A STUDY OF REINFORCED CONCRETE DESIGN CODE IS : 456-78 AND ITS APPLICATION TO DESIGN OF LIQUID RETAINING STRUCTURES

P-48

Project Work

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Guide

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INTRODUCTION

Tamil Nadu Water Supply and Drainage Board (TWAD BOARD) has proposed and constructed a Multipurpose Service Reservoir of 30 lakhs litres capacity for Trichi Water supply Improvement scheme. The unique features of the proposal is its effective space utilization in the light of growing demand for space. When service Reservoirs are required to be located at the heart of the town, the commercial utilization of that part of land is lost permanently. In order to overcome this deficiency, the TWAD has planned a Service Reservoir of 30 lakhs litres capacity and office cum Commercial floors beneath the reservoir, i.e. Ground and First floor.

The first kind of such construction has been taken up by TWAD and is nearing completion at Trichi. The design of Multipurpose service Reservoir has been worked out based on IS: 456/1964. Since the code has now been revised, the design of Multipurpose reservoir based on new code requirements have been taken up in this project.

SYNOPSIS

Is:456-78, deals with three fundamentally different design principles for the design of structural elements.

1. Limit State Method
2. Working Stress Method
3. Method based on experimental investigations.

The design principles particularly applicable to Liquid Retaining Structures (Service Reservoir) have been chosen for analysis and attempts have been made in this project to evaluate a systematic design approach for the first two of the design methods laid down by Is:456-78.

A comparative study of design principles, theory involved, structural and serviceability requirements and the corresponding code provisions have been studied and presented. The revised version of IS 456-78 and its merits and demerits are also analysed.

Special attention has been given to Limit State Design, since IS 3370/1965 and other codes connected with structural design of Liquid Retaining structures have not yet been modified to suit the design requirement of Limit State Method, which is a new introduction in design procedures. An earnest attempt has been made in this project to study the international standards, code specification of other countries and the application of such provisions to suit the IS 456-78 design requirements in respect of Limit State Design for Liquid Retaining Structures.

The design of a multipurpose service Reservoir of 30 lakhs litres capacity have been considered and model designs using the above two methods have been worked out and presented in this volume.

:GENERAL DESIGN PROCEDURES:

4.1 The traditional approach to the design of R.C.C. members has been based on working stress method. However, the utlimate strength method has gained importance during the period of 1950-1970. From the experience and with wide spread applications, the deficiencies noticed in these two design approaches are now corrected by the adoption of Limit State design. In addition to the above two methods of design, IS 456/78 specifies also a third method based on experimental investigations. (1[illegible].2/IS:456-78)

4.2 IS:456/1964 allows only two methods of Design.

1. Working Stress method
2. Ultimate Strength method.

4.3 In the Revised Version of IS:456/78, three methods have been prescribed.

1. Limit State method
2. Working Stress method
3. Methods based on experimental investigations.

Special emphasis has been made for Limit State method which is a modified form of ultimate strength method.

4.4 Working Stress Design ensures satisfactory performance under service loads and assumes a [illegible] strength against failure, whereas the ultimate strength Design ensures safety of the structure against collapse and assumes satisfactory behaviour at working loads. The Limit State Design is a rationalised ultimate strength method which considers the over all behaviour of the structure at all stages of loading right upto collapse, a check for both safety against collapse and serviceability at Working load.

4.5 A comparative study of these three design methods will lead to the better appreciation of these design methods, highlighting the merits and demerits of each method to choose the optimum one according to requirement and limitations imposed.

4.6 As a Comparision, two Design methods, viz. Working stress and Limit State method, as applicable to Liquid retaining structures, have been chosen and worked out separately.

WORKING STRESS METHOD

5.1 THEORY AND PRINCIPLE:

5.1.1 In elastic stress (modular ratio) theory, the moments and forces acting on a structure are calculated from the actual values of the applied loads, but the limiting permissible stresses in the concrete and the reinforcement are restricted to only a fraction of their true strength, in order to provide an adequate safety factor. In addition to ensure that if any likelihood of failure does occur, it is in a desirable form (eg. due to the reinforcement yielding and thus giving advance warning that failure is imminent, rather than the concrete crushing which may happen unexpectedly and explosively) a greater factor of safety is employed to evaluate the maximum permissible stress in Concrete than that used to determine the maximum permissible stress in the reinforcement.

5.1.2 The modular-ratio method is based on a consideration of the behaviour of the section under service loads only. The strength of concrete in tension is neglected, (except in certain cases in the liquid retaining structures) and it is assumed that for both concrete and Steel, the relationship between Stress and Strain is linear (that the materials behave perfectly elastically). The distribution of strain across a Section is also assumed to be linear (sections that are plane before bending remain plane after bending). Thus the strain at any point on a section is proportional to the distance of the point from the neutral axis and, since the relationship between stress and strain is linear, the stress is also proportional to the distance from the neutral axis. This gives a triangular distribution of stress in the concrete, ranging from zero at the neutral axis to a maximum at the compression face of the section. Assuming that no slipping occurs between the steel and surrounding concrete, the strain in both materials at that point is identical and since the modulus of elasticity 'E' of a material is equal to the stress 'f' divided by the strain 'e' the ratio of the stresses in the materials thus depends only on the ratio of the elastic moduli of steel and concrete. This ratio is known as modular ratio. The value of 'E' for steel is about $210 \times 10^3 N/mm^2$ but for concrete the value of 'E' depends on several factors (the modulus of elasticity of Concrete increases with increase in Cement Content, age, repetition of stress and various other factors.) The actual values range from 500 to 1600 times the Compressive strength.

5.1.3 The internal moment of resistance of a member is assumed to result from the internal resisting couple due to the compressive resistance of the concrete (acting through the centroid of the triangular dist4ibution of compressive stress) and the tensile resistance of the tension reinforcement. The arm of this resisting couple i.e. the distance between the lines of action of the resultant forces is known as the lever arm.

5.1.4 According to modular ratio theory for members reinforced in tension only, each of the ratios involving the depth to the neutral axis (x/d) the lever arm(a) the proportion of reinforcement, the ratio of maximum stress (fst/fc) and the moments of resistance of the section in terms of the maximum stress in concrete or steel and may be expressed directly in terms of each other individual ratio and b,d and m only.

5.2 CODE PROVISION AND REQUIREMENTS

IS:456-1978(clauses 43 to 48) specifies the exact Codel provisions and requirements of Working stress method of structural design under 'Section.6'.

IS:3370 Part II/1967 provides the limitting stresses in respect of Liquid retaining structures.

The permissible stress in Concrete in Direct Tension Compression, Bond and Shear for non Liquid retaining structures are given in clauses 44.1.1, Table 15 and Table 17 of IS:456/78

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The Permissible stresses in Concrete in calculations relating to resistance to cracking for liquid retaining structures for grades of Concrete are given in TABLE I. IS:3370 PtII/1965.

The permissible stresses in Steel reinforcement are given in TABLE-16(IS:456-78) and TABLE-2(IS:3370 PtII-1965) for both Non-Liquid and Liquid retaining structures respectively.

The Torsional provisions are dealt in clause 48 and Appendix C 1-8 of IS:456/78.

The tensile strength of concrete is described in clause 5.2.2. of IS:456/78.

