA



Simply supported steel beam

Calculations are in accordance with BS5950-1:2000.

Beam span	L=3.5 m
203 x 102 x 23 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)=3.5 m
Dead load (unfactored)	Gku(1)=1.42 kN/m
Imposed load (unfactored)	Qku(1)=0.84 kN/m
Maximum span bending moment	5.1021 kNm
Design shear force	Fv=5.831 kN

UNIVERSAL BEAM DESIGN SUMMARY

203 x 102 x 23 UB Grade S 275

	Maximum shear force	**5.831 kN**
	Shear capacity	**181.05 kN**
	Max. applied moment	**5.1021 kNm**
	Moment capacity	**64.35 kNm**
	Buckling resistance	**23.202 kNm**
	Moment factor (mLT)	**0.925**
	Resistance (Mb/mLT)	**25.083 kNm**
	Unfactored DL defln	**0.64145 mm**
	Unfactored LL defln	**0.37945 mm**
	Limiting deflection	**9.7222 mm**
Unfactored end shears	**DL shear at LHE**	**2.485 kN**
	LL shear at LHE	**1.47 kN**
	DL shear at RHE	**2.485 kN**
	LL shear at RHE	**1.47 kN**

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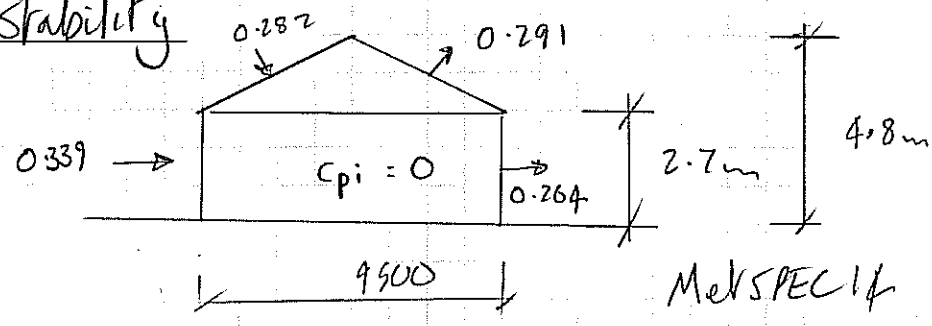
Project:	CHEDDAR PARISH PAVILION	Job No:	u141
	STRUCTURAL CALCULATIONS	Sheet No	W1
		Prepared By	DCB
		Date	Aug-19
		Checked	

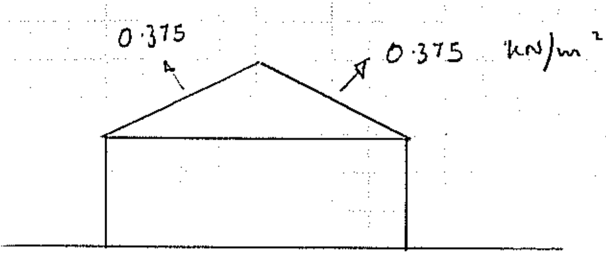
WIND LOAD.

Lateral Stability



MetSPEC Ref W2-4

Wind Uplift



Ref W5, 6

Vertical Component $= 0.375 \cos 32.5°$

$= 0.32$ kN/m²

DL = 0.555 kN/m² Ref L2

∴ No Nett uplift on trussed Rafters

W2

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Wind Assessment to BS6399-2

Data Entry:-

Site Altitude	*18.000*	m	Reference Height (Hr)			Diagonal Dimension (a)		
Basic Wind Speed	*20.900*	m/s	Roof	*4.800*	m	Roof	*9.000*	m
Seasonal Factor (Ss)	*1.000*		Side Walls	*2.700*	m	Sides	*2.700*	m
Probability Factor(Sp)	*1.000*		Gables	*4.800*	m	Gables	*2.700*	m
Site ID	*ST448535*							

Dynamic Pressure Results

Wind Direction (deg)		0	30	60	90	120	150	180	210	240	270	300	330
Direction Factor Sd		0.78	0.73	0.73	0.74	0.73	0.80	0.85	0.93	1.00	0.99	0.91	0.82
Topographic Location Factor s		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Altitude Factor Sa		1.018	1.018	1.018	1.018	1.018	1.018	1.018	1.018	1.018	1.018	1.018	1.018
Effective Height He(m)	Roof	4.80	4.80	4.80	1.92	4.80	4.80	4.80	4.80	4.80	4.80	1.92	1.92
	Sides	2.70	2.70	2.70	1.08	2.70	2.70	2.70	2.70	2.70	2.70	1.08	1.08
	Gable	4.80	4.80	4.80	1.92	4.80	4.80	4.80	4.80	4.80	4.80	1.92	1.92
Terrain & Building Factor Sb	Roof	1.452	1.452	1.452	1.056	1.452	1.467	1.490	1.452	1.452	1.547	1.148	1.076
	Sides	1.308	1.308	1.308	1.056	1.308	1.322	1.344	1.308	1.308	1.395	1.148	1.076
	Gable	1.452	1.452	1.452	1.056	1.452	1.467	1.490	1.452	1.452	1.547	1.148	1.076
Site Wind Speed Vs(m/s)	Roof	16.60	15.53	15.53	15.74	15.53	17.02	18.08	19.79	21.28	21.06	19.36	17.45
	Sides	16.60	15.53	15.53	15.74	15.53	17.02	18.08	19.79	21.28	21.06	19.36	17.45
	Gable	16.60	15.53	15.53	15.74	15.53	17.02	18.08	19.79	21.28	21.06	19.36	17.45
Effective Wind Speed Ve(m/s)	Roof	24.09	22.54	22.54	16.63	22.54	24.97	26.96	28.72	30.88	32.59	22.22	18.78
	Sides	21.71	20.32	20.32	16.63	20.32	22.50	24.30	25.89	27.83	29.37	22.22	18.78
	Gable	24.09	22.54	22.54	16.63	22.54	24.97	26.96	28.72	30.88	32.59	22.22	18.78
Dynamic Pressure q (kN/m²)	Roof	0.356	0.312	0.312	0.169	0.312	0.382	0.445	0.506	0.585	0.651	0.303	0.216
	Sides	0.289	0.253	0.253	0.169	0.253	0.310	0.362	0.411	0.475	0.529	0.303	0.216
	Gable	0.356	0.312	0.312	0.169	0.312	0.382	0.445	0.506	0.585	0.651	0.303	0.216
Size Effect Factor Cae	Roof	0.960	0.960	0.960	0.950	0.960	0.960	0.960	0.960	0.960	0.960	0.950	0.950
	Sides	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
	Gable	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000

