LIQUID RETAINING STRUCTURES

BASIC CONSIDERATIONS

SCOPE: Storage tanks, reservoirs, swimming pools, elevated tanks, ponds, basement walls etc.

GENERAL DESIGN OBJECTIVE

The structure designed to retain liquids must fulfill the requirements for normal structures like STRENGTH, DURABILITY, LIMITED DEFLECTION and CRACKING. In addition, the liquid should not be allowed to LEAK or PERCOLATE though concrete structures.

FUNDAMENTAL REQUIREMENTS

The requirements for the structure without UNDUE MAINTENANCE and ADEQUATE CONCRETE COVER to reinforcement are essential. Potable water from the moorland areas may contain free carbon dioxide or dissolved salts, which attack normal concrete. Similar difficulties may occur with tanks, which are used to store sewage or industrial liquids. The concrete must be of good quality and it may be necessary to use increased cement contents or special cements.

FUNDAMENTAL DESIGN METHODS

Historically the liquid retaining structures have been designed by elastic theory for working loads. More recently limit state methods have been introduced, providing a more realistic basis for determining factors of safety.

The liquid retaining structures designed by elastic theory are subjected to so small material stresses that no flexural cracks are developed. But this is achieved at the cost of too thick sections with excessive amounts of reinforcement. The probability of shrinkage and thermal cracking is not dealt with properly.

The designer has choice to use either of the methods. The limit state design is more logical and some saving in materials can be achieved. The elastic design is simple to carry, but there should be no problems in using limit state theory as design chart and tables are available.

BASIS OF DESIGN

STRUCTURAL ACTION

All liquid retaining structures are required to resist horizontal forces due to liquid pressures. There are two ways in which the pressures can be contained:

i) By forces of direct TENSION or COMPRESSION

ii) By flexural resistance.

Structures designed using tensile or compressive forces are normally circular and may be prestressed. Rectangular tanks or reservoirs, on the other hand, are designed using flexural action as cantilever, propped cantilever walls or walls spanning in two direction.

Structural element acting in flexure to resist liquid pressures reacts on the supporting elements and causes direct forces to occur.

STRUCTURAL LAYOUT

The lay out of the proposed structure and the estimation of member sizes must be made prior to detailed analysis. Structural schemes should be considered from the viewpoints of STRENGTH, SERVICEABILITY, ease of CONSTRUCTION and COST. It may be noted that sudden changes in sections must be avoided because they enhance the possibility of cracking

It is preferable to design cantilever wall as tapering slabs rather then as counterfort walls with slabs and beams. It is essential for the designer to consider the method of construction and to specify on the drawings the location of all construction and movement joints. Important considerations are the provision of KICKERS against which formwork may be tightened, and the size of wall and floor panels to be cast in one operation.

LOADING

Liquid retaining structures are subjected to loading by pressure from the retained liquid. These values can be obtained readily from any handbook. The designer must consider whether sections of the complete reservoir may be empty when other sections are full, and design each structural element for the maximum bending moments and forces that can occur. Several loading cases may have to be considered. Internal partition walls should be designed for liquid loading on one side only.

External reservoir walls are often required to support soil fill. When the reservoir is empty, full allowance must be made for the active soil pressure and any surcharge from vehicles. It may be noted that when reservoir pressure is considered with the reservoir full, no RELIEF is allowed from passive pressure of the soil fill.

FOUNDATION

It is desirable that a liquid retaining structure is founded on good uniform soil, so that differential settlements can be avoided. On sites with non-uniform soils, it may be, necessary to consider DIVIDING the structure into completely separate section.

The use of cantilever walls depends on passive resistance to the applied pressure, resistance to sliding being provided by the foundation soil. If the soil under the foundation is inundated by ground water, it may be possible to develop the necessary soil pressure under the footing. In these circumstances, a cantilever design is not appropriate, and the overturning forces should be resisted by a system of beams balanced by the opposite wall. Or the walls must be designed as spanning horizontally if possible.

FLOTATION

As empty tank constructed in water bearing soil will tend to move upwards, in the ground or float. Ensuring that the weight of the empty tank is greater than the uplift must counteract this tendency. The factor of safety varies between 1.05-1.25. The weight of the tank may be increased by thickening the floor or by providing a heel on the perimeter of the floor.

RECTANGULAR PANELS WITH TRIANGULARLY DISTRIBUTED LOADS

The intensity of pressure on the walls of containers is uniform at any given level, but vertically may very from zero near the top to a maximum at the bottom. If there is a support along the top of a rectangular panel spanning in two directions, the curves and expressions in the Chart 53, which are based on elastic analyses, enable the probable maximum bending moment on vertical and horizontal strips of unit width to be calculated whether the slab is fixed (Case 3) or freely supported (Case 2) along the top edge. Similar curves are given in the Case 1 of Chart 53 for the condition when there is no support along the top edge. In all cases it is assumed that the slab is continuous over the two vertical edges and fixed along the bottom edge.

