Ebenezar Devadason – Chartered Engineer
12/29 2nd Street Salt Colony, Egmore, Chennai 600 008
Phone: 044 28194057, Mob: 9444686955

18.05.15

STRUCTURAL STABILITY CERTIFICATE

This is to certify that the PROPOSED CONSTRUCTION OF CBSE SCHOOL FOR DHARAM EDUCATIONAL TRUST IN VANDAVASI-TINDIVANAM ROAD R.S.No. 113/1A,114/8,114/4,114/6B,112/6C,112/6B, KARUVAMBAKKAM VILLAGE AND PANCHAYAT, OLAKUR PANCHAYAT UNION, TINDIVANAM TALUK, VILLUPURAM DISTRICT .has been designed for Ground floor + 2 floors to the following CODE OF PRACTICES.

I.S 875 – 1987 CODE OF PRACTICES for design loads (part 1,2 ,3 and 4)
I.S 456 – 2000 CODE OF PRACTICES for Plain and Reinforced Concrete.
I.S 1786 – 1985 CODE OF PRACTICES specified for high strength deformed steel bars & wires for concrete reinforcement.
I.S 1904 – 1986 CODE OF PRACTICES for design of construction of foundation in soils.
I.S 1893 – 2002 CODE OF PRACTICES for design of earthquake resistant Structures

The building is a RCC framed structure supported on isolated footings. The loads on each floor are transferred through beams and columns arranged to achieve stability to the structure and finally distributed through the isolated foundation to firm soil at 1.83 m below ground level. Plinth Beams are tied with individual Column at natural ground level. Also the building is designed to resist seismic forces due to earth quake as per relevant code.

Hence we certify that the above building is designed to be structurally stable and safe.

Ebenezar Devadason B.E, .M.S MIE C.E

Chartered Engineer

EBENEZAR DEVADASON
B.E.M.S. (Struct) U.S.A. M.I.E. A.S.C.E.
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Project: PROPOSED CONSTRUCTION OF CBSE SCHOOL FOR DHARAM EDUCATIONAL TRUST IN VANDAVASI-TINDIVANAM ROAD R.S.No. 113/1A,114/8,114/4,114/6B,112/6C,112/6B, KARUVAMBAKKAM VILLAGE AND PANCHAYAT, OLAKUR PANCHAYAT UNION, TINDIVANAM TALUK,	Design		ED	
	Date		018.05.15	
	Approved		ED	

C24

				L		B		
	Load Influence Area			4.60	m	5.08	m	
Roof								
Slab	770 x	4.60 x	5.08 =	17993.36 Kg				
Other Beam	288 x	4.60 x	=	1323 Kg				
Other Beam	288 x	5.08 x	=	1461 Kg				
parapet wall	276 x	5.08 x	=	1402 Kg				
Col			=	Kg				
Sub Total			=	**22178 Kg**	x		1 =	**22178 Kg**
first floor								
Slab	770 x	4.60 x	5.08 =	17993.36 Kg				
Other Beam	288 x	4.60 x	0 =	1323 Kg				
Other Beam	288 x	5.08 x	0 =	1461 Kg				
9"wall	1122 x	5.08 x	=	5702 Kg				
Col	1380		1 =	1380 Kg				
Sub Total			=	**27858 Kg**	x		2 =	**55716 Kg**
Gr. Floor								
Slab	x	4.60 x	5.08 =	0 Kg				
Main Beam	288 x	7.38 x	1 =	2122 Kg				
9"wall	1122 x	7.38 x	1 =	8283 Kg				
Col	1380		1	1380 Kg				
Sub Total				**11785 Kg**				
Below GL					x		1 =	**11785 Kg**
Plinth beam	x	9.68 x	=	0 Kg				
9"wall	276 x	7.38 x	=	2037 Kg				
4.5"wall	0 x	0 x	=	0 Kg				
Col	690	1		690 Kg				
Sub Total				**2727 Kg**				
Grand Total								
							1 =	**2726.9 Kg**
				Actual stress on bw			=	**92407 Kg**
						Say		**92400 t**
								93 t

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Dado Lands
Terrace slab

Self wt	0.14 x	2500 x	1 x	1 =	350	Kg/m
Weathering course	0.11 x	2000 x	1 x	1 =	220	Kg/m
Live Load				=	200	Kg/m
Total				=	770	Kg/m

Roof Slab

Self wt	0.14 x	2500 x	1 x	1 =	350	Kg/m
flor finish	0.04 x	2000 x	1 x	1 =	80	Kg/m
Live Load				=	400	Kg/m
Total				=	830	Kg/m