W3

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Date 15/08/2019
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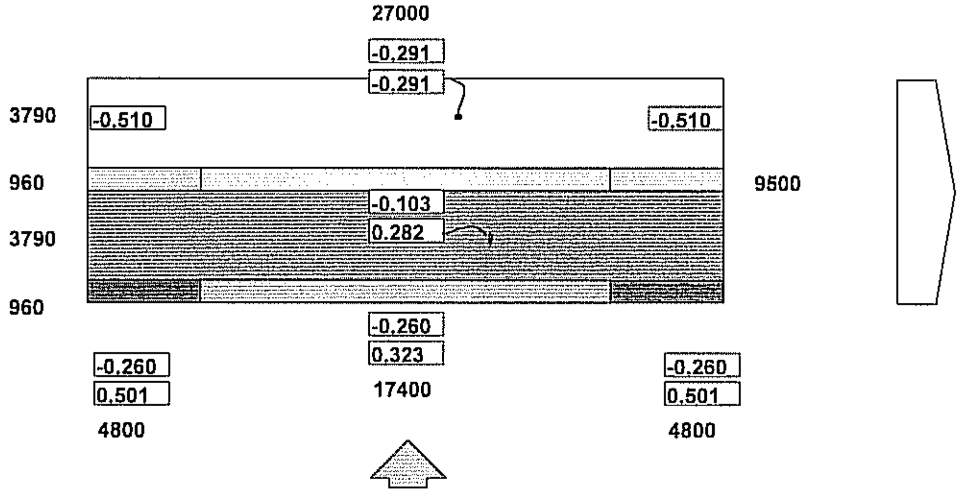
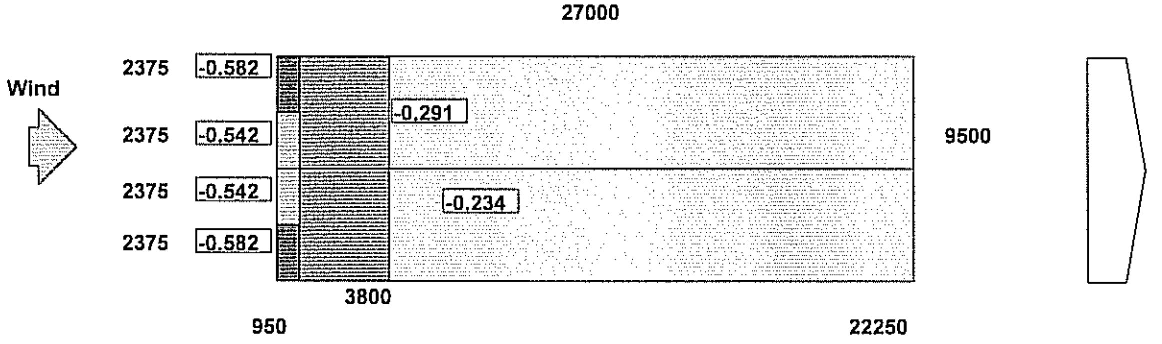
Wind Analysis to BS6399-2 - Wind Loads for Roofs

DATA ENTRY:-

Width of Bay	9.500 m	**Reference Height**	4.800m
Length of Bay	27.000 m	**Roof Pitch**	32.500 deg.
Roof Type	Ridged Duopitch roof		
Bay type	Single bay building		





NB: All dimensions are in millimetres and wind loads in kN/m²

W4

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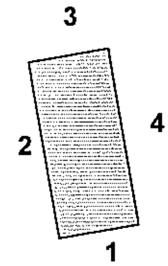
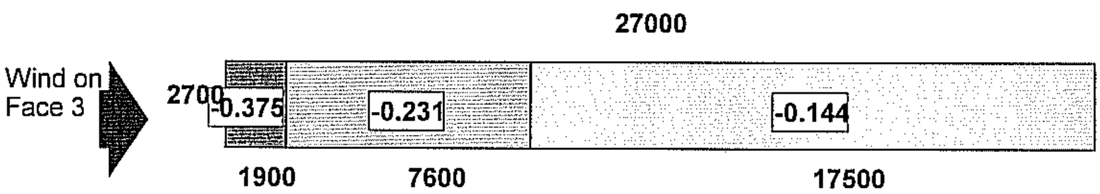
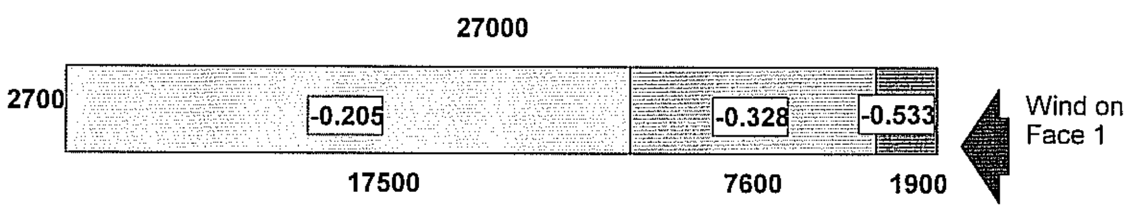
Wind Analysis to BS6399-2 - Wind Loads for Walls

