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A Concise Structural Design Procedure of a Multi-Cell-Tank of a Waste Water Treatment Plant

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Abstract: Waste water treatment plants tanks are invariably built below grade and are constructed of a series of adjacent smaller tanks properly linked to suit the water purification process. Design of such structures under static loadings poses little or no challenge. However, in seismic prone zones the structural design is rather involved since it involves the prediction of all seismic forces induced by the liquid on the walls of the tank comprised of a number of cells in addition to considering the various construction stages. This is a code obligation in order to guarantee that the treatment plant remains watertight under all projected loads and load combinations. It is prudent to mention that such considerations are overlooked at times under the pretense that seismic actions are of minor importance for structures built below grade. ACI 350.3 and ACI 350.06 outline the general requirements of the design of regular water tanks. The following is a detailed numerical account of the design undertaking of an actual project. The structural analysis procedure that follows is brief yet it addresses adequately the fundamental principles involved. Sectional slab or wall reinforced concrete design poses a trivial exercise hence it is not dealt with. The following discourse is limited to vernacular upright rectangular water treatment tank. The Tank's dimensions are relatively small leading to modest magnitude of forces; yet it is the procedure that forms the present focus.

Keywords: Multi-Cell-Tanks; Water Treatment Plants; Impulsive Pressure; Convective Pressure.

I. Problem Statement, Tank's Topology and Material:

The present undertaking presents a structural analysis and design exercise of a 5-cell Waste Water Treatment Plant Tank Structure under static and dynamic loadings with due attention to strength requirements, crack control and durability. Crack width for liquid retaining structure is limited to 0.1 mm. [ACI224-01; table 4.1];. which is achieved by reducing the stresses in the reinforcement bars. The objective is therefore to present a detailed procedure that satisfies ACI requirements. The particular focus is on the application of ACI 350 methodology for the prediction of seismic forces resulting from the internal fluid. Section design is not included within the scope of the present study; however crack control analysis, long term effects on durability and performance remain of prime concern.

The Waste Water Treatment Tank prototype selected is a rectangular one in plan of about 9 meters in length and 4 meters in width; the tank is divided into 5 cells of various widths as shown in Figure 1. The main chamber has a depth of about 3 meters; other smaller cells have a shallower depth of about 2.2 m. The cells are all roof covered and divided by reinforced concrete walls. The entire structure is below grade; it is constructed of reinforced concrete having an f'_c = 28MPa and Reinforcing Steel Bars with F_y= 410 MPa.

