# CHAPTER FOUR
# THEORETICAL
# METHODOLOGY

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# Theoretical Methodology

## 4.1 General

The design of both underground water tank structures and retaining walls is based on analysis and design techniques which have been discussed in previous chapters.

Retaining wall structures are used to resist earth and hydrostatic loading. There are a number of differing types, some of the more common forms of which are described. Walls must be designed to be stable with adequate resistance to overturning and sliding, allowable ground bearing pressures beneath the base of the wall must not be exceeded and all parts of the wall must be of adequate structural strength.

## 4.2 Earth pressure on retaining walls

### (a) Active soil pressure

Active soil pressures are given for the two extreme cases of a cohesion less soil such as sand and a cohesive soil such as clay (Fig. 4.1). General formulae are available for intermediate cases. The formulae given apply to drained soils and reference should be made to textbooks on soil mechanics for pressure where the water table rises front the wall. The soil pressures given are those due to a level backfill. If there is a surcharge of q $kN/m^2$ on the soil behind the wall, this is equivalent to an additional soil depth of $z=q/\gamma$ where $\gamma$ is the density in kilonewtons per cubic metre.

(i) Cohesion less soil, c=0 (Fig. 4.1(a)) The pressure at any depth z is given by

$$P = \gamma z \frac{1-\sin\theta}{1+\sin\theta} \quad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (4.1)$$

Where $\gamma$ is the soil density and $\phi$ is the angle of internal friction. The force on the wall of height H is:

$$\partial_1 = q * \frac{1-\sin\theta}{1+\sin\theta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.2)$$

$$\partial_2 = (q + \gamma H) * \frac{1-\sin\theta}{1+\sin\theta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.3)$$

$$Ea_1 = \partial_1 * H \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.4)$$

$$E_{a2} = 0.5(\partial_2 - \partial_1) * H \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.4)$$

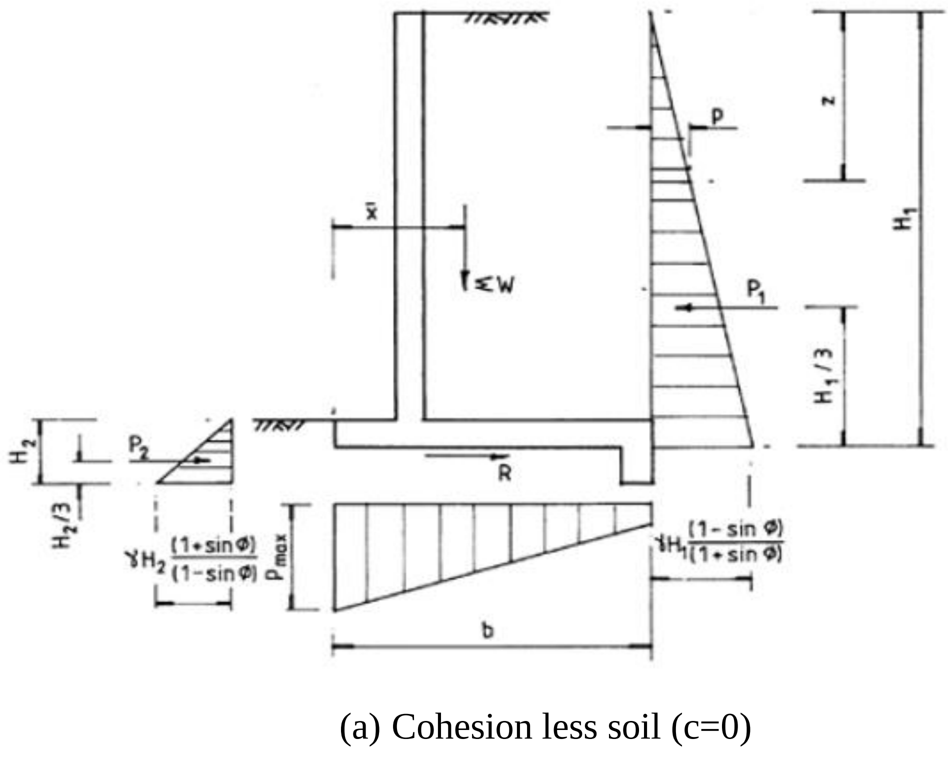
Where: $E_{a1}$ is the horizontal force from surcharge load.

$E_{a1}$ is the horizontal force from back fill load.