At a nominally freely supported top edge (Case2), resistance to negative bending moment equal to two-thirds of the positive bending moment in the vertical span should be provided.

If the slab is assumed to span entirely vertically or entirely horizontally, the amount of reinforcement provided horizontally and vertically respectively, and in other cases at sections where the calculated bending moment is small, should be not less than the minimum amount required in a slab. Since it is common in a container to provide 45^o splays at the corners, it should be noted that the critical negative bending moments are not necessarily at the edges of the splays.

A pressure, which is distributed trapezoidally, can be dealt with by adding the bending moment due to a triangularly distributed load (Chart 53) to the bending moment due to a uniformly distributed load (ACI Coefficients). This applies to the negative bending moments exactly, but only approximately to the positive bending moments.

Example 1: Find the maximum service bending moments in a wall panel of a rectangular tank that can be considered as simply supported along the top edge and continuous along the bottom edge and along the two vertical sides. The height l_z of the panel is 10' and the horizontal span l_x =15'. The intensity of pressure is 625 lb/ft width along the bottom edge and decreases uniformly to zero at the top edge.

Ratio of spans: $k = l_x/l_z = 15/10 = 1.5$. For case 2 on the Chart 53-2, the bending moments are as follows:

Maximum Positive Moment in Vertical Span: $0.023\times625\times10^2=1437.5$ lb-ft. This occurs at $0.45\times10=4.5$' height. (Read λ_1 and C^+_V)

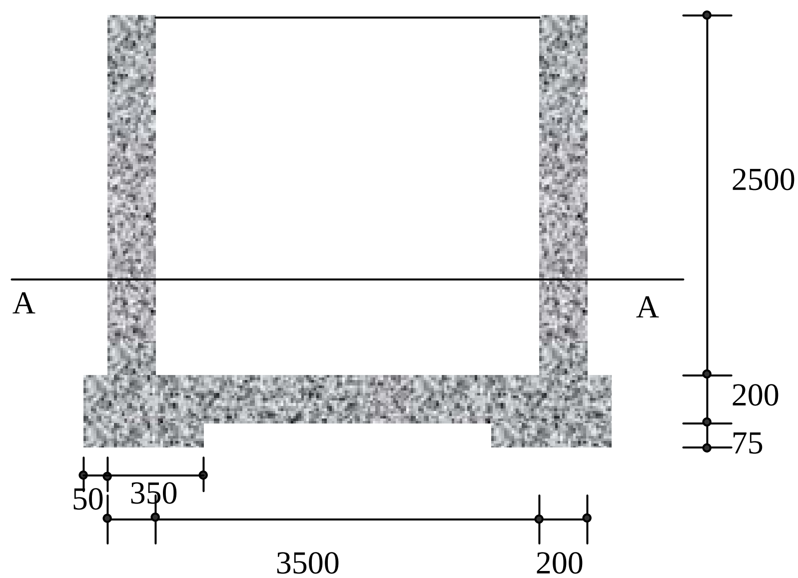
Maximum Negative Moment in Vertical Span: $0.042\times625\times10^2=2625$ lb-ft, which occurs at midpoint along the bottom of panel. (Read C^-_V)

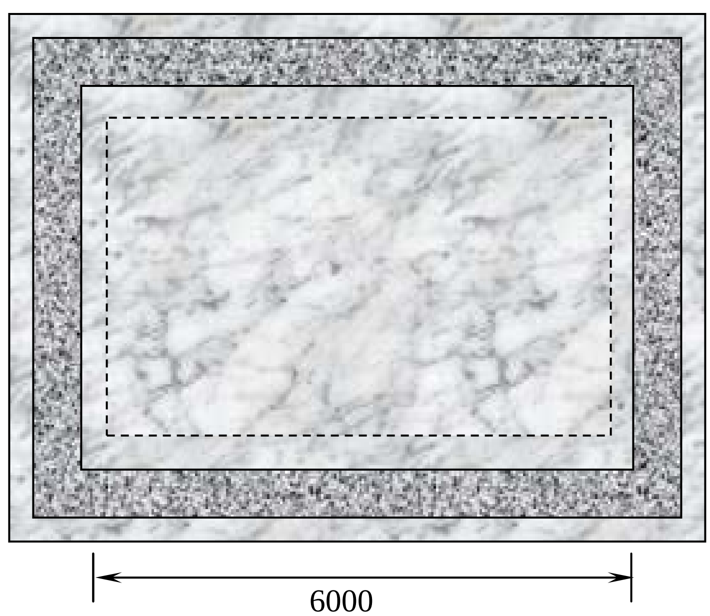
Maximum Positive Moment in Horizontal Span: $0.0045\times625\times15^2=632.8$ lb-ft, which occurs at midspan and height $\lambda_3 l_z = 0.48\times10=4.8$' (Read λ_3 and C^+_H)

Maximum Negative Moment in Horizontal Span: $0.011\times625\times15^2=1546.9$ lb-ft, which occurs at height $\lambda_2 l_z = 0.405\times10=4.05$' at supports of horizontal span.(Read λ_2 and C^-_H)

(The above moments are calculated for a 12" wide strip of wall.)