5.2.1 The general design requirements are as follows:-

I. Slabs : Clause C 1.1 IS:456/78

II. For balanced singly Reinforced Rectangular Sections

1. Moment of Resistance: $M = \frac{1}{2}\, b n \sigma_{cb}\ (d-n/3)$

$= 8.75\ bd^2$ for M_{15}

$= 12.00\ bd^2$ for M_{20}

2. Depth Required : $d = 0.338\sqrt{M/b}$ for M_{15}

$d = 0.289\sqrt{M/b}$ for M_{20}

3. Main reinforcement : $Ast = 8.21 \times 10^{-4}\ M/d$ for M.S.Rods

$Ast = 5.22 \times 10^{-4}\ M/d$ for Tor 40

$M = 0.87\ Ast\ \sigma_{st}.d.$

III. Doubly Reinforced and Flanged Sections:

1. Total Moment : $M_1 + M_2$

$M_1 = 8.75\ bd^2$ for M_{15}

(a) Mr. of additional reinforcement. : $M_2 = (M-1)\ Asc\,\sigma\,cb(n-d^1/d^1)(d-d^1)$

$= Asc\,\sigma\,Sc(d-d^1)$

(b) Equivalent M_R of additional reinforcement :

$= 1300\ Asc\ (d-d^1)$ for M.S. Rods.

$= 1750\ ASc(d-d^1)$ for Tor 40.

2. Flanged Sections (T or L sections)

When stress block $\leqslant$ ds

$$M = Bn\ \sigma\ \frac{cb}{2}\ (d-n/3)$$

When stress block $\geqslant$ ds

$$M = Bds\ \frac{(\sigma\ cb + \sigma\ cb^1)}{2}\ jd$$

$$M(flange) = (B-bw)ds\,(d-ds/2)\frac{(cb+cb^1)}{2}$$

3. MR due to flange   Addtional MR $(B-dw)\ ds\left(d-\frac{ds}{2}\right)\frac{(\sigma\ cb+\sigma cb^1)}{2}$

IV. Short axially loaded column:

Total load carrying capacity : $P = \sigma C\ \ Ac\ +\sigma SC\ \ Asc.$

Grade of Concrete (P)	1% Steel		2% Steel		3% Steel	
	Ms	Tor 40	MS	Tor 40	MS	Tor 40
M_{15}	52.6bD	57.1D	65.2bd	74.2 bd	77.8bd	91.3bD
M 20	62.5bD	67 bD	75. bD	84 bD	87.5bD	101.bD

V. Shear

1. Nominal Shear Stress : q = Q/bjd.
2. Allowable shear stress (with nominal shear reinforcement) — 5 KSC M 15; 7 KSC M 20
3. Max. allowable design shear Strength — 20 KSC M 15 ($4q$); 28 KSC M 20 ($4q$)
4. Shear taken by concrete : Nil
5. Shear resistance ~~of inclined bar~~ by vertical Stirrups $Q = \frac{\sigma_{SS}\ A_w\ j^d}{S}$
6. Shear resistance of inclined bar Q = ss A ss Sinα
7. Maximum spacing of vertical stirrups — jd = 0.87 d
8. Minimum area of stirrups : Not specified

VI. Torsion:

1. Shear due to torsion : According to shape of Cross Section
2. Shear reinforcement for resisting entire SF : $q + q^1 \not> 4q$

   If $> 4q$ Redesign the section
3. Provision for : (i) Vertical Stirrups

   (ii) Additional longitudinal reinforcement.

4. Area of Vertical Stirrups : $AW = M_t S / 0.8\sigma_{ss} \times x_1 \times y_1$ (for Rectangular section)

$$AW = \frac{M_t . S}{\sigma_{ss} x_1 \times Y_1}$$ (for tube of box sections)

5. Area of longitudinal bars: Same volume of Steel per unit length of members on the volume of stirrups provided in legs for tension.

5.2.2 The principal design formulae for rectangular and flanged beam are taken from TABLE 114/REYNOLDS HAND BOOK.

5.2.3 The properties of flanged sections are given in TABLE 100/RHB.

5.2.4 <u>Rectangular Panels with triangularly distributed load</u> :

5.2.4.1 Though IS:3370/Part IV/1967 provides necessary design moments for designing the side walls of a liquid retaining structure, it does not provide these moments for "all four sides fixed " and conditions. Therefore, the Bending Moment Coefficients have been referred from Reynolds and Steedman Hand Book.

5.2.4.2 The intensity of pressure on the walls of container is normally uniform at any given level, but vertically it may very lineraly from zero at or near the top to a maximum at the bottom. From the curves (vide/P.225 of Reynold's Hand Book) the probable critical service bending moments on vertical and horizontal strips of unit width can be calculated when the slab is fully fixed or freely supported or unsupported along the top edge.

5.2.4.6 It is advantageous, wherever possible to provide 45° splays at the internal corners of containers and it should be noted that the critical negative bending moments may not necessarily occur at the edges of the splays.

5.2.4.7 A trapezoidally distributed pressure can be considered by adding the MB due to the triangularly distributed load given by the curves to the BM due to a uniformly distributed load. The resulting negative moments are accurate but the positive moments are only approximate.

5.2.4.8 In designing sections to resist the forgoing moments the bending moments on the horizontal span must be combined with the direct tension due to service loads.

5.2.4.9 The final diagram illustrates a typical distribution of vertical and Horizontal moments in a half slab subjected to a triangularly distributed load where the bottom and side edges are fully fixed and the top edge is unsupported. The actual values and distribution of the BM in each direction depends on the ratio of vertical to horizontal span but the general distribution is basically similar in all cases.

5.3 WORKING STRESS DESIGN

The following shall be the individual component of Design.

5.3.1 General calculations and Constants

5.3.2 Design of roof slab of the tank

5.3.3 Design of Side Wall

5.3.3.1 Design of Butresses

5.3.4 Design of floor slab of water tank

5.3.5 Design of office floor slab

5.3.6 Frame analysis

5.3.7 Design of 'T' beams

5.3.8 Design of Column inside the tank

5.3.9 Design of main column

5.3.10 Design of foundation (footing)

5.3.1 General calculations and Constants

Capacity of tank : 30 lakhs litres

= $3000\ m^3$

Length = 42 m

Breadth = 30 m

∴ Storage depth = 3000/42x30 = 2.38 m

Provide 2.40 m. storage.

Total depth:

Storage depth	= 2.40 m.
Scour depth	= 0.15 m.
Free Board	= 0.30 m.
Total	2.85 m

Individual Pannel size = 4 x 4m (PLATE 1)

Constants:

$M = Qbd^2$

To find Q

$Q = \frac{1}{2}\, cjk$

$K = \frac{Mc}{Mc+t}$ $\quad j = (1 - \frac{K}{3})$

For M_{20} st = 1500 ksc, cb = 70 ksc, m = 13

$K = \frac{13 \times 70}{13 \times 70 + 1500} = 0.378$

$j = (1 - \frac{0.378}{3}) = 0.874$

$\therefore Q = \frac{1}{2} \times 70 \times 0.874 \times 0.378$

$Q = 11.55$

'M' value for uncracked section

$$\frac{M}{I} = \frac{f}{y} \qquad M = \frac{fI}{y} = \frac{17 \times \frac{1}{12} \times bd^3}{d/2}$$

$$= \underline{2.83\ bd^2}$$

5.3.2 DESIGN OF ROOF SLAB OF TANK

Size of Panel = 400 mmx400mm

Loads:

Referring IS:456/78 and based on stiffness consideration, assure 15 cm thick slab.(clause 22.2.1)

D.L.	3600 N/m^2	
LL	1500 "	
FF	750 "	
Total	5850 N/m^2	(585 kg/m^2)

The slab is constructed monolithically with beams and columns, it is assumed that slab are continuous on all sides.

L/B = 4/4 = 1

Referring IS:456/78 (Interior Panel Table p.138)

Max -ve BM : $0.032 \times 585 \times 4^2$: 300 kg.m.

Max +ve BM : $0.024 \times 585 \times 4^2$: 225 kg.m.

Using M_{20} Concrete and RTS rods.

Q = 11.55 j : 0.874

Eff.depth d $\sqrt{\frac{300 \times 100}{11.55 \times 100}}$ = 5.096 cm or 50.96 mm.

