Submitted sir,

Sub: RWS&S-TDWSP- Wagaithanda 90KL OHBR (25mtr) in Utnoor Mandal– Komarambheem Asifabad Segment-Adilabad District-Designs -Approval-Reg.

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Kindly pursue the Designs of the following 90KLOHBR at Wagaithanda (V), Utnoor (M), submitted by the Executive Engineer TDWSP Asifabad Division, Adilabad district for approval.

1. 90 KL OHBR.

The Executive Engineer TDWSP Asifabad Division has submitted Structural Designs & Drawings of 90KL OHBR based on the field conditions and as per the estimate provisions , the structural designs & drawings for the above structure is verified and submitted for approval.

The following design parameters were considered:

- Capacity : 90kL
- Net SBC of Soil : 15.0 t/sqm
- Grade of concrete & Steel : M 30 & Fe 500
- Height of staging : 25 mts
- Dia of Shaft Inner to Inner :6.15 mts
- Dia of Tank Inner to Inner :6.15 mts
- Thickness of shaft :250mm
- Top Slab thickness: 125mm
- Bottom Slab thickness : 250 mm
- Raft Slab thickness: 650mm
- Depth of Foundation : 3.00 mts

As per the above parameters the structural design and drawings of the OHBR is verified, duly following IS codes, IS: 456-2000, SP: 16, 34, IS:3370 and IS 1893-2002 (seismic codes).The sizes and steel proposed in the designs and drawings of all components are safe and sufficient.

The additional points noted after checking the designs are:

- Detailed Estimate of the Structure with these specifications has to be prepared and compared with the provision made in sanctioned estimate. Such that deviation if any is within authorized limits. If any deviations noticed, the Estimate should be submitted for obtaining approval from the Competent Authority.

Subject to approval a draft memo addressed to the EE, TDWSP Asifabad Division , for communicating approved Structure is put up for kind perusal and approval.

30/4/16

AEE (Designs)	DEE (Designs)	Superintending Engineer,
TDWSP,Nirmal Circle	TDWSP,Nirmal Circle	TDWSP,Nirmal Circle

REV. NO.	DESCRIPTION	DESIGNED	CHECKED	APPROVED
A	For Approval	24.03.16 AKHB	30.03.16 RR	31.03.16 BRJ

REVISIONS

LARSEN & TOUBRO LIMITED
CONSTRUCTION DIVISION
Water,Smart World & Communication IC

CLIENT:	CONSULTANT :
TELANGANA DRINKING WATER SUPPLY PROJECT, GOVERNMENT OF TELANGANA	

PROJECT :	**Providing drinking water to habitations in Komarambheem-Asifabad Segment in Adilabad District**
SUPPLIER / CONTRACTOR	**L&T CONSTRUCTION** Water & Effluent Treatment SBG

JOB Ref. No. : LE150883

	NAME	SIGN	DATE
DSGN	AKHB		24.03.16
CHKD	RR		30.03.16
APPD	BRJ		31.03.16

TITLE :

90 KL capacity OHBR -25m staging height - Design Calculations

DOC./DRG. No.

LE150883-C-WS-CW-DC-3023

SIZE: A4 | REV.: A

RELEASED FOR: ☐ PRELIMINARY ☐ INFORMATION ☑ APPROVAL ☐ CONSTRUCTION

LARSEN & TOUBRO LIMITED
ECC Division – EDRC

PROJECT:	Providing drinking water to habitations in Komarambheem-Asifabad Segment in Adilabad District	DOCUMENT NO. LE150883-C-WS-CW-DC-3023		DATE 24/03/2016
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Design of Over head Reservior

(1)	**DATA:**			
	Capacity of Tank		90	m^3
	Unit weight of RCC=		25	kN/m^3
	Unit weight of PCC=		24	kN/m^3
	Unit weight of soil =		18	kN/m^3
	Unit weight of sand filling inside bottom of shaft =		18	kN/m^3
	Unit weight of water=		10	kN/m^3
	Staging Height		25	m
	Net S.B.C of Soil =		150	kN/m^2
(2)	**PERMISSIBLE STRESS:**			
	Grade of concrete;	f_{ck} =	M30	N/mm^2
	Grade of steel;	f_y =	Fe500	N/mm^2
Ref Table 1 of IS:3370	Allowable stress as per IS:3370 relating to resistance to cracking			
	Allowable direct tensile stress in concrete	σ_{at} =	1.5	N/mm^2
	Allowable bending tensile stress in concrete	σ_{bt} =	2.0	N/mm^2
Ref Table 4 of IS:3370	Allowable stress in steel under direct tension, bending & shear = σ_{st} =		130	N/mm^2
	Allowable stress in steel under direct compression = σ_{sc}=		140	N/mm^2
		σ_{st2} =	150	N/mm^2
IS 456:200	Allowable stress in steel under direct tension, bending & shear = σ_{st} =		230	N/mm^2
	Allowable stresses as per IS:456 for strength calculations			
Ref Table 21 of IS:456	Allowable direct compressive stress in concrete	σ_{cc} =	8	N/mm^2
	Allowable bending compressive stress in concrete	σ_{cbc} =	10	N/mm^2
	Modular ratio = $m = \frac{280}{3\sigma_{cbc}} =$	m =	9.33	
	Neutral axis co-efficient; $n = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} =$	n =	0.42	
	Lever arm coefficient; j = 1-n/3 =	j =	0.86	
	Moment coefficient = K=0.5 x σ_{cbc} x (n x j) =		1.81	N/mm^2
(3)	**Volume calculation**			
	Diameter of tank, D =		6.40	m
	Rise of Top Dome, h = =D/5 =6.4/5		1.30	m

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	Diameter of supporting shaft = D =		6.40 m
	Rise of bottom dome , h = =D/5	=6.4/5	1.30 m
	Height of water column in cylindtical portion of tank, H =		4.10 m
	Free board, F.B =		0.30 m
	Total Height of tank wall = H+FB-(1.8-h)	=4.1+0.3+(1.8-1.3)	4.90 m
	C/C Diameter of internal shaft		1.20 m
	Outer Diameter of Internal shaft = (Dia+thk of wall)=	1.2+0.2	1.40 m
	Radius of Inner Shaft =	=1.2/2	0.60 m
	Total height of Internal shaft = H-h+FB=	=4.1-1.3+0.3	3.10 m
	Inner diameter of the tank = D-shaft thk+(wall thk/2)	=6.4-0.25+(0.25/2)	6.15 m
	Volume of Cylindrical portion =V_1= (π/4)×(inner dia)^2×H =	(π/4)×(6.15)^2×4.1	121.79 m^3
	Radius of curvature of bottom dome = R =[(D/2)^2+h^2]/(2h)		
		=[(6.4/2)^2+1.3^2)]/(2×1.3)	4.59 m
	Volume of bottom dome =V_2= (π/3)×(r^2×(3R-h))		
		=(π/3)×(1.3^2×(3×4.59-1.3)	22.07 m^3
	Volume of internal shaft =V_3= (π/4)×(dia^2× (H-h))		
		=(π/4)×[1.4^2×(4.1-1.3)]	4.31 m^3
	Total volume of tank without free board = V_1-V_2-V_3	=121.79-22.07-4.31	95.41 m^3
			OK
	Total volume of tank with free board =		103.86 m^3
(4)	**Design of Top dome:**		

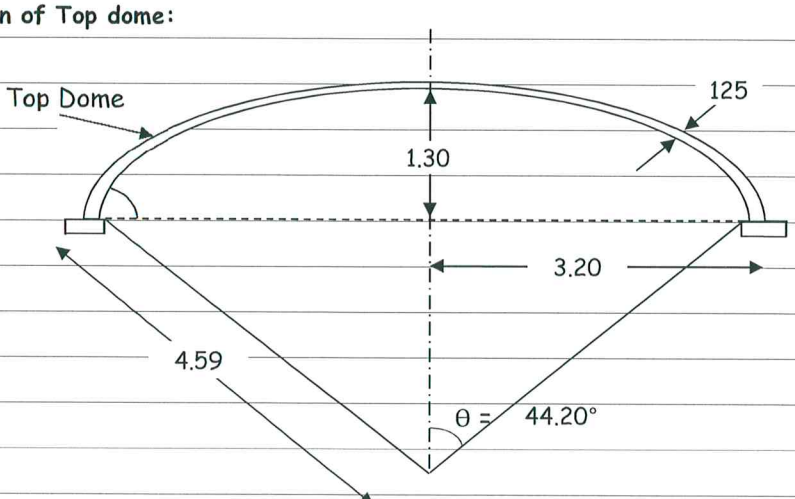

Figure 2: Top Dome.