DATA ENTRY:-	
Short Face 1 or 3	9.500 m
Long Face 2 or 4	27.000 m
Reference Face	Face 2 (Side)
Reference Height	2.700 m
Gap Between Buildings	0.000 m



Reference Face





Suction(kN/m²) On Reference Face2

Pressure (kN/m²) On Reference Face = +0.339

Pressure/Suction (kN/m²) On Opposite(Leeward) Face = -0.264

Note: The above loads are not applicable to parapets which must be designed separately.

NB: All dimensions are in millimetres and wind loads in kN/m²

W5

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Date: 16/08/2019

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Wind Assessment to BS6399-2

Data Entry:-

Condition *Without Dominant Opening*

Internal Volume *135.000* m^3

Cpi	-ve	*0.000*
	+ve	*0.200*

Reference Height (Hr)

Roof	4.800	m
Side Walls	2.700	m
Gables	4.800	m

Size Effect Factors For Cpi

Wind Direction (deg)	0	30	60	90	120	150	180	210	240	270	300	330
Size Effect Factor Cai (Roof)	0.824	0.824	0.824	0.788	0.824	0.824	0.824	0.824	0.824	0.824	0.788	0.788

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Date **16/08/2019**
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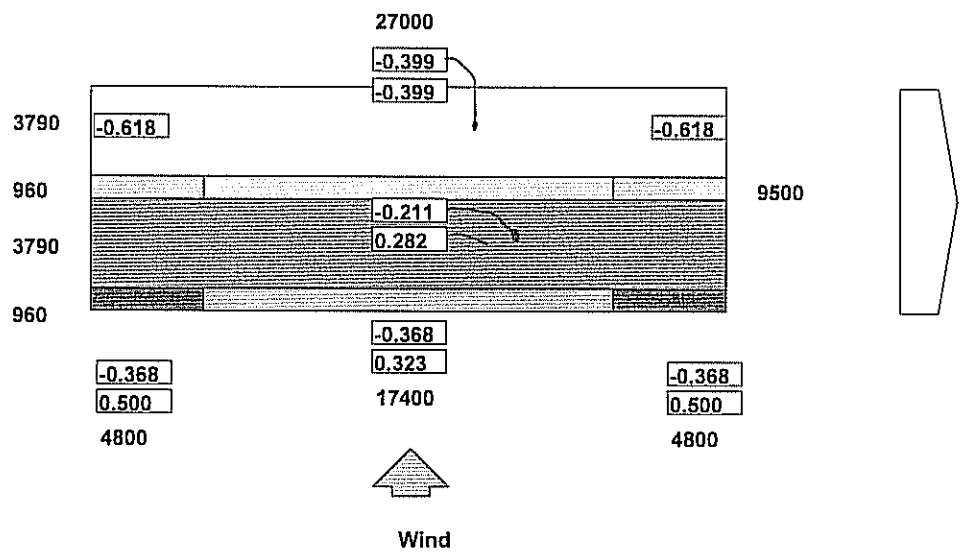
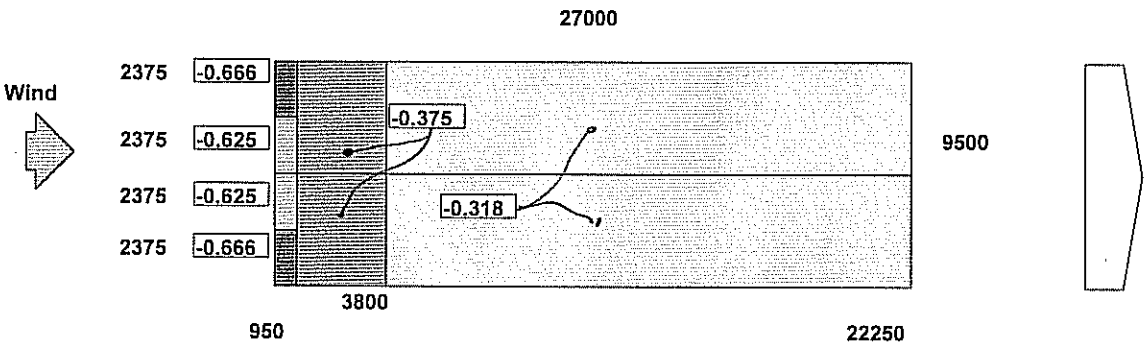
Wind Analysis to BS6399-2 - Wind Loads for Roofs

DATA ENTRY:-

Width of Bay	9.500 m	**Reference Height**	4.800m
Length of Bay	27.000 m	**Roof Pitch**	32.500 deg.
Roof Type	Ridged Duopitch roof		
Bay type	Single bay building		





NB: All dimensions are in millimetres and wind loads in kN/m²

Sand Engineering Consultants		Job No:	u141
		Sheet No	WP 1
		Prepared By	DCB
Project:	CHEDDAR PARISH PAVILION	Date	Aug-19
	STRUCTURAL CALCULATIONS	Checked	

WIND POSTS

Resolve WL to horizontal force at eaves level.