∴ Provide overall depth : 15 cm or 150 mm

Eff. depth : 13 cm or 130 mm

-ve BM : $\frac{300 \times 100}{1500 \times 0.874 \times 13}$ = 1.76 cm^2 or 17.6 mm^2

+ve BM : $\frac{225 \times 100}{1500 \times 0.874 \times 13}$ = 1.32 cm^2 or 13.2 mm^2

Minimum reinforcement for water retaining structure Clause 7.11 (IS:3370(Pt.II) 1965.

$0.3 - \frac{0.1 \times 5}{35}$

0.280%

0.28 x 15 = 4.29 cm^2 or 42.9 mm^2

Provide 10 mm dia rods at 18 cm c/c (4.36 cm^2) in both directions at bottom, at mid span and crank alternative bars at 1/4 th span on either side and take them to the top at edges to give reinforcement (10 mm dia at 18cm c/c) for -ve BM at top (PLATE 2)

Torisional reinforcement on corners:

Ast = ¾ x 4.36 (C 1.8/p140/IS)
= 3.27 cm^2

Provide 8 mm dia bars at 15 cm c/c along both directions at top and bottom floor at a distance of l/5 = 0.8 m at corners.

5.3.3 DESIGN OF SIDE WALL :(PLATE 3)

Storage depth	:	2.4 m
Scour depth		0.15 m
		2.55 m.

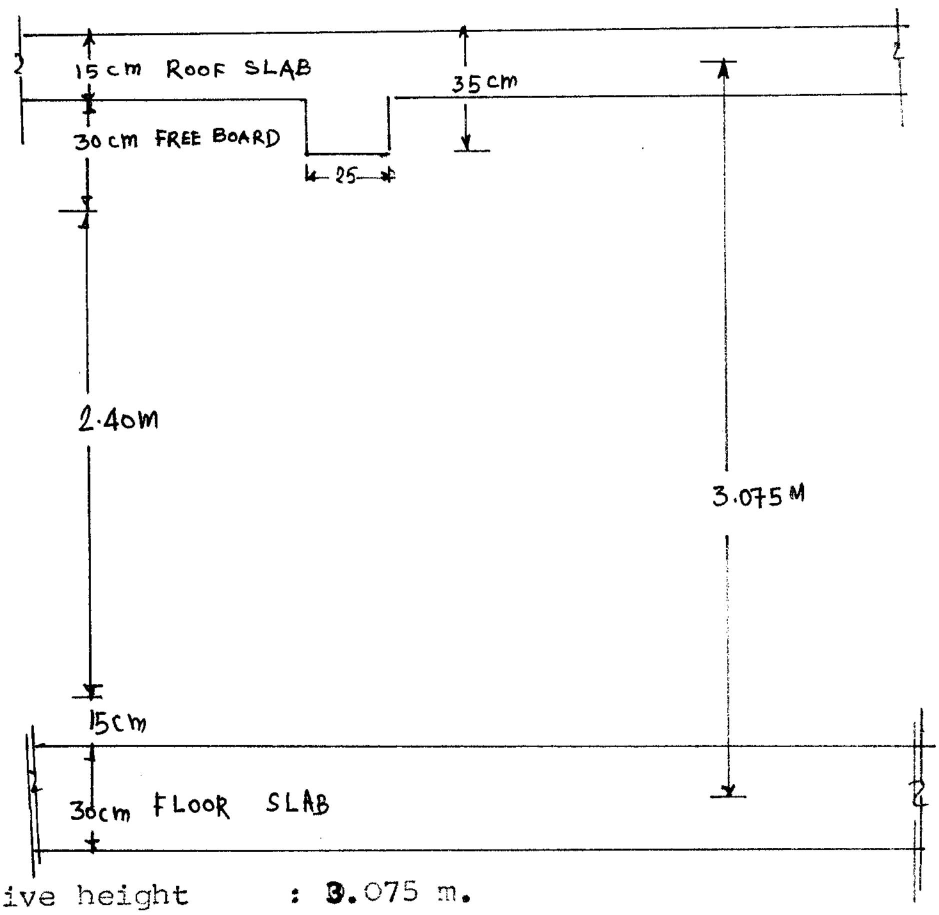
Side wall is designed as wall panels fixed on all four sides in between buttresses and beams.

Assume all Panels as fixed.

Thickness of roof slab : 15 cm.

Roof beam : 20 x 35 cm

Size of floor slab of water tank : 30 cm.



Effective height : 3.075 m.

L_V : 3.075 m.

L_H : 4.0m.

$$\frac{L_H}{L_V} = \frac{4}{3.075} = 1.3$$

Referring Reynold's hand book 1 No 40 for B.M. Coefficient
(fixed on four sides)

Horizontal span:

-ve BM at support 0.013

+ve BM at centre 0.012

Vertical span :

-ve BM at bottom 0.025

-ve BM at top 0.016

+ve BM at VLV 0.016

(W = 1000x2.55 = 2550)

Bending moments:

Max -ve BM in horizontal span: $0.013 \times 2550 \times 4^2 = 531$ kgm

-ve " " $0.012 \times 2550 \times 4^2 = 490$ kgm

Vertical span :

-ve BM(bottom) $0.025 \times 2550 \times 3.075 \times 2.55 = 500$ kgm.

- ve BM(top) $0.016 \times 2550 \times 3.075 \times 2.55 = 320$ kgm.

+ ve BM at VLV $0.016 \times 2550 \times 3.075 \times 2.55 = 320$ kgm.

Max -ve B.M = 531 kgm.

depth required against cracking : $\sqrt{53100/2.83 \times 100}$

$= 13.69$ cm.

Provide overall depth 20 cm and effective depth 16 cm.

Ast $= 53100/0.874 \times 1500 \times 16$

Minimum reinforcement : $0.3 - \frac{0.1 \times 10}{35} = 0.271$ %

$= 0.271 \times 20 = \underline{5.4 \text{ cm}^2}$

Since the moments are -ve, provide reinforcement for +ve & -ve i.e. Provide 10 mm dia rods at 12 cm c/c for all moments.

Vertical stirrups will carry triangular load which will cause Bending moments of 500 kgm and 320 kgm at bottom and top respectively.

$$KA =$$

$$= \frac{a}{3} + \frac{a^2}{3} = \frac{a^3}{3} = -0.103$$

Bottom -ve BM = -0.103x2px3.075 = 500

$$p = 789 \text{ kg/m}^2$$

$$K_B = \frac{a^2}{6} + \frac{a^3}{6} = -0.06$$

BM = 0.06 x 2p x 3.075 = 320

$$\therefore p = 867 \text{ kg/m}^2$$

Average p = $\underline{828 \text{ kg/m}^2}$

Load taken by bottom hor.strip = 2550-828 = $\underline{1722 \text{ kg/m}^2}$

5.3.3.1 DESIGN OF BUTIRESSES : (PLATE.3)

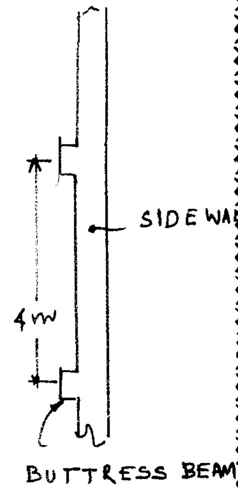
Buttress beams are introduced at 4m span between side wall panels. The beam will span vertically and it will take some amount of water load.and will be constructed monolithic with roof and floor beam of water tank.

W = ½x1722x2.55x4 = 8782 kg.

a = 0.874

V B = $a^2/10(5+29)$ = 0.25

KA = -0.103 KB = 0.06



Ke = γBC = $\frac{-1}{3a^2}(a+e-1)^3+K_B$

= $0.25 \times 0.561 - \frac{1}{0.878^2 \times 3}$

= (0.878+0.561-1)+0.06

Ke = 0.043

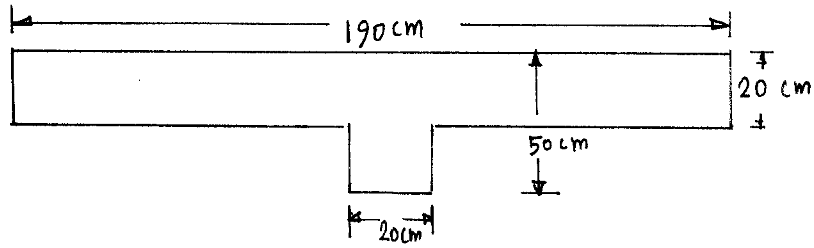
Bending Moments:

Bottom = -0.103x8782x3.075 = 2781 kg.m

Top = 0.0437x8782x3.075 = 1180 kg.m.