	Radius of the chord, r = 6.4/2	3.20 m

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	Rise of the top dome, h =	1.30	m
	Radius of the shell surface = $(r^2 + h^2)/2h$ = (3.2^2+1.3^2)/(2*1.3)	4.59	m
	Semi-central angle is given by		
	Sin θ = r_3/R = 0.70 that is, θ =	44.20°	
	=	0.771	rad
	Thicknes of the dome =	125	mm
	Self weight of dome (wg) = 0.125 X 25	3.125	kN/m^2
	Live load w_l =	1.50	kN/m^2
	Total load, w = = 1.5 + 3.125 =	4.63	kN/m^2
	Weight of the dome = $2\pi Rhw_g$ = 2π x 4.59 x 1.3 x 3.125 =	117.16	kN
	Live load on the dome = $2\pi Rhw_l$ = 2π x 4.59 x 1.3 x 1.5 =	56.24	kN
	Total load on top dome = 117.16 + 56.24 =	173.40	kN
	Meridional thrust = N_o = (wR)/(1+cos θ) =	12.36	kN/m
	Meridional Stress = 0.01236/0.125 =	0.10	MPa
	0.1 < 1.5 (OK)		
	As the stress is only nominal, provide the min. reinforcement of	0.24	%
	A_{sm} = 0.24×(125)×(1000)/100	300.00	mm^2/m
	Dia of bar =	10	
	Spacing of bar required =	260	mm
	Provide 10 mm dia bar @ 125 mm c/c in meridional direction		
	Circumferential force = wR[cos θ-(1/(1+cos θ))] =	2.85	kN/m
	Hoop stress = 0.00285/0.0015	0.02	MPa
	0.02 < 1.5 (OK)		
	As the stress is only nominal, provide the min. reinforcement of	0.24	%
	A_{sm} = 0.24×(125)×(1000)/100	300.00	mm^2/m
	Dia of bar =	10	mm
	Spacing of bar required =	260	mm
	Provide 10 mm dia bar @ 125 mm c/c in circumferential direction		
(5)	**Design of beam at balcony level and balcony slab**		
	<u>Design of balcony</u>		
	Clear width of walkway	0.75	m
	Width of beam at this level	350	mm
	Cantilever span of balcony from beam	0.40	m

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	Thickness of slab	150	mm
	Self weight of slab =(0.15)×25×0.4=	1.50	kN/m
	Live load on slab	1.50	kN/m^2
	Load due to finishes	1.20	kN/m^2
	Total load acting on the walkway slab =0.15x25+1.5+1.2=	6.45	kN/m^2
	Max BM at Support =6.45x0.4²/2=	0.52	kN-m
	Effective Depth required = √((BM×10^6)/(k×1000) = √((0.52×10^6)/(1.81×1000)	16.97	mm
	Provided 150 mm uniform thickness for walkway slab		
	Cover to the reinforcement	25	mm
	Diameter of bar	12	mm
	effective depth provided =150-25-12	119	mm
	Area of steel required =(0.52×10^6)/(0.86×119×130)	39.09	mm^2/m
	Minimum percentage of steel required =	0.24	%
	Minimum Area of steel required on center of slab = =0.0024×150×1000 =	360.00	mm^2/m
	Spacing of 12 mm dia steel =	250	mm c/c
	Spacing provided	200	mm c/c
	Area of steel provided =	565.49	mm^2/m
	percentage of steel provided =	0.48	
	Diameter of distribution bar =	10	mm
	Spacing of 10 mm dia tor steel =	200	mm c/c
	10 mm dia tor steel @ 200 mm c/c as distribution steel		
	Provide 12 mm main bar @ 200 mm c/c		
	Total weight of slab = 2×π×(6.4/2+350/1000+0.4/2)×0.4×(150/1000)×25	35.34	kN
(6)	**Design of Top ring Beam**		
	Hoop thrust on ring beam is same as the horizontal component of		
	the meridional thrust from the top dome. The hoop tension in the		
	ring beam is, therefore, equal to		
	Hoop Tension = T= N_o cos(θ)R =	28.36	kN
	Where R =	3.20	m
	Size of the web of the ring beam:		
	b=	350	mm
	D=	300	mm

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	Area of tension steel required, As=	=(28.36×1000)/130	218.154	mm^2
	Minimum percentage of steel =		0.24	%
	Minimum steel A_{min} =	=(0.0024)×350×300	252.00	mm^2
	Cover to the reinforcement =		25	mm
	Dia of bar =		16	mm
	Number of bars required		2	Nos.
	Number of bars provided		3	Nos.
	Area of steel provided =		603	mm^2
	Stress in concrete = $T/[A_g + (m-1)A_{st}]$ =			
	= (28.36x1000)/[(350x300)+(9.33-1)x603.19] =		0.26	N/mm^2
	0.26 < 1.5 (Safe)			
	Provide a ring beam of size 350 mm by 300 mm.			
	Provide 3Y16 at top and 3Y16 at bottom			
	Provide 8 mm dia stirrups at 250 mm centres.			
	Self weight of beam = 2π (3.375) (0.35 X 0.3) (25) =		55.67	kN
(7)	**Design of vertical wall of tank**			
	Total Wall height =		4.90	m
	height of water column =	=4.1+0.3 =	4.40	m
	Radius of tank		3.20	m
	Hoop tension, T = unit weight of water x H x D/2	=10x4.4x3.2	140.8	kN/m
	Thicknesss of wall =		250	mm
	H^2/Dt=	=4.9^2/(6.15x0.25)	15.616	
	Calculating tension and moment from IS 3770 Part 4			
From IS 3370	Hoop tension for hinged base and top free			
	Coefficient from table 9 of IS 3370 Part 4		0.7714	
	Hoop tension =coefficient x w x H x R	=0.77x10x4.9x3.2	120.955	kN/m
	Maximum Hoop tension, T =		140.8	kN/m
	Ast required on each face for max tension =	=140.8×1000/(130×2)	541.54	mm^2
	Minimum Ast required as per IS 3370		0.24	%
	Ast minimum required on each face	=(0.0024×1000×250)/2	300	mm^2
	Dia of bar provided =		10	mm
	Spacing required on each face		145	mm

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	Provide 10 mm dia @ 145 mm centres on both faces		
	Area of steel provided	541.65	mm^2
	Stress in concrete = $T/[A_g + (m-1)A_{st}]$ =		
	=(140.8×1000)/(1000×250+(9.33-1)×541.65)	0.55	N/mm^2
	0.55 < 1.5 (Safe)		
	Vertical Steel		
From IS 3370	Vertical Moment for Fixed base and top free		
	Coefficient from table 10 of IS 3370 Part 4	0.00811	
	Moment =coefficient x w x H^3 =0.00811*10*4.9^3	9.54258	kN-m
	Area of steel required for moment		
	=9.54*10^6/(130*(250-45-12/2)*0.86)	428.915	mm^2
	Minimum area of steel on each face	300	mm^2
	Diameter of bar provided	12	mm
	Spacing required	250	mm
	Spacing provided	200	mm
	Provide 12 mm dia @ 200 mm centres on both faces		
	Area of steel provided =(π/4)×12^2(1000/250)	565.49	mm^2
	Total weight of cylindrical wall =2×π×3.2×4.9×0.25×25	615.75	kN
(8)	**Design of bottom dome and internal shaft**		

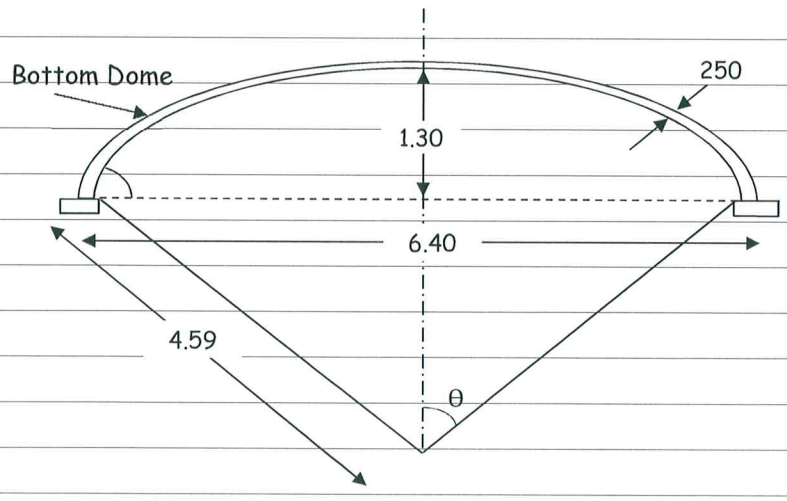

Figure 4: Bottom Dome.