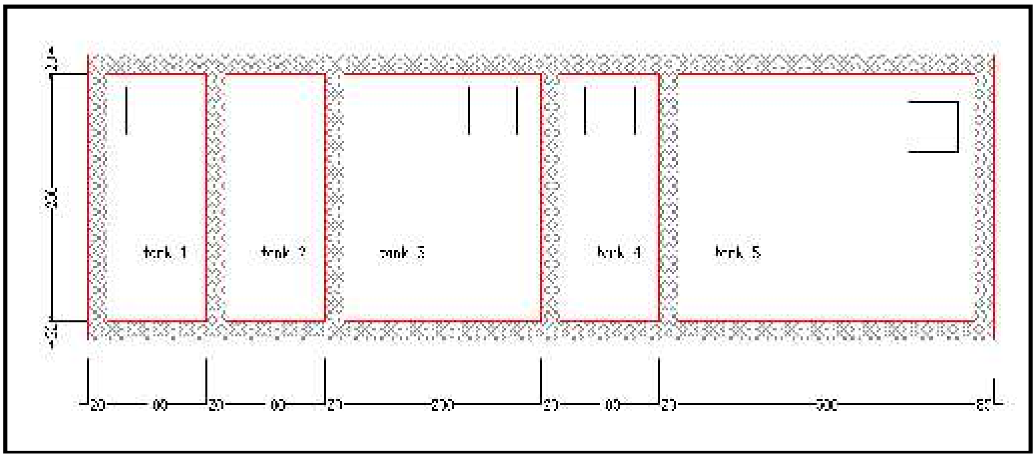


Figure 1: Plan of the Waste water Treatment Tank

II. Methodology:

The structural analysis undertaking is traditionally initiated by carrying out hand calculations for the overall global stability in the two principal directions. Similarly the wall design for elements in the respective direction of analysis follows. Furthermore, the detailed analysis and design for the tank under the various loading scenarios is carried out using the more accurate Finite Element Method techniques. For the exercise the universally acclaimed Extended Three Dimensional Analysis for Building Systems program ETABS2015 is utilized. The analysis and the design procedures are based on the requirements of ACI 318-08, ACI 350.3-01, IBC 2012 and the AISC 7-05.

III. The Site Dependant Seismic Design Parameters:

- Soil Bearing Capacity = 3.0 kg/cm^2
- PGA [peak ground acceleration] = 0.2 g
- Soil Profile S_b
- Seismic Importance Factor = 1.5 [table 4.c]
- R_i = 4 [ACI 350, table 4.d] ; R_c =1
- F_a = 1 [ASCE 7-05, table 11.4-1]
- F_v = 1 [ASCE 7-05, table 11.4-2]
- S_s = 2.50 x 0.2 = 0.50 [mapped maximum considered spectral response acceleration at short periods] Israeli Standard SI413, 202.1.2 -2a-
- S_1 = 1.25 x 0.2 = 0.25 [mapped maximum considered spectral response acceleration at 1 second period] Israeli Standard SI413, 202.1.2 -2b-
- $S_{MS} = F_a\, S_s$ =0.5
- $S_{M1} = F_v\, S_1$ =0.25
- S_{D1} = 2/3 S_{M1} = 0.17
- S_{DS} = 2/3 S_{MS} = 0.34
- $T_s = S_{D1}/ S_{DS}$ [Transitional Period]
- Based on the respective values of S_{DS} and $S_{D1,}$ the Seismic Importance Factor is 1.5

IV. The Hydrodynamic Pressure Distribution

Stresses in the walls of vertical circular cylindrical liquid storage tanks depend primarily on the distribution of the internal fluid pressure. For the static condition the stress distribution from the fluid at rest is linear and poses a trivial case study; such stresses follow a triangular distribution with the maximum value occurring at the base. However, when lateral ground excitation is of importance more involved considerations become indispensible. Hydrodynamic pressure distribution involves two components of pressure, one is called convective which is dependent on the sloshing frequency of water and the other is impulsive which is proportional to the acceleration of the ground motion but independent of the frequency of the fluid motion. Both occur in addition to the hydrostatic pressure distribution. A presentation of such forces is shown in Figure 2. The impulsive component involves the volume of water in the vicinity of the tank's bottom while the convective component involves the upper volume of water because this is the region where the surface dynamic effects on the fluid motion are more pronounced. A number of investigators have solved such a problem for both the rigid tank case as well as the flexible tank case. The mathematical solution to such problems starts by assuming that the fluid is irrotational and inviscid thus the fluid motion is governed by Laplace's Equation. The mathematical solution is beyond the present illustration; however ACI presents values for the two components graphically and parametrically as shown in Figures 3 and 4.

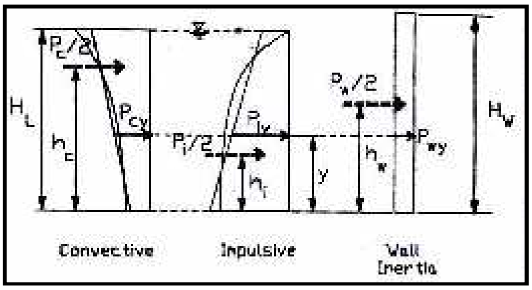
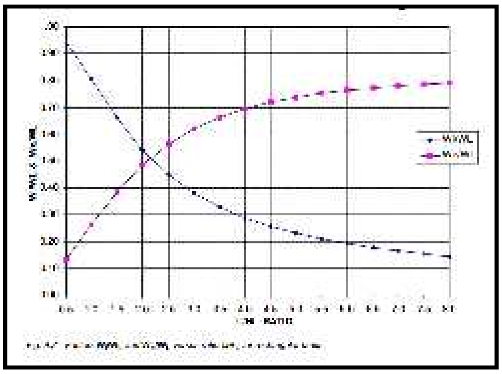