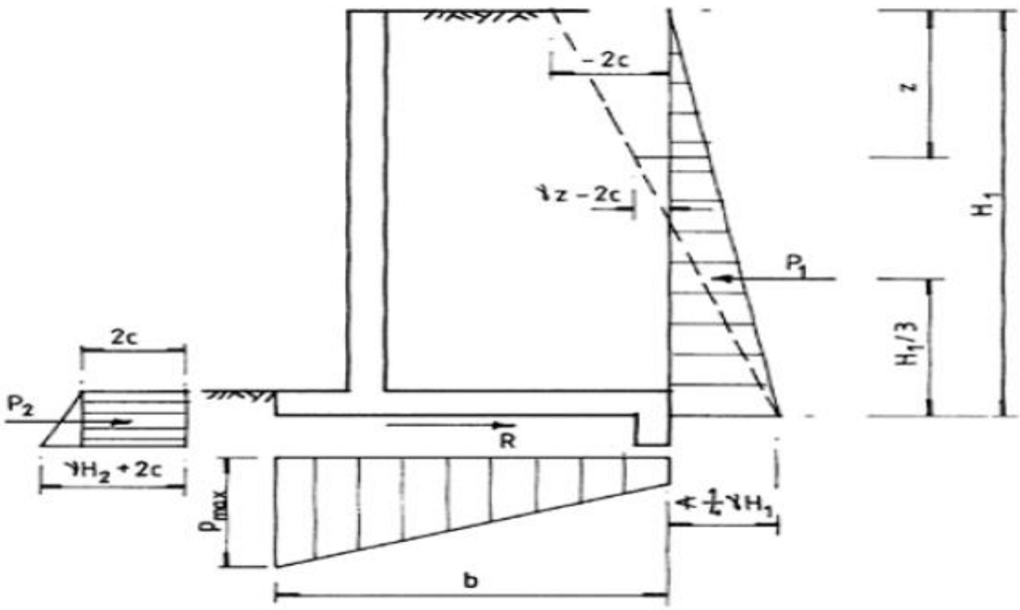
(ii) Cohesive soil, C=ϕ (Fig. 4.2(b)) the pressure at any depth z is given Theoretically by

$$p = \gamma z - 2c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4.5)$$

Where c is the cohesion at zero normal pressure. This expression gives negative values near the top of the wall. In practice, a value for the active earth pressure of not less than is used.




(a) Cohesion less soil (c=0)




(b) Cohesive soil (ϕ=0)

**Fig. 4.1 section of cantilever walls**.

**(b) Wall stability**

Referring to Fig. 4.1 the vertical loads are made up of the weight of the wall and base and the weight of backfill on the base. Front water on the inner base has been neglected. Surcharge would need to be included if present. If the centre of gravity of these loads is x from the toe of the wall, the stabilizing moment is $\Sigma W_x$ with a beneficial partial safety factory $\gamma f=1.4$. The overturning moment due to the active earth pressure is $M_R/M_0$ with an adverse partial safety factor $\gamma f=1.4$. The stabilizing moment from passive earth pressure has been neglected. For the wall to satisfy the requirement of stability (overturning and sliding) above 1.5.

**(c) Vertical pressure under the base**

The vertical pressure under the base is calculated for service loads. For a cantilever wall a 1 m length of wall with base width b is considered. Then

Area $A=b\ m^2$

Modulus $Z=\frac{b^2}{6}m^3$

If ΣM is the sum of the moments of all vertical forces ΣW about the Centre of the base and of the active pressure on the wall then

$$\sum M = \sum W\left(X - \frac{b}{2}\right) \text{ ……………………………….. } (4.6)$$

The passive pressure in front of the base has been neglected again. The maximum pressure is:

$$P_{max} = \frac{\sum W}{A}\left(1 + \frac{6e}{\acute{B}}\right) \text{ …………………………….. } (4.7)$$

This should not exceed the safe bearing pressure on the soil.

**(d) Resistance to sliding (Fig. 4.1)**

The resistance of the wall to sliding is as follows.

I. Cohesion less soil The friction R between the base and the soil is μΣW where μ is the coefficient of friction between the base and the soil (μ= tanϕ ). The passive earth pressure against the outer of the wall from a depth $H_2$ of soil

$$P_2 = 0.5\gamma {H_2}^2 \frac{1-\sin\theta}{1+\sin\theta} \text{ …………………………. } (4.8)$$

II. Cohesive soils the adhesion R between the base and the soil is β where β is the adhesion in kilonewtons per square metre. The passive earth pressure is

$$P_2=0.5\gamma H_2+2cH_2 \text{ ………………………………. } (4.9)$$

A nib can be added, as shown in Fig. 4.2, to increase the resistance to sliding through passive earth pressure.

For the wall to be safe against sliding:

$$F_S = \frac{\mu R_V}{E_{a1}+E_{a2}} \text{ ……………………………………..…. } (4.10)$$

Where P1 is the horizontal active earth pressure on the wall.