Wall $= \frac{2.7m}{2} \times 4.5m \, (0.339 + 0.264)\, kN/m^2$

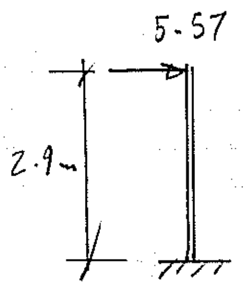
$= 3.66 \; kN$

Roof $= 5.4m \times 4.5m \, (0.282 + 0.291)\, kN/m^2 \times \sin 37.5°$

$= 7.48$

load per post $= 5.57 \; kN$.

$M = 1.4 \times 10.85 = 15.18 \; kNm$



Limit $\delta \not> \frac{h}{300} = 9.7mm$

$152 \times 152 \times 30$ UC $\rightarrow$ $\delta = 8.6mm$ ok

$M_R = 45 \; kNm$ ok.

USE 152 x 152 x 30 UC

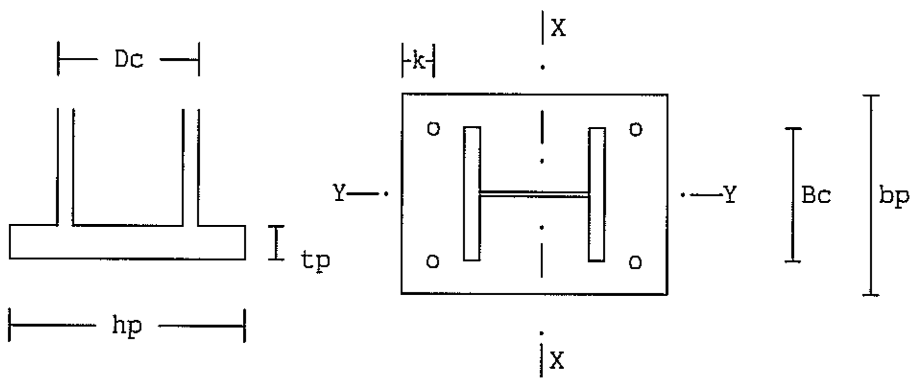
USE 350 x 350 x 15 BASE PLATE

4N° M16 RESIN ANCHORS

Location: WIND POST BASE PLATE

Unstiffened column slab base

Calculations are in accordance with 'Joints in Steel Construction Moment Connections' published by The Steel Construction Institute.



ELEVATION PLAN

Axial load (+ve compression)	N=1 kN
Moment about X-X axis	M=15.18 kNm
Shear on the base in Y direction	Fy=7.8 kN

152 x 152 x 30 UC.

Length of baseplate	hp=350 mm
Breadth of baseplate	bp=350 mm
Edge distance to bolt centre line	k=50 mm
Strength of concrete	fcu=15 N/mm²
Assumed weld size	sw=6 mm
Selected baseplate thickness	tp=15 mm

Total number of bolts to be used	n=4
Assumed diameter of bolt	bd=20 mm
Overall embedded depth	Lo=200 mm
Assumed cover to reinforcement	cv=0 mm
tension reinforcement	Asp=0 %
Selected fillet weld size	sw=6 mm

Web welds

Length of web weld	Lwv=2*dc=247.2 mm
Weld force per mm	Fyw=Fy*10^3/Lwv=31.553 N/mm
Weld size required	sww=Fyw/(0.7*pw)=0.20489 mm
Selected fillet weld size	sfw=6 mm

SUMMARY BASEPLATE REQUIREMENTS	**Size 350 mm x 350 mm x 15 mm**
	Grade S 275 steel
	Edge distance 50 mm
	Number of H.D. bolts 4
	Diameter of bolts M 20
	Grade 8.8
	Overall embedded depth 200 mm
	Anchor plate size:
	100 mm x 100 mm x 15 mm
	Concrete/grout (fcu) 15 N/mm²
WELDS TENSION	**Fillet weld 6 mm**
	Contact areas on the baseplate and column are machined to give a tight bearing contact.
	The specified tension weld will be used for both flanges.
WEB	**Fillet weld 6 mm**

Bolt Pull Out Force.
USE RAWL R-KEM II M16
HDB WITH 110 mm EMBEDMENT
DESIGN TENSION RESISTANCE
42.5 kN IN C20/25.

Pad Foundation

Vertical loads

Roof DL = $\frac{9.8m}{2}$ x 4.5m x 0.25 kN/m² = 5.5 kN

Ceiling DL = $\frac{7m}{2}$ x 4.5m x 0.28 " = 4.41 kN

Wall

Inner DL = 2.9m x 1.8m x (0.1 x 8 kN/m³ + 0.18)

DL = 5.11 kN

Outer DL = 2.9 x 1.8 x 0.1 x 16 kN/m³

DL = 8.35 kN.

Total DL = 23.37 kN.

Try 1.8 x 0.8 x 0.85m x 24 kN/m³ = 29.38 kN.

M_{OT} = (2.9 + 0.85) x 5.57 = 20.88 kNm

M_{REST} = $\frac{0.8m}{2}$ x (23.37 + 29.38) = 21.1 kNm.

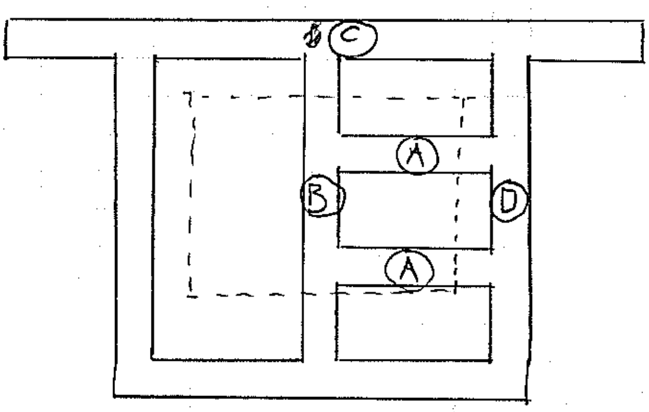
Adopt 1.8 x 1m WIDE.