9" wall Typ floor	0.23 x	2000 x	3.1 x	0.8 =	1122
9" wall Gr. floor	0.23 x	2000 x	3.0 x	0.8 =	1325
Parapet wall	0.23 x	2000 x	0.6 x	1 =	276
Main Beam	0.23 x	2500 x	0.5 x	1 =	288
Side Beam	0.23 x	2500 x	0.4 x	1 =	201
column	0.23 x	2500 x	0.6 x	4 =	1380
column	0.23 x	2500 x	0.6 x	4 =	1380
Main Beam	0.23 x	2500 x	0.6 x	1 =	345

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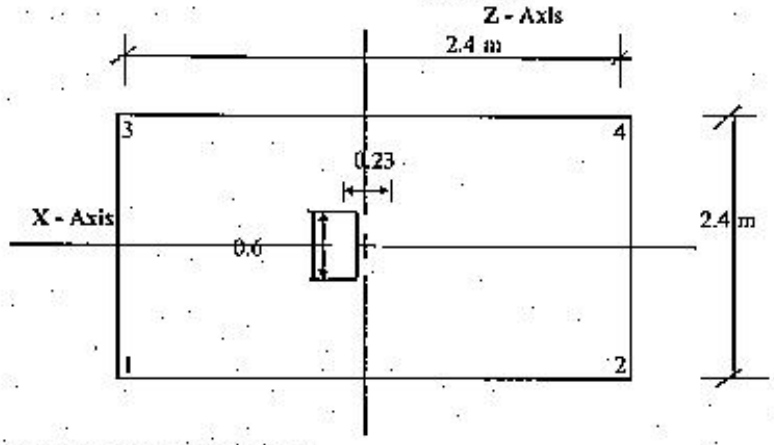
Calculations for				
C24	Divn/Dept	Civil	Job /File No	
	Calculated by	ED	Date 18.03.15	Sheet No.
	Checked by	ED	Date	of

1. DESIGN OF FOOTINGS FOR AXIAL LOADS AND MOMENTS

Data From Staad output

Factored loads

Axial load on the footing	P =	1395 kN	1900X1.5
Moments	Mx =	27.9 kNm	
	Mz =	27.9 kNm	
Shear forces	Fx =	27.9 kN	
	Fz =	27.9 kN	
Load Factor	=	1.5	
Assuming overall depth of footing	D=	650 mm	
Additional moments	Fx * D	=27.9x0.65	
		18.14 kNm	
	Fz * D =	=27.9x0.65	
		18.135 kNm	
Total Moments	Design Mx	=27.9+18.135	
		46.035 kNm	
	Design Mz	=27.9+18.14	
		46.04 kNm	



Mx and Mz are moments about X axis and Y axis of the column

Material Data

fy = 415 N/mm² Density of soil = 18 kN/m³

fck = 20 N/mm²

Bearing capacity of the soil sbc = 250 kN/m² at 2 m below FGL

Gross Bearing capacity of the soil Gsbc = 286 kN/m² at 2 m below FGL

Size of column 0.23 m x 0.6 m

Size of pedestal around the column 0.23 m x 0.6 m

Determination of footing size

The loads considered for determining footing sizes are unfactored service loads and allowable bearing pressure in contrast to the strength design which considers ultimate load and ultimate strength. This is because the safety of the footing is provided by the overall safety factor of 2.5 to 3 applied to the ultimate bearing capacity of the soil to limit the settlement within the tolerable limit.

Service Condition

Axial Load	=1395/1.5	930 kN
Mx	=46.035/1.5	30.69 kNm
Mz	=46.04/1.5	30.69 kNm
Wt.of foundation	=5.76x650/1000x25	
	93.6 kN	
Wt.of soil (Including 0.0 m filling)	=(2.4x2.4-(0.23x0.6))x(2-650/1000)x18	
	136.61 kN	
Total weight	=930+93.6+136.6146	
	1160.21 kN	

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The footing shall be provided such that the pressures below the footing are within sbc
The following combinations shall be checked

Case 1 P/A + Mx/Zx + Mz/Zz < sbc
Assuming B = 2.4 m L = 2.4 m
Zz = L2*B/6
Zx = B2*L/6
A = B*L = 5.76 m²
Base Pr = =1160.2146/5.76+30.69x6/5.76x2.4+30.69x6/5.76x2.4
228.07 kN/m²
Length = 2.4 m

Case 2 P/A + Mx/Zx - Mz/Zz < sbc
Assuming B = 2.4 m L = 2.4 m
Zz = L2*B/6
Zx = B2*L/6
A = B*L = 5.76 m²
Base Pr = =1160.2146/5.76+30.69x6/5.76x2.4-30.69x6/5.76x2.4
201.43 kN/m²
Length = 2.4 m

Case 3 P/A - Mx/Zx + Mz/Zz < sbc
Assuming B = 2.4 m L = 2.4 m
Zz = L2*B/6
Zx = B2*L/6
A = B*L = 5.76 m²
Base Pr = =1160.2146/5.76-30.69x6/5.76x2.4+30.69x6/5.76x2.4
201.43 kN/m²
Length = 2.4 m