Example 2: Design of Rectangular Water Tank



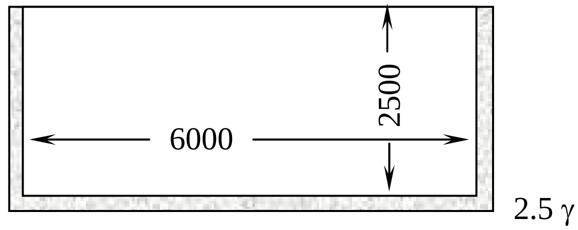


Rotated Section AA

Material Properties: f'_c=30 MPa, f_y=300 MPa

Design of Long Wall

$k = \frac{l_y}{l_z} = \frac{6.00}{2.5} = 2.4 > 2.$ The wall my be designed as a one way slab spanning in vertical direction; fixed at base and free at top.



p= Pressure = 10 × 2.5 = 25 kN/m^2

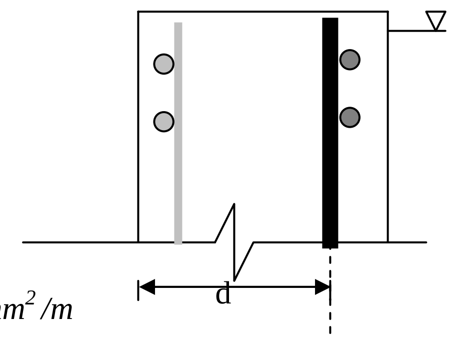
P= Force = 0.5 (25×2.5) = 31.25 kN
M=Moment = 31.25x2.5/3= 26 kN-m

M_u=1.7M=44.3 kN-m (per meter)

The moment causes tension on water face

d=200-40-13-12=135 mm

b=1000 mm



Working Limit State:

$$R_w = \frac{26\times1000^2}{1000\times135^2} = 1.43 \Rightarrow \rho = 0.0115$$

$$A_s = 1552\ mm^2/m$$

Check for Ultimate Conditions:

R=2.43 $\Rightarrow$ ρ=0.0095 A_s=.0095×1000×135=1282 mm^2/m

Shear: V_u=1.7×31.25=53.13 kN;
ϕV_c=0.85/6×$\sqrt{30}\times1000\times135 = 104752$ N > 53.13 kN. OK

Horizontal Steel: (ACI 14.3.3b) ρ=0.0025

A_s=1000×200×.0025=500 mm^2/m (50% on water face at least)

Vertical Steel on exterior face: A_s=0.5(0.0015×1000×200) =300 mm^2

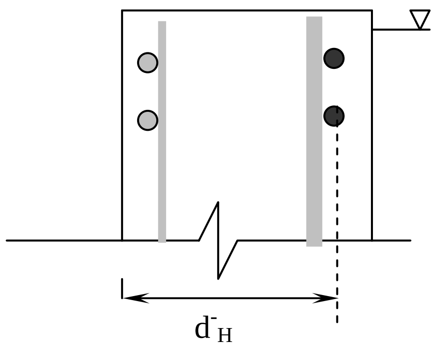
Design of Short Wall:

$k = \frac{l_x}{l_z} = \frac{3.6}{2.6} = 1.4 < 2.$ The wall will be designed as two-way slab subjected to ***uvl.*** The slab is fixed (continuous) on three sides and free at top. (use case 1 of Chart 53.). The spans have been increased by 100 mm for two-way action in wall.

p=Pressure =10×2.5=25 kN/m^2 (max Pressure at base)

P=Force=0.5(2.5×25) =31.25 kN /m length of wall; Max Pressure Force on base

Horizontal Steel



M^-_H =Moment at sides of Horizontal Span=0.031×25×3.6^2= 10.044 *kN-m /m*

M^-_{UH} =1.7×10.044=17.075 *kN-m /m* (This moment causes tension on water face)

d^-_H =200-40-1/2×12= 154 mm

b=1000 mm gives R_w=0.423, ρ=**0.0033** and R=0.72 , ρ=0.0027

A^-_{sH} =0.0033×1000×154=510 mm^2 /m (to be provided on water face near supports)

M^+_H =Moment=0.016×25×3.6^2=5.184 *kN-m /m*

M^+_{UH} =1.7×5.184=8.813 *kN-m /m* (This moment causes tension on exterior face)

$d^+_H = d^-_H$ =154 mm , b=1000 mm R_w=0.22, ρ=**0.0016**, R=0.37, ρ_{min}

A^+_{sH} =(ACI Minimum)=0.0015×1000×200= **300** mm^2 and from R_w the required area is A_s=.0016×1000×154=247

- **Vertical Steel**

Water face: M^-_V =0.058×25×2.6^2=10.26 *kN-m /m*, M^-_{UV} =17.442 *kN-m /m*

$d^-_V = d^-_H$ -12=142 mm R_w=0.51, ρ **=0.004** and R=0.87, ρ=.0033

A^-_{sV} =0.004×1000×142= 568 mm^2 . Provide minimum steel on exterior face.

