DESIGN OF FLOATING FLOOR AND RIBBED FOUNDATION OF BARRAGE USING CONFORMAL MAPPING TECHNIQUE

A DISSERTATION

submitted in partial fulfillment of the
requirements for the award of the degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

By
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CANDIDATE'S DECLARATION

I hereby certify that the work is being presented in the dissertation entitle **"DESIGN OF FLOATING FLOOR AND RIBBED FOUNDATION OF BARRAGE USING CONFORMAL MAPPING TECHNIQUE"** in partial fulfillment of the requirement for the award of degree of Master of Engineering submitted in the Water Resources Development Training Centre of the University of Roorkee is authentic record of my own work carried out during a period from July , 2000 to December , 2000 under supervision of DR. GC. Mishra and DR. Nayan Sharma.

The matter embodied in this dissertation not been submitted by me for the award of any other degree.

(Junaedi)

This is to certified that the above statement made by the candidate is correct to the best of my knowledge.

Roorkee, 4 December , 2000

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(Junaedi)

DESIGN OF FLOATING FLOOR AND RIBBED FOUNDATION OF BARRAGE USING CONFORMAL MAPPING TECHNIQUE

Synopsis

The main problem encountered in structural design of hydraulic structures , such as a barrage, is uplift pressure. In usual design of barrage , which is called gravity floor system, the floor at downstream side is designed as a counter weight to resist the pressure. In such design practice, a considerable thick floor is required. This solution , therefore , will cause new problems to arise, such as foundation failure due to low bearing capacity of soil, the construction problems like large excavation, large concreting, and dewatering . Consequently the construction takes a long time and requires much money.

In design of hydraulic structure founded on clayey soil (soft soil), uplift pressure can be used to reduce stresses on the soil base. In case of barrage with floating floor and ribbed foundation system, the floor is designed as thin as possible, but structurally the floor must strong enough to resist the pressure without cracking and permitted camber/deflection in the permissible range . The dimension of foundation is thus reduced.

The floating floor and ribbed foundation barrage system has light weight but structurally it has high rigidity , because the provision of two or more ribs of reinforced concrete wall below the floor and girders in upper side arranged perpendicular to each other . The lower ribs not only act as foundation but also act as cut off at upstream , down stream and at any intermediate position.

Besides the uplift pressure is counter acted, the whole structure is kept stable against sliding and overturning. In this barrage system the stability is depend on the rigidity of structure to transfer the loads (water at upstream side , self weight and external loads) , as counter weight against uplift force. By equating the equilibrium of that forces, the minimum weight of structure can be determined.

In the design process we can not avoid what we called 'trial and error'. Due to many computations those must be done, a long time will be consumed if it is done manually. In this present study, based on the analytical solution using conformal mapping technique, C++ computer programs aided for preliminary structural design of simple hydraulic structure and barrage have been developed.

Due to the features described above, the barrage with floating floor and ribbed foundation has some advantages :

1. The system is suitable for the alluvial zones which usually have low bearing capacity of soil base.
2. The thickness of floor will be quite thin, total dimension of structure will be less and cost of construction can be significantly reduced.
3. Due to light weight and high rigidity of structure, this system is superior in seismic zones compared to the gravity floor system.
4. The size being very small, the problems encountered in construction stage will be less and construction can be speeded up .

CONTENTS

Chapter I

INTRODUCTION

Design of barrage consist of two approaches: the hydraulic design and the structural design. In this present work, an attempt has been made to study structural design of barrage founded on clayey soil, specifically the stability against subsurface flow (uplift pressure). In the past, most of the barrages are designed with gravity floor concept, stability of the barrage is governed by the weight of floor. The thickness of gravity floor required, therefore, is considerable large. Thus, structural design of barrage floor, plays importance role in achieving economy and technical efficiency.

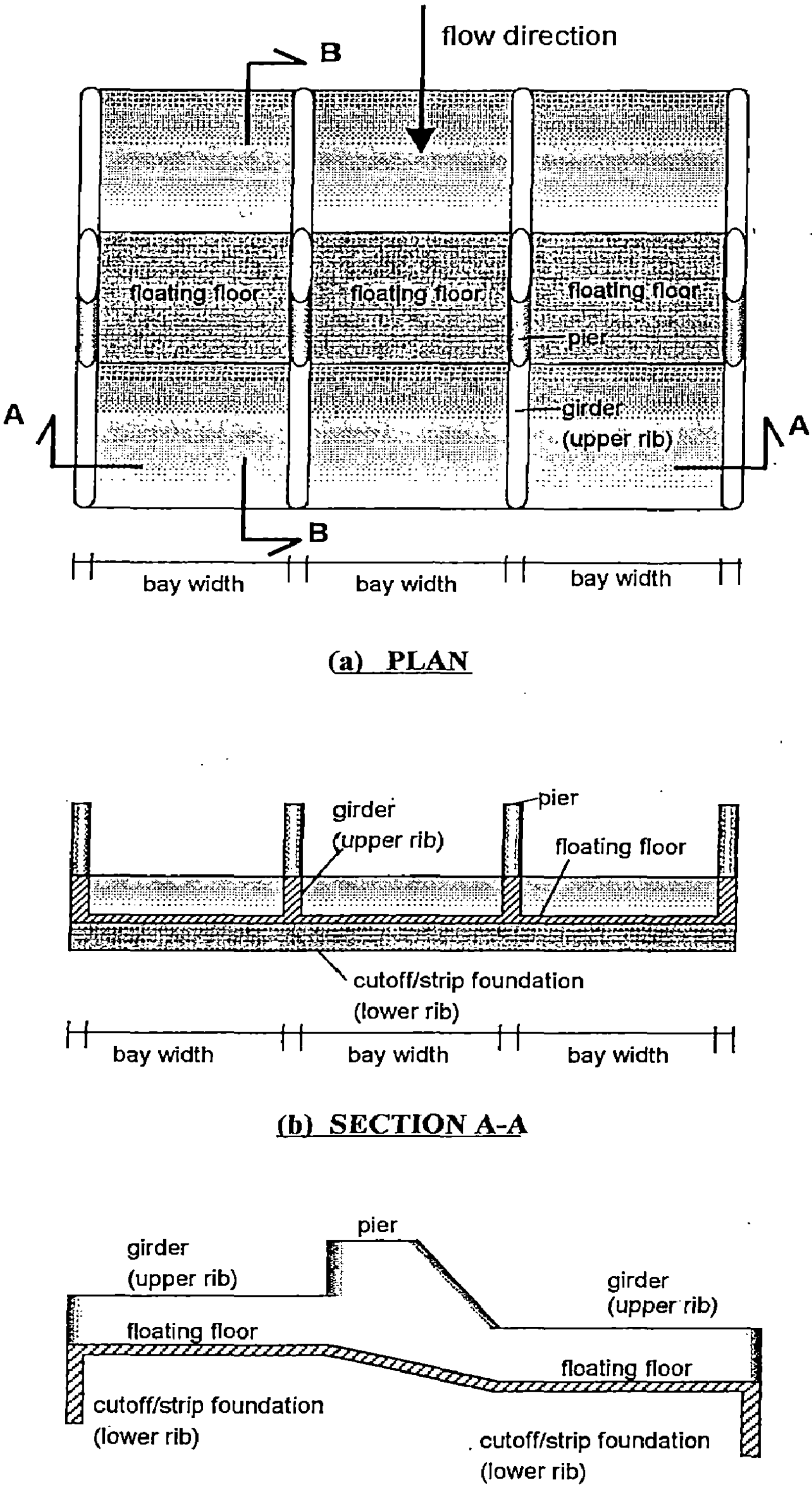
The floating floor and ribbed foundation barrage system , as shown in fig.1, is a barrage having a quite thin of floor but structurally it has high rigidity , because the floor is stiffed by two or more ribs of reinforced concrete wall below the floor and girders in upper side. These ribs are arranged perpendicular to each other.

In the design of hydraulic structure, stability of soil base and the structure are influenced by uplift pressure. An analysis of a simple structure resting on clayey soil is taken as an example to show how the uplift pressure acts below the foundation and influences the stability of the structure. This analysis shows that stability against circular sliding of structure founded on granular soil (ϕ soil) is strongly influenced by pore pressure or uplift pressure along the sliding line, but in the soil having cohesion and frictional characteristics (C-ϕ soil), the contribution of friction in the stability is very small compared to cohesion. Thus, in the stability analysis of structure against circular sliding, the frictional part may be ignored. The soil may be assumed as clayey soil. An analytical method by using conformal mapping is adopted to determine uplift pressure at any location below the structure.

In case of barrage with floating floor and ribbed foundation system, the floor is designed as thin as possible, but structurally the floor must strong enough to resist the pressure without cracking and permitted camber/deflection in the permissible range . The structural analysis for preliminary design of reinforcement of floor and ribs of foundation have been carried out by using ultimate method. The design results are compared with the conventional method (gravity floor system).

In the design process we can not avoid what we called 'trial and error'. Due to many computations that must be done, it will consume long time if it is done manually. *In this present study, based on the analytical solution using conformal mapping* technique, C++ computer programs aided for preliminary structural design of simple hydraulic structure and barrage have been developed.

Fig. 1. A BARRAGE WITH FLOATING FLOOR AND RIBBED FOUNDATION



(c) SECTION B-B

Chapter II

REVIEW OF LITERATURE

II.1. SEEPAGE FLOW

a. General

Hydraulic structure like barrages, weirs, head regulators and cross-drainage works are often built on alluvial soil foundations. In the vast plains of this country (India also Indonesia) the sub soil consist of alluvium whose particle size varies from that of fine clay to coarse sand. Consider now, a hydraulic structure shown in fig. (II.1.1), a dam founded on rock. It is assumed that the foundation and the body of the dam are absolutely water tight, the only force exerted by the water would be $0.5 \gamma w\ h^2$ acting normally to the face of the dam (if there is no tail water). But in fact neither the foundation nor the body of the dam is absolutely water tight.

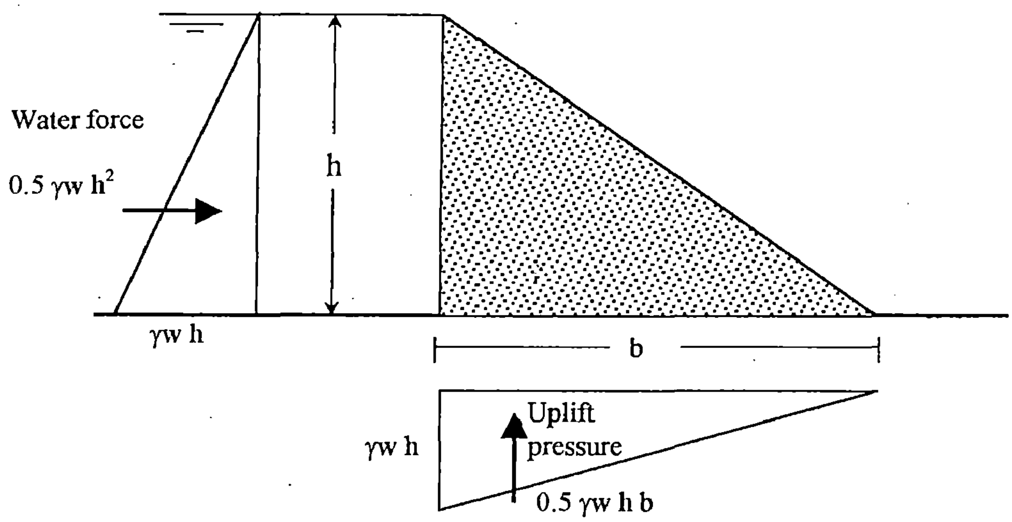


Fig. (II.1.1) Uplift Pressure below a Dam

There is also leakage along the contact of the dam base with the foundations. The water under pressure percolates through cracks, pores and fissures in the foundation and exerts uplift pressure. In the design of the dam on rock, with no foundation gallery, uplift pressure is assumed to act on full base and vary from $\gamma w\ h$ to zero (if there is no tail water). Not much significance is generally attached to the seepage force exerted by percolating water as it not likely to do any damage to the rock. In fact rock or impervious soil may offer so much resistance to the flow of water that the reservoir head may be largely dissipated in overcoming friction before downstream toe of the dam is reached.

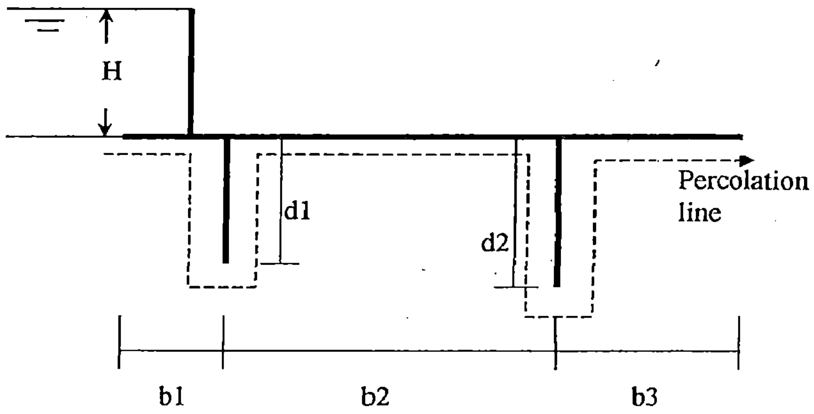
On the other hand, if a dam is built on pervious soil, little resistance may be offer by the soil and percolation may reach the downstream toe of the dam without any substantial loss of head. In such a situation the percolating water may carry soil particle with it and thus, undermine the structure. Prevention to this type of failure, and determination of correct uplift pressure due to sub surface flow have been drawn the attention of engineers since the beginning of this century (W.G Bligh 1907 , E.W Lane 1935 and Khosla 1936).

b. <u>Creep Theory</u>

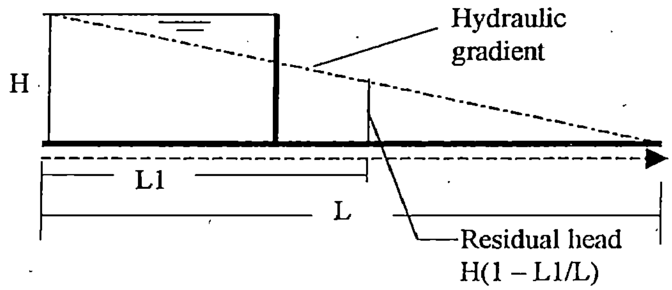
- **Bligh's Creep Theory**

Consider a horizontal floor length (L) metre in fig.(II.1.2), impounding a depth H metre of water. The loss of head per metre length of floor, H/L, is called the hydraulic gradient. Bligh called it percolation coefficient. Mathematically this value is written as bellow:

$$C = \frac{H}{b1 + 2\,d1 + b2 + 2\,d2 + b3} \qquad (II.1.1)$$



(a)



(b)

<u>Fig. (II.1.2) Bligh's Creep Theory – Definition Sketch</u>

The dissipation of the head at any point is supposed to be proportional to the length traveled. At a distance L1 the residual head would be H-(H/L)L1.

Bligh presumed that percolation water creeps along the contact of the base profile of the structure with sub soil and losses head in proportion to the length of its travel. No discrimination was made by him in horizontal and vertical creeps in assessing their effectiveness against undermining or piping. Because of its simplicity, Bligh's theory found general acceptance.

- **Lane's Weighted Creep Theory**

Lane approached the problem by making a statistical examination of large number of structure on pervious foundations. He developed the weighted creep theory which in effect may be called "Bligh's Creep Theory Corrected for Vertical Contacts".

According to Lane's Weighted Creep Theory, the weighted creep length (Lw) is given as

$$Lw = 1/3\ N + V \qquad (II.1.2)$$

Where:

N is the sum of the horizontal contacts and all the sloping contacts less than 45 degree.

V is the sum of all the vertical contacts plus the sloping contacts greater than 45 degree.

While Lane's weighted creep theory is an improvement over Bligh's creep theory, it too suffers from the limitations of an empirical approach.

c. <u>Seepage Flow Theory.</u>

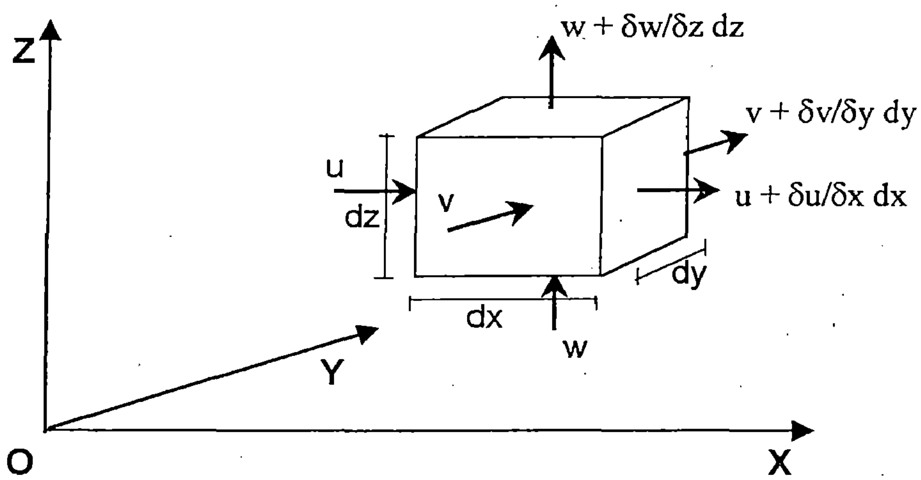
- **Mechanics of Seepage Flow**

It has been known that the flow through porous media follows Darcy's Law which can be expressed by the equation:

$$V = Ki \qquad \text{(II.1.3)}$$

where V = velocity of flow, i = hydraulic gradient and K represent the permeability of the soil.

The Darcy's Law is applicable to laminar flow only. Experiments carried out over a long period have concluded that Darcy's Law is applicable up to maximum velocity of 0.003 to 0.004 metre per second and maximum grain size of the medium between 1.5 to 2.0 mm. In vast alluvium plain in this country (India also Indonesia) these limits are seldom exceeded.