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	Diameter at base of dome =	6.40	m
	Rise of bottom dome = h =	1.30	m
	Thickness of bottom dome, t =	250	mm
	Radius of the shell surface = $(radius^2 + rise^2)/(2 \times rise)$ =	4.59	m
	Weight of the dome slab = $2 \times \pi \times 4.59 \times 1.3 \times 0.25 \times 25$ =	234.32	kN
	Thickness of walls of Internal shaft =	200	mm
	Total Projection of platform required at top of internal shaft	750	mm
	Thickness of platform	150	mm
	Internal diameter of vertical shaft = =(2×0.6)-0.2	1000	mm
	External diameter = 1000 + 2 x 200 =	1400	mm
	Weight of water over bottom dome = (with FB) =103.86×10	1038.6	kN
	Weight of vertical shaft = =π×((1400-200)/1000)×(200/1000)×3.1×25	58.43	kN
	Weight of circular platform		
	=π×(1000/1000+750/1000)×(150/1000)×(750-200)/1000×25	11.34	kN
	Total weight on dome = =234.32+1038.64+58.43+11.34	1342.74	kN
	Load/unit area = w = =1342.74/((π/4)×6.4^2)	41.74	kN/m^2
	Meridional thrust = T_1 = = wR/(1+cos Θ)	111.6	kN
	where, cos Θ =	0.717	rad
	Meridional stress = (111.6*1000)/(130*1000) =	0.858	N/mm^2
	0.858 < 8 (Safe)		
	Circumferential force = wR [cos Θ -(1/(1+cos Θ)] =	25.73	kN
	Hoop stress = (25.73*1000)/(130*1000) =	0.2	N/mm^2
	0.2 < 1.5 (Safe)		
	Provide minimum reinforcement of	0.24	%
	Minimum steel required, A_{st} =	600	mm^2
	Diameter of bar provided =	10	mm
	Spacing of bar required =	125	mm
	Provide 10 mm dia bar at 125 mm centres both radially and in circumferential direction.		
	Maximum hoop compression in the internal shaft =		
	=10 x 3.1 x ((1400 - 200)/1000)/2 =	18.6	kN
	Hoop stress = = (18.6 x 1000)/(130 x 1000) =	0.14	N/mm^2
	0.14 < 8 (Safe)		
	Provide minimum reinforcement of	0.24	%

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	Minimum steel required, A_{st} =	480	mm^2
	Diameter of bar provided =	10	mm
	Spacing of bar required =	160	mm
	Provide 10 mm dia bar at 160 mm centres in both directions.		
(9)	**Design of bottom ring beam**		
	Horizontal thrust from bottom dome = =111.6xCOS(44.2)	79.99	kN
	Net Hoop Tension force in ring beam, H =	79.99	kN
	Hoop compression = = 79.99 (6.4/2) =	255.97	kN
	Dimensions of bottom ring beam :		
	b =	350	mm
	D=	500	mm
	Area of tension steel required =(255.968 x 1000)/130	1968.98	mm^2
	Provide minimum reinforcement of	0.24	%
	Minimum steel required, A_{st} = =(0.24/100)×350×500	420	mm^2
	Diameter of bar provided =	20	mm
	Number of bars required =	8	Nos.
	Area of tension steel provided	2513	mm^2
	Stress in concrete = T/[Ag + (m-1)Ast] =		
	=(255.968×1000)/(350×500+(9.33-1)×2513.27	1.31	N/mm^2
	1.31 < 1.5 (Safe)		
	Provide a ring beam of size 350 mm by 500 mm.		
	Provide 4Y20 at top and 4Y20 at bottom		
	Provide 8 mm dia stirrups at 200 mm centres.		
	Weight of bottom ring beam = π x 6.4 x (0.35 x 0.5) x 25=	87.96	kN
(10)	**Design of supporting cylindrical shaft**		
	Centre to centre Diameter of shaft =	6.40	m
	Height of shaft (above G.L.) =	25	m
	Thickness of shaft wall above G.L. =	250	mm
	Minimum thickness of shaft required as per IS: 11682-1985	151	mm
	Total depth of foundation below G.L. =	3.00	m
	Depth of shaft (below G.L.) = = 3 - 0.65 =	2.35	m
	Thickness of shaft wall below G.L. =	350	mm

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	Self weight of shaft above G.L. = π x 6.4 x 25 x 25 x 0.25 =	3141.59	
	Self weight of shaft below G.L. = π x 6.4 x 25 x 2.35 x 0.35 =	413.43	kN
	Thickness of shaft wall above G.L. =	250	mm
	Loads acting on shaft at ground level:		
	(1) Top dome	173.40	kN
	(2) Top ring beam	55.67	kN
	(3) Balcony	35.34	kN
	(4) Tank wall	615.75	kN
	(5) Bottom spherical dome	234.32	kN
	(6) Internal shaft + platform	69.77	kN
	(7) Bottom ring beam	87.96	kN
	Weight of tank portion =	**1272.22**	kN
	(8) Supporting shaft	3555.02	kN
	Total Dead load on top of footing =	**4827.24**	kN
	(9) Weight of water (Hydro test condition)=	1038.64	kN
	(10) Weight of water (Working condition)=	954.14	kN
	Wind pressure:		
	Basic wind speed, V_b=	50	m/s
	Risk Coefficient, k_1=	1.08	
	Terrain, height and structure size factor, k_2=	1.11	
	Topography factor, k_3=	1	
	Design wind speed, $V_z = V_b \times k_1 \times k_2 \times k_3$ =	59.94	m/s
	$Pz = 0.6 V_z^2$ =	2.16	kN/m^2
Ref Pg.	**Total moment due to wind load about base of footing , M**	**2568.95**	**kN-m**
Wind load	Area of cross section of shaft, A = π [(3.325)² - (3.075)²)] =	5.03	m^2
calculation	Second moment of area, I :		
	I = (π/4) [(3.325^4) - (3.075^4)] =	25.78	m^4
	Stress at base section:		
	Tank empty condition:		
	W =	4827.24	kN
	Outer dia of shaft, D =	6.75	m
	Mean radius of shaft, r =	3.2	m
	M =	2568.95	kN-m
	e = (M/W) =	0.53	m

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	e/r = 0.53/3.2 =	0.166	m
	e/r <= 1/2 (OK)		
IS 11682-	***This section is under compression only***		
1985	σcv = (W/2π r t)[1 + (2e/r)] =	0.91	N/mm^2
	0.91 < 0.38x 30 (Safe)		
	Tank working condition + wind:		
	P =	5781.38	kN
	M =	2568.95	kN-m
	e= M/W =	0.44	m
	e/r = 0.44/3.2 =	0.14	
IS 11682-	**e/r <= 1/2 (OK)**		
1985	σcv = (W/2π r t)[1 + (2e/r)] =	1.05	N/mm^2
	1.05 < 0.38x 30 (Safe)		
	Tank Hydro test condition		
	W =	5865.88	kN
	M =	0	N-mm
	e = M/W =	0	
IS 11682-	σcv = (W/2π r t)[1 + (2e/r)] =	0.83	N/mm^2
1985	**0.83 < 0.38 x 30 (Safe)**		
IS 11682-	Provide minimum longitudnal reinforcement of	0.25	%
1985	Area of steel required on each face, A_{st} =	312.5	mm^2
	Diameter of bar provided =	12	mm
	>=10 mm (OK)		
	Spacing of bar required =	360	mm
	Spacing of bar provided =	200	mm
	Provide 12 mm dia bar at 200 mm centres vertically on each faces.		
	Area os steel provided on each face =	565.5	mm^2
	Circumferential reinforcement in shaft:		
IS 11682-	Provide minimum circumferential reinforcement of	0.2	%
1985	Area of steel required on each face, A_{st} =	250	mm^2
	Minimum steel required per meter length on each face =	200	mm^2
	Diameter of bar provided =	10	mm
	Spacing of bar required =	310	mm

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	Spacing of bar provided =	200 mm
	Area of steel provided per metre length of shaft=	392.70 mm^2
		> 200 (OK)
	Provide 10 mm dia bar at 200 mm centres circumferentially on each faces.	
	Area of steel provided =	392.7 mm^2
	Check for seismic forces	
	Height of staging above groung level =	25.00 m
	Stiffness of shaft, k = 3 EI/l^3 =	
IS 456-2000	E = 5000(f_{ck})$^{0.5}$ =	27386.13 N/mm^2
	I = (π/4) [(3.325^4) - (3.075^4)] =	25.78 m^4
	l= length of staging =	25.00 m
	k=	135528.5 kN/m
	Seismic coefficient is given by : $A_h = \frac{Z}{2}\frac{I}{R}\left(\frac{S_a}{g}\right)$	
IS: 1893-2002	where, Zone Factor, Z =	0.1
	Importance Factor, I =	1.75
	Response reduction Factor R =	3
	Spectral Acceleration , (S_a/g)	
	Tank Empty condition :	
	Weight of tank Container =	1272.22 kN
	Weight of 1/3 of staging = (1/3) x (3141.59) =	1047.20 kN
	Seismic weight for tank empty condition, W_s=	2319.42 kN
	Time period when tank empty, T_e = $2\pi\ [(W_s/9.81) / k]^{0.5}$	
	= 2π x {(2319.42/9.81)/(135528.47)}^0.5=	0.26 sec
IS: 1893-2002	For rocky, or hard soil sites, corresponding S_a/g =	2.50
	The design horizontal seismic coefficient, A_h =	0.07
	Maximum horizontal seismic force acting at top of staging =	169.12 kN
	Moment due to seismic forces at top of footing:	
	Total load, W =	4827.24 kN
	Moment, M=	4625.55 kN-m
	e= M/W =	0.96 m
	e/r = 0.96/3.2 =	0.3