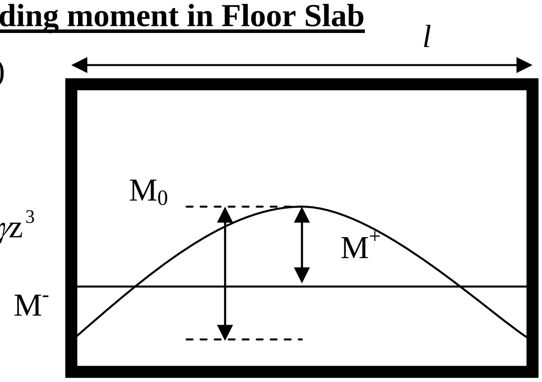
The depth of water for maximum +ve bending moment in Floor Slab

Loading on floor is udl=γz, (on wall it is uvl)

$$\text{Simple Span Moment} = M_0 = \frac{\alpha}{8}\gamma z l^2$$

$$\text{Moment at the Wall-Floor Joint} = M^- = -\beta\gamma z^3$$

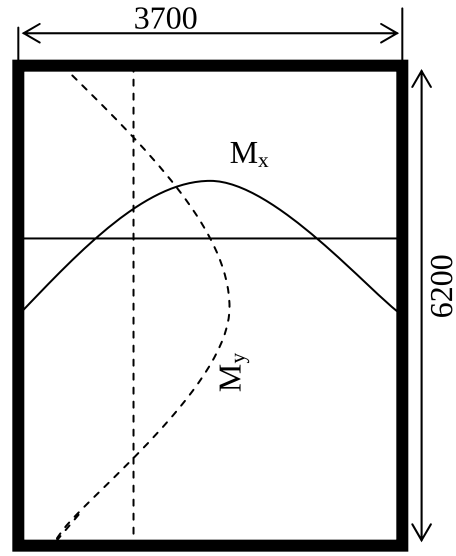
$$M^+ = \frac{\alpha\gamma z l^2}{8} - \beta\gamma z^3$$



$$\frac{\partial M^+}{\partial z} = \frac{\alpha\gamma l^2}{8} - 3\beta\gamma z^2 = 0 \Rightarrow z_c = \sqrt{\frac{\alpha}{24\beta}}\ l \quad \text{(where } z_c \le l_z\text{)}$$

Therefor the Max positive moment in floor: $M^+_{max} = \frac{\alpha\gamma z_c l^2}{8} - \beta\gamma z_c^3$

Design of Floor Slab



$k = \frac{6.20}{3.70} = 1.68$ (the floor is a two way slab)

(Table 1 and Chart 53 are used)

- **Short Span**

Maximum positive moment in span occurs at water level= $z_c = \sqrt{\frac{0.89}{24 \times \frac{1}{6}}} l_x = 0.4717 l_x = 1.745\, m$

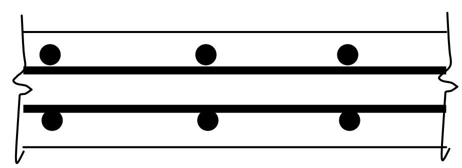
(Note: α_x= 0.89 and α_y=0.11 from Table 1.)

$$M_{ox} = \frac{0.89}{8} \times 3.7^2 \left[1.745 \times 10 + 0.2 \times 25\right] = 26.577 + 7.615 = 34.192\, kN - m$$

$$M_x^+ = 26.577 + 7.615 - (1.745 \times 10) \times \frac{1.745}{2} \times \frac{1.745}{3} = 26.577 + 7.615 - 8.856 = 25.336\, kN - m$$

M_{ux}^+ =1.7×26.577+1.4×7.615-1.7×8.856= .

. 40.787 kN-m



d_y=200-40-9=151 and [d_x=d_y-18=133 mm]

R_w=1.43; **ρ=0.0113** and R=2.31; ρ=0.009 (A_s=0.0113x1000x133 =1503 mm^2) **On Ground Face**

Max Negative Moment: $M_x^- = 26\ k\text{N - m}$ (at the joint of long wall and floor)

With d_x^-=133, R_w=1.47 and **ρ=.012**. And R= 2.5 with ρ=..01

Therefore <u>A_s= 0.012×1000×133=1596 mm^2</u> This steel shall be provided **on water face**.