REINFORCED CONCRETE GROUND BEAMS

Loads

Screed 1.73 kN/m²
Insulation 0.05 "
PC Beam & Block 2.37 kN/m²
DL = 4.15 kN/m²
IL = 5.0 kN/m²

Arrangement



Beam A

UDL's

Beam S/W = $0.45 \times 0.6 \times 25$ kN/m³ DL = 6.75 kN/m

Wall DL = $3.3\text{m} \times (0.1\text{m} \times 16$ kN/m³ + 0.36 Plaster)

DL = 6.47 kN/m

Span

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Date	Sep-19
Checked	

Project: CHEDDAR PARISH PAVILION
STRUCTURAL CALCULATIONS

Beam A - Continued.

Span = 3.2.

End Reaction = DL = 21.52.

Beam B

UDL's

Beam S/W DL = 6.75 kN/m

Dwarf Wall DL = $0.35 \times 0.215 \times 18 kN/m^3 = 1.35$ kN/m

Beam & Block

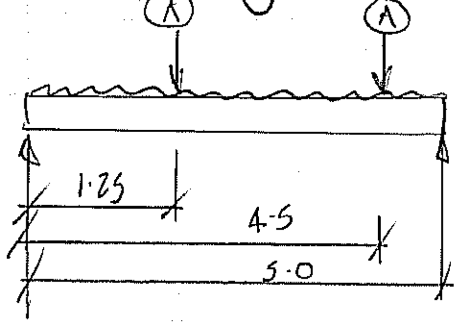
DL = $\frac{7.4m}{2} \times 4.15 kN/m^2$ = 15.36 kN/m

IL = " $\times$ 5.0 = 18.5 kN/m

Wall DL = $3.3m \times (0.1 \times 16 kN/m^3 + 0.36) = 6.47$

Summation $\Sigma DL = 29.93$ kN $\Sigma IL = 18.5$ kN/m

Consider simply supported over zone disturbed by septic tank excavation



M = 249.81 kNm V = 213.4 Ref F3, 4

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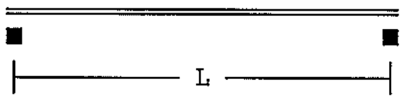
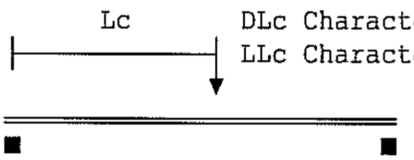
Location: GROUND BEAM B



Elastic modulus	E=28E6 kN/m²
Beam span	L=5 m
Inertia about major axis	I=3.42E-3 m⁴
Dead load factor	gamd=1.4
Live load factor	gami=1.6

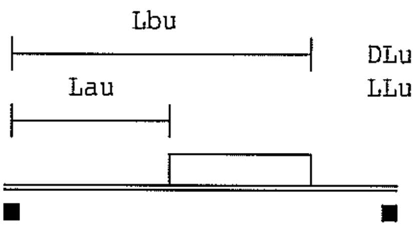
All loads are positive downwards, reactions are positive upwards, sagging moments are positive.



DLc Characteristic dead point load
LLc Characteristic live point load

Distances are measured from left hand support

Distance from left support	Lc(1)=1.25 m
Characteristic dead load	DLc(1)=21.52 kN
Characteristic live load	LLc(1)=0 kN
Distance from left support	Lc(2)=4.5 m
Characteristic dead load	DLc(2)=21.520 kN
Characteristic live load	LLc(2)=0 kN



DLu Characteristic dead UDL
LLu Characteristic live UDL

Distances are measured from left hand support

Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=5 m
Characteristic dead load	DLu(1)=29.93 kN/m
Characteristic live load	LLu(1)=18.5 kN/m

BMs at 20th points, from left to right (sagging is positive)

0	48.857	93.244	133.16	168.61	199.59
	218.57	233.09	243.13	248.7	249.81
	246.44	238.61	226.31	209.54	188.3
	162.59	132.41	97.763	51.116	0

Factored maximum span bending moment 249.81 kNm
(the factors are 1.4 & 1.6)

End shears

Shear force at left hand end	204.36 kN
Shear force at right hand end	213.4 kN
Unfactored dead shear at LHE	93.117 kN
Unfactored live shear at LHE	46.25 kN
Unfactored dead shear at RHE	99.573 kN
Unfactored live shear at RHE	46.25 kN

Midspan deflections

Unfactored dead load deflection	0.0031191 m
Unfactored live load deflection	0.0015722 m
Total dead & live deflections	0.0046913 m
Span:defln ratio for dead load	1603
Span:defln ratio for live load	3180.3
Span:defln ratio for total load	1065.8

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Sheet No	F5
Prepared By	DCB
Date	Sep-19
Checked	

Project: CHEDDAR PARISH PAVILION
STRUCTURAL CALCULATIONS

USE 450 x 450 C30 CONC
5 N° H25 BARS
3 N° H8 LINKS @ 250. Ref F6, 7

Beam (C)

Design to span 3m soft spot.
Reaction from (B) DL = 93.12 kN Ref F3
IL = 46.25

UDL's

Ground Beam s/w DL = 6.75 kN/m
Inner leaf DL = 3.3 x (0.1m x 12 kN/m³ + 0.18)
DL = 4.56 kN/m
Outer leaf DL = 3.3 x 0.12 x 16 kN/m³ = 6.34 kN/m
Roof DL = $\frac{9.9m}{2}$ x 0.55 kN/m² = 2.72 kN/m
IL = " x 0.80 " = 3.96 kN/m

ΣDL = 20.37 ΣIL = 3.96 kN/m
M = 192 kNm V = 154.5 kN Ref F8, 9

USE 600 x 450 C30 CONC Ref F10, 11
5 N° H20 BARS
3 N° H8 LINKS @ 250.