Width of flange : $\left\{\frac{P\ 53}{IS456}\right\}\left(\frac{l_o}{6} + bw + 6\ Df.\right)$

$\frac{3.075}{6} + 0.2 + 6 \times 0.2$ = 190 cm.



Assume approximate lever arm = $46 - \frac{20}{2}$ (d_e ds/2)

= 36 cm

Ast for -ve B.M $= \dfrac{278100}{0.874 \times 1500 \times 36}$

$= 5.89\ cm^2$

Provide 3 rods of 16 mm dia.

At for + ve BM $= \dfrac{11800}{0.874 \times 1500 \times 36}$

$= 2.5\ cm^2$

Provide 2 rods of 16 mm dia rods.

Max S.F. 8782 = -0.25 x 8787

= 6587 kg.

Shear stress : 6587/20x36 $= 9.14\ kg/cm^2$ $7\ kg/cm^2$

Provide 20 mm dia bars at 15 cm c/c at top and bottom and 30cm c/c at mid span (PLATE 3)

5.3.4 <u>Design of floor slab of water tank</u> (PLATE 4)

1.	Wt. of water	:	2.55 x 1000	=	$2550\ kg/cm^2$
2.	Wt. of slab	:	30x 24	=	720 "
	Assuming 30 cm thick)				3270

Panel size : 4m x 4m.

Since the slab will be constructed monolic with beams and over all edges.

L/B = 1

Referring IS : 456/1978

Max -ve BM $= 0.032 \times 3270 \times 4^2$ = 1674 kgm.

Max + BM $= 0.024 \times 3270 \times 4^2$ = 1256 kgm.

Depth of slab required against cracking stress consideration } $d = \sqrt{\frac{1674 \times 100}{2.83 \times 100}}$

$= 24.32$ cm.

Provide overall depth of 30 cm and provide effective depth = 26 cm.

$$Ast\ (-ve)\ BM = \frac{167400}{0.874 \times 1500 \times 26} = 4.91\ cm^2$$

$$Ast\ (+ve)\ BM = \frac{125600}{0.87 \times 1500 \times 26} = 3.68\ cm^2$$

Minimum reinforcement : $0.3 - \frac{0.1 \times 20}{35}$: 0.243%

: $0.243 \times 30 = \underline{7.29\ cm^2}$

$\therefore$ Provide 7.29 cm^2 reinforcement for both -ve and +ve bending moments.

Provide 12 mm dia RTS rods at 15 cm c/c (7.53 cm^2) in both directions at mid span at bottom . Crank alternate rods at L/5 distance from support and take them to the top from either side so as to to have necessary reinforcement (i.e. 12 mm dia at 15 cm c/c)at top to carry -ve B.M.

<u>TORSION:</u>

Ast : $3/4 \times 7.53 = 5.65\ cm^2$

Provide 10 mm dia rods at 13 cm c/c along both directions at top and bottom face at a distance of $l/5$ = 0.8 m at corners.

5.3.5 <u>Design of office floor slab</u> : (FLATE 5)

LL	=	400 kg/M^2
DL (15 cmt)	=	360 "
Partition	=	125 "
FF	=	75 "
	=	960 "

Panel size 4 m x 4 m. Referring IS: 456/78,

Max - ve BM = $0.032 \times 960 \times 4^2$: 492 kgm.

Max + ve BM $0.024 \times 960 \times 4^2$: 369 Kgm.

Use M 150 concrete : and tor steel:

$t = 2300\ kg/cm^2$

$c = 50\ kg/cm^2$

$m = 19$

$j = 0.903$

$Q = 6.6$

effective depth, $d = \sqrt{\dfrac{492 \times 100}{6.6 \times 100}} = 8.63$ cm

Hence provide overall depth = 15 cm and eff.depth 13 cm.

Ast (-ve BM) $= \dfrac{49200}{0.903 \times 2300 \times 13}$

$= 1.82\ cm^2$

Ast (+Ve BM) $= \dfrac{36900}{0.903 \times 2300 \times 13}$

$= 1.33\ cm^2$

Mis.reinforcement $= 0.12 \times 15 = 1.80\ cm^2$ (25.5.2.1)

Provide 8 mm bars @ 25 cm c/c in both direction at bottom in the middle span, crank alternate bars at L/5 distance support and take them to the top on either side to [illegible] reinforcement 8 mm dia @ 25 cm c/c to to resist -Ve Bm at top

Torsion:

$$Ast = 3/4 \times 2.0096 = 1.5072 \text{ cm}^2$$

Provide 6 mm rods @ 18 cm c/c along both directions at top and bottom for a distance at $l/5 = 0.8$ m at corners.

5.3.6 Frame Analysis (KANIS METHOD) PLATE 6.

Assume

1. Size of roof beam of water tank 20 x 35 cm.
2. Size of floor beam of water tank 35 x 60 cm.
3. Size of office floor beam 30 x 50 cm.

Flange width:

1. Roof beam of water tank:

$$bo = \frac{lo}{6} + Bw + 6\ Df$$

$$lo = 0.7 \times 4 = 2.8 \text{ m.}$$

$$= \frac{2.8}{6} + 0.2 + 6 \times 0.15$$

$$= 1.56 \text{ m.}$$

2. Floor beam of water tank:-

$$= \frac{2.8}{6} + 0.35 + 6 \times 0.3$$

$$= 2.62 \text{ m.}$$

3. Office floor beam :-

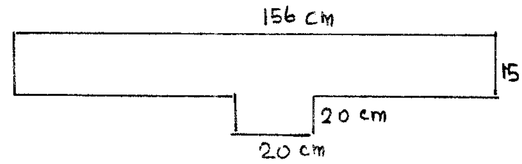
$$= \frac{2.8}{6} + 0.3 + 0.9$$

$$= 1.66 \text{ m.}$$

To find value of Ixx:

1. Roof beam of Water tank:

$\frac{ds}{d} = \frac{15}{35} = 0.43$



$\frac{br}{b} = \frac{20}{156} = 0.13$

Refer Reynolds hand book (P.311)Table : 12

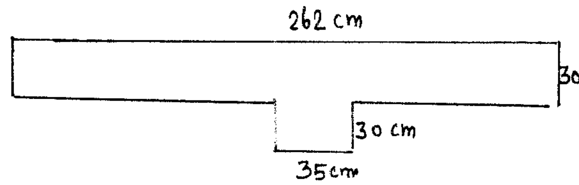
$Cr = 0.187 \quad I = Kbw\ h3$

$I = 0.187 \times 20 \times 35^3 = 160353\ cm^4$

$= 16004 \times 10^4 cm^4$

2. Floor beam of water tank:

$\frac{ds}{d} = \frac{30}{60} = 0.5$



$\frac{br}{b} = \frac{35}{} = 0.134$

from Tables $= Cr = 0.198$

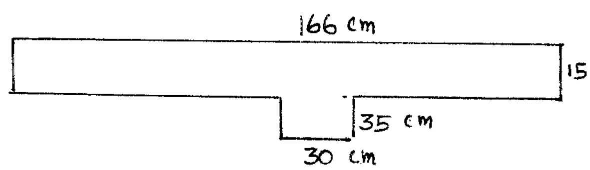
$Ixx = 0.198 \times 35 \times 60^3$

$= 1496880\ cm^3$

$= 149.688 \times 10^4\ cm^4$

3. Office floor beam:-

$\frac{ds}{d} = \frac{15}{50} = 0.3$



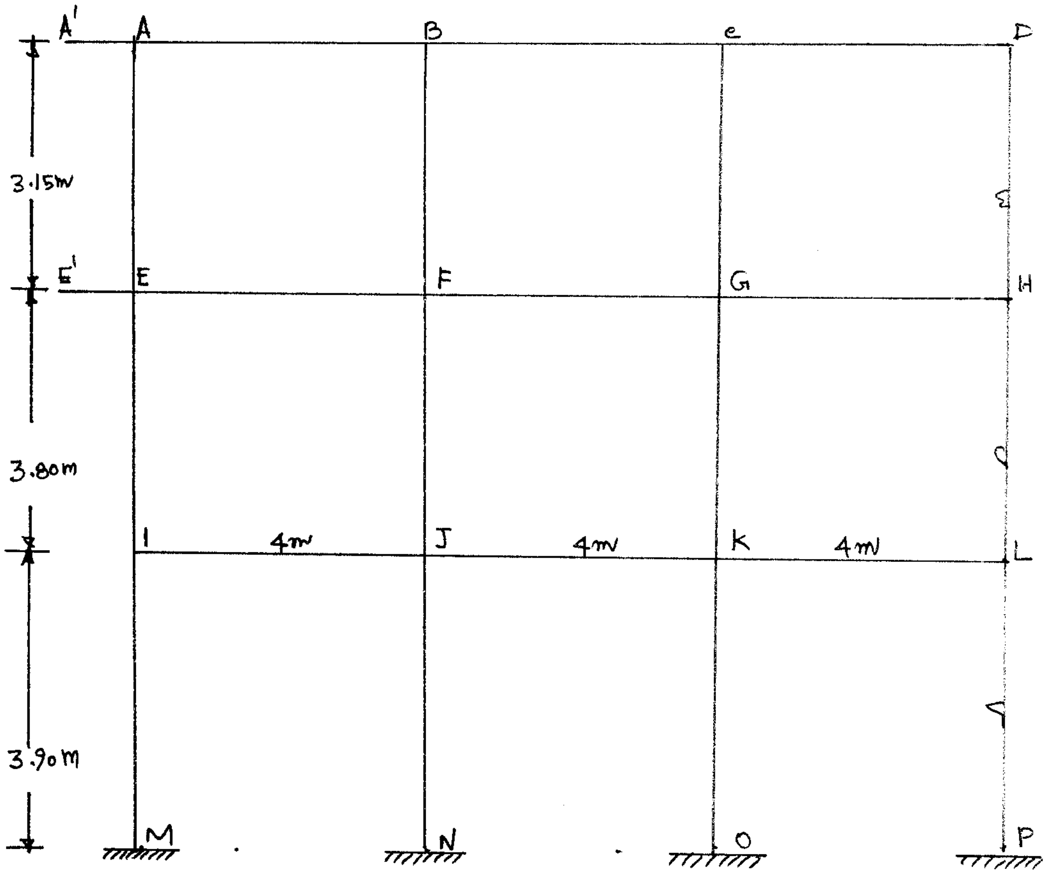
$\frac{br}{b} = \frac{30}{166} = 0.18$

$Cr = 0.164$

$Ixx = 0.164 \times 30 \times 50^3$

$= 615\ 000\ cm^4$

$= 61.5 \times 10^4 cm^4$



Step 1 = To find FEM:-

$$MAA^1 = \frac{585 \times 4 \times 1^2}{2} + \frac{96 \times 1^2}{2} = 1218 \text{ kg.m.}$$

$$-MFAB = MFBA = \underset{\text{(Live)}}{\frac{5\ Wl}{48}} + \underset{\text{(Dead)}}{\frac{wl^2}{12}}$$

$$= \frac{5}{48} \times 1/2 \times 4 \times 2 \times 2 \times 585 \times 4 + \frac{96 \times 4^2}{12}$$

$$= 2078 \text{ kg.m.}$$

End moment EE $= 0.2 \times 2.85 \times 2400 + 0.2 \times 0.3 \times 2.85 \times 2400 \times 1$

$$+ \frac{3270 \times 4 \times 1^2}{2} + \frac{2.52 \times 1^2}{2}$$

$$= 5472 + 144 + 6540 + 5.29$$

$$= \underline{12685 \text{ kgm.}}$$

$$-M_F EF = M_F FE = \frac{5}{48} WL + \frac{wl^2}{12}$$

$$\frac{5}{48} \times \frac{1}{2} \times 4 \times 2 \times 2 \times 3270 \times 4 + \frac{252 \times 4^2}{12}$$

$$= 10900 + 336$$

$$= \underline{11236} \text{ kgm}$$

$$-M_f IJ = M_F JI = \frac{5}{48} \times 960 \times 8 \times 4 + \frac{252 \times 4^2}{12} +$$

$$\frac{0.125 \times 3.8 \times 2000 \times 4^2}{12}$$

$$= 3200 + 336 + 1267$$

$$= \underline{4803} \text{ Kgm}$$

Assume col. inside water tank $= 20 \times 20$ cm.

$$I = \frac{1}{12} \times 20 \times 20^3 = 1.33 \times 10^4 cm^4$$

Assume main column $= 40 \times 40$ cm.

$$I = \frac{1}{12} \times 40 \times 40^3 = 21.33 \times 10^4 cm^4$$

<u>Stepe Rotation factors:</u> Rotation Factor $= \frac{-\frac{1}{2}(Ib/l)}{\Sigma I/l}$

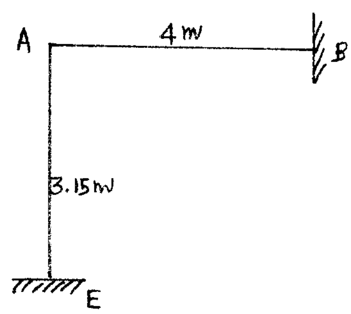
<u>Joint A</u>

$$RAB = -\frac{1}{2} \left\{ \frac{16.04 \times 10^4/4}{16.04 \times 10^4/4 + 1.33 \times 10^4/3.15} \right\}$$

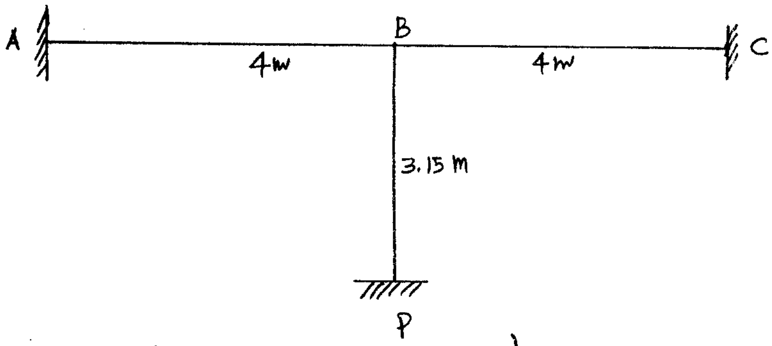
$$= -\frac{1}{2} \left\{ \frac{4.01}{4.01 \times 0.42} \right\}$$

$$= 0.453$$

$$RAE = 0.047$$



Joint B



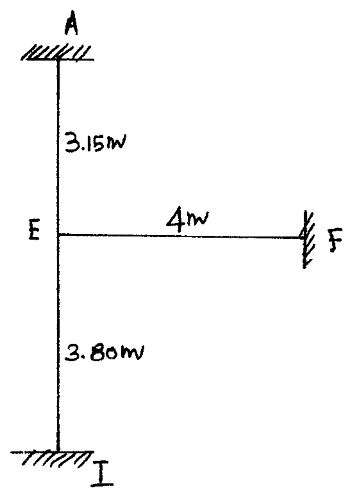
$$RBA = -\tfrac{1}{2}\left\{ \frac{4.01}{4.01+4.01+0.42} \right\}$$