Case 4 P/A - Mx/Zx - Mz/Zz < sbc
Assuming B = 2.4 m L = 2.4 m
Zz = L2*B/6
Zx = B2*L/6
A = B*L = 5.76 m²
Base Pr = =1160.2146/5.76-30.69x6/5.76x2.4-30.69x6/5.76x2.4
174.79 kN/m²
Length = 2.4 m

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Check for GSBC

	Case 1	Case 2	Case3	Case4	Max	
Base Pr =	228.07	201.43	201.43	174.79	228.07 kN/m²	
Length =	2.4	2.4	2.4	2.4	< than GSBC, Hence Safe	

Check for SBC

	Case 1	Case 2	Case3	Case4	Max	
Base Pr =	188.1	161.46	161.46	134.82	188.1 kN/m²	
					< than SBC Hence Safe	

Provide footing of size L = 2.4 m
B = 2.4 m

Area of footing provided = 5.76 m²

Check for Eccentricity

Eccentricity in X-direction ex = Mx/P
0.026 m No tension , e< B / 6 = 0.4

Eccentricity in Z-direction ez = Mz/P
0.026 m No tension ,< B / 6 = 0.4

CALCULATION OF PRESSURES ON THE SOIL

p1 =1395/5.76+(46.035x6/(2.4x2.4^2))+(46.04x6/(2.4x2.4^2)) 282.15 kN/m²
p2 =1395/5.76+(46.035x6/(2.4x2.4^2))-(46.04x6/(2.4x2.4^2)) 242.19 kN/m²
p3 =1395/5.76-(46.035x6/(2.4x2.4^2))+(46.04x6/(2.4x2.4^2)) 242.19 kN/m²
p4 =1395/5.76-(46.035x6/(2.4x2.4^2))-(46.04x6/(2.4x2.4^2)) 202.22 kN/m²

ALONG X

Pmax = =(282.15+242.19)/2 262.17 kN/m²
Pmin = =(242.19+202.22)/2 222.21 kN/m²

ALONG Z

Pmax = =(282.15+242.19)/2 262.17 kN/m²

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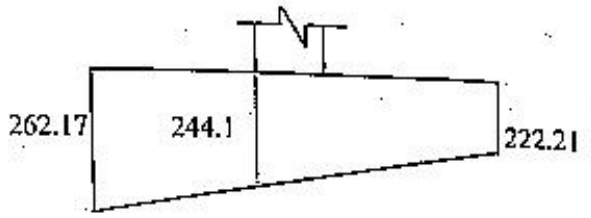
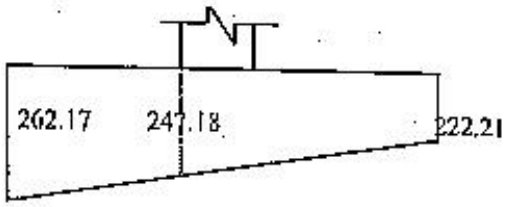
Pmin = =(242.19+202.22)/2 222.21 kN/m²

PRESSURE ALONG X-AXIS



PRESSURE ALONG Z-AXIS



CALCULATION OF BENDING MOMENTS AT FACE OF PEDESTAL

ALONG X

Determination of bending moments at the face of pedestal

Cantilever projection along longer face (1-1) = a =(2.4-0.23)/2) = 1.085 m

Pressure at face of pedestal =222.205+((262.17-222.205)/2.4)x(2.4-1.085))

244.1 kN/m²

Maximum Bending moment at face of pedestal = =[(244.1x1.085^2)/2+((262.17-244.1)x0.5x1.085x2/3x1.085)x2.4

361.85 kNm

Thickness of footing required d = Sqrt{Mu/Qb} =SQRT(361.85x10^6)/(0.138x20x2.4x1000)

233.72 mm

Provide overall depth D = 650 mm

clear cover = 50 mm

Eff. depth d = 650-50-12/2 594 mm

Mu1/bd²= 361.85x10^6/(2.4x1000x594^2) 0.43

From Table 2 - SP16 page 48 for fy=415N/mm² pt = 0.122%

ALONG Z

Determination of bending moments at the face of pedestal

Cantilever projection along shorter face (2-2) = b =(2.4-0.6)/2 = 0.900 m

Pressure at face of pedestal = =222.205+((262.17-222.205)/2.4)x(2.4-0.9)

247.18 kN/m²

Maximum Bending moment at face of pedestal = =[(247.18x0.9^2/2)+((262.17-247.18)x0.5x0.9x2/3x0.9)x2.4]

249.97 kNm

Thickness of footing required d = Sqrt{Mu/Qb} =SQRT(249.97x10^6/(0.138x20x2.4x1000))