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IS 11682-1985	**e/r <= 1/2 (OK)**	
	σcv = (W/2π r t)[1 + (2e/r)] =	1.1 N/mm²
	1.1 < 0.40 x 30 (Safe)	
	Tank Full condition :	
	Weight of tank Container =	1272.22 kN
	Weight of 1/3 of staging = (1/3) x (3141.59) =	1047.20 kN
	Weight of water =	954.14 kN
	Seismic weight for tank full condition =	3273.56 kN
	Time period when tank full, T =	0.31 sec
IS: 1893-2002	For rocky, or hard soil sites, corresponding Sa/g =	2.5
	The design horizontal seismic coefficient, A_h =	0.07
	Maximum horizontal seismic force acting at top of staging =	238.70 kN
	Moment due to seismic forces at top of footing:	
	Total load, W =	5781.38 kN
	Moment, M= =238.7*(25+2.35)	6528.37 kN-m
	e= M/W =	1.13
	e/r = 1.13/3.2 =	0.35
IS 11682-1985	**e/r <= 1/2 (OK)**	
	σcv = (W/2π r t)[1 + (2e/r)] =	1.4 N/mm²
	1.4 < 0.40 x 30 (Safe)	
	Check for stress at openings:	
	Size of opening : width =	1 m
	height =	2 m
	Maximum vertical compressive stress in concrete at outside diameter of shaft shell is given by :	
IS 11682-1985	$\sigma_{cv} = \frac{W}{2(\pi-\beta)\,r\,t}\left[1 + \frac{2\left\{\frac{e}{r} + \frac{\sin\beta}{\pi-\beta}\right\}\{(\pi-\beta)\cos\beta + \sin\beta\}}{(\pi-\beta) - \frac{1}{2}\sin 2\beta - \frac{2\sin^2\beta}{(\pi-\beta)}}\right]$	

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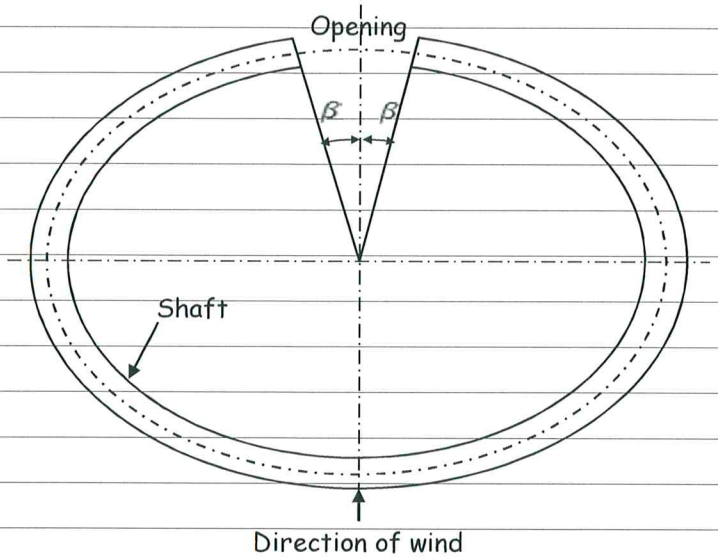

	Where,		
	ß = half the angle substended by neutral axis as a chord on the circle of radius r =	0.16	rad
	W = Total vertical load above section under consideration in N =	5781	KN
	M = Moment in vertical plane at the section under consideration in N-mm =	6.53E+03	KN-m
	e = M/W =	1.129	m
	r = Mean radius of circular shaft in m =	3.2	m
	t = Thickness of shaft in mm =	250	mm
	e/r =	0.353	
IS 11682-1985	substituting values in the above formula , we get σcv =	2.29	N/mm^2
		2.3 < 0.40 x 30 (Safe)	
(11)	**Design of raft foundations**		
	Total load from tank and shaft = (Dead load on top of footing + weight of water working condition)		
	=4827.24KN+954.14KN - (a)	5781.38	kN
From staad	Total weight of staircase : (seismic case) =	1337	kN
	Load from staircase = - (b)	1337	kN
	Diameter of raft slab, D_r =	9.6	m
	Thickness of raft slab, t =	650	mm
	Self weight of footing = $(\pi/4)\times D_r^2 \times t$ =		

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	=(π/4)×9.6^2×0.65×25 - (c)	1176.21	kN
	Weight of Earth filling inside the shaft upto G.L.		
	= [π (6.05²)/4] X 2.35 X 18 = - (d)	1216.02	kN
	Weight of earth filling over the raft slab upto G.L.		
	= [π (9.6²-6.75²)/4] X 2.35 X 18 = - (e)	1548.08	kN
	Total load acting on raft slab, W = =(a)+(B)+(c)+(D)+(e)	11058.69	kN
	Net S.B.C. of soil =	150	kN/m²
	Gross S.B.C at depth of 3 m below G.L. (For normal load)=		
	=150+3×18	204	kN/m²
	Gross S.B.C at depth of 2.35 m below G.L. (For seismic/wind load)=		
	=150×1.25×3×18	241.5	kN/m²
	Area of footing, A = =(π/4)×9.6^2	72.38	m²
	Direct load, W =	11058.69	kN
	Moment M = (Tank full condition under seismic)	6528.37	kN-m
From staad	Moment from staircase column (seismic case) =	50.00	
	Total moment =	6578.37	
	Section modulas, Z=	86.86	m⁴
	Maximum intensity of soil pressure at base = [W/A + M/Z] =	228.52	kN/m²
	228.52 < 241.5 (Safe)		
	Minimum intensity of soil pressure at base = [W/A - M/Z] =	77.05	kN/m²
	77.05 >0 (No tension)		
	Adopt Diameter of raft slab = 9.6 m		
	Projection of raft beyond face of shaft =	1.425	m
	Maximum net soil pressure, w =		
	=228.52-(650/1000*25)-(18*2.35)	169.97	kN/m²
	The loading at base is taken as annular loading on the mean diameter of the shaft.		
	Diameter of raft slab = 2a =	9.6	m
	Diameter of the shaft = 2b =	6.40	m
	Radial moment at centre of foundation is given by:		
	$M_r = \frac{W}{8\pi}\left[2\log_e\left(\frac{a}{b}\right) + 1 - \left(\frac{b}{a}\right)^2\right] - \frac{3}{16}w.a^2$ =	-133.00	kN-m/m
	Moment at junction of footing and tank walls at a radius of 3.2 m is given by:		

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	$M_{max} = \frac{W}{8\pi}\left[2\log_e\left(\frac{a}{b}\right) + 1 - \left(\frac{b}{a}\right)^2\right] - \frac{3}{16}w(a^2 - b^2)$ =	193.34	kN-m/m
	Design ultimate moment = M_{ur} = (1.5 x 193.34) =	290.01	kN-m/m
	Effective depth required d = $[M_u/.133\ f_{ck}\ b]^{0.5}$ =	269.60	mm
	Effective depth provided at the section =	592.00	mm
		(OK SAFE)	
	Compute parameter:		
	M_u/bd^2 =	0.828	
	Refer Table-4 of SP : 16 and read out the percentage reinforcement as:		
	p_t= 100 A_{st} / bd =	0.19672	
	Area of steel required, Ast =	1164.58	mm^2/m
	Diameter of bar provided =	16	mm
	Cover to the reinforcement =	50	mm
	Actual effective depth at the section =	592	
	Spacing of bar required =	150	mm
	Provide 16 mm dia bar at 150 mm centres both ways at bottom of footing.		
	Area of steel provided =	1340.41	mm^2/m
	Design ultimate moment = M_{uc} = (1.5 x -133) =	-199.5	kN-m/m
	Compute parameter:		
	M_u/bd^2 =	0.57	
	Refer Table-4 of SP : 16 and read out the percentage reinforcement as:		
	p_t= 100 A_{st} / bd =	0.1338	
	Area of steel required, Ast =	794.77	mm^2/m
	Diameter of bar provided =	12	mm
	Cover to the reinforcement =	50	mm
	Effective depth at the section =	594	
	Spacing of bar required =	125	mm
	Provide 12 mm dia bar at 125 mm centres both ways at top of footing.		
	Check for shear :		

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	Intensity of soil pressure, p =	169.97	kN/m^2
	Cantilever projection of raft = (9.6-6.4-0.35)/2 =	1.425	m
	Max. SF at a distance 'd' from the support = 169.97*(1.425-0.592) =	141.59	kN
	Shear stress , t_v = V/bd = 141.59x 1000/1000(592) =	0.24	N/mm^2
	percentage steel at the section, 100As/bd =	0.197	%
	The allowable shear stress for 0.197 % tension reinforcement is		
	$k \times t_c$ =	0.32	N/mm^2
	Slab is safe in shear (OK)		

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Wind Load Calculation:

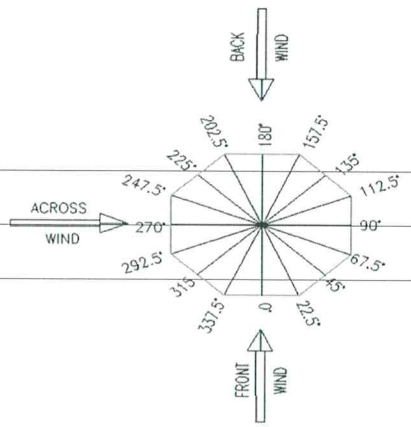
Basic Wind Speed V_b (m/s) =	50	m/s
Risk Coefficient K_1 =	1.08	
Terrain Factor K_2 (For Category-1 & Class-B) =	1.11	
Topography factor K_3 =	1	
Design Wind Speed V_z = $V_b \times K_1 \times K_2 \times K_3$ =	59.94	m/s
Design Wind Pressure acting $P_z = 0.6 \times V_z^2$ =	2155.68	N/m^2
	2.16	kN/m^2

External Pressure Coefficient on shaft and top Cylindrical wall of bowl:

Refer Table-18 (IS: 875 (Part-3) - 1987)

Height of the Tank above ground level (h) =	27.45	m
Outer Diameter of the shaft (D) =	6.65	m
Ratio h/D = 27.45/6.65 =	4.13	