<u>Fig. (II.1.3) Velocity Components on an Element of Soil</u>

- **Seepage Flow as Potential Flow**

Darcy's Law V = Ki may be generalized in the form

$$V = -K\frac{dh}{dx} \qquad \text{(II.1.4)}$$

where V = velocity, h is the head in the fluid driving the water particles in the length of dx. Considering an element of soil of dimension dx and dz in the plane of the paper and dy perpendicular in the plane as shown in fig. (II.1.3). Let the velocity components at the entrance to the element along the axis of x, y, z be u, v and w respectively. Then according to Equation (II.1.4) we obtain:

$$u = -K\frac{\partial h}{\partial x}, \quad v = -K\frac{\partial h}{\partial y}, \quad w = -K\frac{\partial h}{\partial z} \qquad \text{(II.1.5)}$$

It may be seen from these equation that the velocities u, v, w can be regarded as the partial derivatives with respect to x, y, and z of a quantity $\phi = -k\,h$ known as a velocity potential. Equation (II.1.5) may be written as:

$$u = \frac{\partial \phi}{\partial x}, \quad v = \frac{\partial \phi}{\partial y}, \quad w = \frac{\partial \phi}{\partial z} \qquad \text{(II.1.6)}$$

The quantity of water entering the element in unit time is

$$u\,dz\,dy + v\,dx\,dz + w\,dx\,dy \qquad \text{(II.1.7)}$$

The quantity that leaves it, is

$$(u+\frac{\partial u}{\partial x}dx)\,dz\,dy+(v+\frac{\partial v}{\partial y}dy)\,dx\,dz+(w+\frac{\partial w}{\partial z}dz)\,dx\,dy \qquad \text{(II.1.8)}$$

If the flow is steady and the fluid is incompressible, the quantity of water that enters the element is equal to that leaves it. Therefore, equating expression (II.1.7) and (II.1.8) we get:

$$(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z})\ dz\,dx\,dy=0$$

Since dx, dy, dz are dimensions of the element, the expression in the bracket must be equal to zero, or

$$(\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}+\frac{\partial w}{\partial z})=0 \qquad \text{(II.1.9)}$$

Substituting values of u, v, w in equation (II.1.6) we obtain:

$$(\frac{\partial^2 \phi}{\partial x^2}+\frac{\partial^2 \phi}{\partial y^2}+\frac{\partial^2 \phi}{\partial z^2})=0 \qquad \text{(II.1.10)}$$

Equation (II.1.10) is the well known Laplace's equation which is encountered in many branches of applied mathematics.

It must be remembered that since the Potential Law ,equation (II.1.10), has been derived from Darcy's Law eq.(II.1.3), it is subjected to all its limitations. Equation (II.1.10) deals with the three dimensional flow. In weirs and other similar structures, where the width of the river is considerable, the flow may be considered as two dimensional, as the flow at any cross section of the structure is not appreciably influenced by any cross flow from the sides. In case of drainage works, undersluice adjacent to head regulator and other narrow structures, the influent from the sides will be considerable, and the flow will approximate to three dimensional. For two dimensional flow equation (II.1.10) reduces to

$$\left(\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2}\right) = 0 \qquad \text{(II.1.11)}$$

Graphically, equation (II.1.11)) can be represented by two sets of curves that intersect at right angles. The curves of one set are called flow lines, whereas the curves of the other set are known as equipotential lines. The combine representation of two sets of lines is called a flow net.

- **Potential and Flow function**

Introducing the concept of the velocity potential function ϕ (x , z) defined as

$$\phi = - kh \qquad \text{(II.1.12)}$$

the velocity component are given by

$$Vx = \frac{\partial \phi}{\partial x} \quad \text{and} \quad Vz = \frac{\partial \phi}{\partial z} \qquad \text{(II.1.13)}$$

and Laplace's equation may be expressed as

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial z^2} = \nabla^2 \phi = 0 \qquad \text{(II.1.14)}$$

If second function ψ (x , z) is introduced such that

$$Vx = \frac{\partial \psi}{\partial z} \qquad \text{and} \qquad Vz = -\frac{\partial \psi}{\partial x} \qquad \text{(II.1.15)}$$

Then it may be seen that

$$\frac{\partial^2 \psi}{\partial x^2} + \frac{\partial^2 \psi}{\partial z^2} = 0 \qquad \text{(II.1.16)}$$

Thus the function ψ, called the flow function or the stream function, also satisfies the Laplace's equation. The two functions ψ and ϕ which satisfy the Laplace's equation are, in fact, both parts of solution of solution of the equation. Their physical significant is examined below. The potential function ϕ (x , z) gives distribution of ϕ in the xz plane. Consider a curve along which ϕ (x , z) = constant = $\phi 1$, as shown in fig. (II.1.4). To obtain the tangent on a point on the curve, the total differential of the function is written as

$$d\phi = \frac{\partial \phi}{\partial x} dx + \frac{\partial \phi}{\partial z} dz$$

$$= Vx\ dx + Vz\ dz$$

Then for constant ϕ, $d\phi=0$ and the slope of the tangent is given by

$$0 = \frac{\partial \phi}{\partial x} dx + \frac{\partial \phi}{\partial z} dz \ , \quad Vx\ dx = - Vz\ dz$$

$$\frac{dz}{dx} = -\frac{Vx}{Vz} \qquad \text{(II.1.17)}$$

The curve along which ϕ is constant is called the equipotential line. It is a line of constant head 'h' or energy.

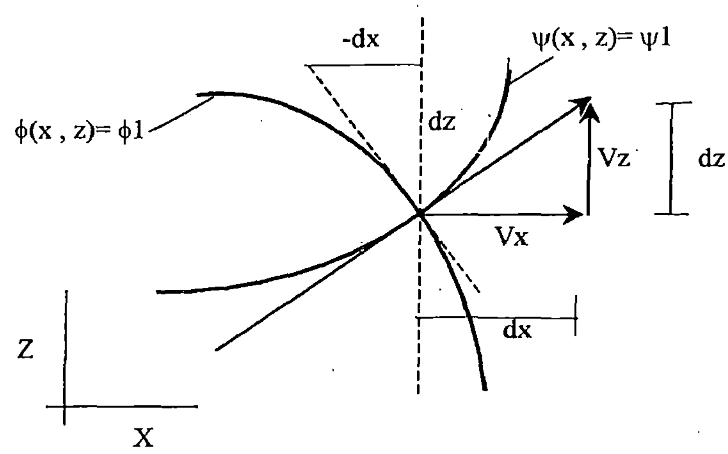


Fig. (II.1.4) Intersection between Flow line and Potential line

Considering a second curve defined by a constant flow function, the total differential of the function $\psi(x, z)$ is

$$\partial\psi = \frac{\partial\psi}{\partial x}dx + \frac{\partial\psi}{\partial z}dz$$

$$= -Vz\ dx + Vx\ dz$$

As the function $\psi(x, z) = \text{constant} = \psi 1$, $d\psi = 0$ and the slope of the tangent is given by

$$\frac{dz}{dx} = \frac{Vz}{Vx} \qquad \text{(II.1.18)}$$

The curve of constant ψ is called the flow line or stream line. Compring eq. (II.1.17) and eq. (II.1.18) it is apparent that the product of gradient of an equipotential line and flow line is (-1) ie. the two lines the equipotential line and the flow line are orthogonal curves.. The form of these curves is governed by the boundary condition of the problems.

II.2. DETERMINATION OF UPLIFT PRESSURE BY MEANS OF CONFORMAL MAPPING TECHNIQUE

a. Flat Bottom Weir without Sheet Pile Resting on a Porous Medium of Infinite Depth

In this case we consider the simplest example of confined flow of ground water under hydraulic structure -the flow about a flat bottom weir on soil of infinite depth- as shown in fig. (II.2.1a). This figure describes the physical domain in Z-plane, the weir rests on a homogeneous isotropic porous medium of infinite depth. The Schwarz-Christoffel transformation that maps the Z-plane on to the lower half of the t-plane is

$$Z = M\int \frac{dt}{(t+b)^{1-B/\pi}(t-b)^{1-C/\pi}} + N$$

Since the intervals angles at vertices B and C are equal to π, we get,

$$Z = M\int \frac{dt}{(t+b)^{0}(t-b)^{0}} + N$$

$$Z = M\,t + N \qquad \text{(II.2.1)}$$

To determine the constants M and N, we set the corresponding conditions, at point O values of Z=0 and t=0 then N=0, at point C, values of Z=L and t=b, we get value of M=L/b. Substituting these values into eq. (II.2.1), we get the required transformation of Z plane onto lower half of t plane as:

$$Z = \frac{L}{b}\,t \qquad \text{(II.2.2)}$$

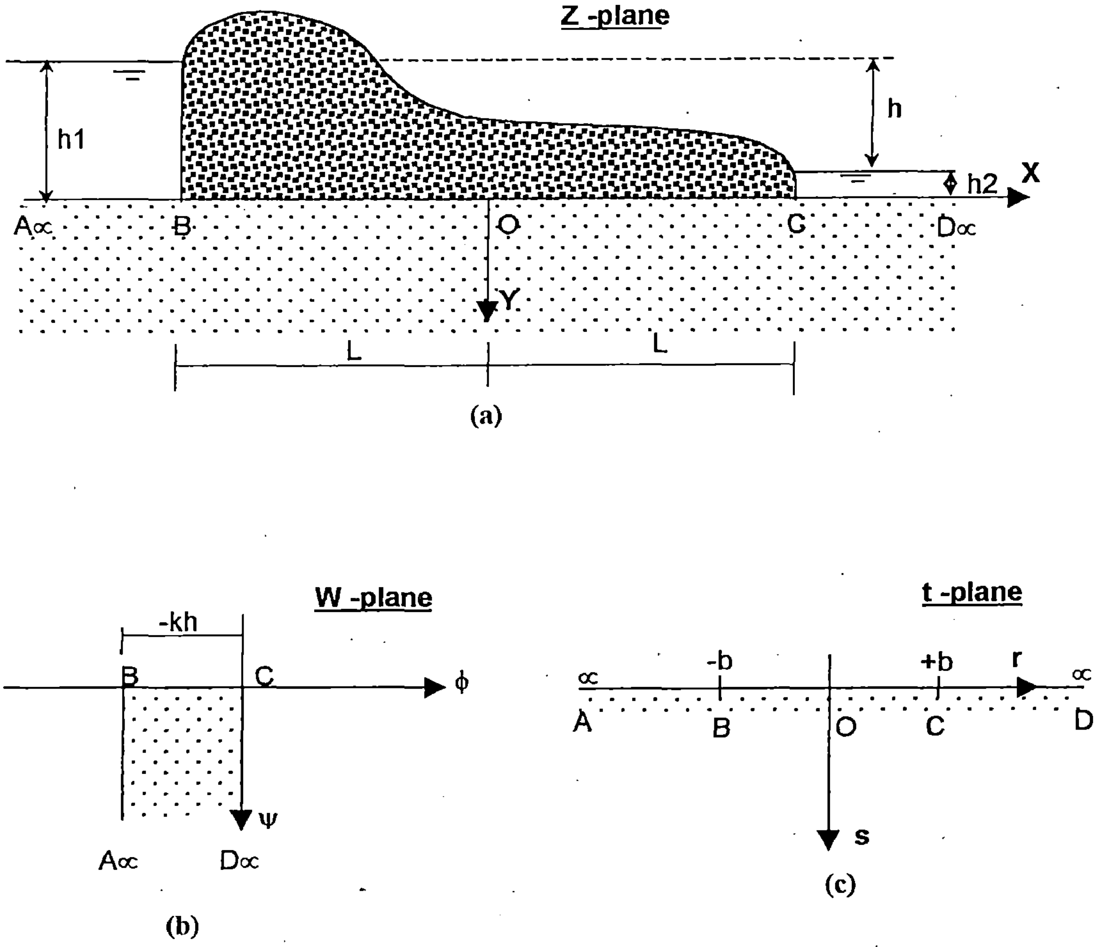


Fig. (III.2.1) Flat Bottom Weir without Sheet Pile

The transformation of w plane onto t plane is given by,

$$w = M1\int \frac{dt}{(t+b)^{0.5}(t-b)^{0.5}} + N1$$

$$= -iM1\int \frac{dt}{\sqrt{b^2 - t^2}} + N1$$

$$= M2\left(\text{asin}\,(\frac{t}{b}) \right) + N1 \qquad \text{(II.2.3)}$$

At point C, values of w=0 and t=b, hence N1= -0.5π M2. At point B, values of w= -kh and t=-b, hence M2= -kh/π. Substituting these values into eq. (II.2.3), we get transformation of w plane onto lower half of t plane as:

$$w = -\frac{kh}{\pi}\,\text{asin}\,(\frac{t}{b}) + \frac{kh}{2} \qquad \text{(II.2.4)}$$

or

$$(\frac{t}{b}) = \sin\left(\frac{\pi}{2} - \frac{w\pi}{kh}\right)$$

$$t = b\,\cos\left(\frac{w\pi}{kh}\right) \qquad \text{(II.2.5)}$$

Finally, substituting eq.(II.2.5) into eq.(II.2.2), we get for the required transformation between all points in the Z and w planes, that is

$$Z = L\cos\left(\frac{w\pi}{kh}\right) \qquad \text{(II.2.6)}$$

From eq.(II.2.6) we can develop equations for stream lines and equipotential lines, substituting $w = \phi + i\psi$ into that equation, we get,

$$Z = L\cos\left(\frac{\phi\pi}{kh} + i\frac{\psi\pi}{kh}\right)$$

$$Z = L\cos\left(\frac{\phi\pi}{kh}\right)\cos\left(i\frac{\psi\pi}{kh}\right) + L\sin\left(\frac{\phi\pi}{kh}\right)\sin\left(i\frac{\psi\pi}{kh}\right)$$

substituting $\phi' = \left(\frac{\phi\pi}{kh}\right)$ and $\psi' = \left(\frac{\psi\pi}{kh}\right)$ we get,

$$Z = L\cos\phi'\cosh\psi' + i\,L\sin\phi'\sinh\psi' \qquad \text{(II.2.7)}$$

Equating the real and imaginary parts of eq. (II.2.7) we get,

$$X = L\cos\phi'\cosh\psi'$$

$$Y = L\sin\phi'\sinh\psi'$$

$$\frac{X^2}{L^2\cosh^2\psi'} + \frac{Y^2}{L^2\sinh^2\psi'} = 1 \qquad \text{(II.2.8)}$$

and

$$\frac{X^2}{L^2\cos^2\phi'} - \frac{Y^2}{L^2\sin^2\phi'} = 1 \qquad \text{(II.2.9)}$$

Equation (II.2.8) gives the locus of a stream line for given ψ' and equation (II.2.9) gives the locus of equipotential line for given ϕ'. Now assuming ψ' equal to a sequence of a constant , say ψ'_n, we find the stream lines to be the ellipses with foci at the points +L and –L. The equipotential lines will be confocal hyperbolas. A portion of the resulting flow net is shown in fig. (II.2.2) below.

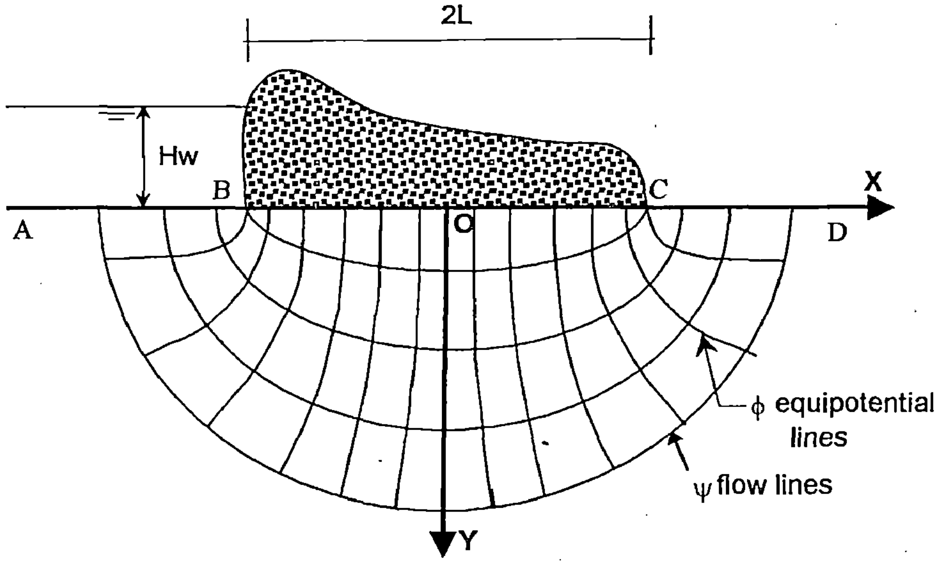


Fig. (II.2.2) Flow net

Let us consider now, the nature of uplift pressure acting along the base of the structure. Calling the pressure excess of tail water pressure Δp and noting that ϕ was taken as zero along CD (where y=0), from eq.(II.2.6) we have,

$$\Delta p = \frac{h\,\gamma w}{\pi} \text{ acos } \left(\frac{x}{L}\right) \qquad \text{(II.2.10)}$$

A plot of Δp/γw as a function of x/L is given in fig.(II.2.3). The excess pressure force P (per unit length) acting on the base of the structure is

$$P = \int_{-L}^{L} \Delta p \;\, dx = \frac{h\,\gamma w}{\pi}\int_{-L}^{L} \operatorname{acos}(\frac{x}{L})\, dx = h\, L\, \gamma w \qquad \text{(II.2.11)}$$

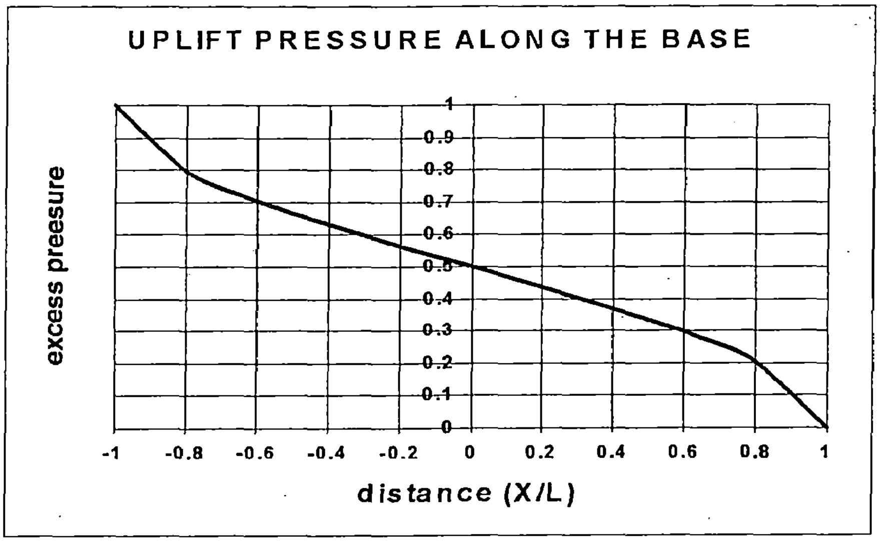


Fig. (II.2.3)

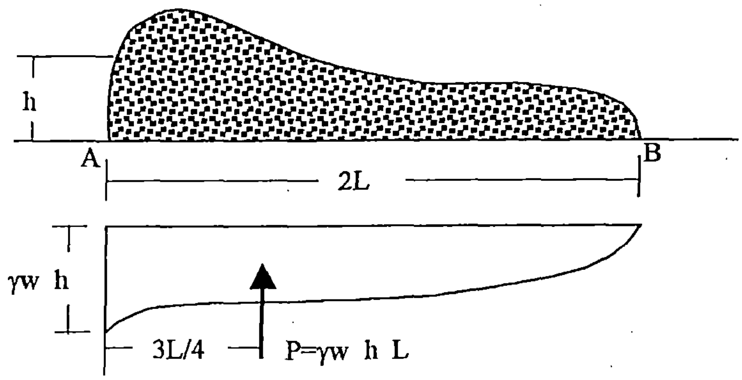


Fig. (II.2.4) Uplift Force

The moment of these uplift forces relative to the heel of structure (point A) is

$$MA = \int_{-L}^{L} (x+L)\,\Delta p \;dx = \frac{h\,\gamma w}{\pi}\int_{-L}^{L} (x+L)\;\operatorname{acos}\left(\frac{x}{L}\right) dx = \frac{3}{4} h\,L^2\,\gamma w \qquad (II.2.12)$$

where the moment arm X of the pressure force (measured from the heel, point A) is found to be

$$X = \frac{MA}{P} = \frac{3L}{4} \qquad (II.2.13)$$

b. <u>Flat Bottom Weir with Single Sheet Pile Resting on a Porous Medium of Infinite Depth.</u>

As the next example of the solution of confined flow problems we shall consider a structure with a single sheet pile (depth s) resting on the surface of an infinite depth of porous medium. To provide the general solution to this problem, we shall choose the Z plane as shown in fig. (II.2.5a). In this figure, BCDEF represents the bottom of the contour of the structure including sheet pile CDE. Once general solution is determined, various special cases can be obtained by adjusting the length of L1,L2 and s.