194.26 mm

Provide overall depth D = 650 mm

clear cover 50 mm

Eff. depth d = 650-50-12-12/2 582 mm

Mu1/bd²= 249.97x10^6/(2.4x1000x582^2) 0.31

From Table 2 - SP16 page 48 for fy=415N/mm² pt = 0.087%

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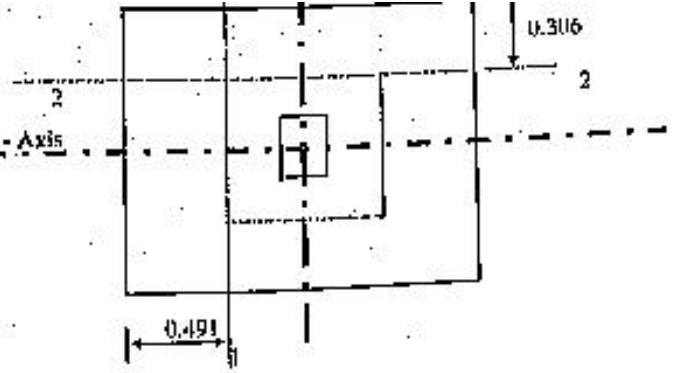
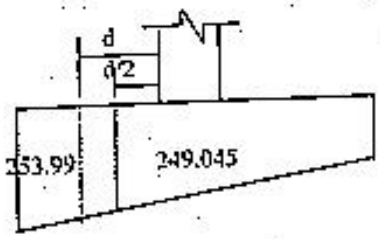
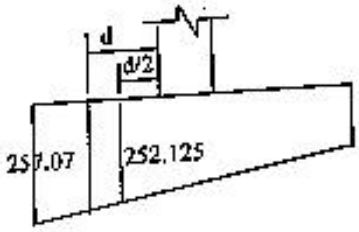
Check for one way



PRESSURE ALONG X-AXIS



PRESSURE ALONG X-AXIS



Along X direction For 1-1 x1 = a - d = 1.085-594/1000 0.491 m
Along Y direction For 2-2 x2 = b - d = 0.9-594/1000 0.306 m

Pressure at 1-1 = =222.205+(262.17-222.205)/2.4x(2.4-0.491)
= 253.99 kN/m²

Pressure at 2-2 = =222.205+(262.17-222.205)/2.4x(2.4-0.306)
257.07 kN/m²

Maximum SF 1-1 = p * B * x =253.99x2.4x0.491+((0.5 x 0.491x (262.17-253.99)x2
= 304.12 kN

Maximum SF 2-2 = p * L * x2 =(257.07x2.4x0.306)+ (0.5x0.306(262.17-257.07)x2.4)
= 190.66 kN

Effective depth at the critical section d 1-1 = 594 mm
Effective breadth at the critical section b 1-1 = 2400 mm
Effective depth at the critical section at d 2-2 = 582 mm
Effective breadth at the critical section b 2-2 = 2400 mm

Maximum shear at 1-1 SF1-1 = 304.12 kN Vu = 304.12 kN
Tv 1-1= 304.12x10^3/(594x2400) 0.2 N/mm²
Provided reinf pt = 0.190% for which the shear strength of concrete from
TABLE 61 - SP16, page 178, Tv' = 0.319 N/mm² > Tv1-1 hence safe

Maximum shear at 2-2 SF2-2 = 190.66 kN Vu = 190.66 kN
Tv 2-2= 190.66x10^3/(582x2400) 0.1 N/mm²
Provided reinf pt = 0.194% for which the shear strength of concrete from
TABLE 61 - SP16, page 178, Tv' = 0.322 N/mm² > Tv2-2 hence safe

Check for Punching shear
The critical section for shear occurs at a distance of d/2 from the face of the pedestal
d/2 = 297 mm

Pressure at d/2 from pedestal in X direction = (244.1+253.99)/2 249.045 kN/m²
Pressure at d/2 from pedestal in Z direction = (247.18+257.07)/2 252.125 kN/m²
Shear area b1= 0.824 m d1 = 1.194 m
Perimeter of punching area = (0.824+1.194)x2
= 4.036 m
Punching shear = 1395-[(0.824x1.194)x(249.045+252.125)x0.5]
= 1148.46 kN
Depth at the section d1 = 594 mm
τv = 1148.46x1000/(4.036x1000x594)
τv = 0.48 N/mm²