Figure 2: The Impulsive and the Convective Components of the Hydrodynamic Pressure [ACI 350.03. Figure R5.5]

In summary the internal hydrodynamic forces are comprised of the following two components:

1) The impulsive component which represents the portion of the fluid that moves in unison with tank's structure.
2) The convective component which represents the effect of the sloshing action of the contained fluid.

For reinforced concrete water storage tanks which, for all practical purposes, are generally considered rigid, this amounts to a single degree of freedom system associated with the first eigenvector. Further supporting the argument for using the procedure of Equivalent Lateral Load Method.



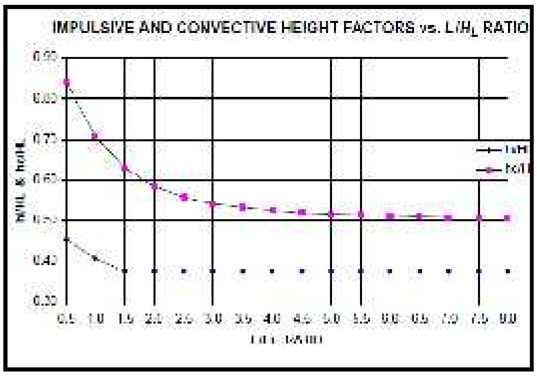
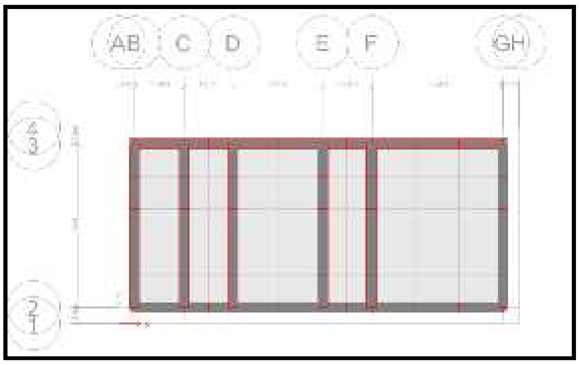


Figure 3 **Figure 4**

Figure 3: Impulsive and convective mass factors [ACI 350.3R]

Figure 4: h_i/H_L and h_c/H_L vs L/H_L

V. The ETABS Numerical Model:



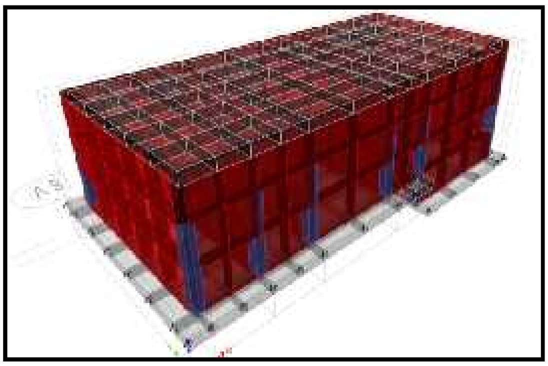


Figure 5: Plan of the Numerical Model

Figure 6: 3D Extruded View of the Model

The numerical model shown in Figures 5 and 6 above is made by ETABS2015. It is comprised of an assemblage of shell elements and column line elements. The columns are 25x25, situated at the wall junctions and corners, meant for better load transfer while roof slabs are defined as shell elements of 27 cm thickness. The wall thickness of 30 cm is the recommendation of the ACI code yet it is valid when no elaborate analysis is carried out. Similar elements are defined for all walls and the grade slab except that the later are built of shell elements of 35 cm thickness. The foundation is prescribed as pin supports at the nodes in order to result in a more conservative design; the foundation slab on grade extends 40 cm from all sides; its planar dimensions are decided by the underlying soil pressure under service loading conditions. A cracked section factor of 0.25 is introduced in all shell elements.