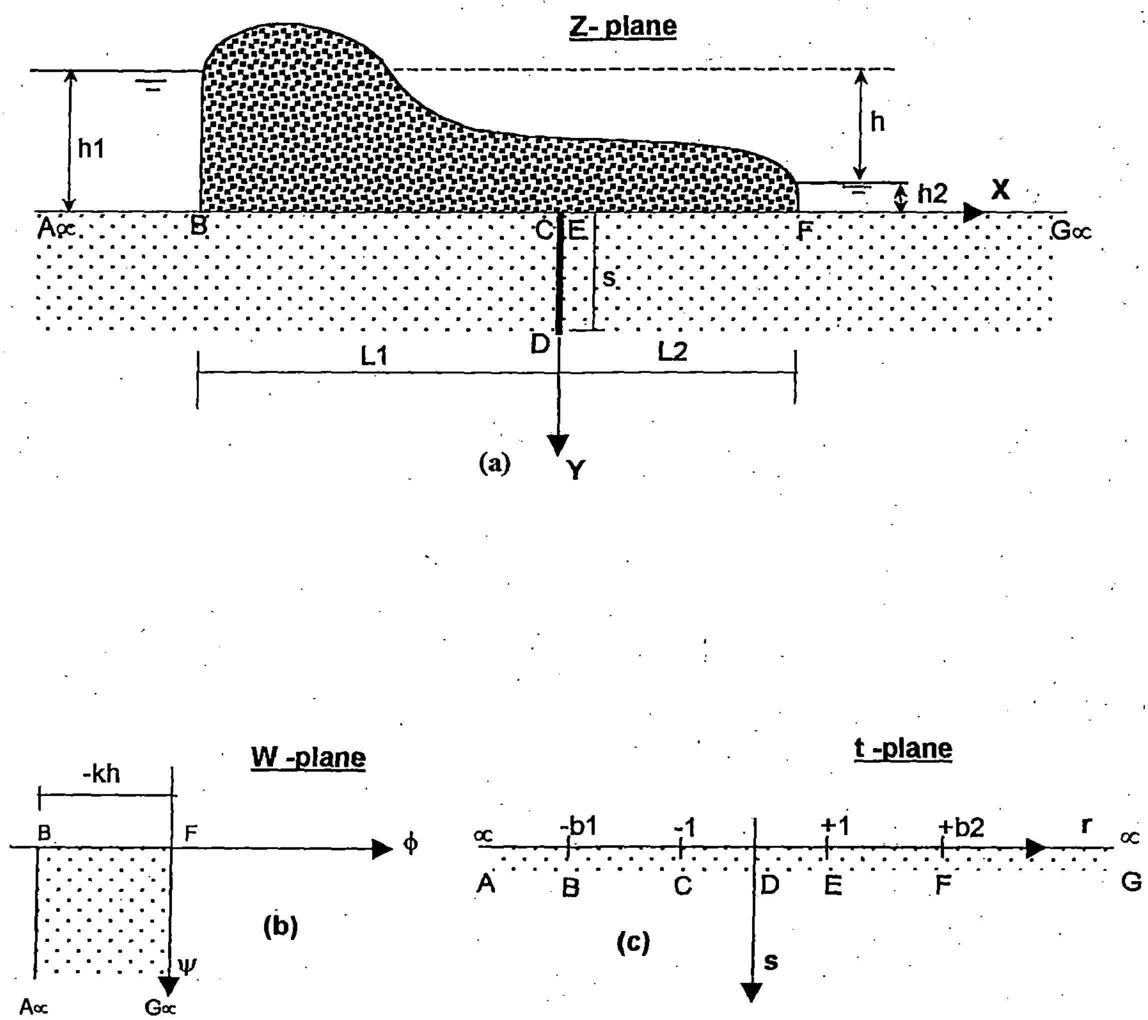


Fig. (II.2.5) Flat Bottom Weir with Single Sheet Pile

We begin by mapping the Z plane onto the lower half of t plane . Following the mapping rule the points A∞, C, D, E, G∞ are mapped onto - ∞, -1 , 0, 1, +1, + ∞ on the real axis of the t plane. The points B, and F will be located on the points t=-b1 and t=+b2, which are to be determined. The Schwarz-Christoffel transformation is

$$Z = M\int \frac{dt}{(t+1)^{1-0.5}\,(t-0)^{1-2}\,(t-1)^{1-0.5}}$$

$$= M\int \frac{t\,dt}{(t+1)^{0.5}\,(t-1)^{0.5}} + N$$

$$Z = M\sqrt{t^2-1} + N \qquad (II.2.14)$$

To evaluate the constants N and M, we consider the corresponding values of Z and t at the vertices E and D. At point E, value of Z=0 and t=1, hence N=0. At point D value of Z=is and t=0, therefore M=s. Now if these values are substituted into equation (II.2.14) we get:

$$Z = s\sqrt{t^2-1} \qquad (II.2.15)$$

The complex potential plane for the flow domain is shown in fig. (II.2.5b). Mapping of the w plane fig. (II.2.5b) onto the lower half of the t plane is given by

$$w = M1\int \frac{dt}{(t+b1)^{0.5}(t-b2)^{0.5}} + N1$$

$$w = M1\ \operatorname{asin}\left(\frac{t+\lambda 1}{\lambda}\right) + N1 \qquad (II.2.16)$$

where $\lambda=(b1+b2)/2$ and $\lambda1=(b1-b2)/2$. At point F , value of w=0 and t=b2, substituting these values into eq.((II.2.16) we get $N1= - (\pi M1)/2$. The condition at point B gives value of w=-kh and t=-b1, substituting this into eq.((II.2.16) we get value of $M1=-kh/\pi$. Now we can rewrite eq.(II.2.16) become:

$$w = -\frac{kh}{\pi}\,\mathrm{asin}\left(\frac{t+\lambda 1}{\lambda}\right)+\frac{kh}{2} \qquad \text{(II.2.17)}$$

or

$$\left(\frac{\lambda 1+t}{\lambda}\right) = \sin\left(\frac{\pi}{2}-\frac{w\pi}{kh}\right)$$

$$t = \lambda\ \cos\left(\frac{w\pi}{kh}\right)-\lambda 1 \qquad \text{(II.2.18)}$$

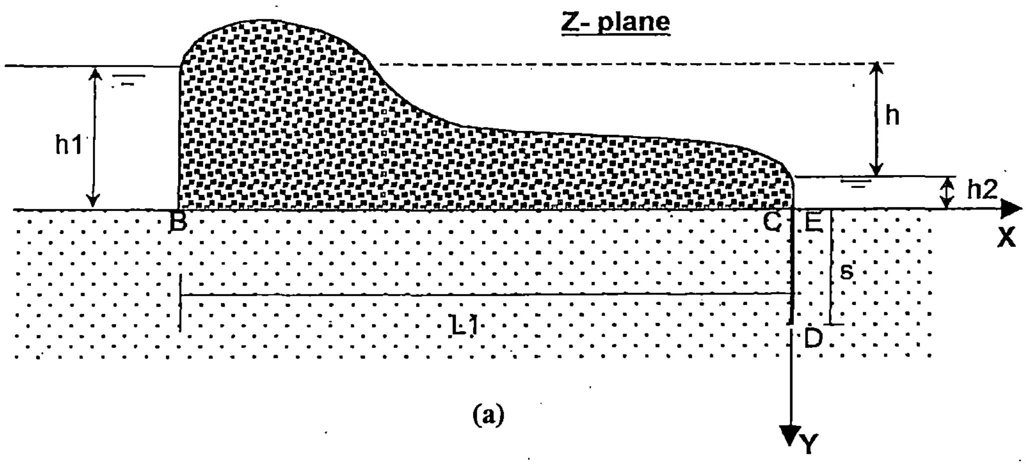
Finally, substituting eq.(II.2.18)) into eq.(II.2.15), we get for the required transformation between all points in the Z and w planes,

$$Z = s\left[\left(\lambda\cos\left(\frac{w\pi}{kh}\right)-\lambda 1\right)^{2}-1\right]^{0.5} \qquad \text{(II.2.19)}$$

Where

$$\lambda = 0.5\left[\sqrt{\left(\frac{L1}{s}\right)^{2}+1}\ +\sqrt{\left(\frac{L2}{s}\right)^{2}+1}\right] \qquad \text{(II.2.19a)}$$

$$\lambda 1 = 0.5\left[\sqrt{\left(\frac{L1}{s}\right)^{2}+1}\ -\sqrt{\left(\frac{L2}{s}\right)^{2}+1}\right] \qquad \text{(II.2.19b)}$$



(a)

Fig. (II.2.6) Sheet Pile at Toe of Structure

For the particular structure in which a sheet pile is located at the toe of as shown in fig. (II.2.6), equating eq.(II.2.15) at point B, Z=-L1 and t=-b1. At point E, Z=0, t=1, we get:

$$b1 = \sqrt{\left(\frac{L1}{s}\right)^2 + 1} \qquad \text{(a)}$$

$$\lambda = 0.5\left[\sqrt{\left(\frac{L1}{s}\right)^2 + 1} \;\; + 1\right] \qquad \text{(b)}$$

$$\lambda 1 = 0.5\left[\sqrt{\left(\frac{L1}{s}\right)^2 + 1} \;\; - 1\right] \qquad \text{(c)} \qquad \text{(II.2.20)}$$

For design purpose we need to know the hydrodynamic acting along the various section of the structure. We consider first the computation of pressure distribution. As the y axis has been taken positive downward, the velocity potential function is given by

$$\phi = -k\left(\frac{p}{\gamma w} - y\right) + C \qquad \text{(II.2.21)}$$

Now, noting that at y=0, ϕ=-kh and p/γw=h1, we also find C = -k (h - h1). Along the impervious boundary ψ=0, w=ϕ. Hence from eq. (II.2.18) we get

$$t = \lambda \cos\left(\frac{\phi\pi}{kh}\right) - \lambda 1$$

$$\phi = -\frac{kh}{\pi} \operatorname{acos}\left(\frac{t + \lambda 1}{\lambda}\right) \qquad \text{(II.2.22)}$$

$$0 \leq \operatorname{acos}\left(\frac{t + \lambda 1}{\lambda}\right) \leq \pi$$

from eq. (II.2.21), we get:

$$-k\left(\frac{p}{\gamma w} - y\right) - k\,(h - h1) = -\frac{kh}{\pi} \operatorname{acos}\left(\frac{t + \lambda 1}{\lambda}\right)$$

$$\frac{-p}{\gamma w} = -\frac{h}{\pi} \operatorname{acos}\left(\frac{t + \lambda 1}{\lambda}\right) + (h - h1) - y$$

$$p = \gamma w\left[\frac{h}{\pi} \operatorname{acos}\left(\frac{t + \lambda 1}{\lambda}\right) + y - h + h1\right] \qquad \text{(II.2.23)}$$

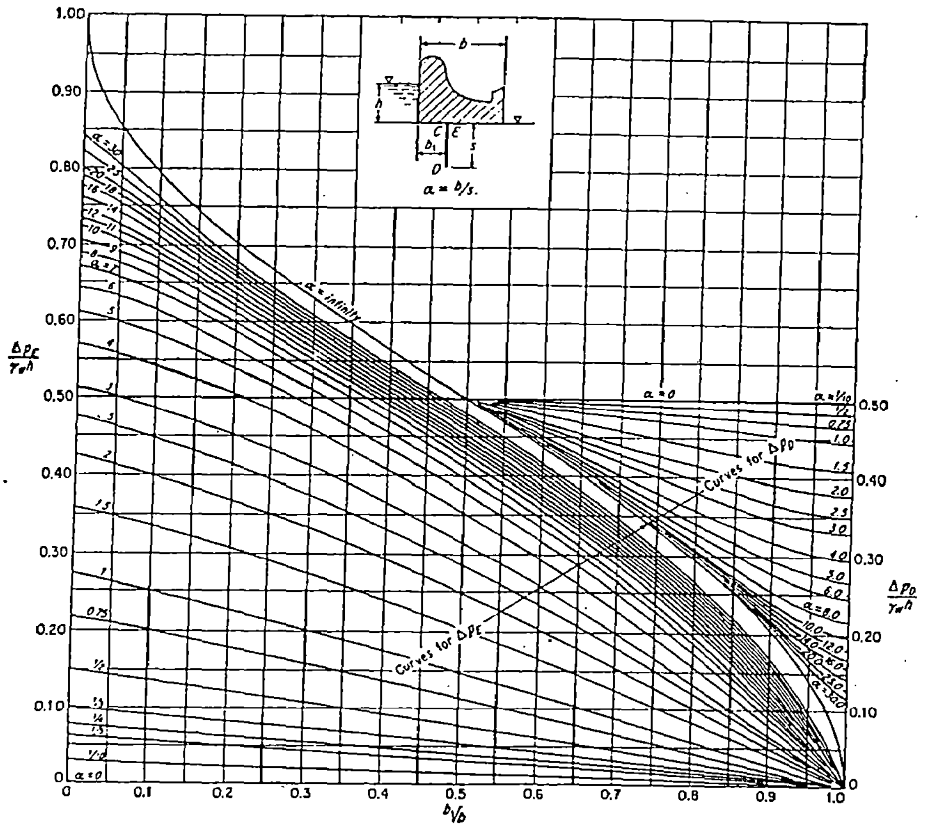
Eq. (II.2.23) is the pressure at any point along the impervious boundary. For a flat bottom weir with a sheet pile at any intermediate point, by putting y=0 and substituting eq. (II.2.15) into eq. (II.2.23) we get the pressure along the base equal to

$$p = \gamma w\left(\frac{h}{\pi}\operatorname{acos}\frac{\lambda 1\, s \pm \sqrt{s^2 + x^2}}{\lambda\, s} + h2\right) \qquad \text{(II.2.24)}$$

And the pressure along sheet pile by putting x=0, we get

$$p = \gamma w\left(\frac{h}{\pi}\operatorname{acos}\frac{\lambda 1\, s \pm \sqrt{s^2 - y^2}}{\lambda\, s} + y + h2\right) \qquad \text{(II.2.25)}$$

In fig. (II.2.7) are shown curves prepared by Khosla, Bose, and Taylor that give pressure in the water Δp in excess of hydrostatic pressure relative to tail water elevation at point C, D, E of fig. (II.2.5) for any position of the sheet pile. The values of Δp at point E (ΔpE) can be obtained directly from the plot. For example, for sheet pile of s = b/2 (α=b/s=2) located at one-fourth of the base length from heel of the structure (b1/b=1/4), the pressure in the water at point E, in excess of tail water hydrostatic pressure, will be ΔpE = 0.35 h γw. In like manner, pressure at point D for b1/b >= ½ can be obtained directly from the curves labeled 'curves for ΔpD' on the figure. For values of b1/b < ½ the abscissa is entered with values of (1 – b1/b), and the desired value of ΔpD/ h γw is obtained by subtracting the determined value from unity. Thus, from the example given above, ΔpD = 0.57 h γw. To find the excess pressure at point C, the abscissa is



<u>**Fig. (II.2.7) Uplift Pressure at Points C D E**</u>

entered with (1 – b1/b), and the value of $\Delta pE / h\ \gamma w$ is determined for the proper α from ΔpE curves. Then the desired $\Delta pC / h\ \gamma w$ is obtained by subtracting this value from unity. For the example above, $\Delta pC = 0.87$.

c. Khosla's Theory

The usual barrage and weir sections do not conform to a simple elementary form and direct solution of the Laplace equation is not available. To apply the analytic solution to any practical composite profile of a weir or a barrage, Khosla and his associate (1936) evolved the method of independent variables. In this method a composite barrage or weir section split up into a number of simple standard forms of known analytical solutions. The most useful standard forms among these are

- A straight horizontal floor of negligible thickness with a sheet pile at either end fig. (II.2.8a) and fig. (II.2.8b).
- A straight horizontal floor depressed below the bed but with no vertical cutoff fig. (II.2.8c).
- A straight horizontal floor of negligible thickness with a sheet pile at some intermediate position fig. (II.2.8d)

In general the usual barrage or weir section consist of a combination of all the three forms mentioned above, the entire length of the floor with any a pile lines, etc. making up one such form. Each elementary form is then treated as independent of the other. The pressure at the key points are then read off from the curve fig. (II.2.9).