The shear capacity of the section is ks x τc.
ks = 0.5 + βc

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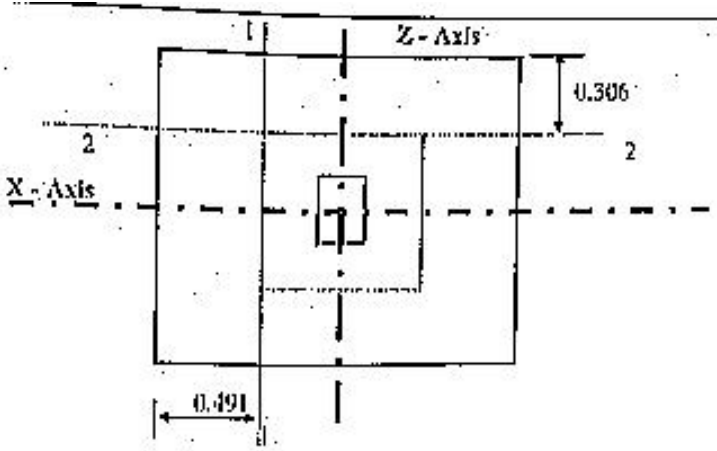
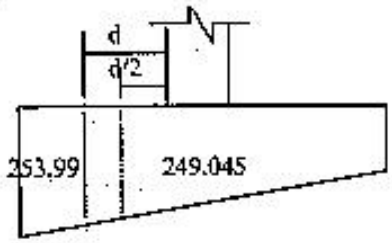
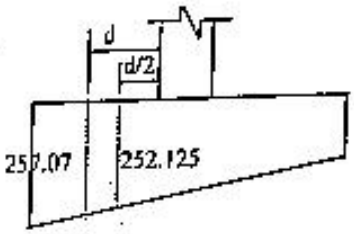
Check for one way shear



PRESSURE ALONG X-AXIS



PRESSURE ALONG X-AXIS



Along X direction — For 1-1 — x1 = p - d = 1.085-594/1000 — 0.491 m

Along Y direction — For 2-2 — x2 = b - d = 0.9-594/1000 — 0.306 m

Pressure at 1-1 = =222.205+(262.17-222.205)/2.4x(2.4-0.491)
= 253.99 kN/m²

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257.07 kN/m²

Maximum SF 1-1 =p * B * x =253.99x2.4x0.491+((0.5 x 0.491x (262.17-253.99)x2
= 304.12 kN

Maximum SF 2-2 = p * L * x2=(257.07x2.4x0.306)+(0.5x0.306(262.17-257.07)x2.4)
= 190.66 kN

Effective depth at the critical section d 1-1 = 594 mm

Effective breadth at the critical section b 1-1 = 2400 mm

Effective depth at the critical section at d 2-2 = 582 mm

Effective breadth at the critical section b 2-2 = 2400 mm

Maximum shear at 1-1 — SF1-1 = 304.12 kN — Vu = 304.12 kN

Tv 1-1= 304.12x10^3/(594x2400) — 0.2 N/mm²

Provided reinf pt = 0.190% for which the shear strength of concrete from

TABLE 61 - SP16, page 178, Tv' = 0.319 N/mm² > Tv1-1 hence safe

Maximum shear at 2-2 — SF2-2 = 190.66 kN — Vu = 190.66 kN

Tv 2-2= 190.66x10^3/(582x2400) — 0.1 N/mm²

Provided reinf pt = 0.194% for which the shear strength of concrete from

TABLE 61 - SP16, page 178, Tv' = 0.322 N/mm² > Tv2-2 hence safe

Check for Punching shear

The critical section for shear occurs at a distance of d/2 from the face of the pedestal

d/2 = 297 mm

Pressure at d/2 from pedestal in X direction = (244.1+253.99)/2 — 249.045 kN/m²

Pressure at d/2 from pedestal in Z direction = (247.18+257.07)/2 — 252.125 kN/m²

Shear area b1= 0.824 m — d1 = 1.194 m

Perimeter of punching area = (0.824+1.194)x2
= 4.036 m

Punching shear = 1395-[(0.824x1.194)x(249.045+252.125)x0.5]
= 1148.46 kN

Depth at the section d1 = 594 mm

τv = 1148.46x1000/(4.036x1000x594)

τv = 0.48 N/mm²

The shear capacity of the section is ks x τc

ks = 0.5 + βc

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	βc	0.383333333
	ks =	0.883333333
	Allowable, ks =	0.883333333 N/mm²
τc = 0.25 x Sqrt (fck)	0.25xsqrt(20)	1.12 N/mm²

The shear capacity of the section

Tv allow = 0.989333333 N/mm²

Tv1-1 < Tv hence okay

CHECK FOR SLIDING

Horizontal force	=	18.6 kN	
Vertical load	=	1160.21 kN	
F.O.S.	=	21.83	Safe

CHECK FOR OVERTURNING

Total moment about x-axis	=	30.69 kNm	
Resisting moment	=	1392.26 kNm	
F.O.S.	=	45.37	Safe
Total moment about x-axis	=	30.69 kNm	
Resisting moment	=	1392.26 kNm	
F.O.S.	=	45.36	Safe