<u>Shear in Short Span</u>

$$V_u = 0.89 \times \frac{3.5}{2}\left[1.7 \times 25 + 1.4 \times 5\right] = 77.1\, kN < .85/6\sqrt{30} \times 1000 \times 133 = (103{,}200 N) \text{ OK}$$

<u>Shear in long span</u> <u>(Not critical)</u>

$$V_u = 0.11 \times \frac{6}{2}\left[1.7 \times 25 + 1.4 \times 5\right] = \qquad kN$$

- **Long Span**

The ***maximum positive moment*** in the long span occurs at the water level=z_c, where

$$z_c = \sqrt{\frac{0.11}{24\times 0.058}}\ l_y = 0.2811 l_y = 1.687m \ (< 2.5 \text{ m})$$

$$M_{oy} = \frac{0.11}{8}\times 6.2^2\left[1.687\times 10 + .2\times 25\right] = 8.92 + 2.64 = 11.56\ kN\text{-}m$$

$$M_y^+ = 8.92 + 2.64 - 0.058\times(1.687\times 10)\times 1.687^2 = 8.92 + 2.64 - 2.78 = 8.78\, kN\text{-}m$$

M_{uy}^+ =1.7×8.92+1.4×2.64-1.7×2.78=14.32 *kN-m*

d_y=200-40-9=151 mm

R_w=0.385; **ρ=0.003** and R=0.683; ρ=0.0027 (A_s=0.003x1000x151=453 mm^2) > A_{smin}

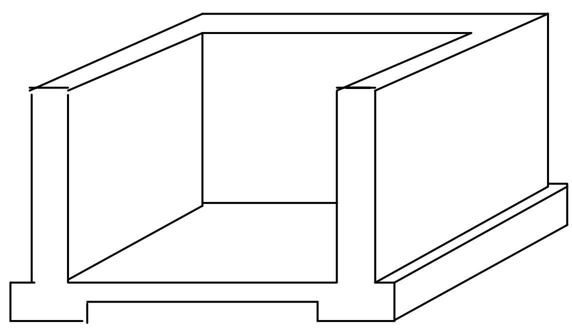
(on ground face)

Max Negative Moment: $M_y^- = 10.26\, kN - m$ (at the junction of short wall and floor)

With d_y^-=151 R_w=0.45 and ρ=.**0035**. And R=0.765 with ρ=.003. Provide 529 mm^2 **on water face.**

DIRECT TENSION IN WALLS AND FLOOR

The direct tension in the short walls and x-direction of the floor can be found considering a free body of the tank cutting a section along vertical plan parrallel to long walls as shown in the Figure on right. The calculations are presented in the following table.



Description	Area A_i	z_i	$A_i z_i$	I_0	$A_i(z_i - \bar{z})^2$
2-Short wall	2×0.2×2.5=1.000	1.525	1.525	0.05208	0.6116
floor	6.5×0.2=1.300	0.175	0.227	0.0043	0.4194
2-Supports	2×0.4×0.075= 0.060	0.037	0.002	0.0003	0.0299
	2.360		1.754	0.5254	1.0608

$\bar{z} = \frac{\sum A_i z_i}{\sum A_i} = 0.743\, m$ with A= 2.36 m^2 and I=I_0+ $A_i(z_i - \bar{z})^2$ =1.586 m^4

Max. Pressure on long wall=25 *kN/m²*. The total Pressure Force P=25×2.5/2×6=187.5*kN* and it acts at 0.833+0.275 *m* from ground level. The eccentricty of this force from the centroid of the section is 1.108-0.743=0.365*m*, which gives a moment =187.5x.365 *kN-m*

The stress distribution at the top and bottom of the tank walls is obtained as follows:

$$f_{top} = \frac{187.5}{2.36} + \frac{187.5 \times 0.365}{1.586} = -----kN/m^2 \quad 122.6 \quad (167)$$

and

$$f_{bot} = \frac{187.5}{2.36} - \frac{187.5 \times 0.365}{1.586} = -----kN/m^2 \quad 36.3 \quad (59)$$

$f_{avg} = \dfrac{f_{top} + f_{bot}}{2}$, which gives Tension in each of the short wall equal to T_{wall}, where $T_{wall}=f_{avg}\times0.20\times2.5=$ *kN* (or $f_{avg}\times0.20$ *kN/m* height of the wall)

The tension in the floor can be obtained from $T_{floor}=Total\ tension-2\ T_{wall}$. This tension acts along the short direction of the floor.

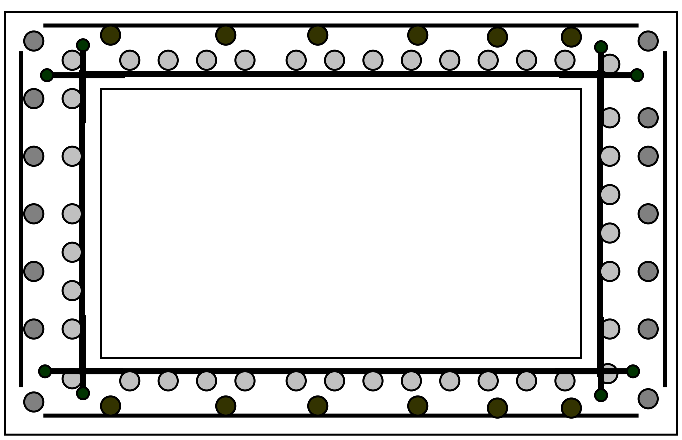
Similar calculation may be performed to obtain the tension in long walls and the floor in y-direction. The steel areas required to meet these direct tensions can be calculated as:

$A_s(Tension)=T/f_s$. The half of the area is to be provided on either face of the wall (or floor). Therefore these areas can be superimposed with those obtained earlier from the strength considerations. Where the strength area of steel was minimum, the superposition is to made not with A_{min} but with actual A_s(strength) in the following way:

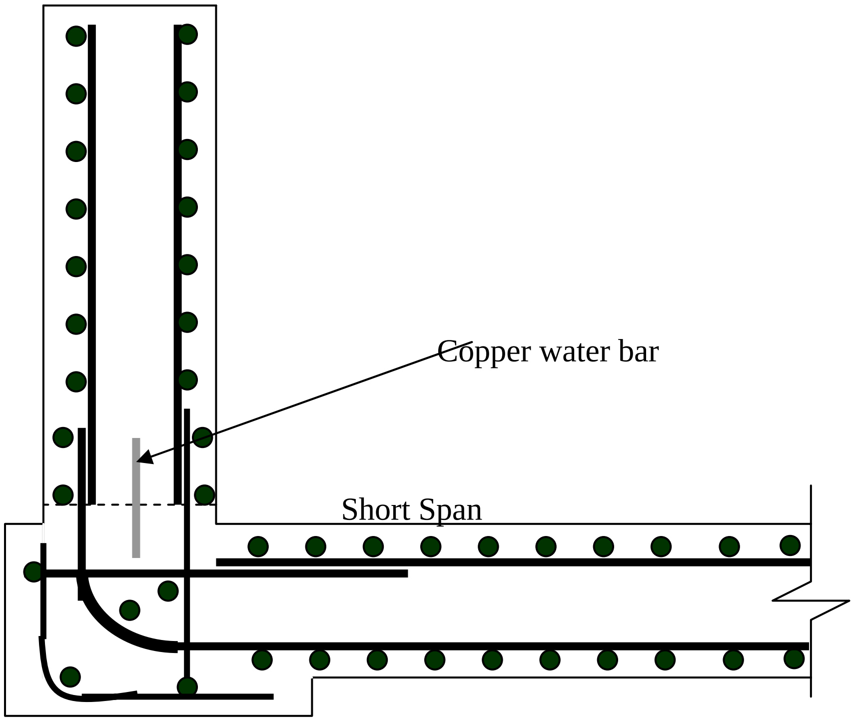
$A_s=A_s(\text{Strength})+0.5A_s(\text{Tension})$, where A_s(Strength) is the actual area of steel required for strength disregarding A_{min}. If A_s is still < A_{min}, provide the A_{min}.

The (section through walls) sketch showing steel in walls

Detail on last page.



The (vertical section) sketch showing steel in walls and floor



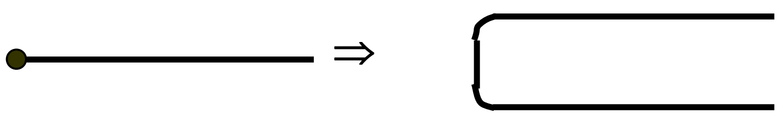
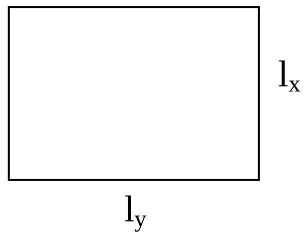


Table 1. The fraction of load taken by short and long spans

$k = \frac{l_y}{l_x}$	1	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
α_x	.500	.594	.674	.741	.793	.835	.867	.893	.913	.929	.941
α_y	.500	.406	.326	.259	.207	.165	.133	.107	.087	.071	.059

Note: $\alpha_x = \frac{k^4}{1+k^4}$ and $\alpha_y = \frac{1}{1+k^4}$

Simple span moment= $M_{0i} = \frac{\alpha_i}{8} w l_i^2$



Crack Widths

The basic equation for relating crack width to strain in the reinforcement is

$$w = \alpha\, a_c^{\beta} \varepsilon_s^{\gamma} \qquad 1$$

The effect of the tensile strain in the concrete between the cracks is neglected as insignificant. α_c is the crack spacing, ε_s the unit strain in the reinforcement, and α, β, and γ are constants. As a result of this fracture hypothesis, the mathematical model in Eqn 1, and the statistical analysis of the data of 90 slabs tested to failure, the following crack-control equation emerged:

$$w = K\beta f_s \sqrt{\frac{d_{b1} s_2}{Q_{i1}}} \qquad 2$$

where the quantity under the radical, $G_1 = d_{b1}s_2/Q_{i1}$, is termed the grid index and can be transformed into

$$G_1 = \frac{8 s_1 s_2 d_c}{\pi d_{b1}} \qquad 3$$

where

K = fracture coefficient, having a value of $K=2.8\text{x}10^{-5}$ for uniformly loaded, restrained, two-way action square slabs and plates. For concentrated loads or reactions or when the ratio of short to long span is less than 0.75, but larger than 0.5, a value of $K=2.1\text{x}10^{-5}$ is applicable. For a span aspect ratio of 0.5, $K=1.6\text{x}10^{-5}$. Units of coefficient K are in in^2/lb. (may be interpolated linearly)