$$= -0.238 = REC$$

$$RBF = -\left\{ 0.5 - (0.238 \times 2) \right\}$$

$$= \underline{0.024.}$$

Joint E



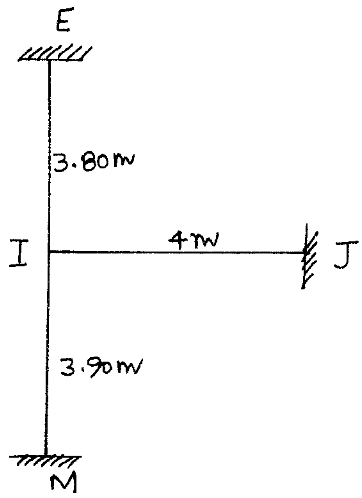
$$IAE = 1.33 \times 10^4 cm^4$$

$$IEI = 21.33 \times 10^4 cm^4 \qquad IB = 149.688 \times 10^4 cm^4$$

$$REF = -\tfrac{1}{2}\left\{ \frac{\frac{149.688}{4} \times 10^4}{\frac{149.688 \times 10^4}{4} + \frac{1.33 \times 10^4}{3.15} + \frac{21.33 \times 10^4}{3.8}} \right\}$$

$$= -\tfrac{1}{2}\left\{ \frac{37.422}{37.422 + 0.42 + 5.613} \right\}$$

Joint I



$I_{EI} \quad = \quad 21.33 \times 10^4 cm^4$

$I_{Ij} \quad = \quad 61.5 \times 10^4 cm^4$

$I_{Im} \quad = \quad 21.33 \times 10^4 cm^4$

$$RIJ = -\tfrac{1}{2} \left\{ \frac{61.5 \times 10^{3/4}}{\frac{21.33 \times 10^4}{3.8} + \frac{21.33 \times 10^4}{3.9} + \frac{61.5 \times 10^4}{4}} \right\}$$

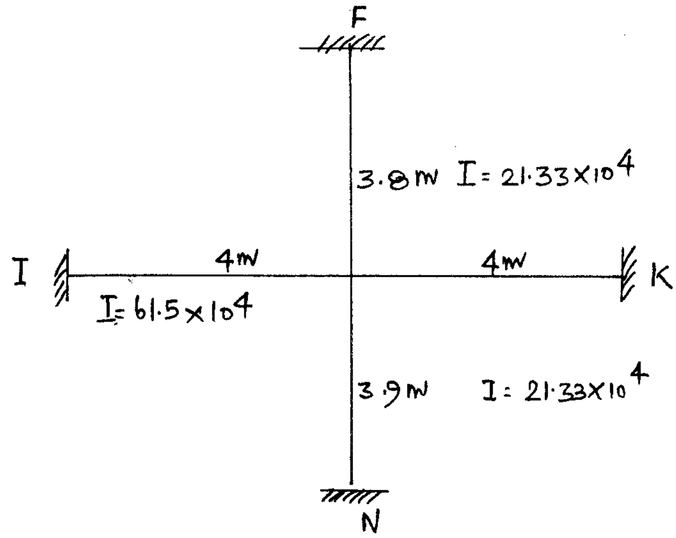
$$= -\tfrac{1}{2} \left\{ \frac{15.375}{5.613 + 5.47 + 15.375} \right\}$$

$$= -\tfrac{1}{2} \left\{ \frac{15.375}{26.458} \right\} \quad = \underline{0.291}$$

Joint B $= -\tfrac{1}{2} \left\{ \frac{5.613}{26.458} \right\} = 0.106$

RIM $= -\tfrac{1}{2} \left\{ \frac{5.47}{26.458} \right\} = 0.103$

Joint J



$$RJI = RJK = -\tfrac{1}{2}\left\{\frac{15.375}{26.458 + 15.375}\right\} = -0.184$$

$$RJF = -\tfrac{1}{2}\left\{\frac{5.613}{41.833}\right\} = -0.067$$

$$RJN = -\underline{0.065}$$

Column moments

Max.moments in the col.inside water tank

$$= 2 \times 167 + 130 = 464 \text{ Kgm.}$$

Max.moment in the column below water tank

$$= 2 \times 1669 + 343 = 3681 \text{ Kgm.}$$

Step 4 Free BM

1. For roof beam of water tank

$$= \frac{WL}{6} + \frac{wl^2}{8}$$

$$= \frac{\frac{1}{2} \times 4 \times 2 \times 2 \times 585 \times 4}{6} + \frac{96 \times 4^2}{8}$$

$$= 3120 + 192 = \underline{3312} \text{ Kgm}$$

2. For water tank floor beam

$$= \frac{3270 \times 32}{6} + \frac{252 \times 4^2}{8}$$

$$= 17440 + 504 = 17,944 \text{ Kgm.}$$

3. For office floor beam

$$= \frac{960 \times 32}{6} + \frac{252 \times 4^2}{8} + \frac{0.125 \times 3.8 \times 2000 \times 4^2}{8}$$

$$= 5120 + 504 + 1900$$

$$= \underline{7524} \text{ kgm.}$$

5.3.7 Design of 'T' Beams

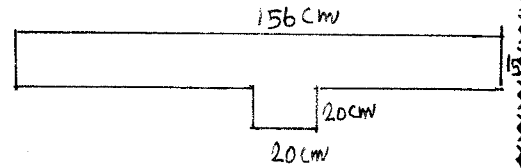
5.3.7.1 For roof beam of water tank (PLATE 2)

Max --ve Bm = 2893 kgm.

Max+ve BM $= 3312 - \left\{\frac{1905+2130}{2}\right\} = 1295$ kgm.

Edge moment at cantilever portion -ve = 1218 Kgm.

<u>Section of beam</u>



Let cover $= 4$ cm.

Assume lever arm $= 31 - \frac{15}{2}$

$= 23.5$ cm.

Ast (+) $= \frac{129500}{23.5 \times 1500} = 3.67\ cm^2$

Provide 2 nos.16 mm rods @ bottom throughout

<u>Section at support</u>

The beam is designed as rectangular beam.

Moment of Resistance of Section

$= 11.55 \times 20 \times 31^2$

$= 2,21,991$ kg.cm.

Actual BM $= 2,89,300$ kg.cm.

BM $>$ RM $\therefore$ design as doubly reinforced beam.

MR to be provided by comp.steel $= 289300 - 221991$

$= \underline{67309}$ kg.cm.

Steel on tension side (top)

$$= \frac{221991}{0.904 \times 31 \times 1500} + \frac{67309}{27.5 \times 1500}$$

$= 5.28 + 1.63 = \underline{6.91\ cm^2}$

Provide 4 bars of 16 mm ϕ

Depth of N.A. $= 0.4 \times 31 = 12.4$ cm.

Area of steel on compression side

$$= \frac{67309}{70 \times 13 \times \frac{12.4 - 4}{12.4} \times 27.5} = 3.97\ cm^2$$

Provide 2 bars of 16 mm ϕ

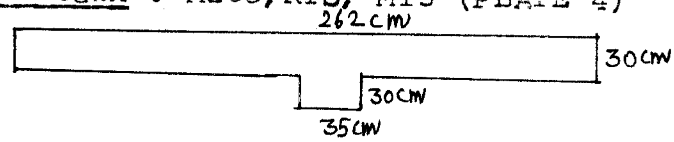
Check for strees

S.F. $= \frac{1}{2} \times 4 \times 2 \times 585 = 1940$ kg.

Stear stress $= \dfrac{1940}{0.904 \times 31 \times 20} = 3.46$ ksc $<$ allowable 7 kgc.

However provide 8 mm ø two legged stirrups @ 20 cm c/c at supports and 30 cm C/c at mid span.

5.3.7.2 Floor beam of water tank : M200, RTS, M13 (PLATE 4)



Max -ve BM = 17022 kgm.