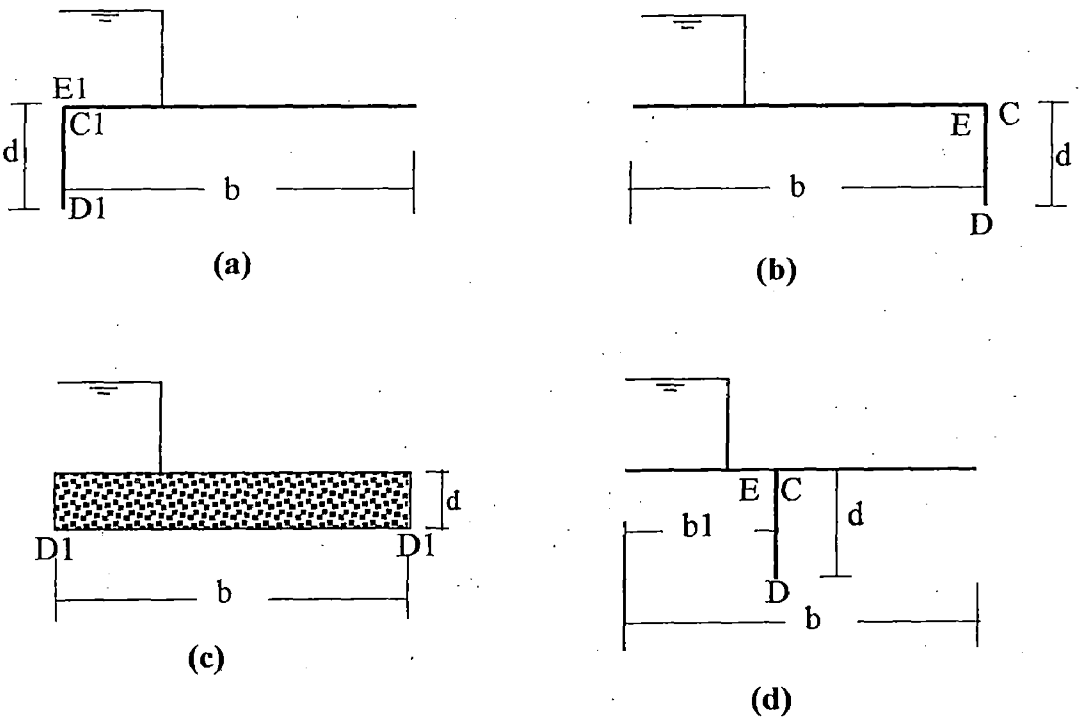
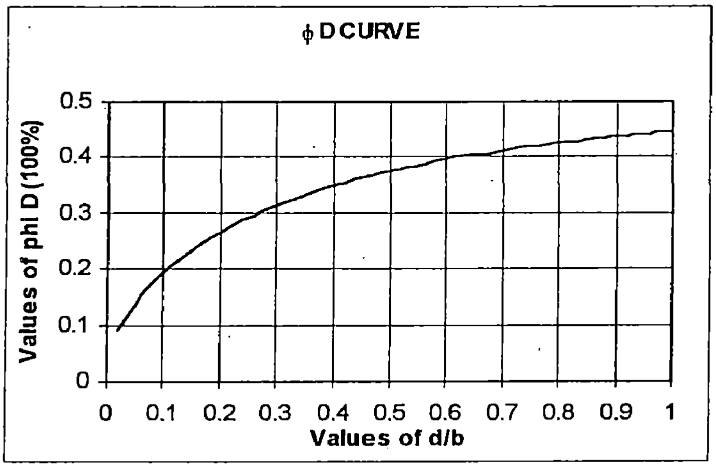
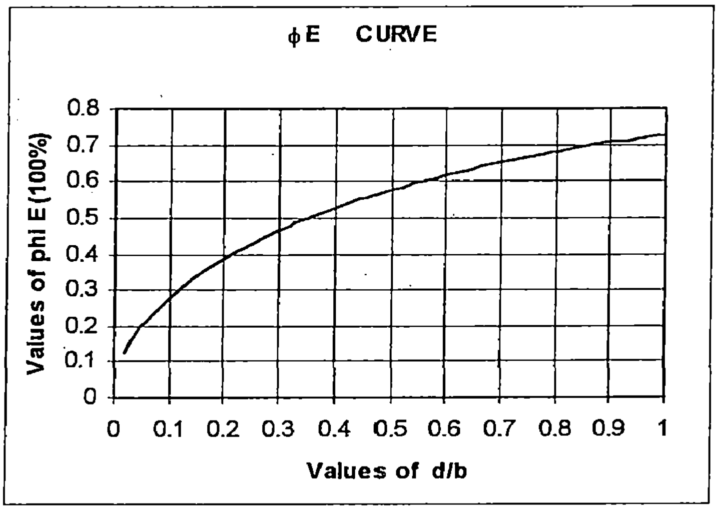


Fig. (II.2.8) Standard Forms – Khosla's Method of Independent Variable





$\phi E = 1/\pi \ \text{acos} \ (\lambda - 2)/\lambda$

$\phi D = 1/\pi \ \text{acos} \ (\lambda - 1)/\lambda$

$\phi C1 = 100 - \phi E$

$\phi D1 = 100 - \phi D$

$\lambda = 0.5 \ (1 + \sqrt{(1+\alpha^2)})$

Fig. (II.2.9) Khosla's Chart

II.3. STABILITY ANALYSIS OF SOIL BASE

a. Stability Against Rotational Sliding Failure

a.1. Rotational Sliding due to Land Slope

The concept of rotational sliding along a circular failure surface originated in Sweden where failure surfaces in soft clays were observed to closely resemble arch of a circle. Fellenius presented in 1918 an analysis of slope failure using a circular slip surface, where the soil was regarded as purely cohesive material ($\phi t=0$). He further extended it to "c-ϕ" soils. This analysis based on a circular slip circle has been known since about 1930 as the "Swedish Slip Circle Method".

The assumption of circular failure surface gives the simplest solution, although it may lead to larger values of safety factor (2 to 7 %), because in most cases the critical shear failure surface for the minimum safety factor is non circular.

The basic approach in the slip circle method is to consider the overall moment equilibrium of a sliding mass of soil above a trial failure arc. The sliding mass is divided into a reasonable number of slices (6 to 10), a typical slice is shown in fig. (II.3.1) Taking moment about the centre of rotation, the overall moment of equilibrium can be expressed as:

$$\Sigma W x = SF (\Sigma S L R) \qquad (II.3.1)$$

where: W = weight of slice

x = horizontal distance between cg of slice and centre of slip circle

L = length of slice base

R = radius of slip circle

S = shear stress for equilibrium

SF = safety factor

The side forces on the slices are not included in the moment equation, because when all the slices are considered, the net moment of side forces will be zero.

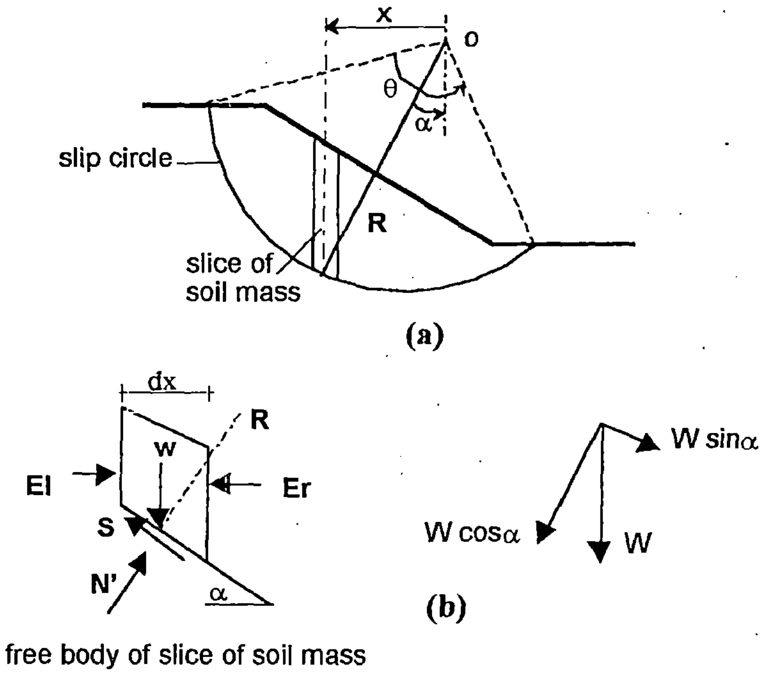


Fig. (II.3.1) Forces on a Slice in The Slip Circle

Assuming SF is constant along the entire circular arc, using eq. (II.3.1), we can rewrite the equation in the form as follows:

$$SF = \frac{\sum S\ L R}{\sum W\ x} = \frac{Mr}{Md} \qquad (II.3.2)$$

where Mr and Md are resisting moment and driving moment respectively, thus the safety factor is ratio between the average shear strength of soil to the average shear stress required for equilibrium along a potential failure surface.

For slope in clays where $\phi=0$ condition applies Su=Ct (undrained shear strength) and it is independent of the normal stress. Eq. (II.3.2) can be written as:

$$SF = \frac{\sum S\ L\ R}{\sum W\ x} = \frac{\sum Ct\ L}{\sum W\ \sin\alpha} \qquad \text{(II.3.3)}$$

where α is inclination of slice base to the horizontal, Ct is soil cohesion and L is length of slice base. For soil having C and ϕ values, the shear strength of soil not only depends on cohesion but also depends on the normal stress acting on it. Now, the weight of soil W resolved into normal and tangential components, $N=W\cos\alpha$ and $T=W\sin\alpha$, as shown in Fig. (II.3.1). The sliding force is tangential component (T) and the resisting force is the shear strength, which is the sum of the cohesive and the frictional strengths at the base of each slice. Thus,

$$SF = \frac{Mr}{Md} = \frac{R\ \sum(C\ L + N\ \tan\phi)}{R\ \sum W\ \sin\alpha}$$

$$SF = \frac{\sum(C\ L + N\ \tan\phi)}{\sum W\ \sin\alpha} \qquad \text{(II.3.4)}$$

If effective parameters are used, C' and ϕ', the normal force is reduced by the water force N'=p L, where p is the average pore water pressure on the slice bottom. The safety factor is expressed as:

$$SF = \frac{\sum (C' L + N \tan \phi')}{\sum W \sin \alpha} \qquad \text{or}$$

$$SF = \frac{C' Lt + \tan \phi' \sum (N - N')}{\sum W \sin \alpha} \qquad (II.3.5)$$

where Lt is total length of sliding line.

a.2. Rotational Sliding due to External Load

Now let us apply the above approach to a problem as shown in fig. (II.3.2). In this case, the driving load is induced by external load (structural loads), which consist of vertical loads of structure, moment due to eccentric loads of structure and moment due to horizontal water thrust (earthquake not considered). These loads will influence stresses either in the soil base (interface between the structure and the soil strata) or base of slice. Thus, the normal force acting on the base of slice must be added by the external load q'. The driving moment due to soil weight within the slip circle is equal to zero, because it is symmetrical about vertical line C-O through the centre of the circle. The driving moment in eq.(II.3.5) is ,therefore, replaced by total external force multiplied by its distance to the centre line of circle. Now we can rewrite eq.(II.3.5) as

$$N = (w + q') \cos\alpha$$

$$SF = \frac{\sum R\,(C'\,Lt + (N - N') \tan \phi')}{P\,x} \qquad (II.3.6)$$

To calculate stresses beneath the foundation level, we will use a method called 'sixty-degree approximation'. This method is widely used in making rough estimates of subsurface stresses resulting from a loaded foundation area. In this method, it is assumed that the subsurface vertical stresses spread out uniformly with depth, the stress area increasing at aslope of 1 horizontally for each 2 of depth measured from the edges of foundation as shown in fig. (II.3.2)

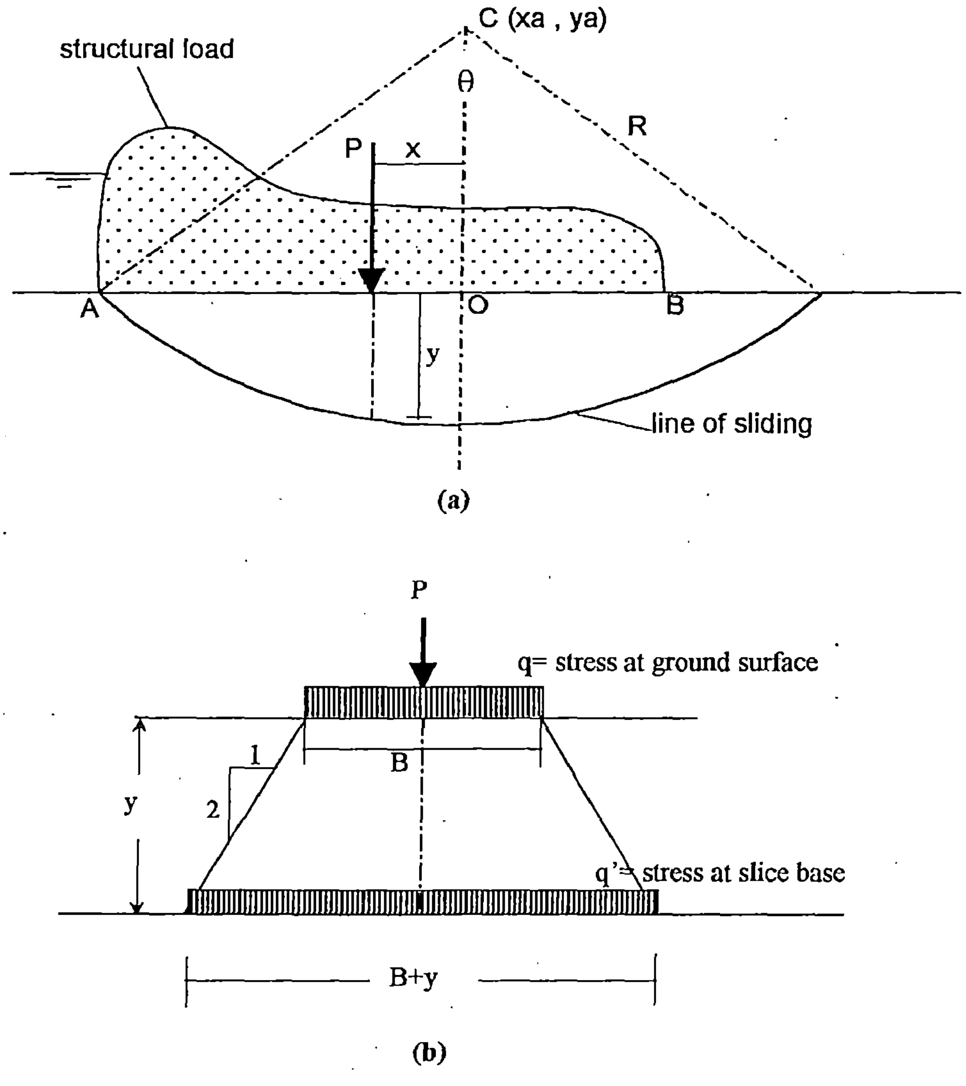


Fig. (II.3.2) Stress Distribution in The subsurface due to External Load

At a given depth, the subsurface stress q' is assumed to be uniform over the area stressed. Then the stress at depth y becomes

$$q' = \frac{P}{B+y} \qquad (II.3.7)$$

a.3. Effect of Seepage and Submergence.

Soil in which water is flowing is made up of a frame work of mineral particles interspersed with water. Taylor (1948) points out that the resultants body forces of the soil and water can be determined in several ways.

1. Using the submerged soil weight and the seepage force.
2. Using the total weight of soil and water and a resultant boundary neutral force.
3. Using the true weight of soil grains, the true weight of water, and a resultant boundary neutral force.

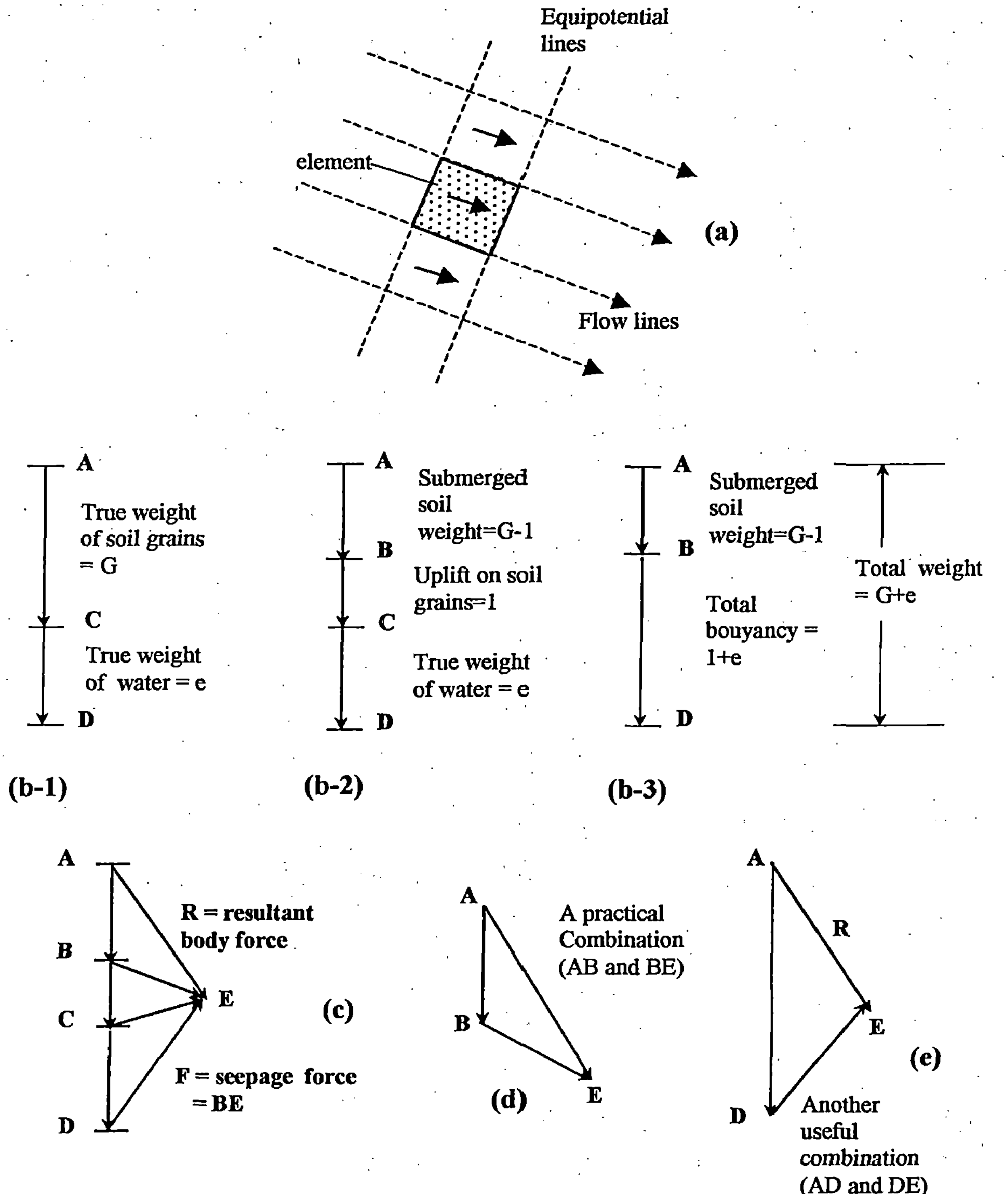
According to standard soil mechanics procedure, the true weight of soil grain in saturated soil can be expressed numerically by G, the specific gravity of the grains, and the true weight of water contained in the soil voids can be expressed by , e , the void ratio. In the metric system the total unit weight of saturated soil is (G+e) in a volume equal (1+e). The unit saturated weight is therefore (G+e)/(1+e), the unit dry weight is G/(1+e) , and the unit submerged weight is (G-1)/(1+e).