From Table-18 use the coefficients for the nearest curve of h/D =7



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θ in degrees	Shaft (C_{pe})	Wall (C_{pe})
0	1	1
15	0.8	0.8
30	0.1	0.1
45	-0.8	-0.8
60	-1.7	-1.7
75	-2.2	-2.2
90	-2.2	-2.2
105	-1.7	-1.7
120	-0.8	-0.8
135	-0.6	-0.6
150	-0.5	-0.5
165	-0.5	-0.5
180	-0.5	-0.5
195	-0.5	-0.5
210	-0.5	-0.5
225	-0.6	-0.6
240	-0.8	-0.8
255	-1.7	-1.7
270	-2.2	-2.2
285	-2.2	-2.2
300	-1.7	-1.7
315	-0.8	-0.8
330	0.1	0.1
345	0.8	0.8

Internal Pressure Coefficient :

Refer Clause 6.2.3.1 (IS: 875 (Part-3) - 1987)

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	Internal Pressure coefficients for openings not more than 5% (C_{pi}) =		+0.2	
			-0.2	
	Wind Load acting on the shaft (Case-1)			

θ in degrees	Shaft (C_{pe})	Shaft (C_{pi})	wind force /m h	$F_{along\ wind}$	$F_{across\ wind}$
0	1	0.2	1.5	1.5	0
15	0.8	0.2	1.13	1.091	0.292
30	0.1	0.2	-0.19	-0.165	-0.095
45	-0.8	0.2	-1.88	-1.329	-1.329
60	-1.7	0.2	-3.57	-1.785	-3.092
75	-2.2	0.2	-4.51	-1.167	-4.356
90	-2.2	0.2	-4.51	0	-4.51
105	-1.7	0.2	-3.57	0.924	-3.448
120	-0.8	0.2	-1.88	0.94	-1.628
135	-0.6	0.2	-1.5	1.061	-1.061
150	-0.5	0.2	-1.32	1.143	-0.66
165	-0.5	0.2	-1.32	1.275	-0.342
180	-0.5	0.2	-1.32	1.32	0
195	-0.5	0.2	-1.32	1.275	0.342
210	-0.5	0.2	-1.32	1.143	0.66
225	-0.6	0.2	-1.5	1.061	1.061
240	-0.8	0.2	-1.88	0.94	1.628
255	-1.7	0.2	-3.57	0.924	3.448
270	-2.2	0.2	-4.51	0	4.51
285	-2.2	0.2	-4.51	-1.167	4.356
300	-1.7	0.2	-3.57	-1.785	3.092
315	-0.8	0.2	-1.88	-1.329	1.329
330	0.1	0.2	-0.19	-0.165	0.095
345	0.8	0.2	1.13	1.091	-0.292

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				SUM =	6.8	0
	Wind Load acting on the shaft (Case-2)					
	θ in degrees	Shaft (C_{pe})	Shaft (C_{pi})	wind force /m h	$F_{along\ wind}$	$F_{across\ wind}$
	0	1	-0.2	2.26	2.26	0
	15	0.8	-0.2	1.88	1.816	0.487
	30	0.1	-0.2	0.56	0.485	0.28
	45	-0.8	-0.2	-1.13	-0.799	-0.799
	60	-1.7	-0.2	-2.82	-1.41	-2.442
	75	-2.2	-0.2	-3.76	-0.973	-3.632
	90	-2.2	-0.2	-3.76	0	-3.76
	105	-1.7	-0.2	-2.82	0.73	-2.724
	120	-0.8	-0.2	-1.13	0.565	-0.979
	135	-0.6	-0.2	-0.75	0.53	-0.53
	150	-0.5	-0.2	-0.56	0.485	-0.28
	165	-0.5	-0.2	-0.56	0.541	-0.145
	180	-0.5	-0.2	-0.56	0.56	0
	195	-0.5	-0.2	-0.56	0.541	0.145
	210	-0.5	-0.2	-0.56	0.485	0.28
	225	-0.6	-0.2	-0.75	0.53	0.53
	240	-0.8	-0.2	-1.13	0.565	0.979
	255	-1.7	-0.2	-2.82	0.73	2.724
	270	-2.2	-0.2	-3.76	0	3.76
	285	-2.2	-0.2	-3.76	-0.973	3.632
	300	-1.7	-0.2	-2.82	-1.41	2.442
	315	-0.8	-0.2	-1.13	-0.799	0.799
	330	0.1	-0.2	0.56	0.485	-0.28
	345	0.8	-0.2	1.88	1.816	-0.487
				Σ	6.76	0

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	Total horizontal force acting on the shaft due to wind :		
	Horizontal Wind load acting on the shaft per m height =	6.8	kN/m
	Height of Shaft above the GL upto bottom of Top Bowl =	25	m
	Depth of Shaft below the GL upto top of footing =	2.35	m
	Total Horizontal wind load acting on the shaft = 6.8x25 =	170	kN
	Maximum moment acting at the base of the shaft =		
	= 170x(25/2+2.35) =	**2524.5**	kN-m
	Force acting on the roof of the structure:		
Ref. Table 19	Structure according to the shape is (c)		
	H=	31.2	
	D=	6.4	
IS: 875	H/D =	4.88	
(Part 3)-1987	C_{pe} for above value of H/D =	-1	
	C_{pi} =	0	
	Force acting on the roof of the structure, $P = 0.785\ D^2\ (p_i - C_{pe}\ p_d)$ =	69.45	kN
	e = 0.1 D =	0.64	
	Moment about the base of tank =	**44.45**	kN-m
	Total Moment =	**2568.95**	kN-m

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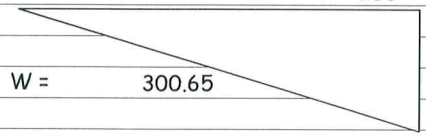
DESIGN OF STAIR CASE

DESIGN OF STAIR CASE

* Maximum span of flight is designed and the same reinforcement is provided for all flights and landing slab.

Design data :

fck	=	25 N/mm²
fy	=	500 N/mm²
Tread , T	=	250 mm
Rise , R	=	167 mm
Thickness of Waist slab , D	=	150 mm



Dead load :

On landing area, Self wt.of slab	=	3.75 KN/m²
Finish load	=	1.2 KN/m²
Total dead load	=	4.95 KN/m²

On Stair area,

Flight load = 1/ T (D * W + T * R / 2) * 25

= 1 / 0.25 (0.15 * 0.30 + 0.25 * 0.17 / 2) * 25

= 6.60 KN/mm²

Span for stair area = 2.5 m

Span for landing area =

l_1 = 1 m

l_2 = 1 m

Clause 33.1., IS : 456,

Effective span,

ES = A + B + C = 2.5 m

Live load :

Live on landing & stair area = 4 KN/m²

Factored loads,

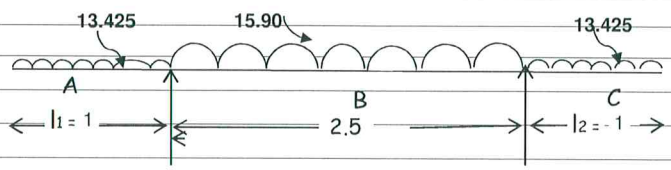
On landing area, = 1.5 * (DL + LL)

= 13.43 KN/m²

On stair area, = 1.5 * (DL + LL)

= 15.90 KN/m²

Loading diagram ,



From staad

Ra	=	33.33 KN
From staad Rb	=	33.33 KN

Maximum B.M.

Mu = 7.00 KN-m

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Clear cover in mm	=		30 mm
Assuming dia of bar as	=		10 mm
Effective depth, d	=		115 mm
Table , SP : 16			
Reinforcement :			
Mu/bd^2	=		0.53 N/mm^2
pt	=		0.12 %
Ast(req)	=		143.51 mm^2
Required 10 Dia.	@		547 mm c/c
Provide 10 Dia.	@		125 mm c/c
therefore,			
pt(prov)	=		0.55 %
Ast(prov)	=		628.3 mm^2
Minimum reinforcement required	=(0.12/100)*1000*150		187.2 mm^2
Provide 8 mm dia 200 mm spacing c/c			251.2 mm^2
Reinforcement provided			
pt(prov)	=		0.17 %
Check for shear :			
Actual shear stress,Vu	=		33.33 KN
T_V	=		0.29 N/mm^2
for pt	=		0.55
Allowable shear stress,T_c	=		0.507 N/mm^2
			> T_V
NO SHEAR REINFORCEMENT IS REQUIRED			
Check for deflection :			
(From IS:456:2000 clause 23.2)			
Allowable span /depth ratio	=		20.00
% of tension reinforcement	=		0.55
fs = 0.58*415*(143.51/628.32)	=		54.98
From Fig 4 Modification factor for tension Rft (Mft)	=		2.00
From Fig 5 Modification factor for tension Rft (Mfc)	=		1.00
Modified span /depth ratio	= l/d x Mft x Mfc	=	40.00
Actual span/depth ratio	2.5*1000/115	=	21.74
Actual span/depth ratio < Modified span/depth ratio		=	safe