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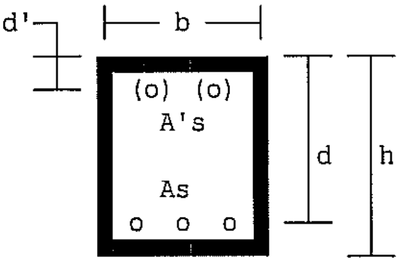
Location: RC GROUND BEAM B

Bending in rectangular beams with optional calculations for shear, lap lengths, bar curtailment & limiting span/effective depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.



Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Ultimate BM before redistribution Mbef=249.81 kNm
Beam being analysed is considered as non-continuous.

Characteristic concrete strength	fcu=30 N/mm^2
Max.aggregate size (for bar spc)	hagg=15 mm
Char.strength of long'l bars	fy=500 N/mm^2
Longitudinal reinforcement is high-yield steel.	
Diameter of tension bars	dia=25 mm
Diameter of link legs	dial=8 mm
Char.strength of link steel	fyv=500 N/mm^2
High-yield steel shear reinforcement.	
Nominal concrete cover	cover=50 mm
Overall depth of section	h=450 mm
Effective depth of section	d=379 mm
Breadth of section	b=450 mm

Longitudinal reinforcement

TENSION	**Characteristic strength**	**500 N/mm^2**
REINFORCEMENT	**Diameter of bars**	**25 mm**
SUMMARY	**Number of bars**	**5**
	arranged in a single layer.	
	Cover to all steel	**50 mm**
	Area of steel required	**1833.4 mm^2**
	Area of steel provided	**2454.4 mm^2**
	Percentage provided	**1.212 %**
	Weight of steel provided	**19.267 kg/m**
	Max.permissible spacing	**188 mm**
	Link size assumed	**8 mm**

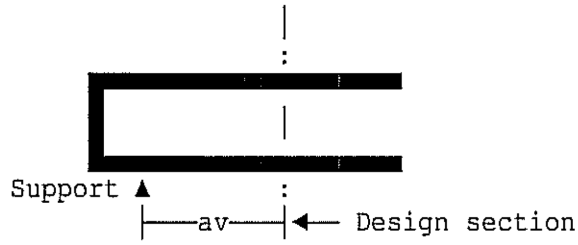
Check on span/effective depth ratio

Basic ratio for simp.-sup.beam	bs'd=20 see Table 3.9
As applied-moment factor	M'bd2=M*1000*1000/(b*d^2)=3.8647 N/mm^2
Mod.factor for tension steel from equation 7 (Table 3.10)	modf1=0.55+(477-fs)/(120*(.9+M'bd2)) =0.94878

```
Mod.factor for no comp.steel       modf2=1
Maximum permissible
span/effective-depth ratio         ps'd=bs'd*modf1*modf2=18.976
Span of beam (see Cls.3.4.1.2-4)   span=5 m
Actual span/effective-depth ratio  as'd=1000*span/d=13.193
As this does not exceed                 18.976,  this is Acceptable.
```

Shear reinforcement

Shear calculations are in
accordance with Clauses 3.4.5
of Code



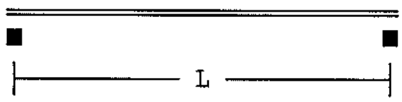

```
Location for shear calculation:    AWAY FROM SUPPORT
Effective breadth for shear        bv=450 mm
Shear force due to ultimate load   V=213.4 kN
Distance from support              av=2000 mm
No.of tension bars effective at section nbars=5
As bv exceeds 350 mm note the conditions in Clause 3.4.5.5:
  i) that no longitudinal bar should be more than 150 mm from a
     vertical leg, and
 ii) (because bv exceeds d), that the transverse spacing of the
     legs must not exceed the effective depth d ( i.e. 379 mm ).
Number of legs to be provided      nlegs=3
Proposed spacing sv exceeds 0.75*d= 0.75*379 =284.25 mm
Reset link spacing to limiting value of 280 mm (rounded).
Chosen link spacing                sv'=250 mm
```

Use 8 mm links (3 legs), spaced at 250 mm ctrs.along beam.

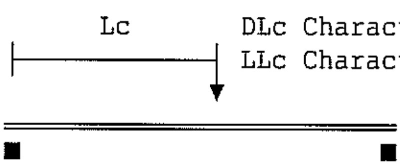
When detailing steel, watch carefully the requirements of Cl.3.4.5.5.

SHEAR REINFORCEMENT SUMMARY	**Characteristic strength**	**500 N/mm²**
	Diameter of links	**8 mm**
	Number of legs	**3**
	Spacing	**250 mm**
	Approx.weight of links	**3.3001 kg/m**

Location: GROUND BEAM C



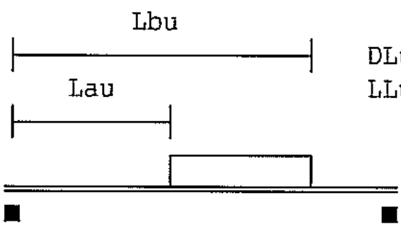
Elastic modulus E=28E6 kN/m²
Beam span L=3 m
Inertia about major axis I=0.00342 m⁴
Dead load factor gamd=1.4
Live load factor gami=1.6
All loads are positive downwards, reactions are positive upwards, sagging moments are positive.