Consider the forces acting on a small element of soil subjected to seepage in fig. (II.3.3a) . In fig. (II.3.3b-1) the downward forces due to the weight of the soil and water can be shown as made up of the true weight of the soil grains (distance AC) and the true weight of water (distance CD); or in fig. (II.3.3b-2), the submerged soil weight (distance AB), the uplift in the soil grains (distance BC), and the weight of the water (distance CD). In fig. (II.3.3b-3) the total downward force is represented by the submerged soil weight (distance AB) and total buoyancy (distance BD).

In fig. (II.3.3c) the resultant body force (distant AE) may be obtained by combining the forces in several different ways. The two most practical combination are:

1. The submerged soil weight and the seepage forced.
2. The total weight of soil and water and the resultant boundary neutral force.

The way these combination can be made is shown in fig. (II.3.3d) and fig. (II.3.3e). In (d) the submerged soil is combined with the seepage force, and in (e) the saturated soil weight is combined with the resultant boundary neutral force. The resultant R is exactly the same in both cases.



Resultant body force with seepage can be obtained several ways:

- Forces AB and BE
- Forces AC and CE
- Forces AD and DE

Fig. (II.3.3) Resultant Body force in Soil with Seepage

b. Stability against Bearing Failure

A foundation is a part of structure which transmit loads from the structure to soil base. Stability of soil base, therefore, plays an importance part of whole structure. There are two basic types of foundations: shallow and deep foundation. A shallow foundation usually derive its support from the soil or rock close to the lowest part of the structure which it supports. The depth of the bearing area below the adjacent ground is usually about equal to or less than the width of bearing area. The foundation that has length considerably greater than its width is called strip foundation.

A deep foundation transmit all or portion of the loads to some significant depth below the base of the structure. The deep foundation is ordinarily intended to by pass weak or an undesirable material and carry the loads to some stratum below the unsuitable soil. A part or whole of the structural loads may also be carried by side friction or adhesion of a deep foundation.

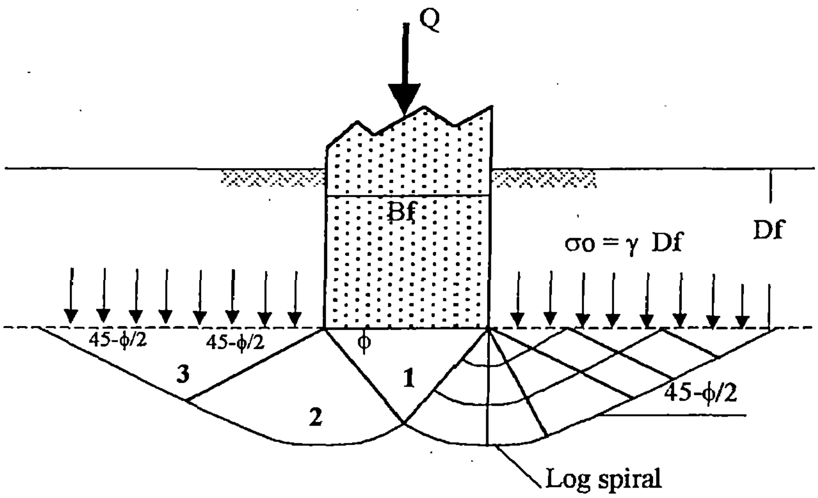


Fig. (II.3.4) Failure Surface and Zones in The Terzaghi Analysis

The bearing capacity of soil base depends on the shearing resistance on the boundary of the failure zones. This shearing resistance can be conveniently divided into three parts:

1. Cohesive resistance.
2. Frictional resistance resulting from surcharge σo at the footing level.
3. Frictional resistance resulting from the weight of soil with in the failure zones.

Although these quantities are not entirely independent, they may be considered separately and on superposition the ultimate bearing capacity may be expressed by eq. (II.3.8) given below (Tomlinsion 1980). This superposition is believed to lead to error on the safe side, (not exceeding 17% to 20% for ϕ=20 - 40 (degree), while being equal to zero when ϕ=0, Vesic 1973). [ref. 4]

$$qf = Ct\ Nc + \sigma o'\ Nq + 0.5\,\gamma\,B\,N\gamma \qquad (II.3.8)$$

Where Ct = undrained cohesion of soil,

$\sigma o'$ = effective overburden pressure at foundation level ($\gamma'D$ if submerged)

γ = unit weight of soil below foundation level (use γ' if submerged) .

Nq, Nγ, and Nc are bearing capacity factors depending only on the value of ϕ.

In case of a deep strip foundation, the potential surface of sliding for fairly isotropic soil mass may be assumed to be a logarithmic spiral starting under the foundation and ending with a vertical tangent, as shown in fig. (II.3.5).

The overburden pressure $\sigma o'$ is assumed to act at the base level of the footing. With the conservative assumption that the mass of soil within the logarithmic spiral is weightless, Zeevaert (1983) analyzed the limiting plastic equilibrium condition and obtained the values of the bearing capacity factor Nc and Nq which are given in Table (II.3.1). Further, neglecting for all practical purpose the contribution of the third term of the bearing capacity eq. (II.3.8), the ultimate bearing capacity for a deep footing may be written as:

$$qf \geq s\ [Ct\ \ Nc + \sigma o']\ \ (Id + 0.1) \qquad (II.3.9)$$

where s= shape factor

= 0.1 for strip and 1.2 for square and circular footing, and (Id+0.1)=1 for dense state. [ref. 4]

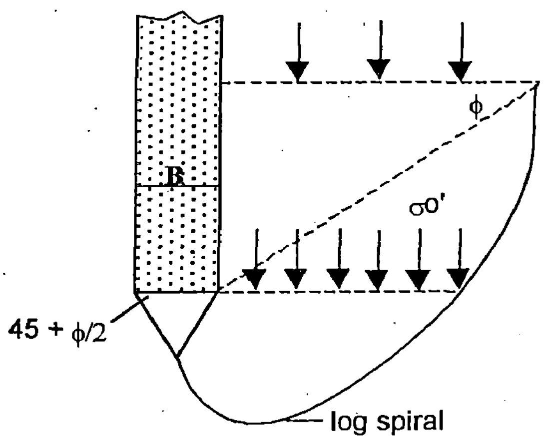


Fig. (II.3.6) Surface Failure of Deep Strip Foundation (Zeevaert 1983)

Table II.3.1

ϕ (degrees)	Nc	Nq
0	5.7	1.0
5	7.8	1.7
10	10.5	2.7
15	15	4.8
20	24	8.1
25	40	15
30	70	30
35	140	65
40	300	150
45	-	429

The capacity of horizontal resistance of deep foundation is calculated by assuming the net ultimate soil pressure distribution at the front and the back faces of the wall as shown as fig (II.3.7) [ref.4]. The point rotation is assumed at 0.2 D above the base. The resisting moment Ms per unit length of the wall is calculated as follow:

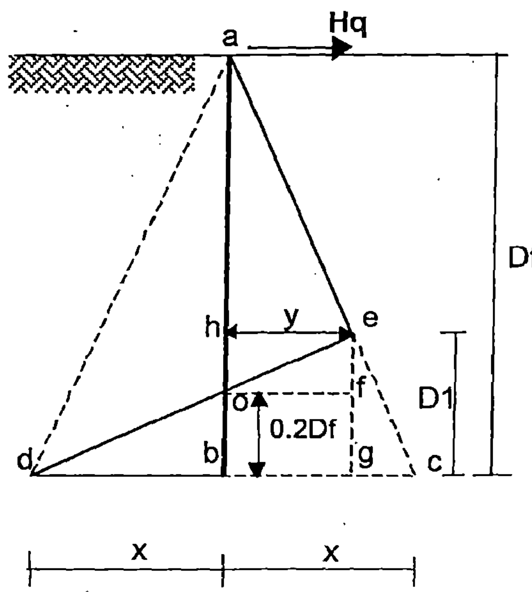


Fig. (II.3.7) Assumed Soil Distribution

From triangle dob and deg :

$$\frac{ob}{db}=\frac{0.2Df}{x}=\frac{eg}{dg}=\frac{D1-0.2Df}{y} \quad \text{or} \quad \frac{Df}{x}=\frac{5D1-Df}{y}$$

from triangle abc and ahe, we get:

$$\frac{Df}{x}=\frac{Df-D1}{y}=\frac{5D1-Df}{y} \quad \text{and} \quad D1=Df/3 \qquad \text{(II.3.10)}$$

The moment of side resistance about O is the algebraic sum of moments of triangle abc and dec :

$$Ms = 0.5Df \ x\left(\frac{Df}{3}-\frac{Df}{5}\right)+0.5\left(\frac{Df}{3}\right)\ 2x\left(\frac{Df}{5}-\frac{Df}{9}\right)$$

$$=\frac{x\ Df^2}{15}+\frac{4x\ Df^2}{135}=0.096x\ Df^2$$

$$\approx 0.1x\ Df^2$$

$$Ms = 0.1\gamma Df^3\ (Kp-Ka) \qquad \text{(II.3.11)}$$

From eq. (II.3.11), we can find out the horizontal resistance of strip foundation by putting sum moments about O:

$$Hq\ 0.8Df = 0.1\gamma\ Df^3\ (Kp-Ka)$$

$$Hq = 0.125\gamma\ Df^2\ (Kp-Ka) \qquad \text{(II.3.12)}$$

Now let us look at triangle bdo and taking moment about o , we get moment ultimate at body of strip foundation equal to

$$Mo = \gamma\ Df\ (Kp-Ka)(0.5Df\ 0.2Df)(0.2Df\ 2/3)$$

$$Mo = 0.0133\gamma\ Df\ (Kp-Ka) \qquad \text{(II.3.13)}$$

The skin friction, in term of total stress, is the ultimate adhesion Ca between the clay and the pile shaft (concrete, steel etc.), which is generally related to the undrained shear strength Ct as follows

$$fs = Ca = \alpha \ (Ct)av \qquad (II.3.14)$$

Where α = adhesion factor

Ct_{av} = average undrained cohesion over pile shaft

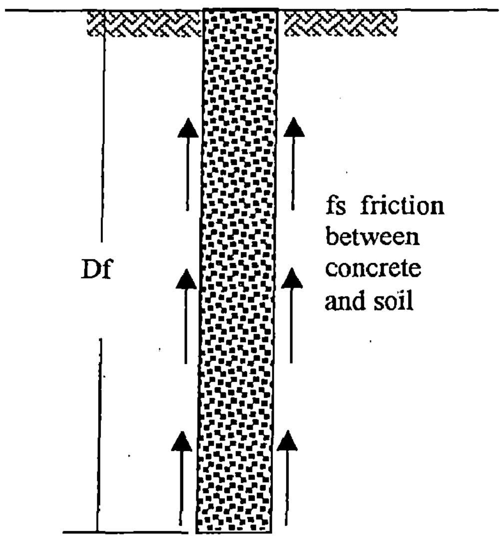


Fig. (II.3.8) Skin Friction

The adhesion factor depend on the type of the clay, the method of installation and the pile material. The appropriate value of α is obtained from the results of load tests. For driven piles, the value of α range on the average roughly from unity for soft clay to 0.5 or less for stiff clay, while for bored piles in stiff clay α is roughly from 0.3 to 0.5.

In the absence of other data, the following values of α may be adopted for preliminary design purpose:

$$\alpha = 0.3 \text{ for } Ct \geq 250 \text{ kPa}$$

$$\alpha = 0.4 \text{ for } Ct = 100 \text{ kPa}$$

$$\alpha = 0.5 \text{ for } Ct \leq 50 \text{ kPa} \qquad \text{(II.3.15)}$$

with a maximum value of $fs = 100$ kPa. A value of $\alpha = 0.45$ for the skin friction of firm to stiff clays may be adopted (with fs not exceed 100 kPa), when no previous experience is available (Skempton 1979, Tomlinson 1980).[ref. 4]

Chapter III

DESIGN OF HYDRAULIC STRUCTURE FOUNDED ON CLAYEY SOIL

III.1. A SIMPLE HYDRAULIC STRUCTURE

a. Minimum Base width Requirement

If we consider only hydrostatic force, the elementary profile of hydraulic structure will be triangular in section, as shown in fig. (III.1.1), having zero width , where water pressure is zero, and a maximum base width b, where the maximum water pressure acts. Thus the section of the elementary profile is similar to the shape of the hydrostatic pressure distribution diagram. The same profile will provide the maximum possible stabilizing force against overturning without causing tension at the base. In triangular profile, in case of empty reservoir condition, the resultant of the gravity load only acts at a distance of b/3 from the heel i.e. at the extreme upstream middle third point. Let us now consider stability of the structure in fig. (III.1.1). Taking moment about toe (point B), we get:

- moment due to self weight, hydrostatic force and uplift pressure respectively

$$Mw = \frac{\gamma c\ m^2\ h^3}{3} \qquad \text{(III.1.1a)}$$

$$Mh = \frac{\gamma w\ h^3}{6} \qquad \text{(III.1.1b)}$$

$$Mu = \frac{\gamma w\ m^2\ h^3}{3} \qquad \text{(III.1.1c)}$$

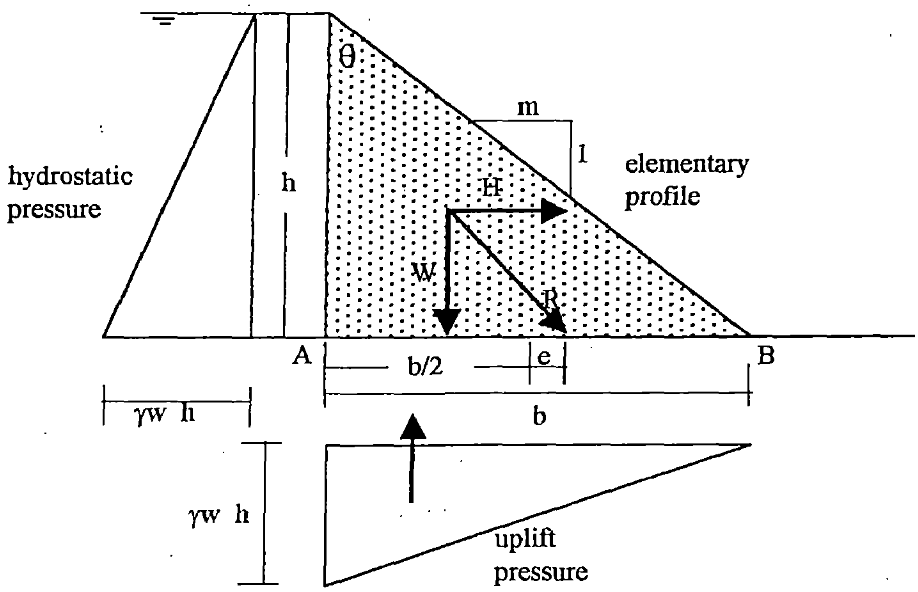


Fig. (III.1.1) Elementary Profile

- Consider the safety of the structure against overturning, it leads to condition

$$\frac{\gamma c\, m^2 h^3}{3} - \frac{\gamma w\, h^3}{6} - \frac{\gamma w\, m^2 h^3}{3} \geq 0$$

$$m \geq \sqrt{\frac{\gamma w}{2(\gamma c - \gamma w)}} = 0.598 \quad \text{or} \quad b = 0.598\, h \qquad \text{(III.1.2)}$$

- against horizontal sliding (if sf=0.75)

$$\frac{H}{W - U} \leq 0.75 \qquad \text{(III.1.3)}$$

$$\frac{\frac{\gamma w\, h^2}{2}}{\frac{\gamma c\, m\, h^2}{2} - \frac{\gamma w\, m\, h^2}{2}} \leq 0.75 \quad \text{or} \quad m \geq \frac{4\gamma w}{3(\gamma c - \gamma w)} \geq 0.952 \text{ or } b = 0.952\, h$$

- and if there is no tension at the base (uplift not considered)

    - stress due to gravity load

$$\sigma\, zu = \frac{W}{b}\left(1+\frac{6e}{b}\right) = \frac{2\,W}{b} = \frac{2\,\gamma c\, h\, b}{2\, b} = \gamma c\, h \qquad \text{(III.1.4a)}$$

$$\sigma\, zd = 0$$

    - stress due to horizontal water thrust

$$\sigma\, zu = -\frac{\gamma w\, h^3}{6}\cdot\frac{6}{b^2} = -\frac{\gamma w\, h}{m^2} \qquad \text{(III.1.4b)}$$

$$\sigma\, zd = \frac{\gamma w\, h}{m^2}$$

by combining the two values we get

$$\sigma\, zu = 0 = \gamma c\, h - \frac{\gamma w\, h}{m^2} \quad \text{or} \quad m \geq \sqrt{\frac{\gamma w}{\gamma c}} \geq 0.645 \qquad \text{(III.1.4c)}$$

- and if there is no tension at the base (uplift considered)

$$\sigma\, zu = 0 = (\gamma c - \gamma w)\, h - \frac{\gamma w\, h}{m^2} \quad \text{or} \quad m \geq \sqrt{\frac{\gamma w}{(\gamma c - \gamma w)}} \geq 0.845 \qquad \text{(III.1.4d)}$$

b. Design of Simple Hydraulic Structure Founded on Clayey Soil

A simple hydraulic structure Fig. (III.1.2) is to be constructed on a clayey soil. Dimension of structure and data of soil are as follows:

- Concrete structure, γc is taken equal to 2.4 t/m3.
- Height of structure i.e water level Hw=10 m.
- Base width is taken with respect to sliding consideration, b = 0.952 Hw (taken b=Hw=10 m).
- Saturated unit weight $\gamma sat = 2.1$ t/m3.
- Cohesion of soil Ct = 10 t/m2 (undrained).
- Angle of internal friction $\phi = 30$.
- Unit weight of water $\gamma w = 1$ t/m3.