"Designs Vetted"
M/s. S.K. ASSOCIATES Hyd-08

APPROVED
30/11/16
SE, NIRMAL

Asst. Executive Engineer
TDWSP Asifabad

Dy. Executive Engineer
TDWSP Asifabad

Executive Engineer
TDWSP Asifabad

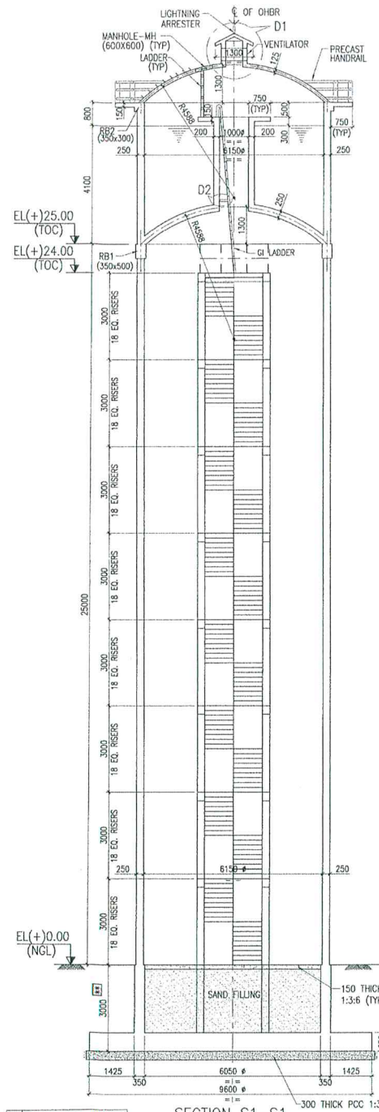
WAPCOS LIMITED

LARSEN & TOUBRO LIMITED
Water,Smart World & Communication IC

PROJECT:	Providing drinking water to habitations in Komarambheem-Asifabad Segment in Adilabad District	DOCUMENT NO. LE150883-C-WS-CW-DC-3023		DATE 07-Apr-2016
TITLE :	90 KL Capacity OHBR - 25 m staging height	DESIGNED AKHB/RRG	CHECKED RR	PAGE

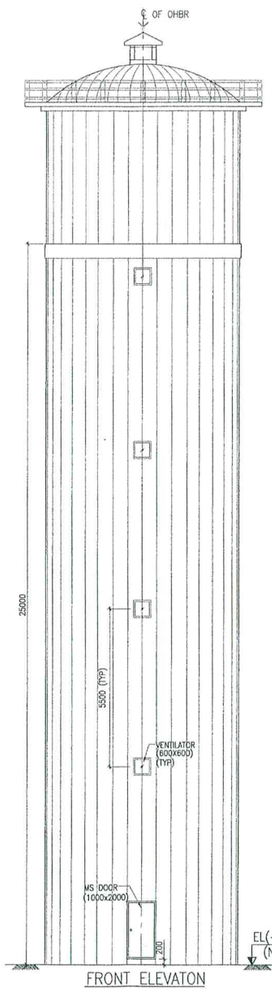
APPENDIX

(1)	**Stability Check** - Tank empty conditon		
	Wind force		214.45 kN
	Moment due to wind force		2568.95 kN-m
	Seismic force		169.12 kN
	Moment due to seismic force		4625.55 kN-m
	Max. horizontal force		214.45 kN
	Max. overturning moment = OM		4625.55 kN-m
	Total vertical DL		
	=(Top container(without water) + shaft + stair case + raft + earth inside and outside)		10104.55 kN
	0.9 DL	=0.9 x 10104.55	9094.10 kN
	Restoring moment = RM =DL x (raft dia)/2	=10104.55 x 9.6/2	48501.85 kN-m
	Check for safety against overturning		
	Factor of Safety =OM/RM	= 4625.55/48501.85 =	10.49
			>1.5 safe Ok
	Check for safety against sliding		
	Factor of Safety =(0.9DL x μ)/(Max horizontal force)	=9094x0.4/214	16.96
			>1.25 safe Ok
(2)	**Stability Check** - Tank full conditon		
	Seismic force		238.70 kN
	Moment due to seismic force		6528.37 kN-m
	Max. horizontal force		238.70 kN
	Max. overturning moment = OM		6528.37 kN-m
	Total vertical DL		
	=(Top container (with water) + shaft + stair case + raft + earth inside and outside)		11058.69 kN
	0.9 DL	=0.9 x 11058.69	9952.83 kN
	Restoring moment = RM =DL x (raft dia)/2	=11058.69 x 9.6/2	53081.73 kN-m
	Check for safety against overturning		
	Factor of Safety =OM/RM	= 6528.37/53081.73 =	8.13
			>1.5 safe Ok
	Check for safety against sliding		
	Factor of Safety =(0.9DL x μ)/(Max horizontal force)	=9953x0.4/239	16.68
			>1.25 safe Ok

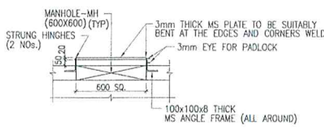


SECTION S1–S1
(NUM. DETAILS)

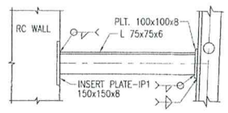
DEPTH OF FOUNDATION SHALL BE 3.0m BELOW FGL OR NGL WHICHEVER IS LOWER.



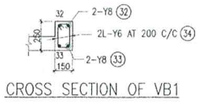
FRONT ELEVATON



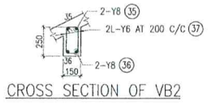
TYPICAL DETAIL OF MANHOLE–MH



DETAIL – D2



CROSS SECTION OF VB1



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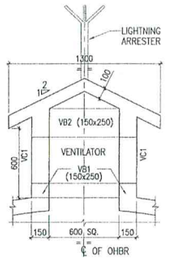
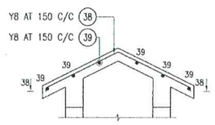
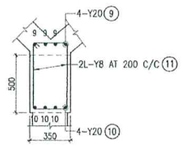


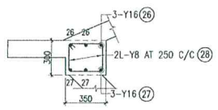
DETAIL–D1



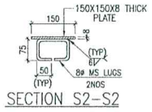
RC DETAIL OF VENTILATOR SLAB



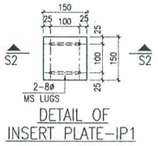
CROSS SECTION OF RB1



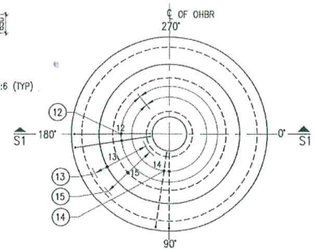
CROSS SECTION OF RB2



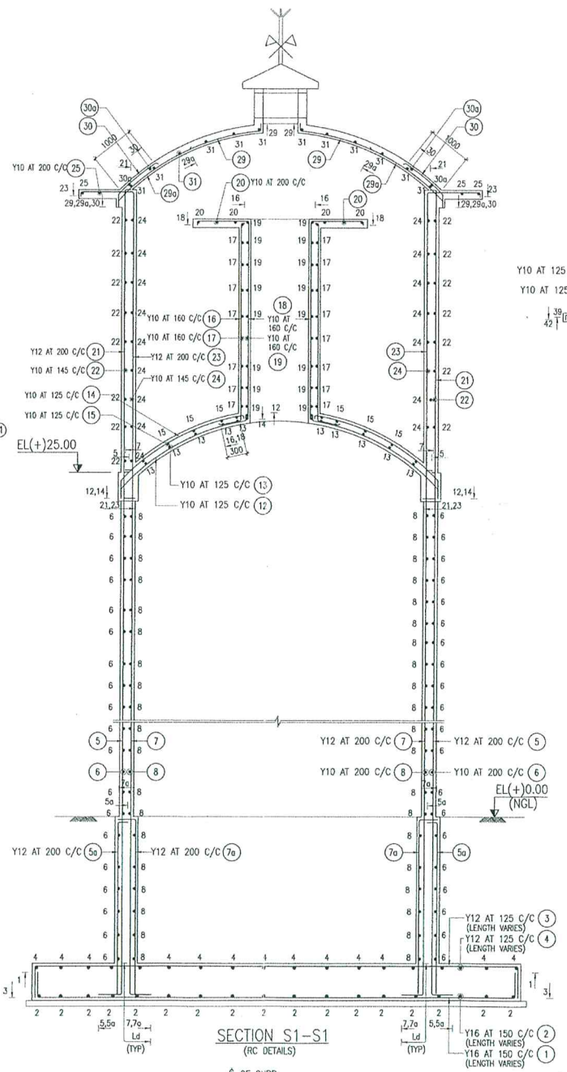
SECTION S2–S2



DETAIL OF INSERT PLATE–IP1

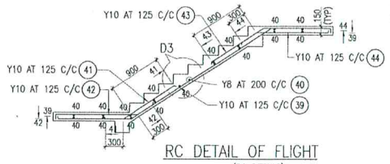


RC DETAILS OF BOTTOM DOME
(RC DETAILS)

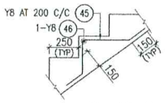


SECTION S1–S1
(RC DETAILS)

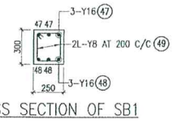
FOR LOCATION AND SIZE OF INLET, OUTLET, SCOUR & OVERFLOW PIPES REFER APPROVED G.A DRAWING.