DLc Characteristic dead point load
LLc Characteristic live point load

Distances are measured from left hand support

Distance from left support Lc(1)=1.5 m
Characteristic dead load DLc(1)=93.12 kN
Characteristic live load LLc(1)=46.25 kN



DLu Characteristic dead UDL
LLu Characteristic live UDL

Distances are measured from left hand support

Dist. from left support to start Lau(1)=0 m
Distance from left support to end Lbu(1)=3 m
Characteristic dead load DLu(1)=20.37 kN/m
Characteristic live load LLu(1)=3.96 kN/m

BMs at 20th points, from left to right (sagging is positive)

0	22.778	44.771	65.98	86.405	106.05
	124.9	142.97	160.26	176.77	192.49
	176.77	160.26	142.97	124.9	106.05
	86.405	65.98	44.771	22.778	0

Factored maximum span bending moment 192.49 kNm
(the factors are 1.4 & 1.6)

End shears

Shear force at left hand end 154.46 kN
Shear force at right hand end 154.46 kN

Unfactored dead shear at LHE 77.115 kN
Unfactored live shear at LHE 29.065 kN
Unfactored dead shear at RHE 77.115 kN
Unfactored live shear at RHE 29.065 kN

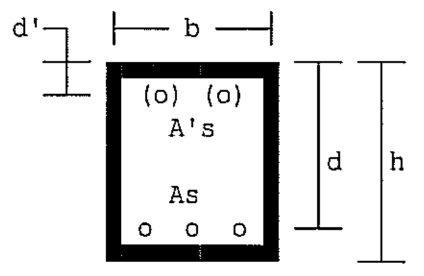
Midspan deflections

Unfactored dead load deflection	0.77134E-3 m
Unfactored live load deflection	0.31529E-3 m
Total dead & live deflections	0.0010866 m
Span:defln ratio for dead load	3889.3
Span:defln ratio for live load	9515
Span:defln ratio for total load	2760.8

Location: RC GROUND BEAM C

Bending in rectangular beams with optional calculations for shear, lap lengths, bar curtailment & limiting span/effective depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.



Design to BS8110(1997) with partial safety factor for steel gammaS=1.15	
Ultimate BM before redistribution	Mbef=192 kNm
Beam being analysed is considered as non-continuous.	
Characteristic concrete strength	fcu=30 N/mm^2
Max.aggregate size (for bar spc)	hagg=15 mm
Char.strength of long'l bars	fy=500 N/mm^2
Longitudinal reinforcement is high-yield steel.	
Diameter of tension bars	dia=20 mm
Diameter of link legs	dial=8 mm
Char.strength of link steel	fyv=500 N/mm^2
High-yield steel shear reinforcement.	
Nominal concrete cover	cover=50 mm
Overall depth of section	h=450 mm
Effective depth of section	d=382 mm
Breadth of section	b=600 mm

Longitudinal reinforcement

TENSION	**Characteristic strength**	**500 N/mm^2**
REINFORCEMENT	**Diameter of bars**	**20 mm**
SUMMARY	**Number of bars**	**5**
	arranged in a single layer.	
	Cover to all steel	**50 mm**
	Area of steel required	**1269.2 mm^2**
	Area of steel provided	**1570.8 mm^2**
	Percentage provided	**0.58178 %**
	Weight of steel provided	**12.331 kg/m**
	Max.permissible spacing	**174 mm**
	Link size assumed	**8 mm**

Check on span/effective depth ratio

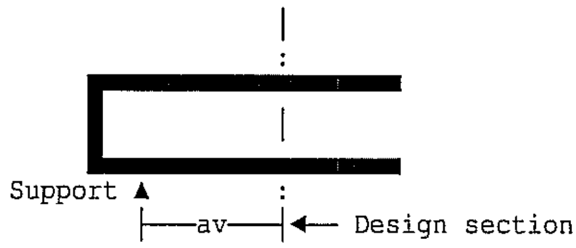
Basic ratio for simp.-sup.beam	bs'd=20 see Table 3.9
As applied-moment factor	M'bd2=M*1000*1000/(b*d^2)=2.1929 N/mm^2
Mod.factor for tension steel from equation 7 (Table 3.10)	modf1=0.55+(477-fs)/(120*(.9+M'bd2)) =1.1095

Mod.factor for no comp.steel	modf2=1
Maximum permissible span/effective-depth ratio	ps'd=bs'd*modf1*modf2=22.191
Span of beam (see Cls.3.4.1.2-4)	span=3 m
Actual span/effective-depth ratio	as'd=1000*span/d=7.8534

As this does not exceed 22.191, this is Acceptable.

Shear reinforcement

Shear calculations are in accordance with Clauses 3.4.5 of Code



Location for shear calculation:	AWAY FROM SUPPORT
Effective breadth for shear	bv=600 mm
Shear force due to ultimate load	V=154.5 kN
Distance from support	av=2000 mm
No.of tension bars effective at section	nbars=5

As bv exceeds 350 mm note the conditions in Clause 3.4.5.5:
  i) that no longitudinal bar should be more than 150 mm from a vertical leg, and
 ii) (because bv exceeds d), that the transverse spacing of the legs must not exceed the effective depth d (i.e. 382 mm).

Number of legs to be provided	nlegs=3
Chosen link spacing	sv'=250 mm

Use 8 mm links (3 legs), spaced at 250 mm ctrs.along beam.

When detailing steel, watch carefully the requirements of Cl.3.4.5.5.

SHEAR REINFORCEMENT SUMMARY

Characteristic strength	**500 N/mm^2**
Diameter of links	**8 mm**
Number of legs	**3**
Spacing	**250 mm**
Approx.weight of links	**3.8919 kg/m**

Bearing Pressure

Consider Beam C spreads load from Beam B over 3m Length.