Design of the section 2.4 x 2.4 x 0.65

Longer direction pt = 0.122%

Ast = pt bd/100 = 725.81 mm²

Spacing of 12 dia bars required is = 156 mm

Provide 12 dia bars @ 100 mm spacing (# 1131 mm²)

pt provided= 0.1904%

Shorter direction Min pt = 0.120%

Ast = pt bd/100 = 712.8 mm² 780

Spacing of 12 dia bars required is = 159 mm

Provide 12 dia bars @ 100 mm spacing (# 1131 mm²)

pt provided= 0.1943%

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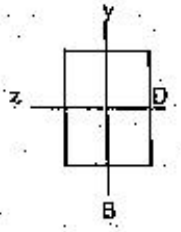
COLUMN DESIGN-C24

	fck	=	25	N/mm²
Size of column :				
Depth of column	B	=	230	mm
Breadth of column	D	=	530	mm
Effective length factor	ky	=	0.85	
	kz	=	0.85	
Unsupported length	Ly	=	3050	mm
	Lz	=	3050	mm
Eff. length about y - direction	ley	=	2593	mm
Eff. length about z - direction	lez	=	2593	mm
Load :				
Axial load	Pu	=	1395.00	kN
Moment about y direction	Muy	=	41.85	kN.m
Moment about z direction	Muz	=	41.85	kN.m
Percentage of reinforcement :				
	pty	=	2.6	%
	ptz	=	2.6	%
cross sectional area	Ag	=	121900	mm²
Area of reinforcement	Asty	=	3169	mm²
	Astz	=	3169	mm²



Provide 4 -25 +4-20

Additional moment due to slenderness :

	ley/B	=	11.27	Short column
	lez/D	=	4.89	Short column
Eccentricity	e_{ey}	=	(B/2000)*(ley/B)²	
		=	15 mm	< emin = 20mm
	e_{ez}	=	(D/2000)*(lez/D)²	
		=	6 mm	< emin = 20mm
	May	=	Pu*e_y	
		=	20.38 kN.m	
	Maz	=	Pu*e_z	
		=	8.85 kN.m	

Calculation of reduction factor :

	Puy	=	0.45*fck*(Ag-Asty)+0.75*fy*Asty				
		=	2322.20 kN				
	Puz	=	0.45*fck*(Ag-Astz)+0.75*fy*Astz				
		=	2322.20				
	d'/B	=	(40+12.5)/230		d'/D	=	(40+12.5)/530
		=	0.23			=	0.10
From table 60 of SP 16,							
	k1y	=	0.18	k2y	=	-0.07	
	k1z	=	0.21	k2z	=	0.33	
	Pby	=	(k1y+k2y*p/fck)fck*b*D				
		=	517.59 kN				
	Pbz	=	(k1z+k2z*p/fck)fck*b*D				
		=	736.05 kN				
	kry	=	Puy-Pu / Puy-Pby	=	0.51	(lesser or equal to 1)	
	krz	=	Puz-Pu	=	0.58	(lesser or equal to 1)	

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Puz-Pbz

May	=	kry*May	=	10.47 kN.m	
Maz	=	krz*Maz	=	5.17 kN.m	

Additional moment due to eccentricity :

Eccentricity	e_y	=	ly/500+B/30	
		=	14 mm	< emin = 20mm
	e_z	=	lz/500+D/30	
		=	24 mm	> emin = 20mm
	Maey	=	$Pu*e_y$	
		=	19.20 kN.m	
	Maez	=	$Pu*e_z$	
		=	33.15 kN.m	

Total moments for which the column is to be designed are :

Muy	=	Muy + Maey	=	61.05 kN
Muz	=	Muz + Maez	=	75.00 kN

Moment capacity of section

Pu / fckBD	=	0.458				
	d'/B	=	(40+12.5)/230	d'/D	=	(40+12.5)/530
		=	0.23		=	0.10
Refer chart43	pty/fck	=	0.104	ptz/fck	=	0.104
Mucy	=	0.105*fck*DB2		Mucz	=	0.105*fck*BD2
	=	73.6 kN.m			=	169.59 kN.m
Pu/Puy	=	0.601		Pu/Puz	=	0.60
αy	=	1.668		αz	=	1.6679

Interaction factor	=	$(Muy/Mucy)^{\alpha y}+(Muy/Mucy)^{\alpha z}$	(lesser or equal to 1)
	=	0.989	Safe

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Ebenezar Devadason - Structural Consultant

Project: PROPOSED CONSTRUCTION OF CBSE SCHOOL FOR DHARAM EDUCATIONAL TRUST IN VANDAVASI-TINDIVANAM ROAD R.S.No. 113/1A,114/3,114/4,114/6B,112/6C,112/6B, KARUVAMBAKKAM VILLAGE AND PANCHAYAT, OLAKUR PANCHAYAT UNION, TINDIVANAM TALUK, VILLUPURAM DISTRICT