RC DETAIL OF FLIGHT
(RC DETAILS)

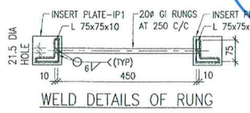


DETAIL–D3



CROSS SECTION OF SB1

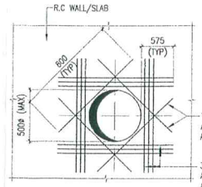
CROSS SECTION OF SB2



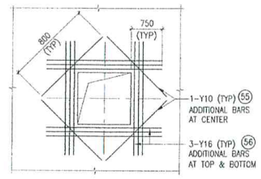
WELD DETAILS OF RUNG

COLUMN MARKED	C1	VC1
TOP OF COLUMN	EL(+)24.000	TOP OF SLAB
VERTICAL REINF.	4 NOS. Y16 (a)	4 NOS. Y8 (c)
TIES	Y8 AT 200 C/C (b) (1 TIE PER SET)	Y6 AT 200 C/C (d) (1 TIE PER SET)
CROSS SECTION OF COLUMN	(b) 250 x 250	(d) 150 x 150
COLUMN SIZE	250 x 250	150 x 150
COLUMN STARTING LEVEL	TOP OF THE RAFT	TOP OF THE DOME

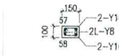
SCHEDULE OF COLUMNS:



TYP. ADDITIONAL REIN. DETAIL FOR CIRCULAR OPENING

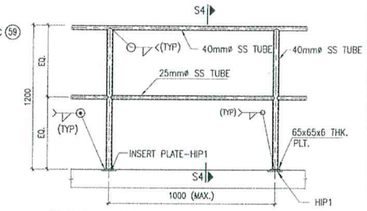


TYP. ADDITIONAL REIN. DETAIL FOR SQUARE OPENING

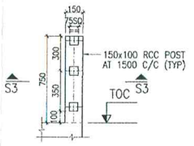


CROSS SECTION OF RCC POST
(RC DETAILS)

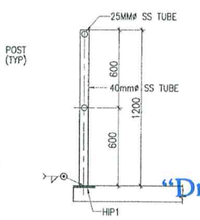
SECTION S3–S3



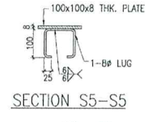
TYPICAL HORIZONTAL HANDRAIL DETAILS



TYPICAL DETAIL OF HAND RAIL POST



SECTION S4–S4

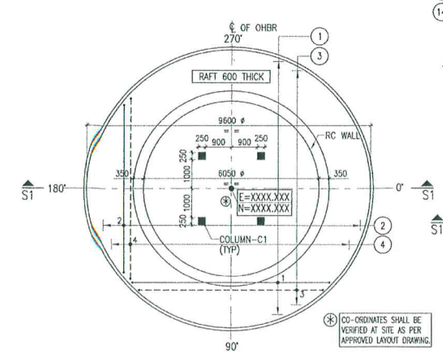


SECTION S5–S5

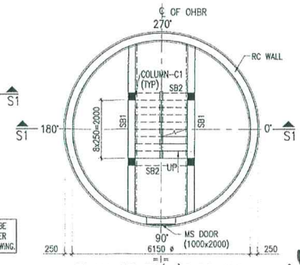
DETAIL OF INSERT PLATE–HIP1

LEGEND:
NGL : NATURAL GROUND LEVEL
TOC : TOP OF CONCRETE

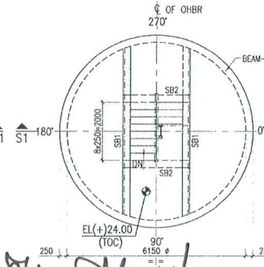
NOTES:–
1. ALL DIMENSIONS ARE IN mm & ALL LEVELS SHALL BE IN METRES.
2. GRADE OF CONCRETE SHALL BE M30 – 43 GRADE CEMENT, 20MM DOWN SIZE AGGREGATE WITH MINIMUM CEMENT CONTENT OF 320 kg/cum.
3. Y INDICATES HYSD BARS OF GRADE Fe500 CONFORMING TO IS: 1786–1985.
4. CLEAR COVER TO MAIN REINFORCEMENT SHALL BE :
   a) RAFT = 50mm
   b) BEAMS (ALL ROUND) = 30mm
   c) COLUMNS = 40mm
   d) SHAFT = 30mm
   e) WALL = 30mm(FREE FACE), 45mm (WATER FACE)
   f) TOP DOME = 30mm
   g) BOTTOM DOME = 30mm(FREE FACE), 45mm (WATER FACE)
5. ALL LAPS SHALL BE 41 TIMES DIA OF BAR AND SHALL BE STAGGERED.
6. THREE COATS OF EPOXY PAINT TO INNER SURFACE OF THE RESERVOIR INCLUDING ROOF AND 2 COATS OF WEATHER PROOF EMULSION PAINTING FOR EXTERNAL SURFACES.
7. SAFE BEARING CAPACITY OF SOIL IS CONSIDERED AS 15T/Sqm.



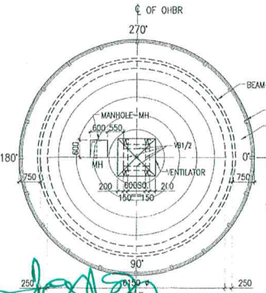
LAYOUT OF COLUMN & FOOTING
(NUM. & RC DETAILS)



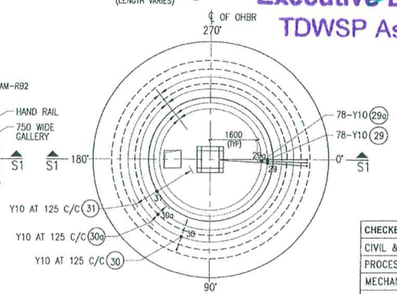
PLAN AT EL(+)0.00 LVL.
(NUM. DETAILS)



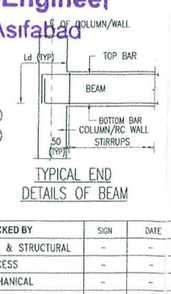
PLAN AT EL(+)25.000 LVL.
(NUM. DETAILS)



TOP PLAN
(NUM. DETAILS)



PLAN AT TOP LEVEL
(RC DETAILS)



TYPICAL END DETAILS OF BEAM

CHECKED BY	SIGN	DATE
CIVIL & STRUCTURAL	–	–
PROCESS	–	–
MECHANICAL	–	–
ELECTRICAL	–	–
INSTRUMENTATION	–	–

"Drawings Vetted"
M/s. S.S.K. ASSOCIATES
Hyd-08.

APPROVED
30/4/16
SE, NIRMAL

Executive Engineer
TDWSP Asifabad

Asst. Executive Engineer
TDWSP Asifabad

Dy. Executive Engineer
TDWSP Asifabad

REV.NO.	DESCRIPTION	DESIGNED	DRAWN	CHECKED	APPROVED
A	FOR APPROVAL	AKHB 24.03.16	MGS 29.03.16	RR 30.03.16	BRJ 31.03.16

LAST BAR NO. 59

L&T Construction
Water, Smart world & Communication

CLIENT : GOVERNMENT OF TELANGANA
RURAL WATER SUPPLY AND SANITATION DEPARTMENT

CONSULTANT :

PROJECT : PROVIDING DRINKING WATER TO HABITATIONS IN KOMARAMBHEEM ASIFABAD SEGMENT IN ADILABAD DISTRICT

SUPPLIER / CONTRACTOR : L&T Construction
Water & Effluent Treatment SBG

JOB No. : LE150883

	NAME	SIGN	DATE
DSGN	AKHB		24.03.2016
DRWN	MGS		29.03.2016
CHKD	RR		30.03.2016
APPD	BRJ		31.03.2016

TITLE :
90 KL CAPACITY OHBR
25m STAGING HEIGHT
NUMERATION & RC DETAILS

SCALE 1:100, 1:50 & 1:25
PROJECTION

DRG. No. LE150883–C–WS–CW–NR–3024
SHEET 1 OF 1
SIZE A1 REV. A

RELEASED FOR ☐ PRELIMINARY ☐ TENDER ☐ INFORMATION ☑ APPROVAL ☐ CONSTRUCTION

GEOTECHNICAL INVESTIGATION REPORT

TELANGANA DRINKING WATER SUPPLY PROJECT

KOMARAM BHEEM - ASIFABAD- SEGMENT 22

ASIFABAD , ADILABAD DISTRICT

90 KL OHBR –BPT AT VAGAI THANDA (V), UTNOOR (M)

CONTRACTOR :

M/s. LARSEN& TOUBRO LIMITED,L&T CONSTRUCTION,

WATER & EFFLUENT TREATMENT SBG, CHENNAI

Drilling By:

M/s. ANJI DRILLING & GROUTING WORKS

Report Prepared by

DR. D. BABU RAO,

M.E.(IIT,Roorkee), Ph.D.(USA), MIGS

MCH Panellist No. 2490 /TP/2000-2

GEOTECHNOLOGIES

CONSULTING GEO TECHNICAL ENGINEER

FORMER PROFESSOR &HEAD OF CIVIL ENGINEERING

OSMANIA UNIVERSITY

Phone: 6663 8830, Mobile : 98490 – 39337

Email :dbaburao2000@yahoo.com

TELANGANA DRINKING WATER SUPPLY PROJECT

90 KL OHBR – BPT AT VAGAI THANDA (V), UTNOOR (M) IN ADILABAD DT.

1. INTRODUCTION

M/s. L &T Construction, Water & Effluent Treatment is proposing to construct 90 KL OHBR – BPT at Vagai Thanda (V), Utnoor (M) .The work is taken up under Segment 22 , Komaram Bheem Project , TDWSP, in Adilabad Dt.