β = ratio of the distance from "the neutral axis to the tensile face of the slab" to "the distance from the neutral axis to the centroid of the reinforcement grid". A value of 1.25 may be taken to simplify the calculations, although it varies between 1.20 and 1.35.

f_s = actual average service load stress level, or 50%-40% of the design yield strength in **ksi**.

d_{b1} = diameter of the reinforcement in direction 1 closest to the concrete outer fibers (inch)

d_c = concrete cover to Centroid of reinforcement (inches)

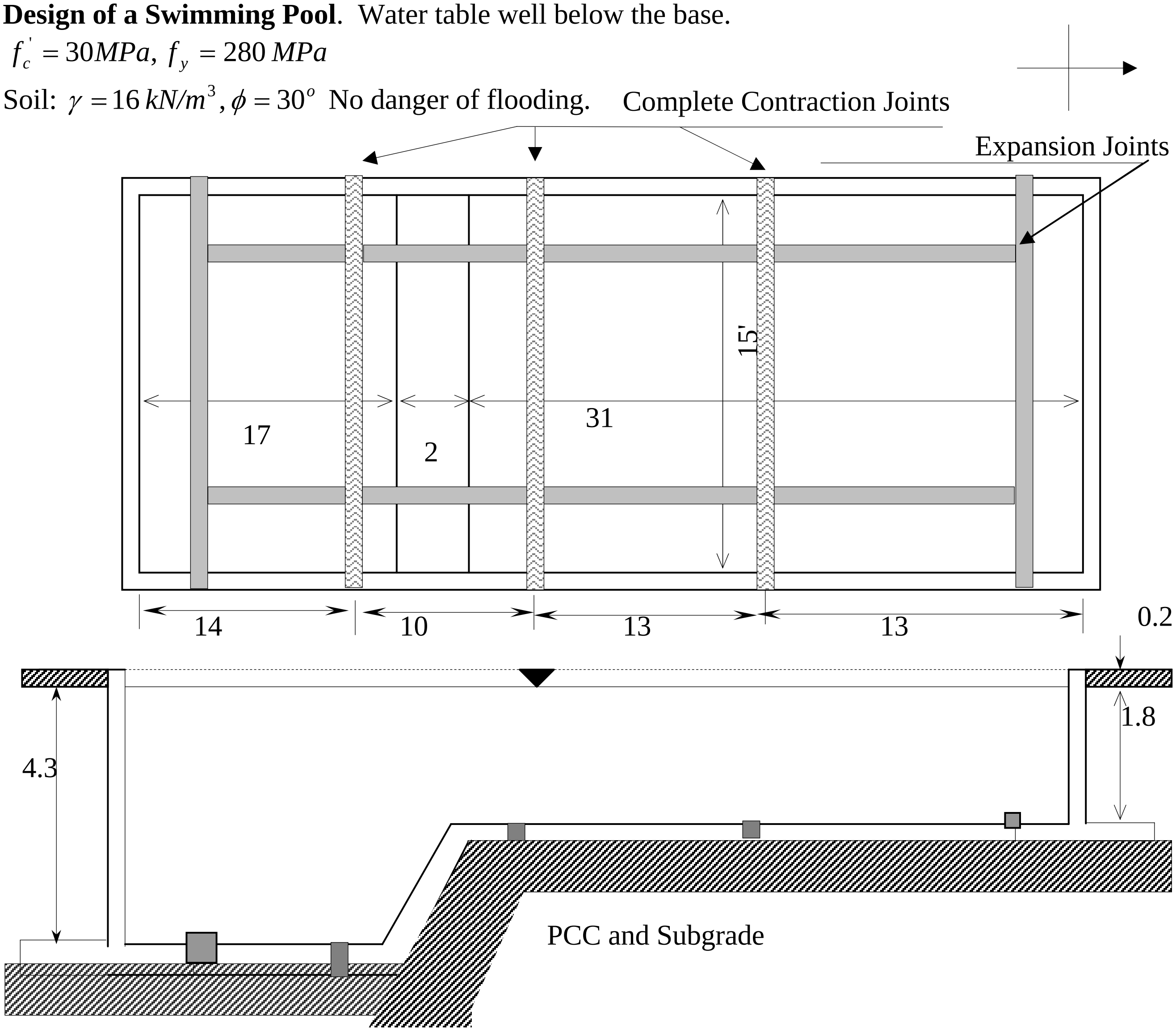
s_1, s_2 = spacing of reinforcement in directions 1, 2. (inches) Direction 1 is the direction of the reinforcement closest to the outer concrete fibers; this is the direction for which crack control check is to be made.