Designed by : EBENEZAR DEVADASON
Date : 19.05.15

BEAM DESIGN

Design Information			
1	Reinforced Concrete		
	Fck	M20 FOR FOOTING BEAMS AND M25 FOR COLUMN	
	Fy	415N/mm²	
2	Design codes		
i	IS 456 - 2000	Plain and reinforced concrete code of practice	
ii	IS 875 - 1987	Parts I to IV for Assesment of loads	
iii	I.S 1786 – 1985	Code of practices specified for high strength deformed steel bars & wires for concrete reinforcement.	
iv	IS 1904 - 1985	Code of practices specified for Design and construction of foundation in soils	
v	IS 1893 - 1975 & IS4326 - 1976 with latest addendum	Explanatory Handbook on codes for Earth quake Engineering	
	Design of Beam B1 Span = 5.74m		
	Provide Beam size 300 x 600		
	Load		
	Self Wt	= 250	Kg
	Slab	= 2800	Kg
	Slab	= 2800	Kg
	Total	= 5850	Kg
	Say	= 5850	Kg
	Moment = W*L²/10	= 29205	Kg-m
	Ast = M*100/2300*.87*d	= 21.783851	sq.cm
	Provide5 -Y25 Bottom and 3 - Y25 Top		
	Shear = W*L/2	= 16760.25	Kg
	M.n Shear cap = 2*30*50	= 3600	Kg
	Net Shear	= 13160.25	
	Provide Y10@125c /c		

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Design of Two way slab	Divn / Dept. CIVIL	Job Nr. / File No.	
	Calculated by BK	Date 27.03.12	Sheet No.
	Checked by	Date	of

LOCATION DATA — REFERENCE

Shorter span , Lx = 3.6 M or 3600 mm

Longer span, Ly = 6.1 M or 6100 mm

cover = 20 mm

Concrete grade ,fck = 20 N/mm²

Steel grade, Fe = 415 N/mm²

DESIGN

1. Type of slab

Ly / Lx = 6.1 / 3.6 1.69 <2hence design as a two way slab

2. Thickness of slab and durability consideration

Assume span/ depth ratio = 26

Effective depth = 3600 / 26 — IS 456 23.1

= 138 mm

Assume h = 140 mm 0.14 m

Diameter of rod 10 mm

Actual d (140 - 5 - 20)

= 115 mm

Span = effective span + d

= 3600+115 = 3715 mm or 3.715 m

Actual span dept = 31

3. Design load

Dead load	0.14 x	25 =	3.2 kN/m²
Live load		=	4 kN/m²
Weathering course		=	0 kN/m²
Finishing		=	0.8 kN/m²
Total load		=	8 kN/m²

Design load 8 x 1.5

12 kN/m²

4. Ultimate moments and shears

BM at centre. $M = \frac{WL^2}{12}$

$= \frac{12 \times 3.6 \times 3.6}{12}$

= 12.96 kNm

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Design of Two way slab	Div.n / Dept. CIVIL	Job Nr. / File No.	
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BM at support

$$= \frac{WL2}{12}$$

$$= 12.96 \text{ kNm}$$

5. Check depth for bending — SP 16 Table C

For Fe 415 steel, M = $0.138 f_{ck} bd^2$

$$d = \sqrt{\frac{12.96 \times 1000000}{0.138 \times 20 \times 1000}}$$

$$= 68.52 \text{ mm}$$ < **Effective depth 115mm**

6. Calculation of steel areas

At centre

$$\frac{Mu}{bd2} = \frac{12.96 \times 1000000}{1000 \times 115 \times 115}$$

$$= 0.98$$

$$= 0.29\%$$

Ast required $= 0.0029 \times 1000 \times 115$

$$= 333.5 \text{ mm}^2$$

At support

$$\frac{Mu}{bd2} = \frac{12.96 \times 1000000}{1000 \times 115 \times 115}$$

$$= 0.98$$

$$= 0.29\%$$

Ast required $= 0.0029 \times 1000 \times 115$

$$= 333.5 \text{ mm}^2$$

7. Main steel

Provide [10] mm dia. bars
Spacing 236 mm c/c

Provide 10 mm diameter at [150] mm c/c

Area of steel provided = 524 mm²

Percentage of steel provided $= \frac{524 \times 100}{1000 \times 115}$

$$= 0.46\ \%$$

Negative reinforcement =

Provide [10] mm dia. bars
Spacing 236

Provide 10 mm diameter at [150] mm c/c

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9. Check for control of cracks