Conducting Modal Analysis revealed, as expected, that the structure is quite rigid with a fundamental frequency of about 55 Hz and a period of 0.018 seconds. This is coupled with no torsion modal shapes which rendered the structure as a regular one. Considering that the period is considerably less than 3.5 T_s as stipulated

by the IBC [T_s being the transitional period], it paves the way towards the adaptation of the Equivalent Lateral Load Method of analysis in order to address seismic loadings. The exercise is carried out in the vulnerable weak direction.

VI. Loading Patterns Considered:

- A Roof Live Load of 5 KN/m^2
- A Lateral Earth Pressure Load assuming [$\gamma_{soil} = 2{,}000$ kg/m^3]
- The Hydrostatic Pressure resulting from the contained fluid
- The Added Masses
- ELLM applied in the x-direction

For a tank structure it is prudent to consider sequential construction scenarios; each stage analyzed under the relevant loading scenario. The design is eventually based on the enveloping force effect.

VII. Load Combinations for the Ultimate Delivery Scenario:

Standard load combinations are defined as shown below. However, an additional safety factor S_d is introduced as per the ACI requirements; this is an industry recommended practice. S_d does not apply when a combination includes earthquake effects The following are the relevant load combinations according to ACI 350-06:

- Comb1 = 1.4 (DL + FL)
- Comb2 = 1.2 DL + 1.3 FL + 1.6 (LL + HL)
- Comb3 = 1.2DL + 1.3 FL + 1.0 (EQX+0.15DL) + 1.6 H + 1.0 LL
- Comb4 = 0.9 DL + 1.3FL + 1.6 HL
- Comb5 = 0.9 DL + 1.3 FL + 1.0 (EQX + 0.15DL) + 1.6H
- Comb6 = DL + LL + FL [service load condition]
- Comb7= Env (S_d.Comb1,S_d.Comb2,Comb3,S_d.Comb4,Comb5)

[DL: Dead Load; LL: Live Load; FL: Fluid Load; H: Lateral Soil Pressure Load; EQX is the Equivalent Lateral Static Load applied in the x direction]

For vertical ground acceleration the following is considered:

$$E = \rho E_h + E_v$$

Where ρ is a redundancy factor, to increase the effects of earthquake loads on structures with few lateral force resisting elements. It is given by the following when ρ = 1.

$$E = E_h + 0.15 \text{ DL}$$

.

It is prudent to mention that a Sanitary Coefficient, $S_d = 1.45$ for flexure is selected and included in the load combinations. The following is the computation of S_d

$S_d = \varphi\, f_y/\gamma\, f_s$ ACI 350-06
$\varphi=0.9$
$\gamma = 1.4$
$f_s = 1794\ x\ 25.4/\beta\sqrt{s^2}\ +\ 4(50 + db/2)^2$
Re-bar used are of 12 mm diameter at 15 cm c.c.
$S_d = 0.9x\ 410/1.4x180$
$= 1.45$

VIII. The Convective and the Impulsive Mass Components:

ACI 350 give the ratio of the convective mass and the impulsive mass as a function of the ratio of the lateral width to the total liquid depth, Figures 3. The respective heights at which these masses act are given in Figure 4. Because the cells are rectangular in plan, the impulsive and the convective parts of the total mass are thus different in the x and the y directions as well as in the different tank cells. The water mass in Cell 1= 8.36 t, in Cell 2=8.36 t, in Cell 4 = 8.36t, in Cell 3 =16.72 t and in Cell 5 = 34.2 t. The total fluid mass is 760 KN. The following are calculations made for the y- direction:

For Cell #1:
The impulsive and the convective masses in the y-direction are as follows:

W_L= 1x3.8x2.2x10 = 84 KN
L/H_L= 3.8/2.2 = 1.72 [Figure 9.2 / ACI 350.3]
$W_i/W_L = 0.6$; $W_c/W_L = 0.4$
Wi = 50 KN
Wc =34 KN