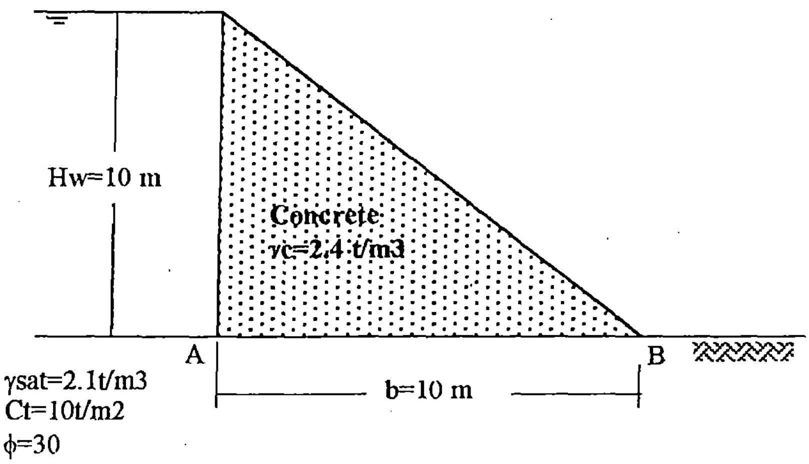


Fig. (III.1.2) Design Example of Simple Hydraulic Structure Founded on Clayey Soil

b.1. Loading Analysis

The loads acting on the structure can be drawn schematically as shown in fig. (III.1.3) below

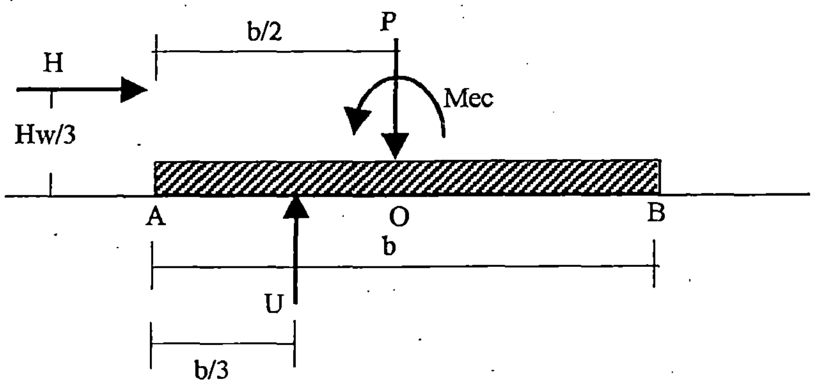


Fig. (III.1.3)

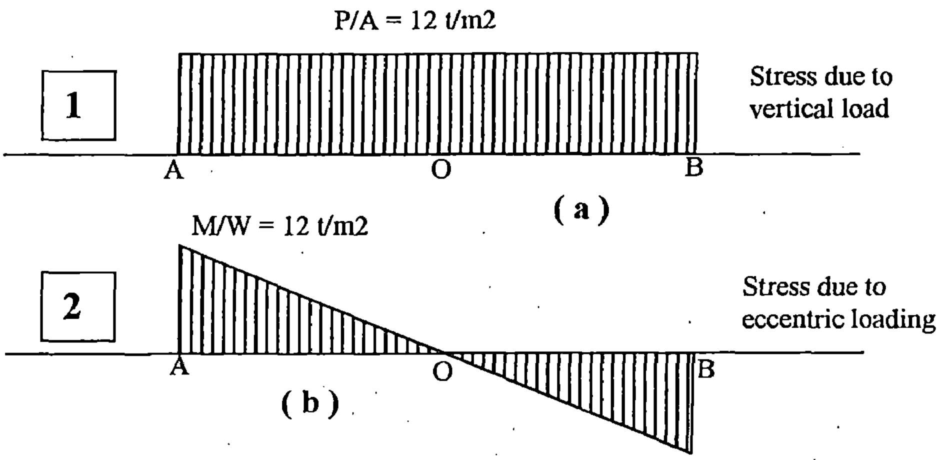
where $P = b*h*0.5*\gamma c = 10*10*0.5*2.4 = 120$ ton

$M = P*e = 120*1.67 = 200$ ton-m; $e=b/6$.

$H = \gamma w*h*h*0.5 = 1*10*10*0.5 = 50$ ton

$U = \gamma w*h*b*0.5 = 1*10*10*0.5 = 50$ ton

b.2. Stresses at Ground Surface



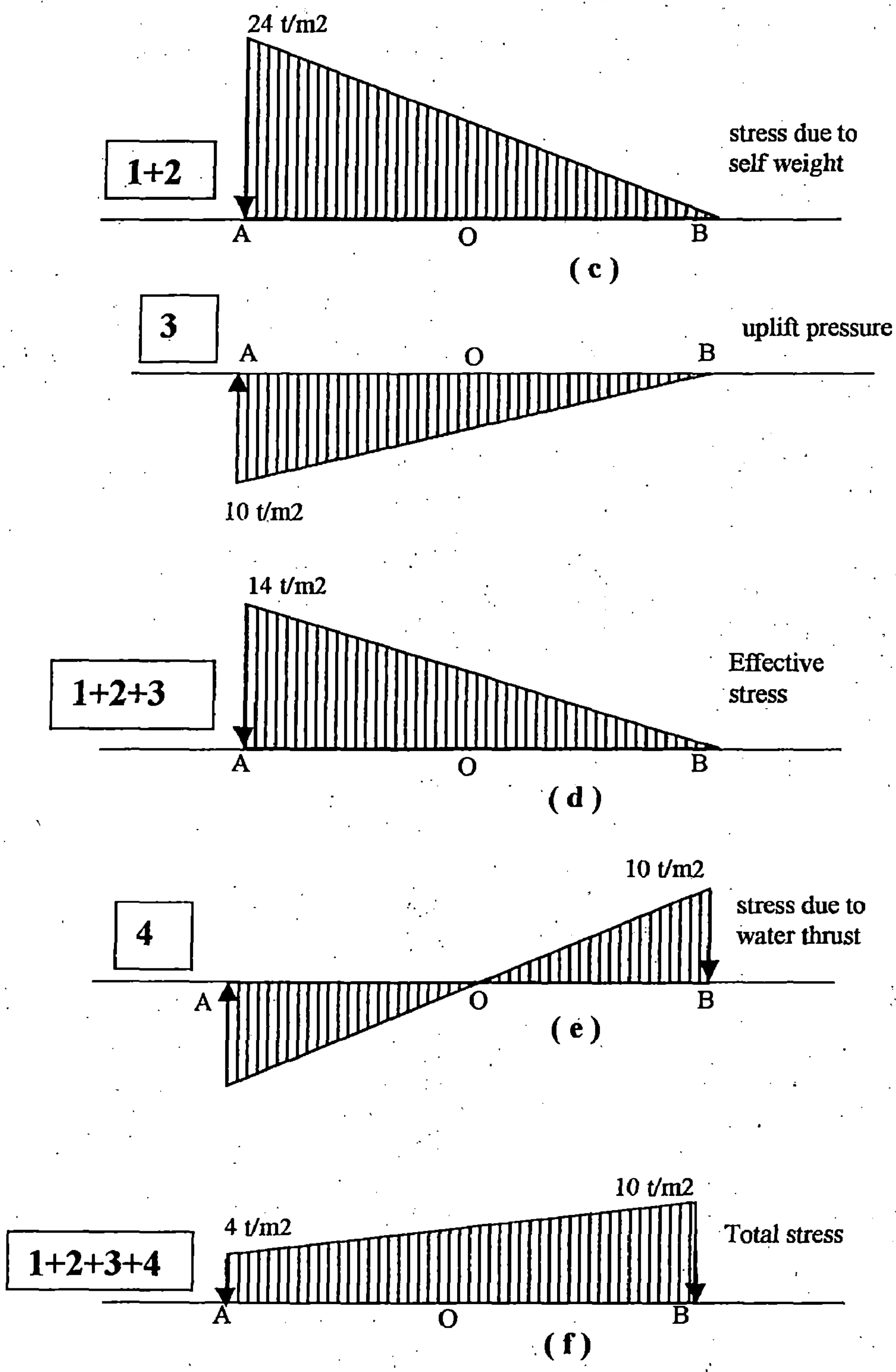


Fig. (III.1.3) Stresses at Ground Surface

b.3. Circular Sliding Analysis

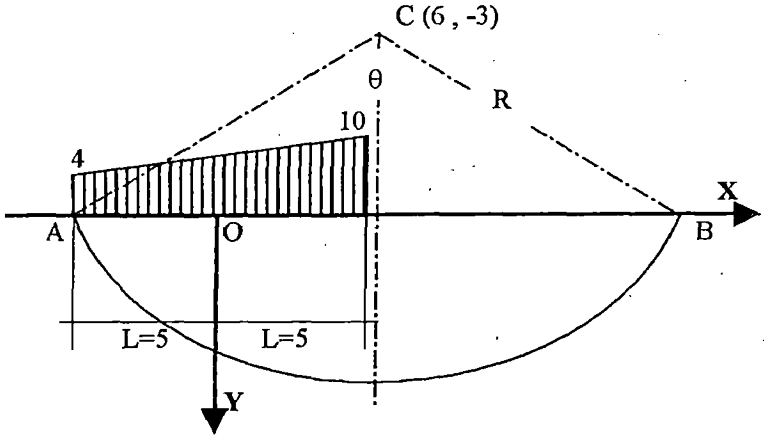


Fig. (III.1.4) A trial Slip Circle

To draw a trial slip circle, let us now select any arbitrary point at the X-Y plane as centre of the circle, taking Y positive downward and the circle must pass through corner of heel of the structure, then we can find radius and length ,central angle , arc of circle as follows

$$R = \sqrt{(L + xc)^2 + yc^2} \qquad \text{(III.I.5)}$$

$$R = \sqrt{(5 + 6)^2 + (-3)^2} = 11.402 \text{ m}$$

$$\theta = 2 * \text{acos}\left(\frac{yc}{R}\right) = 2.609 \text{ rd} \qquad \text{(III.I.6)}$$

$$\text{arc AB} = \theta * R = 29.748 \text{ m}$$

Clay Soil ($\phi = 0$)

If the soil is purely clay, where $\phi = 0$, the shear resisting moment corresponding to the trial slip circle is

$$Msr = arc.AB*Ct*R = 29.748*10*11.402 = 3391.87 \text{ tm}$$

The moment that cause sliding or called "Driving Moment" is multiplying between the total load acting on ground surface and its distance to the line through centre of slip circle, i.e

$$Mdr = (4*10*6 + 6*10*0.5*4.33) = 370.0 \text{ tm.}$$

Thus, the safety factor against circular sliding is

$$Sfs = Msr / Mdr = (3391.87)/(370) = 9.167.$$

The computer analysis has been carried out to compute the safety factor of simple structure founded on clay soil and the results are presented below.

```
DATA
   Centre of Circle (xa,ya):6 ,-3
   Vertical Load of structure (+ downward, t): 120
   Moment Load of structure (+ counter clockwise, tm): 200
   Length of base of structure (m): 10
   Unit weight of soil (γ,t/m3): 2.1
   Angle of internal friction of soil(φ,deg): 0
   Cohesion of soil (Ct,t/m2): 10
   Height of water(m): 10

OUTPUT
   Central angle (rd): 2.609088
```

```
Radius of circle (m): 11.401754
Uplift Pressure (t/m2): 50
Weight of Soil in the circle line (t): 286.996582
Shear resistance due to soil cohesion (t/m2):297.481842
Resisting moment (tm): 3391.814941
Driving moment (tm): 370.125
Safety factor of Circular sliding: 9.163972
```

The computation for various slip circle line to find out the minimum of safety factor is done by means of CISANx (apendix) and the results are presented in the fig. (III.1.5).

- **Bearing Capacity**

The bearing capacity for clay $q_f = 5.14\ Ct + \gamma D$.

Put D=0 for foundation on ground surface, thus the bearing capacity of foundation is

$$q_f = 5.14*10 = 51.4 \text{ ton/m2}$$

The safety factor

$$SF = q_f / q\ max. = 51.4 / 24 = 2.142$$

X 5	5.5	6	6.5	7	7.5	8	Y
9.533	9.340	9.223	9.163	9.145	9.161	9.204	-7
9.426	9.251	9.149	9.100	9.093	9.118	9.170	-6.5
9.338	9.179	9.090	9.053	9.056	9.090	9.150	-6
9.270	9.125	9.049	9.023	9.035	9.078	9.144	-5.5
9.222	9.092	9.027	9.010	9.031	9.081	9.155	-5
9.199	9.080	9.025	9.018	9.046	9.103	9.182	-4.5
9.201	9.093	9.047	9.046	9.081	9.144	9.228	-4
9.231	9.131	9.092	9.098	9.138	9.205	9.293	-3.5
9.293	9.198	9.164	9.174	9.218	9.288	9.379	-3

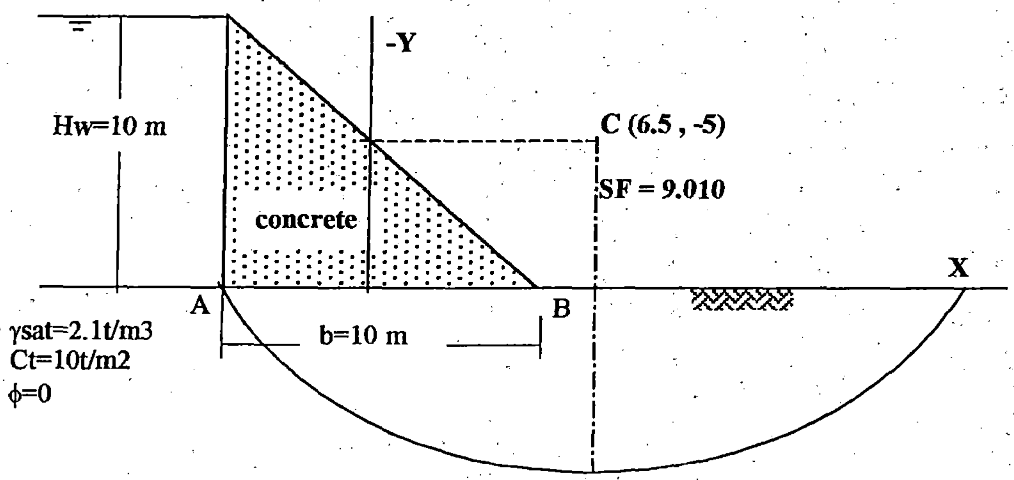


Fig. (III.1.5) The Minimum Safety Factor of Simple Hydraulic Structure Founded on Clay Soil

Cohesionless Soil (Ct = 0)

Now, let us consider if the structure is founded on cohesionless soil, where Ct = 0. In this case, the stresses due to external loads and uplift pressure influence the stresses on surface of the slip circle. The external loads increase the normal stresses in base of the slip circle, that means increasing of shear resistance. The uplift pressure along this line reduce that stresses. In another case, the uplift pressure acting below the base of structure will reduce both the driving moment and vertical loads.

Let the internal angle of soil friction (ϕ) be equal to 30 degree and γsat= 1.98 t/m3. First we determine the stresses in the slip circle induced by external loads, as shown in the fig. (III.1.6) below. We know that the two trapezoids in this figure have same area, also triangle cde and c1d1e1, using this condition we get

$$qo = \frac{ao}{b}Q \quad \text{and} \quad q1 = \frac{a1}{b+y}Q1 \qquad \text{(III.I.7)}$$

$$\frac{q1}{qo} = \frac{a1}{b+y}Q1\,\frac{b}{ao\,Q} \quad \text{or} \quad q1 = \frac{a1}{b+y}\,\frac{Q\,b}{b+y}\,\frac{b}{ao\,Q}\,qo$$

$$q1 = \frac{b^2}{(b+y)^2}\,\frac{(ao+0.5\,y)}{ao}\,qo \qquad \text{(III.I.8)}$$

$$qb = q1 + Qr\frac{b}{b+y} \qquad \text{(III.I.9)}$$

where qb is the stress in the base of slice due to external load that must be added to the stress due to soil weight in the slice for stability analysis.