The analysis and design of retaining walls includes the following:

a. Estimation of primary dimensions of the Tank and wall, then these dimensions should be checked.

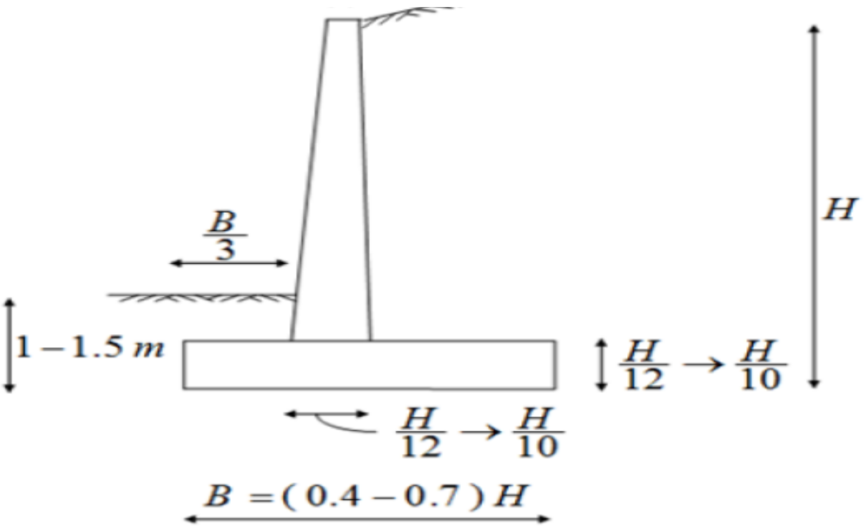
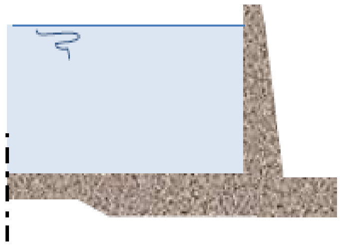
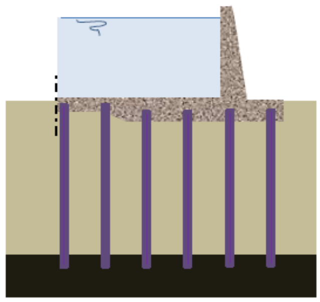



**Fig.4.2 The primary dimension of the wall**

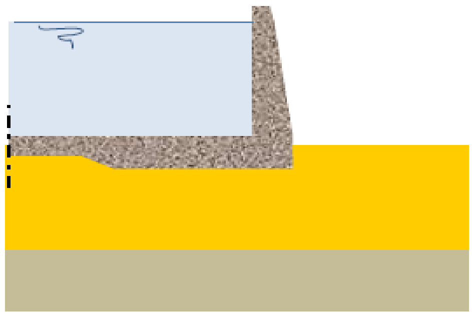
b. Checking external stability of the walls (sliding of retaining walls, overturning stability and bearing stability).

**If stresses on soil are unsafe**:





-Use toe -Use deep foundations (piles)



-Make soil replacement

**Fig.4.3 Treatment of soils if stresses are unsafe.**

c. For reinforced concrete retaining walls main and secondary reinforcement must be calculated.

L= 0.6 H (for clayey soil)

**4.3 Design steps:**

**Step (1)**

Determination of dimension of the tank

Water Quantity Estimation = per demand x Population

**Step (2)**

Estimation of primary dimensions of the wall

The dimensions of the retaining wall will be assumed as follow refer to figure1:

a. The width of the wall base

B= 0.4H to 0.7H

b. The thickness of the stem at the top

t=$\frac{H}{12}$ to $\frac{H}{10}$

**Step (3)**

Calculate the loads and earth pressures acting on the wall

I. The active earth pressures:

$$K_a = \frac{(1-\sin\phi)}{(1+\sin\phi)} \quad \text{.....................................} \quad (4.11)$$

II. The stability analysis:

a. Checking the factor of safety against sliding.

b. Checking the factor of safety against overturning.