The elevations at which the masses act are computed with the aid of Figure 9.3 [ACI 350.3]

$h_i/HL = 0.37$; $h_c/ H_L = 0.6$
$h_i = 0.37x2.2 = 0.8$ m
$h_c = 0.6x2.2 = 1.32$ m

Similarly the values in the x-direction are computed:

W_L= 84 KN
$L/H_L = 1/2.2$
$Wi/W_L = 1$; $Wc/W_L = 0$
Wi = 84 KN
Wc = 0

The impulsive and the convective masses in both directions are computed for all cells. The results are summarized in Table 1 below. It is readily observed that in the x-direction the impulsive mass is appreciably larger than the convective mass, which is not the case in the y-direction. However, in both directions the impulsive mass is invariably dominant. Furthermore, this conclusion acts as a further confirmation that the x-direction of the constructed numerical model with the added masses representing the impulsive masses is adequate. Masses are equally divided at the nodes as ACI 350.3 does not specify the manner of impulsive mass application.

Total weight of walls 710 KN [assuming $\gamma_{\text{concrete}} = 2.5$ t/ m^3]
Weight of Roof 243 KN
Total weight = $[W_w + W_i + W_{roof}]$ =1443 KN

In order to simplify calculations a weighted average height of tank is computed to be 2.45m. This is limited for period and stability calculations only.

	X- direction				Y- direction			
Tank #	Impulsive Mass [KN]	$h_{i\ [m]}$	Convective Mass [KN]	$h_{c\ [m]}$	Impulsive Mass [KN]	$h_{i\ [m]}$	Convective Mass [KN]	$h_{c\ [m]}$
1	84	1	0	0	50	0.8	34	1.32
2	84	1	0	0	50	0.8	34	1.32
3	134.4	0.88	33.6	1.54	100	0.8	68	1.32
4	84	1	0	0	50	0.8	34	1.32
5	273.6	1.2	68.4	2.1	240	1.25	102	2.2
Total	660		102		490		272	

Table 1: Impulsive and Convective Masses in the x and y directions

IX. Stability Analysis:

a) Stability Check in the y- direction:
Stability against Sliding

Base Shear due to the impulsive mass
$Vi = Csi\ (W_w + W_{roof} + W_i)$
$Csi = S_{DS}\ I/\ Ri = 0.34 \times 1.5 /4 = 0.1275$
$Vi = 0.1275 \times 1443 = 184$ KN
$Csc = S_{DS} \times I/Rc = 0.51$
$Vc = 0.51\ (3x34 + 68 + 102) = 138.7$ KN

Total Base Shear = $\sqrt{(184^2 + 138.7^2)} = 230$ KN

The available sliding resistance that includes the weight of the grade slab while assuming a coefficient of friction of 0.6:
= 0.6 x [1443 + (9.8 x 4.8 x 0.35 x2.5 x 10) =1113 KN

This implies that the tank is stable under the shear action in the N-S global direction.

Stability Under Overturning Moment:

$Mi = Csi\ (W_w h_w + W_{roof}\ h_r + W_i h_i)$
= 0.1275 (710x 1.22 + 243x 2.45 + 3x50x1.05+ 100x 1.05 + 240x1.25)
= 258 KN-m
$Mc = Csc\ (W_c \times h_c)$
= 0.51x [34x3(1.32+0.25) + 68 x(1.32+0.25)+ 102x(2.2)]
= 250 KN-m
$M = \sqrt{Mi^2 + Mc^2}$
$=\sqrt{258^2+250^2} = 359$ KN-m
Resisting moment
= Weight x width/2
= (1443 +412) x 2 = 3710 KN-m
The tank is safe against global overturning in the N-S direction.

a) Stability Check in the x- direction:

Weight of wall: 710 KN
Weight of roof: 243 KN
Total weight: 660 +710 +243 = 1613 KN

Stability against Sliding

Base shear $= Csi\ (W_w + W_{roof} + W_i)$
$= 0.1275 \times 1613 = 206$ KN
$Vc = Csc \times Wc$
$= 0.51 \times (33.6 + 68.4) = 52$ KN