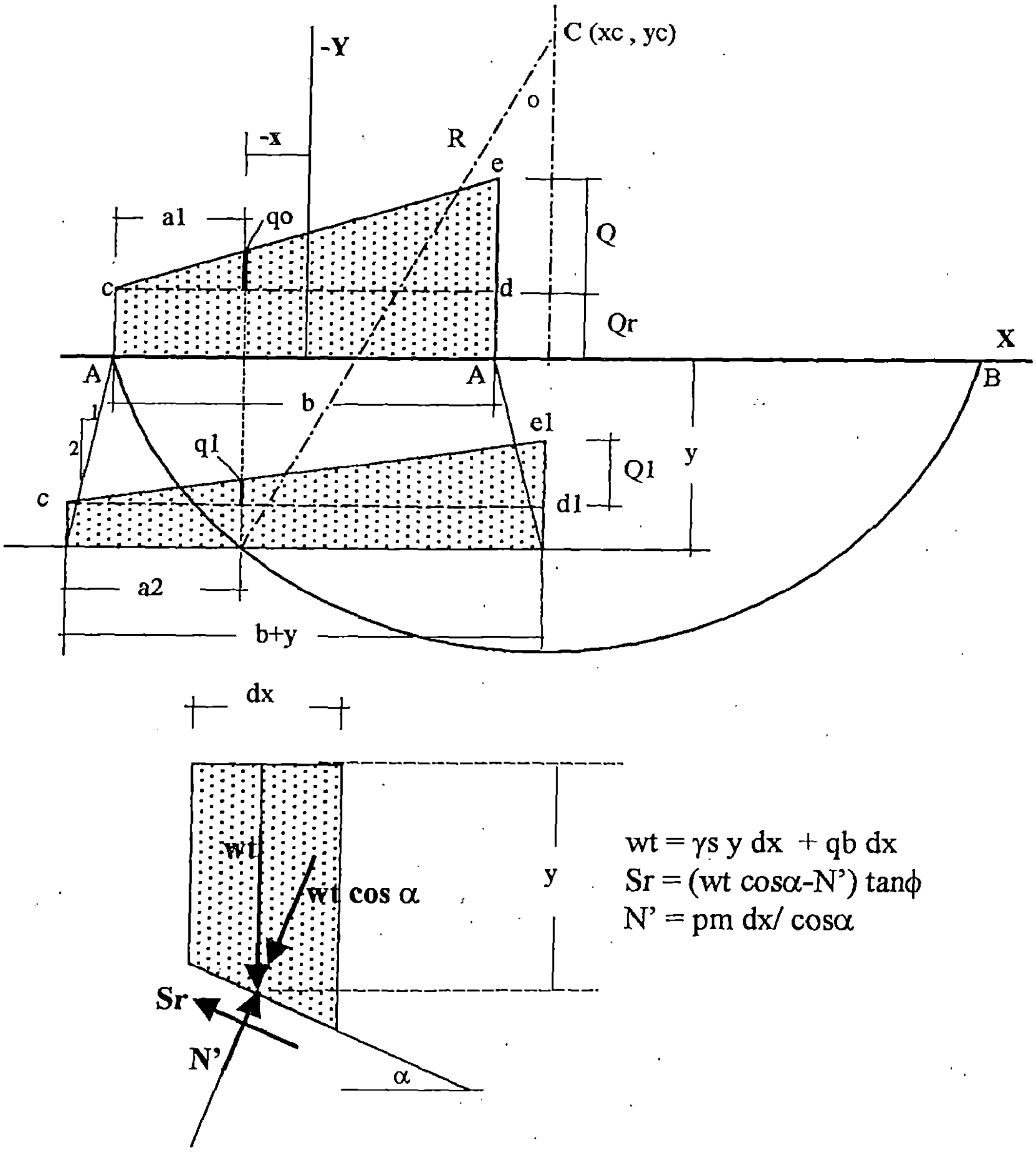


Fig. (III.1.6) Stress Distribution on Base of Slice and Free body

Further, we determine the uplift pressures at any points along the slip circle line. By putting ϕ' = a constant, say A, where value of A lies between $-\pi$ to 0. We can write eq. (II.2.9) as

$$\frac{x^2}{L^2\cos^2 A} - \frac{y^2}{L^2\sin^2 A} = 1$$

For A=A1 , A2,....we can draw the curves of equipotential lines p1, p2.... as shown in fig. (III.1.) below.

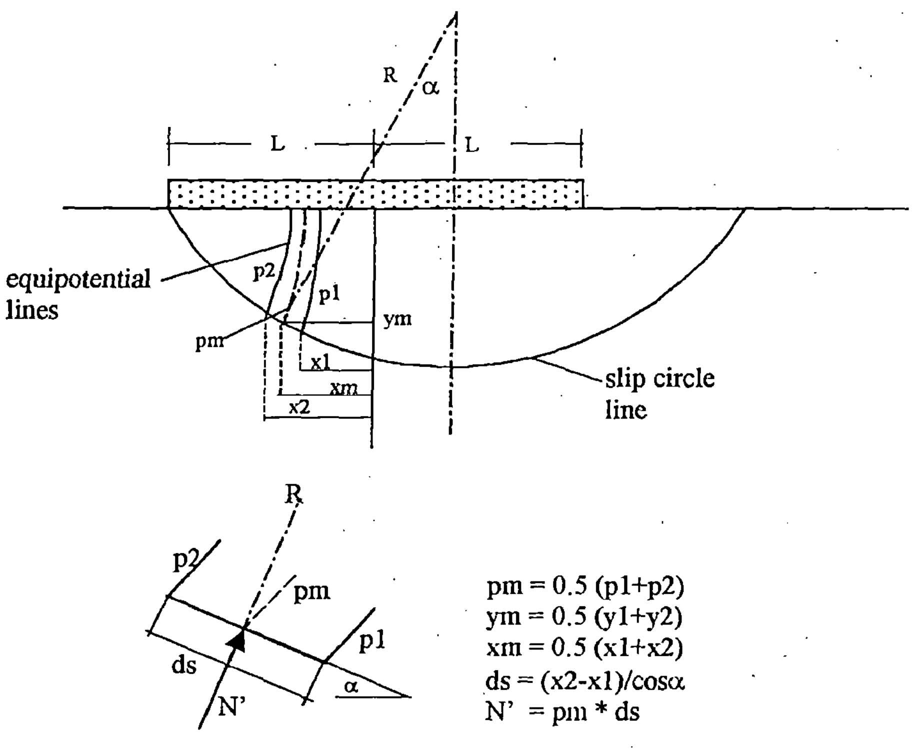


Fig. (III.1.7) Uplift Pressure at Any points along The Slip Circle

Now, to find the points at the circle line, where the uplift pressure will be determined, we must determine equation of the circle line. From the given coordinate of centre of the circle C (xc ,yc) and the base length of the structure, we can calculate the radius of the circle R , then the equation of the circle passing through the corner of heel of the structure is

$$(x - xc)^2 + (y - yc)^2 = R^2 \qquad \text{(III.I.10)}$$

Equating this equation with a series of equipotential lines equation will results a series of points lying in that circle line. The uplift pressure at that points are found from eq. (II.2.17), and putting C=kh2, we have

$$\phi = -k\left(\frac{p}{\gamma w} - y\right) + k\,h2$$

$$p = \gamma w\left(y - \frac{\phi}{k} + h2\right)$$

from eq. (II.2.7) $\phi' = \dfrac{\phi\pi}{kh}$

$$\text{so} \quad p = \gamma w\left(y - \frac{\phi' h}{\pi} + h2\right) \qquad \text{(III.I.11)}$$

If there is no tail water in the down stream side (h2=0), we get

$$p = \gamma w\left(y - \frac{\phi' h}{\pi}\right) \qquad \text{(III.I.12)}$$

where p is the uplift pressure at any point, h is difference head between upstream and downstream and y is the depth of point below the level of the base of structure. ϕ' are the constants having values ranging from $-\pi$ to 0. If we have the values of y of points in the circumference of the slip circle line and the equipotential lines, we can find the uplift pressure at those points by using eq. (III.1.12).

The coordinate of points of intersection between the circle line and the equipotential lines, forces at that points and safety factor have been computed by means of computer program (CISAN) and the results are presented below.

```
DATA
   Centre of Circle (xa,ya):6 ,-3
   Vertical Load of structure (+ downward, t): 120
   Moment Load of structure (+ counter clockwise, tm): 200
   Length of base of structure (m): 10
   Unit weight of soil (γ,t/m3): 2.1
   Angle of internal friction of soil (φ,deg): 30
   Cohesion of soil (Ct,t/m2): 0
   Height of water(m): 10

CIRCLE OF SLIDING
   Central angle (rd): 2.609088
   Radius of circle (m): 11.401754

UPLIFT PRESSURE IN THE SLIDING LINE (-X REGION)
Considered Points of Pressure
```

x	y	φ'	p
-4.974594	0.0973	-2.983	9.597301
-4.891706	0.377559	-2.826	9.377559
-4.739186	0.83065	-2.669	9.33065
-4.501329	1.444097	-2.512	9.444097
-4.152554	2.190077	-2.355	9.690076
-3.670679	3.042387	-2.198	10.042386
-3.03063	3.961257	-2.041	10.461256
-2.213707	4.908643	-1.884	10.908642
-1.204251	5.837555	-1.727	11.337555

dx	α	ds.	pm	Fr=ds*pm	Tot.Fr
0.082888	-1.283102	0.292125	9.48743	2.771514	2.771514

0.152519	-1.248711	0.481824	9.354105	4.507033	7.278547
0.237857	-1.1984	0.653725	9.387373	6.136764	13.41531
0.348776	-1.133105	0.822877	9.567086	7.872533	21.287844
0.481874	-1.053797	0.974914	9.866231	9.618726	30.90657
0.640049	-0.961597	1.118555	10.251822	11.467231	42.373802
0.816923	-0.857508	1.248499	10.684949	13.340144	55.713947
1.009456	-0.742487	1.370073	11.123098	15.23946	70.953407

UPLIFT PRESSURE IN THE SLIDING LINE(+X REGION)
Considered Points of Pressure

x	y	φ'	p
0.006657	6.699688	-1.57	11.699689
1.421163	7.442314	-1.413	11.942314
3.032792	8.008947	-1.256	12.008948
4.824528	8.341048	-1.099	11.841049
6.764406	8.376061	-0.942	11.376062
8.800771	8.052348	-0.785	10.552349
10.857083	7.31503	-0.628	9.315031
12.831296	6.127437	-0.471	7.627438
14.599532	4.482539	-0.314	5.48254
16.033106	2.414687	-0.157	2.914688

dx	α	ds	pm	Fr=ds*pm	Tot.Fr
1.414507	-0.482077	1.596448	11.821001	18.871611	18.871611
1.611628	-0.337274	1.707848	11.975632	20.45256	39.324173
1.791736	-0.182683	1.822055	11.924999	21.72801	61.052185
1.939878	-0.018027	1.940194	11.608556	22.522848	83.575035
2.036365	0.156987	2.061718	10.964206	22.605101	106.180138
2.056313	0.342475	2.183092	9.93369	21.686163	127.866302
1.974213	0.538174	2.299217	8.471234	19.477201	147.343506
1.768236	0.743253	2.40161	6.554989	15.742526	163.086029
1.433574	0.956354	2.486673	4.198614	10.440578	173.526611

SOIL IN THE SLIDING LINE

x	y	α	wo=y*i*Gs	T=wo*cos α
-4.75	0.799671	-1.231044	0.839655	0.279818
-4.25	1.993746	-1.117445	2.093433	0.916883
-3.75	2.910795	-1.025805	3.056335	1.584438
-3.25	3.666146	-0.946336	3.849453	2.25062
-2.75	4.310096	-0.874816	4.5256	2.901534
-2.25	4.870038	-0.808965	5.11354	3.529611
-1.75	5.362864	-0.747381	5.631008	4.130185
-1.25	5.799858	-0.689131	6.089851	4.700139
-0.75	6.188988	-0.633556	6.498437	5.23727
-0.25	6.536116	-0.580164	6.862921	5.73996
0.25	6.845685	-0.528581	7.187968	6.20698

0.75	7.121141	-0.478508	7.477198	6.63738
1.25	7.365206	-0.429705	7.733466	7.030407
1.75	7.580052	-0.38197	7.959054	7.38546
2.25	7.767428	-0.335135	8.155799	7.702058
2.75	7.928747	-0.289052	8.325184	7.979809
3.25	8.065148	-0.243593	8.468406	8.218399
3.75	8.177545	-0.198642	8.586421	8.417573
4.25	8.266654	-0.154094	8.679986	8.577136
4.75	8.333027	-0.109853	8.749678	8.696937
5.25	8.37706	-0.065827	8.795913	8.776862
5.75	8.399014	-0.021928	8.818964	8.816844
6.25	8.399014	0.021928	8.818964	8.816844
6.75	8.37706	0.065827	8.795913	8.776862
7.25	8.333027	0.109853	8.749678	8.696937
7.75	8.266654	0.154094	8.679986	8.577136
8.25	8.177545	0.198642	8.586421	8.417573
8.75	8.065148	0.243593	8.468406	8.218399
9.25	7.928747	0.289052	8.325184	7.979809
9.75	7.767428	0.335135	8.155799	7.702058
10.25	7.580052	0.38197	7.959054	7.38546
10.75	7.365206	0.429705	7.733466	7.030407
11.25	7.121141	0.478508	7.477198	6.63738
11.75	6.845685	0.528581	7.187968	6.20698
12.25	6.536116	0.580164	6.862921	5.73996
12.75	6.188988	0.633556	6.498437	5.23727
13.25	5.799858	0.689131	6.089851	4.700139
13.75	5.362864	0.747381	5.631008	4.130185
14.25	4.870038	0.808965	5.11354	3.529611
14.75	4.310096	0.874816	4.5256	2.901534
15.25	3.666146	0.946336	3.849453	2.25062
15.75	2.910795	1.025805	3.056335	1.584438
16.25	1.993746	1.117445	2.093433	0.916883
16.75	0.799671	1.231044	0.839655	0.279818

DRIVING LOADS

x	y	qb	ap1	T=wn1*cos ap1
-4.75	0.799671	4.038114	-1.231044	0.672857
-4.25	1.993746	4.063694	-1.117445	0.88991
-3.75	2.910795	4.071998	-1.025805	1.055484
-3.25	3.666146	4.078046	-0.946336	1.192134
-2.75	4.310096	4.085904	-0.874816	1.309814
-2.25	4.870038	4.096919	-0.808965	1.413945
-1.75	5.362864	4.11156	-0.747381	1.507856
-1.25	5.799858	4.129982	-0.689131	1.593757
-0.75	6.188988	4.152226	-0.633556	1.673197

-0.25	6.536116	4.178303	-0.580164	1.747309
0.25	6.845685	4.208225	-0.528581	1.816951
0.75	7.121141	4.242026	-0.478508	1.882787
1.25	7.365206	4.279764	-0.429705	1.945342
1.75	7.580052	4.321527	-0.38197	2.005042
2.25	7.767428	4.36744	-0.335135	2.062231
2.75	7.928747	4.41766	-0.289052	2.117195
3.25	8.065148	4.472383	-0.243593	2.170174
3.75	8.177545	4.53185	-0.198642	2.221367
4.25	8.266654	4.596346	-0.154094	2.270942
4.75	8.333027	4.666208	-0.109853	2.319041

```
STABILITY

Uplift Force (t): 50
Weight of Soil(t): 286.996582
Normal Forces due to Weight of Soil(t): 251.432617
Normal Forces due to Weight of Soil(t): 33.867336
Total water force at the circle line(t): 244.480011
Total Normal force at the circle line(t): 285.299957
Effective force (t): 40.819946
Shear resistance due to soil cohesion (t/m2): 0
Total Shear Resistance(t): 23.567381
Resisting moment (tm): 268.709503
Driving moment (tm): 370.125
Safety factor of Circular sliding: 0.725997
```

The Centre of Critical Circle

The centre of the most critical circle giving the minimum safety factor can only be found by trial and error from analyses of various slip circle. By means of computer program (CISANx) we able to carry out analyses of various slip circle for homogenous soil in dry or submerged condition, without or with steady seepage flow. Various shape of hydraulic structure and loading condition can be analyzed by this program by converting the loading mechanism as shown in fig. (III.1.2). For the present study we consider a sample of simple hydraulic structure (elementary profile of Gravity Dam).

X	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7	Y
	1.258	1.001	0.876	0.812	0.785	0.782	0.795	0.821	0.856	0.901	0.953	-5
	1.158	0.927	0.816	0.762	0.740	0.742	0.758	0.788	0.826	0.873	0.926	-4.5
	1.057	0.851	0.754	0.709	0.694	0.701	0.721	0.752	0.794	0.842	0.898	-4
	0.949	0.771	0.688	0.653	0.647	0.657	0.682	0.717	0.760	0.812	0.870	-3.5
	0.839	0.687	0.620	0.596	0.596	0.612	0.640	0.679	0.726	0.781	0.842	-3
	0.721	0.601	0.549	0.535	0.544	0.566	0.601	0.641	0.691	0.748	0.812	-2.5
	0.597	0.507	0.474	0.475	0.489	0.519	0.553	0.601	0.655	0.716	0.779	-2
	0.465	0.409	0.395	0.403	0.429	0.466	0.512	0.560	0.618	0.684	0.749	-1.5
	0.326	0.305	0.312	0.341	0.367	0.412	0.455	0.516	0.571	0.648	0.714	-1

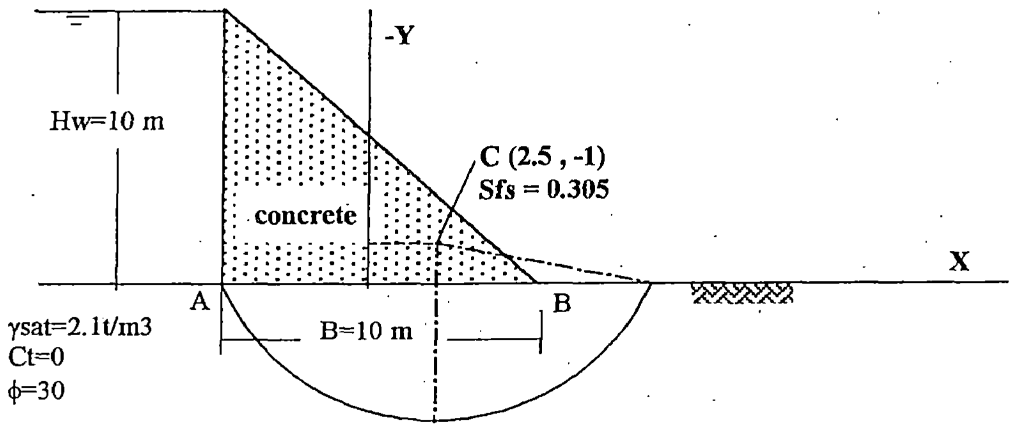


Fig. (III.1.8) The Minimum Safety Factor of Simple Hydraulic Structure Founded on Cohesionless Soil