Total load on C

$= 2 \times 154.46 = 309$ kN.

Bearing Pressure

$= 309 / 3m \times 0.6m = 172$ kN/m²

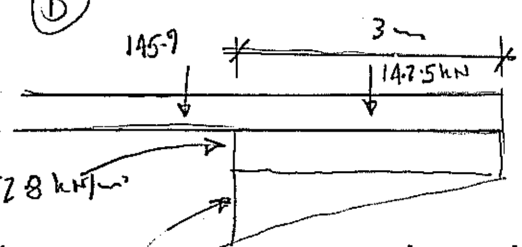
Allow for Overburden = 18 kN/m²

Nett Bearing = 153 kN/m²

Require Firm Clay Cu = 75 kN/m³

Consider this is a conservative estimate of Bearing pressure as Beam C will distribute the load further than 3m and ~~so~~ the full load on Beam B is unlikely to be transfered onto C.

Beam B



$\frac{142.5}{3 \times 0.9m} = 52.8$ kN/m²

$2 \times 145.9 / 3 \times 0.9 = 108$ kN/m²

Consider 900 wide fill

Max Bearing = 160 kN/m² Gross
= 148 Nett.

Beam (D)

UDL as Beam B DL = 29.9 kN/m
IL = 18.5 "

ULS = 71.46 kN/m

Design spanning 3m soft spot.

M = 80.39 kNm V = 107.19

USE 450 x 450 C30
4 No H 16
2 No H8 LINKS

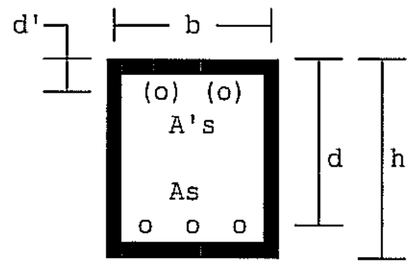
Ref F14, 15

Location: RC GROUND BEAM D

Bending in rectangular beams with optional calculations for shear,

lap lengths, bar curtailment & limiting span/effective depth ratio

Calculations are in accordance with Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block and that the depth to the neutral axis is restricted to 0.5*d.



Design to BS8110(1997) with partial safety factor for steel gammaS=1.15
Ultimate BM before redistribution Mbef=80.39 kNm
Beam being analysed is considered as non-continuous.

Characteristic concrete strength	fcu=30 N/mm²
Max.aggregate size (for bar spc)	hagg=15 mm
Char.strength of long'l bars	fy=500 N/mm²

Longitudinal reinforcement is high-yield steel.

Diameter of tension bars	dia=16 mm
Diameter of link legs	dial=8 mm
Char.strength of link steel	fyv=500 N/mm²

High-yield steel shear reinforcement.

Nominal concrete cover	cover=50 mm
Overall depth of section	h=450 mm
Effective depth of section	d=382 mm
Breadth of section	b=450 mm

Longitudinal reinforcement

TENSION REINFORCEMENT SUMMARY

Characteristic strength	**500 N/mm²**
Diameter of bars	**16 mm**
Number of bars	**4**
arranged in a single layer.	
Cover to all steel	**50 mm**
Area of steel required	**509.5 mm²**
Area of steel provided	**804.25 mm²**
Percentage provided	**0.39716 %**
Weight of steel provided	**6.3133 kg/m**
Max.permissible spacing	**222 mm**
Link size assumed	**8 mm**

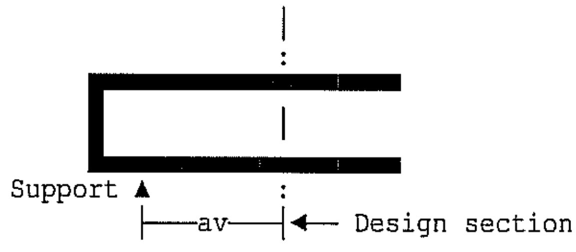
Check on span/effective depth ratio

Basic ratio for simp.-sup.beam bs'd=20 see Table 3.9
As applied-moment factor M'bd2=M*1000*1000/(b*d^2)=1.2242 N/mm²
Mod.factor for tension steel from equation 7 (Table 3.10) modf1=0.55+(477-fs)/(120*(.9+M'bd2))
=1.5928

```
Mod.factor for no comp.steel      modf2=1
Maximum permissible
span/effective-depth ratio        ps'd=bs'd*modf1*modf2=31.857
Span of beam (see Cls.3.4.1.2-4)  span=3 m
Actual span/effective-depth ratio as'd=1000*span/d=7.8534
As this does not exceed                31.857,  this is Acceptable.
```

Shear reinforcement

Shear calculations are in accordance with Clauses 3.4.5 of Code




```
Location for shear calculation:   AWAY FROM SUPPORT
Effective breadth for shear       bv=450 mm
Shear force due to ultimate load  V=107.19 kN
Distance from support             av=2000 mm
No.of tension bars effective at section nbars=4
As bv exceeds 350 mm note the conditions in Clause 3.4.5.5:
  i) that no longitudinal bar should be more than 150 mm from a
     vertical leg, and
 ii) (because bv exceeds d), that the transverse spacing of the
     legs must not exceed the effective depth d ( i.e. 382 mm ).
Number of legs to be provided     nlegs=2
Chosen link spacing               sv'=200 mm
```

Use 8 mm links (two legs), spaced at 200 mm centres along beam.

When detailing steel, watch carefully the requirements of Cl.3.4.5.5.

SHEAR REINFORCEMENT SUMMARY		
	Characteristic strength	**500 N/mm²**
	Diameter of links	**8 mm**
	Number of legs	**2**
	Spacing	**200 mm**
	Approx.weight of links	**2.7935 kg/m**