Total base shear $\sqrt{206^2 + 52^2} = 212$ KN
Sliding stability
Weight of grade slab = 9.8 x 4.8 x 0.35 x2.5 x 10
= 412 KN
Available Resistance = 0.6 (1613+ 412)
1215 KN > 212 KN [safe]

Stability under Overturning Moment

$Mi = Csi\ (W_w h_w + W_{roof} h_r + W_i h_i)$
$= 0.1275\ [710 \times 1.22 + 243 \times 2.45 + 3 \times 84 \times 1.25 + 134.4 \times 1.05 + 273.6 \times 1.45]$
= 295 KN-m
Mc = 0.51 [33.6 (1.54 +0.25) + 68.4 (1.54)]
= 84 KN-m
$M = \sqrt{295^2 + 84^2} = 306$ KN-m
The Resisting Moment is:
(1613 +412) x 4.5 = 9112 KN-m
The tank is therefore stable against global overturning moment in the E-W direction.

X. SCENARIO STUDIES

1) **Scenario 1:**
This scenario represents the leakage test. All cells within the tank are without a roof yet they are full of fluid but the tank is without backfill. The fluid in the tank is placed in alternate cells in order to create a more severe loading condition. The analysis is at rest hence no added masses of any sort.

2) **Scenario 2:**
Here the tank is empty of fluid, with no roof but has backfill material pressure applied at the four exterior walls. The applied soil pressure due to fill is saturated.

3) **Scenario 3:**
The tank is with a roof; it is full of fluid but with no backfill around

4) **Scenario 4:**
The tank is with a roof, no fluid inside yet backfill is placed all around.

5) **Scenario 5:**
The tank is with a roof, no applied live load, subjected to full hydrostatic load, no soils pressure and EQ load applied in the x- direction only.

6) **Scenario 6:**
The tank is full of fluid, with live load on its roof, with backfill all around, closed and subjected to EQ load in the x- direction.

Scenario	Deflection mm	Mxx KN-m/m Mid span	Mxx KN-m/m At edges	Myy KN-m/m Mid span	Myy KN-m/m At edges
1	2	6.3	10	4.4	12.8
2	0	1.2	2.09	0.9	2.5
3	0.1	4.2	8	5.6	11.2
4	0	0.8	1.7	1.1	2.3
5	1	8.23	10.8	10.8	15.8
6	2	7.9	7.4	10.7	19

Table 2: Resulting Forces as Computed by ETABS 2015

XI. Cracked Section Considerations:

The cracked section analysis is performed by SAFE12 of Computers and Structures Corporation. For the undertaking the Dead Load, a Super Dead Load and a Live Load are utilized. Initially an immediate non linear all load case for a cracked section is defined in order to investigated the immediate deflection under service loads combination. For the short term deflection the deflection is 1.15 mm while for the immediate all loads case the deflection is 1.2 mm; the difference is rather small. For the long term deflection which includes creep and shrinkage a long term sustained cracked load case is defined based on 25% of the live load in addition

to an immediate all loads case. All deflection values are very small. This is by virtue of the relatively small size of the structure.

The estimated crack width according to the following Gergely-Lutz Equation

$$w = 0.076\ \beta\ b_h\ f_s\ (d_c A)^{1/3}$$

is 0.1 mm provided the reinforcement bar diameter is 12 mm; β is the ratio of the distance from the neutral axis to the tension face of the member to the distance to the centroid of the tensile reinforcement.

XII. Conclusion:

The present brief structural design exercise for the Waste Water Treatment Plant results in modest force magnitudes resulting from the relatively small induced forces; hence limited reinforcement steel ratio. This is due to the relatively small overall dimensions of the structure and the fact that such tanks are normally built below grade. Crack width is traditionally controlled by proper steel reinforcement details and small bar sizes. The cited example is a recent prototype of an actual small scale tank constructed in the Palestinian Territories. Larger scale structures inevitably lead to larger scale forces, however; buffer walls remain effective for controlling sloshing effects.

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