Q_{i1} = active steel ratio= $\dfrac{\text{Area of steel } A_s \text{ per unit foot width}}{12(d_{b1} + 2c_1)}$, where c1 is the clear cover measured from the tensile face of the concrete to the nearest edge of the reinforcing bar in direction 1.

Design of a Swimming Pool. Water table well below the base.

$f_c^{'} = 30 MPa,\ f_y = 280\ MPa$

Soil: $\gamma = 16\ kN/m^3, \phi = 30^o$ No danger of flooding.



<u>Design Procedure:</u>

Stability: There are two critical zones for stability: North and South end zones. Consider two loading conditions: (A) Empty pool with soil pressure from outside; (B) Full pool with no allowance for soil pressure from outside. The stability against overturning and sliding with FOS=1.5 and 1.25 respectively is just sufficient. For the present case, there is no danger of flooding and rise of water table hence floatation is not a problem.

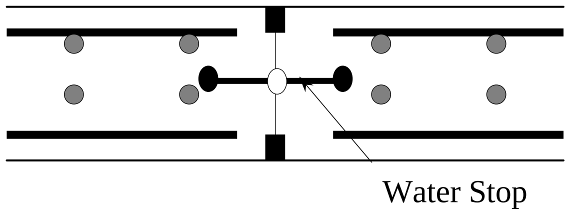
Bearing: The critical loading condition: when pool is full of water.

Strength: The pool will be designed for two loading conditions: (A) Empty pool with soil pressure from outside; (B) Full pool with no allowance for soil pressure from outside.

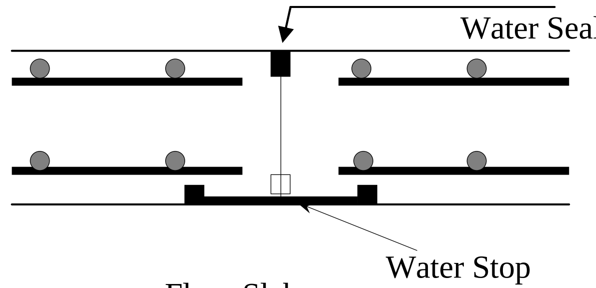
Problem: Check the stability and design for strength <u>the north and south end zone</u> walls and floor slab.

Joints: for liquid retaining structures (No concrete continuity)

Complete Contraction Joints: Maximum distance between joints = 15m (No continuity of steel)

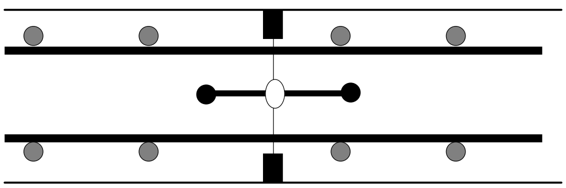


Wall

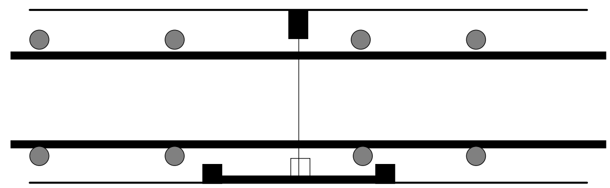


Floor Slab

Partial Contraction Joints: Maximum distance between joints = 7.5m (100% or 50% steel continuity)



Wall



Floor Slab

Expansion joints should be avoided in liquid retaining structures. The construction joints should be prepared for continuity and sealed properly on liquid side.

Design consideration: The floor slab between cracks may be designed as slab on grade. Extra care should be taken for temperature changes during hydration and temperature rise or drop between seasons.

<u>Slab on grad</u>e: $T = 0.5\mu\, w \frac{t}{12} L$ where μ is friction between sub-grade and slab, w is unit weight of RC, t is slab thickness in inches and L is length in ft between two contraction joints (partial or complete). Provide steel for T using working stress theory or apply overload factor to the tension T and use ultimate strength theory. In case of shallow depths the weight of water on slab may be neglected.

<u>Temperature Changes</u>: The estimated maximum crack spacing S_{max} is related to the direct tensile strength f_t, the bond strength f_b, diameter of bar d, and steel ratio ρ as follows:

$$S_{\max} = \frac{f_t}{f_b}\frac{d}{2\rho}$$

The maximum crack spacing is also governed by maximum crack width δ_{max}, and fall from hydration peak temperature to ambient temperature T_h and seasonal fall in temperature T_s as :

$S_{\max} = \frac{\delta_{\max}}{0.5\alpha(T_s + T_h)}$. The two equations may be combined to give a solution for ρ:

$\rho = \frac{\alpha(T_s + T_h)}{4\delta_{\max}}\frac{f_t}{f_b} d$ with α= $12\times10^{-6}/^{o}C$. However, $\rho \geq \frac{f_t}{f_y}$ in any case.(f_t/f_b may be taken=0.80). The effect of temperature T_s may be ignored if complete contraction joints are provided at not more than 50' centers. The steel is provided in two layers.