Maz+ ve BM = 17[illegible]4 kgm $- \left\{ \dfrac{100065 + 11592}{2} \right\}$

$= \underline{7116}$ kgm.

at edge beam +ve BM @ support = 8517 kgm.

cover = 5 cm.

∴ lever arm $= 55 - \dfrac{30}{2}$

$= \underline{35}$ cm.

Ast $= \dfrac{851700}{35 \times 1500} = 16.22\ cm^2$

Provide 6 bars of 20 mm ø rods at bottom

Section at support : Section : rectangular.

MR $= 11.55 \times 35 \times 55^2 = 12,22,856$ kgcm.

Actual BM = 1702200 kg cm $>$ 12, 22, 856

∴ Designed as doubly reinforced beam

MR to be provided by comp.steel = 1702200-1222856

$= \underline{479344}$ kgcm.

Assuming 5 cm cover to C/c of comp.steel and tension

$$Ast \text{ at top} = \frac{1222856}{0.904 \times 55 \times 1500} + \frac{479344}{51 \times 1500}$$

$$= 16.4 + 6.27$$

$$= 22.6 \text{ cm}^2$$

Provide 5 bars of 25 mm Ø rods (top)

depth of N.A. $= 0.4 \times 55 = 22$ cm.

$$Ast \text{ (comp)} = \frac{479344}{70 \times 13 \left(\frac{22-4}{4}\right) \times 51}$$

$$= 12.6 \text{ cm}^2$$

Provide 4 bars of 20 mm Ø rods bottom

Check for sheers:

$$\text{Max. S.F.} = 3270 \times 4 + \frac{252 \times 4}{2}$$

$$= \underline{13584} \text{ kg.}$$

Two bars of 20 mm Ø are bent at L/5 distance and taken to the top. Shear taken by the bent up bars $= 2 \times 3.14 \times 1500 \times 0.707$

$= 6660$ kg.

Net shear $= 13584 - 6660 = 6924$ Kg.

$$\text{Shear stress} = \frac{6924}{0.904 \times 55 \times 35}$$

$= 3.98 \text{ kg/cm}^2 < 7$ Ksc O.K

However provide 10Ø mm Ø two legged stirrups @ 20 cm c/c at supports and 30 cm C/c at mid span.

5.3.7.3 Office floor beam (PLATE 5)

Max -veBM = 5540 kgm.

Max -veBM = $7524-\left(\frac{4799+4811}{2}\right)$

2719 kgm.

Lever arm $+\ 46 - \frac{15}{2} = 385$ cm.

Ast $= \frac{2719\times100}{38.5\times2300} = 3.07\ cm^2$

Provide 3 nos. 16 mm Ø rods at bottom throughout

Section at support : shape : rectangular

MR $= 6.6\times30\times46^2 = 4,18,968$ kgm.

MR to be provided by com.st = 554000-418968

= 135032 kgm.

cover = 4 cm.

Steel on tension (top) $= \frac{418968}{0.903\times46\times2300} + \frac{135032}{42\times2300}$

$= 4.39 + 1.4$

$= 5.79\ cm^2$

Provide 4 rods of 16 mm Ø at top

Depth of N.A. $= 0.4 \times 46 = 18.4$ cm.

Ast comp (Bottom) $= \frac{135032}{50\times18\times\frac{18.4-4}{18.4}\times42}$

$= 4.56\ cm^2$

Provide 3 nos. 16 mm Ø rods

Max. S.F. $= 960\times4+252\times2+0.125\times3.8\times2000\times2$

$3840+504+1900 = 6244$ kg.

Shear stress $= \frac{6244}{0.903\times46\times30} = 5.01 \simeq 5$ Ksc. O.K.

However provide 8 mm ϕ 2 legged stirrups @ 20 cm C/c at supports and 30 cm C/c at mid span.

<u>Plinth beam</u>

Assum the size as 30 x 30 cm.

Load from wall = 0.125 x 3.9 x 2000 x 4 = 975 Kg/m

Assum LL = 3200 x 2 = 400 kg/m

Total load = 1375 Kg/m.

Since plinth beam rest on ground

$$BM = \frac{wl^2}{20}$$

$$= \frac{1375 \times 4^2}{20} = 1100 \text{ kg.m}$$

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Provide total depth = 30 cm and eff. = 27 cm.

$$Ast = \frac{1100 \times 100}{0.903 \times 2300 \times 27}$$

$$= 1.98 \text{ cm}^2$$

Provide 4 Nos 12 mm ϕ rods at top and bottom throughout and 8 mm 12 legged stirrups at 25 cm C/c.

5.3.8 <u>Design of column inside tank</u>

Assume 20 x 20 cm.

Self wt. 0.2x0.2x3.15x2400 = 302 Kg.

Load froom roof slab = 586 x 16 = 9360 kg.

Total 9662 kg.

BM = 464 kgm.

Assume 4 Nos of 16 mm ϕ RTS rods.

equivalent area = 20x20+(13-1)x4x2.01

= 496.48

$$\text{Eq.M.I.} = \frac{20^4}{12} + 12 \times 4 \times 2.01 \times 7^2$$

$$= 18061.33 \text{ cm}^4$$

$C'c = \dfrac{10,000}{496.48} = 20.14$ KSC

$C' = \dfrac{46400 \times 10}{18061.33} = 25.7$ KSC

$\dfrac{\sigma'c}{\sigma c} + \dfrac{C'}{\bar{C}} = \dfrac{20.14}{50} + \dfrac{25.7}{70}$

$= 0.675 < 1 \quad \therefore$ Safe.

5.3.9 <u>Design of main column</u>

Load from roof slab = 585 x 16 = 9360 Kg

Load from water tank slab = 3270x16 = 52320 Kg.

Load from office floor 960 x 16 = 15360 Kg.

Load from water tank beam = 96 x 8 = 768 Kg.

Load from wall 0.125 x (3.8+3.9)x2000x8 = 15400 Kg.

Load from office floor beam = 252x8 = 2016 kg.

Load from plinth beam = 0.3x0.3x2400x8 = 1728 kg.

Load from col.inside tank = 302 kg.

Self wt.assuming 40 x 40 col. = 0.4x0.4x(3.8+3.9+1.6)x2400 = 3571 kg.

102841 kgs.

= <u>1,03,000</u> kg.

$\dfrac{l}{D} = \dfrac{3.9}{0.4} = 9.75 < 12$

$\therefore$ Design as short column (IS.24.1.2)

Moment at bottom = 333 kgm.

Assume 40 x 40 col & 12 Nos. 32 mm ∅ rods.

Equivalent area = 40x40+17x12x8.04

= 3240.16 cm^2

Eq.M.I. = $\frac{40^4}{12}+17x(8x8.04x16^2+4x8x04x8^2)$

= 5,28,244 x cm^4

σ C' = $\frac{1,03,000}{3240.16}$ = 31.78 Ksc.

C' = $\frac{33300x20}{52844}$ = 1.26 Ksc.

σ C' + c' = $\frac{31.78}{40} + \frac{1.26}{50}$ = 0.8197 < 1 ∴ Safe.

5.3.10 <u>Design of foundation</u>

The foundation is subjected to a vertical load of 4099 Kg + a bending moment of 21315 kgm. Let the slef weight of foundation be = 410 kg.

Total vertical load = 4099+410 + 4509 Kg.