Minimum % = 0.12%

As = 0.0012x1000x140

= 168 mm^2

Diameter used = 10 mm — IS 456 25.5.2.2

d = 140 / 8

= 17.5 — **10<17.5 Hence dia. is O.K**

Maximum spaci = 3d

= 345 mm — **150<345 Hence spacing is O.K**

10. Check for deflection

Basic span depth = 26

fs = 0.58fy $\frac{\text{Area of cross section of steel required}}{\text{Area of cross section of steel provided}}$ — IS 456 FIG.3

= 0.58 x 415 x 333.5 / 524

= 153.19 N/mm^2

fi for Pt =0.46 = 2

Allowable L/d = 2x26 = 52

Assumed L/d = 31

= **Hence safe in Deflection**

11. Provide secondary steel

As = 0.12 x (b x h) / 100

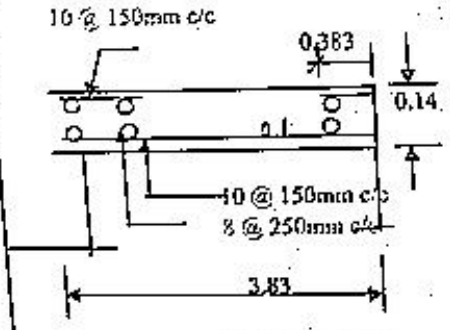
= 0.12 x 1000 x 140 / 100

= 168 mm^2

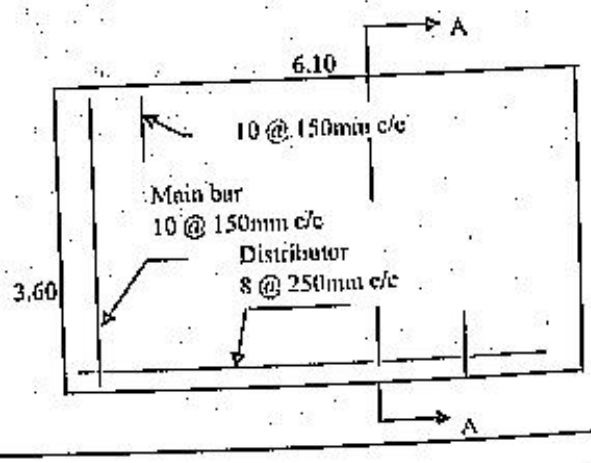
Provide 8 mm dia. bars

Spacing 299 mm c/c

Provide 8 mm diameter at 250 mm c/c



SECTION - AA



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slab design

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Design of Two way slab

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Sheet No. of

9. Check for control of cracks

Minimum % = 0.12%

As = 0.0012x1000x140

= 168 mm²

Diameter used = 10 mm

d = 140

= 8

Maximum spaci = 3d = 17.5 — 10<17.5 Hence dia. is O.K

= 345 mm — 150<345 Hence spacing is O.K

IS 456 25.5.2.1

10. Check for deflection

Basic span depth = 26

fs = 0.58fy

$= 0.58 \times 415 \times \frac{333.5}{524}$ (Area of cross section of steel required / Area of cross section of steel provided)

= 153.19 N/mm²

fl for Pt =0.46 = 2

Allowable L/d = 2x26 = 52

Assumed L/d = 31

= Hence safe in Deflection

IS 456 FIG3

11. Provide secondary steel

As $= \frac{0.12 \times (b \times h)}{100}$

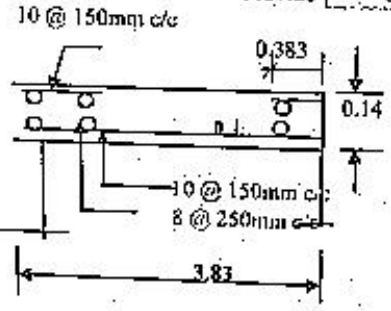
$= \frac{0.12 \times 1000 \times 140}{100}$

= 168 mm²

Provide 8 mm dia. bars.

Spacing 299 mm c/c

Provide 8 mm diameter at 250 mm c/c



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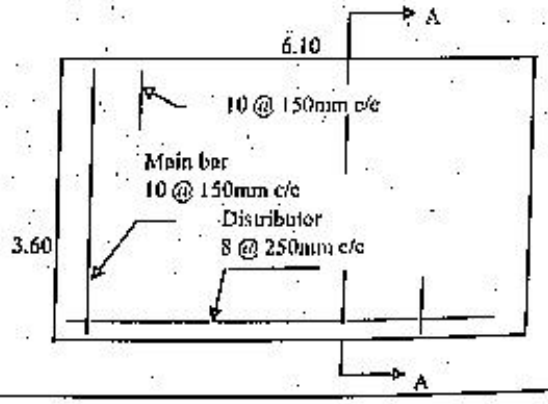

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By virtue of Professional training, experience and Corporate Membership of this Institution

DEVADASON EBENEZAR

is hereby authorised to use the style and title of

Chartered Engineer [India]

Dated this Twenty Fifth day of March 2004

Secretary and Director General