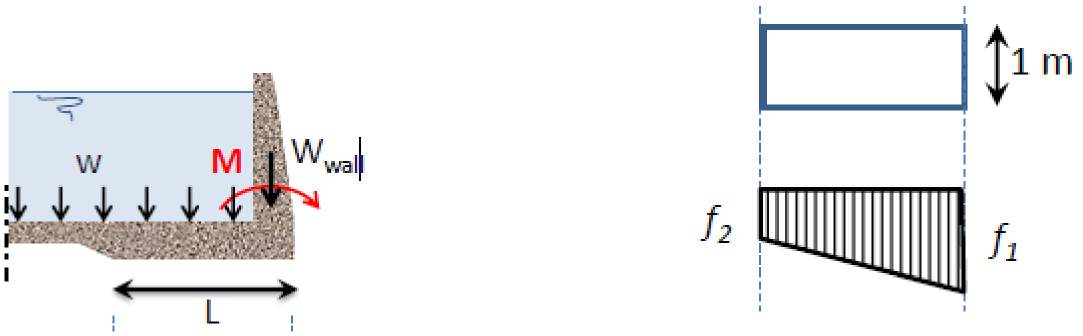



**Fig.4.4 The net pressure on the base**

$$P_{max} = \frac{\sum W}{A}\left(1 + \frac{6e}{B}\right) \quad (4.12)$$

$$P_{min} = \frac{\sum W}{A}\left(1 - \frac{6e}{B}\right) \quad (4.13)$$

$$e = \frac{B}{2} - \bar{x} \quad (4.14)$$

$P_{max}$< soil bearing capacity

$P_{min} > \frac{P_{max}}{2}$ (for clay)

If the stability are unsafe use toe Length of toe (l=B/3)

iii. Checking the pressure under the base of the wall

**Step (4)**

Design of stem reinforcement.

**Step (5)**

Design of the base of the wall.

**Step (6**

Chick crack width

$$a_c = \frac{E_S}{E_C} \quad (4.15)$$

$$\frac{a_c * A_S}{b * d} \quad (4.16)$$

$$\frac{x}{d} \text{ from fig B.3} \quad (4.17)$$

$$\frac{I_c}{b * d^3} \text{ from fig B.4} \quad (4.18)$$

$$\varepsilon_s = \frac{a_c M}{E_s * I_s} * (d - x) \quad \text{…………..………….……..….} \quad (4.19)$$

$$\varepsilon_h = \frac{h-x}{d-x} \varepsilon_s \quad \text{…………..……………..………...….} \quad (4.20)$$

$$\varepsilon_m = \varepsilon_h - \frac{b*(h-x)(a-x)}{3E_S A_S(d-x)} \quad \text{………………………....….} \quad (4.21)$$

$$\varepsilon_1 = \frac{\varepsilon_{s}\ (a-X)}{(d-x)} \quad \text{………..…………………………...….} \quad (4.22)$$

Design surface crack width=

$$\frac{3a_{cr}\varepsilon_m}{1+2\left(\frac{a_{cr}-c_{min}}{h-x}\right)} \quad \text{………..…………………………....….} \quad (4.23)$$

**<u>Step (7)</u>**

Design of the slab

Classification of slab

$\frac{L_Y}{L_X}$= > 2 (One way slab)

$\frac{L_Y}{L_X}$ = < 2 (tow way slab)

Calculate self-weight = $\gamma_c$*H *t

**<u>Design loads</u>**

Consider a strip 1 m wide.

Design load= (1.4×DL) + (1.6×LL) ……………..……..…. (4.24)

**Shear forces and bending moments in the slab**

Moments = $\frac{WL^2}{8}$

Shear = $\frac{W}{2}$

**Design of moment steel**

Assume 10 mm diameter bars with 25 mm cover. The effective depth is

$$d = h - c - \frac{\theta}{2} \quad \text{………..……………..…...….} \quad (4.25)$$

Section at support

$$K = \frac{M}{d*b^2 f_{cu}} \quad \text{..........................................} \quad (4.26)$$

Z= 0.95d

$$A_S = \frac{M}{0.87 F_Y Z} \quad \text{..........................................} \quad (4.27)$$

The minimum area of reinforcement is $\frac{0.13B*H}{1000}$

**Shear resistance**

$$V = \frac{W}{2}$$

$$V = \frac{V}{b*d}$$

$$0.79\left(\frac{100A_S}{b*d}\right)^{\frac{1}{3}}\left(\frac{400}{120}\right)^{\frac{1}{4}}\left(\frac{F_{cu}}{25}\right)^{\frac{1}{3}} \Big/ 1.25 \quad \text{..........................................} \quad (4.28)$$

**Deflection**

Actual = span/effective depth

$$F_S = \frac{5}{8}\left(\frac{F_Y A_{req}}{A_{S\,Prov}}\right) \quad \text{..........................................} \quad (4.29)$$

The modification factor is

$$= 0.55 + \left(\frac{477 - F_S}{120}\right)\left(0.9 + \frac{M}{bd^2}\right) = 1.4 \quad \text{.......} \quad (4.30)$$

Allowable span/d ratio

Actual = span/effective depth

If allowable span ratio < Actual

The slab is satisfactory with respect to deflection

If allowable span ratio > Actual

Increase depth of slab and redesign.