Let 'B' be the side of the column.

maximum pr at the base will be

$$\frac{W}{B^2} + \frac{M}{B^3/6} = 2$$

$4509+2131500x6x1/B = 2\ B^2$

Provide a base of 2x2m

Net upward distribution variation will be given by

$$P = \frac{4509}{200x200} \pm \frac{2131500 \times 6}{200^3}$$

$$= 0.11 \pm 1.60 = 1.49\ KSC$$

Max. BM at edges at column, (at XY)

$$= 1.49 \times 200 \times 85 \times \frac{85}{2} = 1076525 \text{ kgm.}$$

$$\text{eff.depth} \quad d = \sqrt{\frac{1076525}{9.7 \times 30}}$$

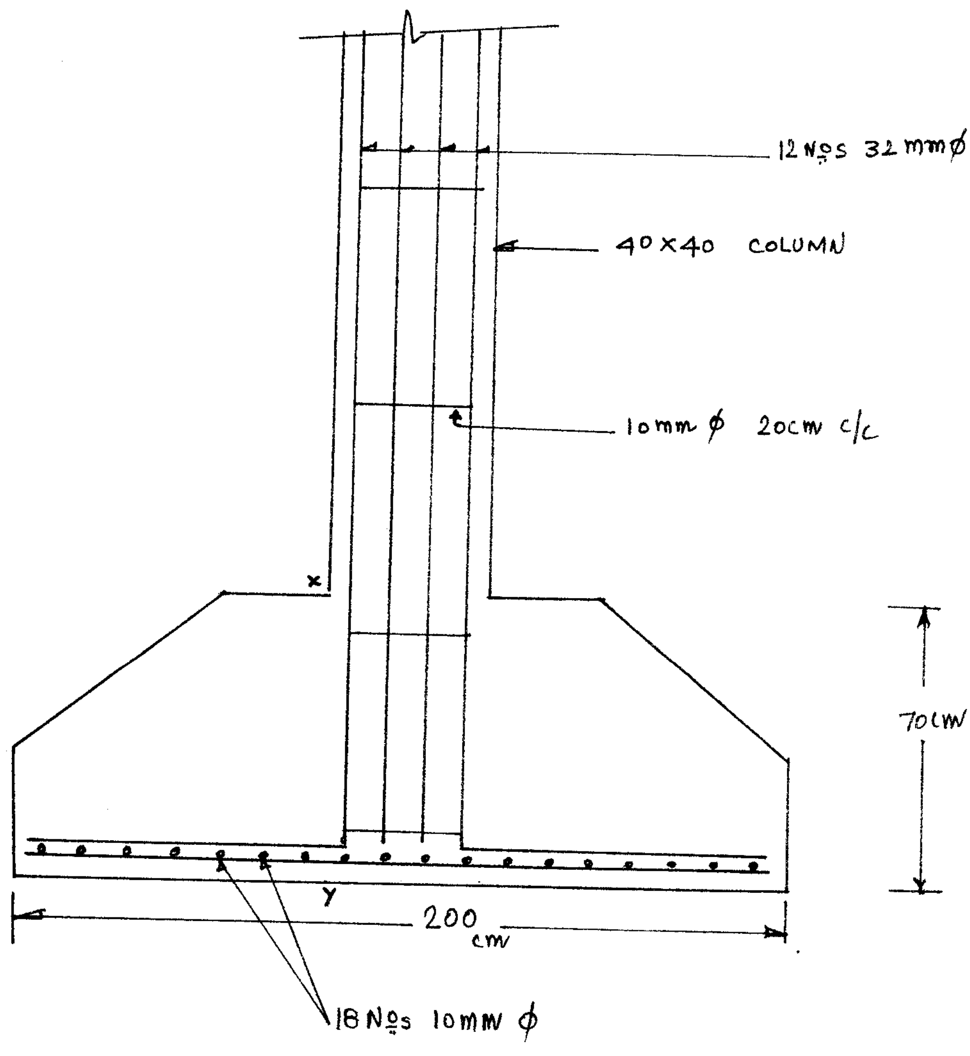
$$= \underline{64}.\text{cm.}$$

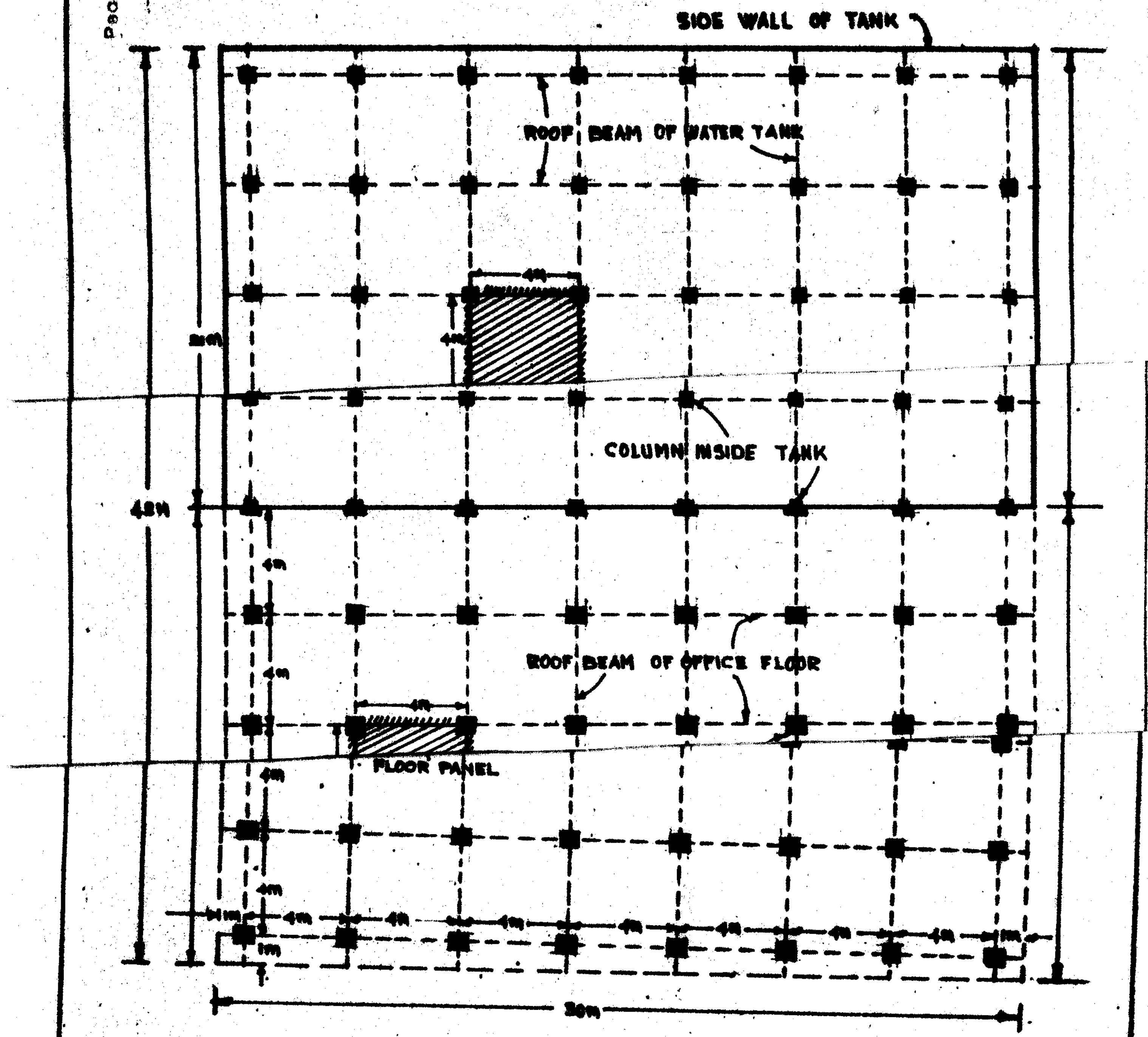
Provide overall depth of 70 cm with effective depth of 65 cm.

$$Ast = \frac{1076525}{0.87 \times 65 \times 1400}$$

$$= 13.59 \text{ cm}^2$$

Provide 18 bars of 100 mm Ø in both the direction.





PLAN (HALF AT TOP AND HALF AT BOTTOM) SHOWING THE ARRANGEMENT OF COLUMN, BEAMS, PANEL ETC.,

PLATE. 1
(NOT TO SCALE)