The present Report presents the results of (1) Bore hole.

M/S Anji Drilling & Grouting works; Anantapur has carried out the drilling of bore holes, collection of soil and rock samples and conduct of Standard Penetration Tests at different levels in the respective bore holes at the proposed site.

Analysis of borehole data , Laboratory tests and geotechnical investigation report have been made by Prof. D Babu Rao, ME (IIT,R) , Ph.D. (USA), MIGS, Empanelled Consulting Geo technical Engineer &,Director, Geo technologies, Former Professor of Civil Engineering, Osmania University.

2. SCOPE OF WORK

The following is the scope of work of M/s. Anji Drilling and Grouting Works:

- Drilling Borehole at (1) location for 90 KL OHBR – BPT at Vagai Thanda (V), Utnoor (M)
- Conducting SPT at regular intervals, where feasible
- Collection of undisturbed / disturbed samples from the Bore holes

GEO TECHNOLOGIES

Dr. D. BABU
M.E., Ph.D.(USA)
Consulting Geotechnical Engineer

- Preparation of Technical Report recommending suitable foundations and safe bearing capacity

Following is the scope of work of Prof. D Babu Rao ,

Testing of soil samples in the Laboratory

Preparation of Technical Report

3. SUB SOIL INVESTIGATION

The sub soil investigation was carried out to determine:

Nature of sub stratum and engineering properties of sub strata which may affect the mode of construction of the proposed work.

FIELD INVESTIGATION PROCEDURE:

The following technique is adopted for sub soil investigations.

a) BORINGS:

Rotary Drilling was done using TC / Diamond bits. The size of the casing used was 125 to 75 mm, yielding samples of NX size.

TC bits were employed for the overburden, and Impregnated Diamond Core bits were used for rock formation.

Drilling was performed on 31Jan/1 Feb ,2016.

The following relevant data was recorded during Rotary drilling operations.

- Nature of strata
- Details of samples

Dr. D. BABU
M.E., Ph.D.(USA)
Consulting Geotechnical Engineer

GEO TECHNOLOGY

- Core Recovery (CR)
- Rock Quality Designation (RQD)

b) STANDARD PENETRATION TEST (SPT):

SPT split spoon sampler of standard dimensions was driven into the soil from the borehole bottom using 63.5 kg hammer with a fall of 75 cm height. The SPT weight was lifted to the specified height and allowed to fall freely on the anvil with the use of cat-head winch with one to one and half turn of the drum. Blow counts for the penetration of every 15 cm were recorded and the 'N' value is reported as the blow counts for 30 cm penetration of the sampler excluding the first 15 cm penetration as seating drive.

When the number of blows exceeded 50 to penetrate the first or second 15 cm length of the sampler, the SPT 'N' is regarded as more than 100 as described in IS 2131 - 1981. The test is terminated in such case and a record of the penetration of the sampler under 50 blows is made. SPT refusal is recorded when there is no penetration of the sampler at any stage and also when a rebound of the sounding system is recorded. These tests were conducted at close intervals of 1.0m so that a continuous SPT 'N' profile is available.

Disturbed soil collected in the SPT sampler was preserved in polythene covers and transported to the laboratory. Additional polythene cover was used to prevent the loss of moisture during the transit period.

c) DEPTH OF BORING: The depth of the Bore hole was as follows:

BH No	Drilled depth
1	3.7 m

GEO TECHNOLOGY

DR. B. BABU RAO
M.E., Ph.D.(USA)
Consulting Geotechnical Engineer

d) LOG OF BORE HOLE:

All the results obtained from the field operations are presented in Log of Bore hole in Fig. 1 .

4. LABORATORY TESTING:

The laboratory tests are conducted in the laboratory of Geotechnolgies, Hyderabad, an ISO- 9000 approved Laboratory.

From GL to 0.7 m, Gravel was seen.

The following tests were conducted on cores from hard rock below 0.7 m depth.:

- Unconfined compressive strength (as per IS: 9143)

Table 1 gives the properties of Cores in hard rock.

5. SUB SOIL PROFILE

Based on Field and Laboratory tests, the following idealized sub soil profile is evolved.

Depth	Strata	N value
0 – 0,7 m	Gravel	-
0.7 – 3.7 m	Hard Rock	Cores

. In Hard rock, no SPT can be conducted. However, in SDR strata, SPT can be conducted with N values tending to be 'refusal'. This is the criterion for distinguishing between Soft rock /Weathered rock and Hard rock.

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6.0 SHALLOW FOUNDATIONS

In general, the following pertains to foundations resting in soils.

. A properly designed foundation has to satisfy the following two limit states.

1) Limit state of collapse (i.e. Shear strength)

2) Limit state of serviceability (i.e. Settlement)

SHEAR CRITERIA:

The first criterion is depends on shear strength. The calculations are based on "TERZAGHI" bearing capacity equation as recommended by IS: 6403 (with factor of Safety) which takes care of L/B ratio (shape), foundation depth etc., along with other parameters.

SETTLEMENT CRITERIA:

The intensity of loading that will cause a permissible settlement or specified settlement of the structure is termed as allowable bearing pressure. The settlement in this type of layer will be elastic settlement.

These foundation settlements are evaluated using elastic theory. The pressure distribution below the footing is assumed as 2 V: 1 H for estimating the settlement. Since rock formation is available at shallow depth. The settlement will be within the permissible limit. Hence open foundation is suitable.

ALLOWABLE BEARING CAPACITY:

Allowable Bearing capacity (ABC) is the net intensity of the loading which the foundation will carry without undergoing settlement in excess of the permissible value for the structure under consideration but not exceeding the net safe bearing capacity (SBC).

7.0 DISCUSSION ON FOUNDATION OPTIONS : From sub soil profile and laboratory test data, it can be seen that hard rock exists from 0.7 m depth . Hence shallow

foundation is feasible and same is recommended.

8.0 RECOMMENDATIONS

Based on Field Investigations and laboratory testing, the following Recommendations are made for construction of 90 KL OHBR – BPT at Vagai Thanda (V), Utnoor (M)

a) Open foundations resting at 2 m below GL ,are recommended. The structure is likely to result in saturation and inundation of the sub soil during long – time operation,

b) SBC is recommended as follows :

Location		BH 1
S. No.	Depth (m)	Recommended SBC t/ sq m
1	2.0	30
2	3.0	35
3	4.0	40

c) The actual size of foundations will be based on loads from the superstructure.

For ANJI DRILLING AND GROUTING WORKS

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Allowable Bearing capacity (ABC) is the net intensity of the loading which the foundation will carry without undergoing settlement in excess of the permissible value for the structure under consideration but not exceeding the net safe bearing capacity (SBC).

7.0 DISCUSSION ON FOUNDATION OPTIONS : From sub soil profile and laboratory test data, it can be seen that weathered rock exists from GL to 4.5 m depth . Hence shallow foundation is feasible and same is recommended.

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TELANGANA DRINKING WATER SUPPLY PROJECT

FIG 1 : Record of Boring, *Bore Hole No : 1*

90 KL OHBR – BPT AT VAGAI THANDA (V), UTNOOR (M) IN ADILABAD DT.

Type of Boring: Core drilling | Dia of Boring: NX

Date : 29 Jan/1 Feb 2016 | Drilled depth : 3.7 m

Depth, m	Profile	Soil	Sample Depth m	N value	CR, %	RQD%
0		Gravel	0			-
1.0			0.7		78	55
2.0		Hard rock	1.7		89	72
3.0			2.7		92	78
4.0						-
5.0						
6.0						
7.0						
8.0						
9.0						
10.0						
11.0						
12.0						
13.0						
14.0						
15.0						
16.0						

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TABLE 1 : RESULTS OF TESTS ON ROCK SAMPLES

90 KL OHBR – BPT AT VAGAI THANDA (V), UTNOOR (M) IN ADILABAD DT.

BH No.	Depth, m	Specific gravity	Porosity %	Water absorption %	UCS Kg / sq cm
1	0.8	2.72	4.0	3.3	422
	1.8		3.9	2.1	465
	2.9		4.2	3.4	460

NOTES : Where core Samples are less than 100 mm long, UCS tests are not conducted.

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APPENDIX

CALCULATION OF SBC

90 KL OHBR – BPT AT VAGAI THANDA (V), UTNOOR (M) IN ADILABAD DT.

TYPICAL CALCULATIONS FOR OPEN FOUNDATIONS AT 2 M DEPTH

As per IS : 12070 -1987,

For q_c = Average uniaxial compressive strength of rock cores

= 400 kg/ sq cm

N = Empirical coefficient =0.4, for discontinuities

Safe bearing pressure $q = q_c N$

= 400 x 0.4 = 160 kg / sq cm = 1600 t / sq m

Considering FS of 10, (in view of likely fissures and fractures in the rock)

SBC = 160 t / sq m

Recommended Safe Bearing Capacity is 30 tonnes per sq m

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