X \ Y	4	4.5	5	5.5	6	6.5	7	
	10.584	10.226	10.017	9.912	9.883	9.912	9.985	-5
	10.483	10.148	9.957	9.868	9.852	9.890	9.973	-4.5
	10.411	10.097	9.922	9.845	9.840	9.888	9.980	-4
	10.371	10.073	9.913	9.848	9.852	9.909	10.008	-3.5
	10.366	10.082	9.933	9.877	9.890	9.955	10.060	-3
	10.400	10.127	9.988	9.937	9.955	10.025	10.135	-2.5
	10.478	10.212	10.071	10.027	10.050	10.123	10.233	-2

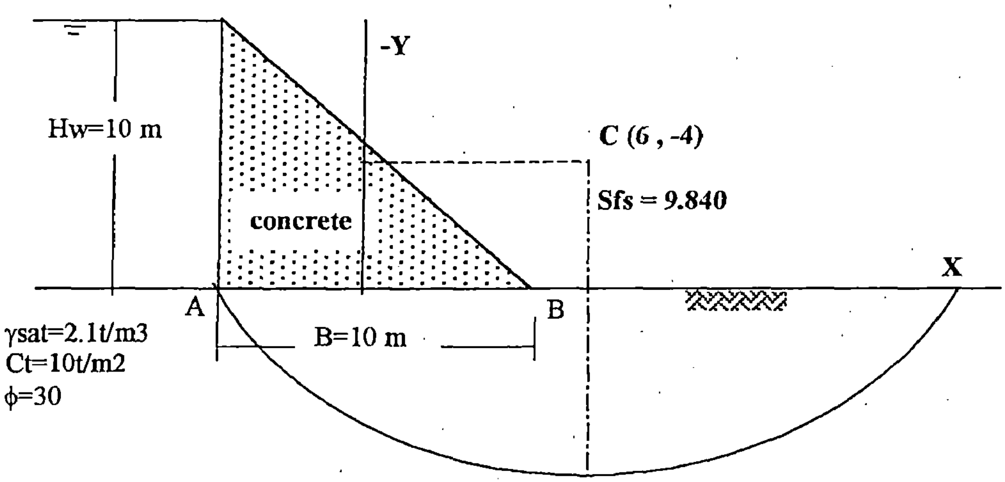


Fig. (III.1.9) The Minimum Safety Factor of Simple Hydraulic Structure Founded on Clayey (C-ϕ) Soil

From the analyses of various slip circle of two kinds of soil, as shown in fig. (III.1.5) the hydraulic structure founded on clay soil (ϕ=0) and fig. (III.1.8) the hydraulic structure founded on cohesionless soil (Ct=0), we can note that the critical of slip circle for cohesionless soil is lying close to the base of structure (shallow seated slides) and the safety factor is very small (unstable), but for cohesive soil or clay soil, the critical slip circle is far from the base (deep seated slides) and the safety factor resulted is high (safe condition). For the clayey soil (C-ϕ soil) the centre of critical slip circle and magnitude of safety factor is slightly less than the clay soil, thus it can be noted that stability of hydraulic structure founded on clayey soil (with seepage condition) is predominantly performed by cohesion of soil. The friction of soil has no significant contribution, so it may be ignored in the analyses.

b.4. Influence of Water Storage Level

The height of water storage influent uplift pressure either at the base level of the structure or at the slip circle level. In the base of the structure, the uplift pressure will counter both vertical load and driving moment due to structural loads, thus, it will be advantage for circular sliding stability. Meanwhile the uplift pressure in the slip circle level will reduce normal pressure and it will be disadvantage for shear resistance of soil along the slip circle line. The computation of sliding safety factor of hydraulic structure founded on clay soil and cohesionless soil for various water storage level have been carried out and the results are presented in table III.1.1 below

Table III.1.1 Influence of water storage level on Safety Factor

Cohesionless Soil		Clay Soil	
Hw	Sfs	Hw	Sfs
10	0.725997	10	9.163972
9.5	0.747332	9.5	8.212774
9	0.768609	9	7.479339
8.5	0.789951	8.5	6.897502
8	0.811427	8	6.425352
7.5	0.83307	7.5	6.035048
7	0.854893	7	5.707352
6.5	0.876889	6.5	5.428534
6	0.899041	6	5.188523
5.5	0.921319	5.5	4.979751
5	0.943686	5	4.796414
4.5	0.966094	4.5	4.63398
4	0.988489	4	4.488843
3.5	1.010808	3.5	4.358101
3	1.03298	3	4.239378
2.5	1.05493	2.5	4.130711
2	1.076574	2	4.030462
1.5	1.097823	1.5	3.93724
1	1.118583	1	3.849867
0.5	1.138754	0.5	3.767332
0	1.158234	0	3.688759

Table III.1.1 shows that as the water level decrease, the safety factor of structure founded on cohesionless soil increase, it reveals that decreasing of uplift pressure below the base of foundation, means increasing of driving moment, is less than increasing of shear resistance of soil in the slip circle line (in this case increasing of normal stress). Decreasing of safety factor of the structure founded on clay soil is caused by decreasing of uplift pressure below the base of the structure (increasing driving moment), meanwhile the shear resistance of soil in the slip circle line constant.

III. 2 . A BARRAGE WITH FLOATING FLOOR AND RIBBED FOUNDATION

a. Stability Analysis

A barrage is a diversion structure in which the crest is kept at a low level and the pounding up of the river is accomplished primarily by means of gates. In India also Indonesia, in many water resources projects, barrage are widely used as diversion structures. Design of barrage consist of two main cases, the hydraulic design and the structural design. In this present study, as mentioned before, we are concerning in structural design of barrage, specifically in stability of barrage founded on clayey soil against subsurface flow (uplift pressure). In the past, most of the barrages are designed with gravity floor concept. Stability of the barrage is governed by the weight of floor. Structural design of barrage floor, therefore, plays important role in achieving economy and technical aspects.

To resist the indicated uplift pressure, the floor is usually designed as pure gravity section, called as "Gravity Floor" as shown in Fig.(III.2.2). For this purpose, a considerable thick of concrete floor require to counter that pressure, sometimes the thickness is more than 3.5 m. This solution, therefore, will cause new problems to arise, like low bearing capacity of soil base, construction problems (large excavation, large concreting and dewatering), consequently, these will consume long time and much money.

A barrage with floating floor and ribbed foundation system as shown in Fig.(III.2.3), consist of:

- Pier. It acts as supporting that transfer loads from bridge and appurtenant structures to the girder (upper rib of the barrage).

- Girder. It will continue the pier loads to foundation (lower ribs).
- Floating floor. This is quite thin of reinforced concrete slab, and designed only to resist uplift pressure. It acts like a slab of water tank loaded by weight of water in side.
- Strip (lower rib) foundation. This is a cutoff made from reinforced concrete and has double function, not only to increase seepage line , but also acts as foundation continuing the structural loads to soil base. Strip foundation and floating floor form a inverted box.

a.1. Weight of Structure Required

In this barrage the uplift pressure is resisted by using the rigidity of the structure to transfer the loads (the weight of the water existing in the upstream side, the weight of structure and external loads) as counter balance. The water existing in upstream side induces both vertical and horizontal thrust. The uplift pressure along with horizontal water thrust cause instability to the structure. To keep the barrage stable, overturning moments, horizontal and vertical movement due to that forces will be resisted by the weight of the structure, inclusive external loads, and the weight of water in upstream side. By taking equilibrium of these forces, we able to determine the weight of the structure required . The mechanism of forces acting on this barrage schematically shown in Fig.(III.2.1). By taking equilibrium of vertical forces, we get:

$$W + P - U = 0$$

$$bu\ Hw\ \gamma w + P - 0.5\ Hw\ (bu+bd) = 0$$

or

$$P = 0.5\ Hw\ (bu+bd) - bu\ Hw\ \gamma w \qquad (III.2.1)$$

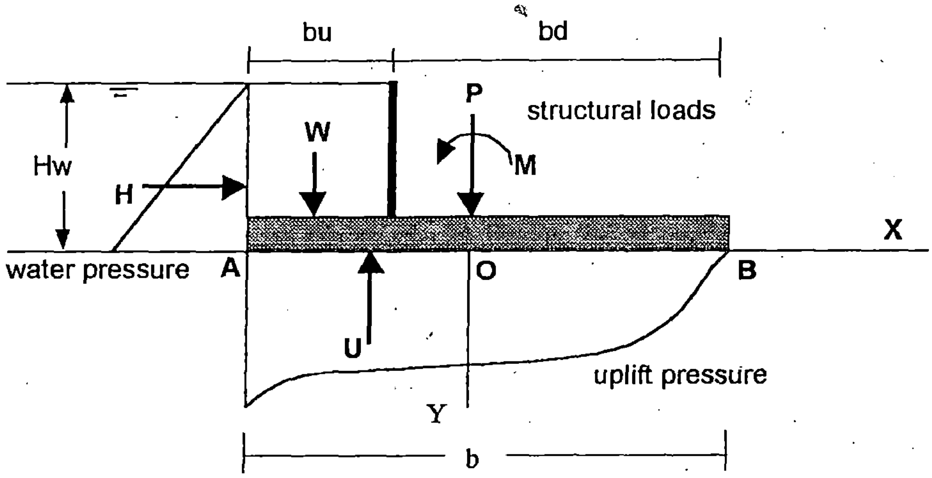


Fig. (III.2.1) Mechanism of Forces Acting on The Barrage

From eq. (III.2.1), we can note that if the values of right hand side , in this case the uplift pressure and the weight of water at upstream side, are same, theoretically, the weight of the structure (P) equal to zero. It means there is no counterweight required (by the structure) balancing the uplift pressure in vertical direction

Now, let us look at the equilibrium of moments about both ends, upstream and downstream. First we take sum moments about point B at downstream, taking positive moments in counterclockwise direction and $\gamma w = 1$ t/m2:

$$M + W(bd + 0.5\,bu) + P\ 0.5b - 0.5b\,Hw\,(5b/8) - (Hw^3)/6 = 0$$

$$P = \frac{0.5b\,Hw\ (5b/8) + (Hw^3)/6 - M - W\,(bd + 0.5bu)}{0.5b} \qquad (III.2.2)$$

Now taking sum moments about point A at upstream side:

$$M - W(0.5bu) - P\ 0.5b + 0.5b\ Hw(3b/8) - (Hw^3)/6 = 0$$

$$P = \frac{0.5b\ Hw(5b/8) - (Hw^3)/6 + M - W(0.5bu)}{0.5b} \qquad \text{(III.2.3)}$$

From the three equations above we can find out the value of P. Some values of P has been calculated by means of computer program with C++ language (Pmin.cpp) and the results is presented in table III.2.1. It shows that, from eq.(III.2.1), if length of the upstream floor equal to 5m (half of the total length), the value of P equal to zero. From eq.(III.2.2) and eq.(III.2.3), at the value of bu higher than 5m, values of P to be negative, it means the structure is not required to counter balance the uplift pressure. On the other hand, the structure can be made as light as possible.

Table III.2.1.

```
Pmin.cpp:Minimum of Structure Weight
Length of floor: 10 m
The External moment: 0
Height of water at U/S floor: 5 m
```

bu	P from eq.(3)	P from wq.(2)	P from eq.(1)
0	14.583333	35.416668	25
1	14.083333	25.916666	20
2	12.583333	17.416666	15
3	10.083333	9.916667	10
4	6.583333	3.416667	5
5	2.083333	-2.083333	0
6	-3.416667	-6.583333	-5
7	-9.916667	-10.083333	-10
8	-17.416666	-12.583333	-15
9	-25.916666	-14.083333	-20
10	-35.416668	-14.583333	-25

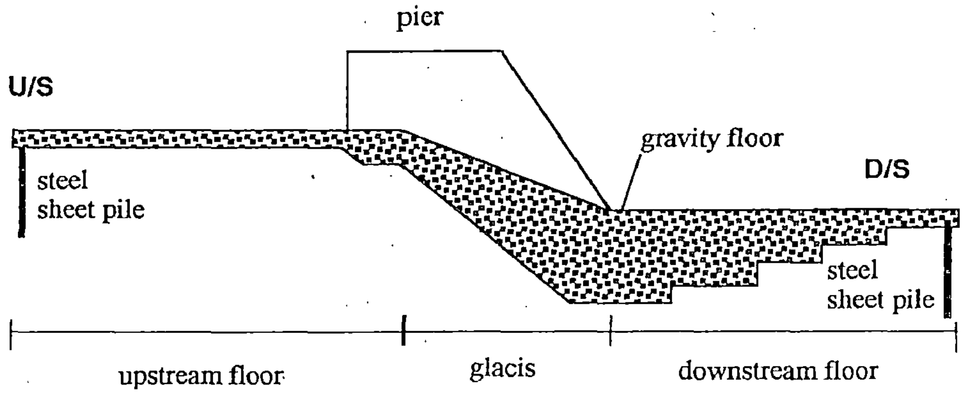


Fig. (III.2.2) Barrage with Gravity Floor

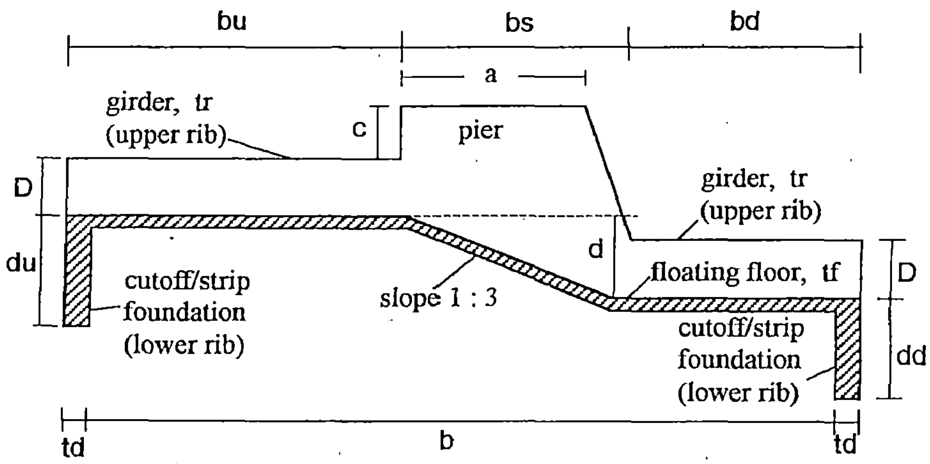


Fig. (III.2.3) Barrage with Floating Floor and Ribbed Foundation

a.2. Horizontal Stability

The horizontal stability of the barrage is governed by the horizontal resistance of the foundation Hq. In this case, horizontal resistance due to friction between floor and soil base equal to zero, since the floor is floating. Now let us look at the figure below.

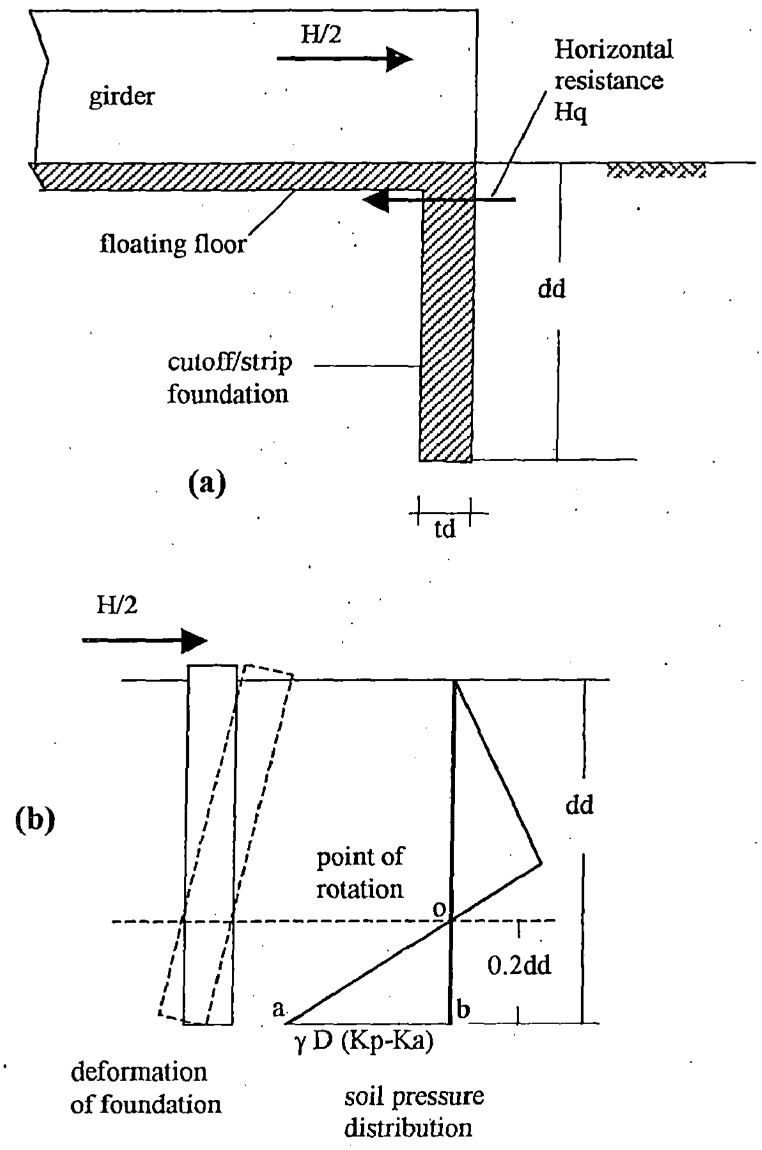


Fig.(III.2.4) Detail of Foundation and Horizontal Resistance

Since the floating floor quite thin, the rigidity of floor in direction perpendicular to its plane is small, but in plane direction the rigidity is very high. To carry out analysis for determining horizontal resistance of foundation, we may assume that due to horizontal force, the foundation will deflect or rotate against a point having distance equal to 0.2D from base of foundation, as shown in fig. (III.2.4). In right side of foundation above point of rotation will act passive soil pressure, meanwhile in left side the passive soil pressure will act at below of point of rotation. These soil pressures cause deformation of the foundation is resisted. Thus, capacity of horizontal resistance of the foundation is equal to the magnitude of resultant of that soil pressure or the strength of foundation its self.

To calculate the magnitude of the horizontal resistance of foundation, we able to use the problem described in the chapter II.3, and for determine the structural strength of foundation we can find out as follows. Let us look at triangle abo and taking moment about o , we get moment ultimate at body of strip foundation equal to

$$Mo = \gamma\, dd\, (Kp - Ka)(0.5dd\ \ 0.2dd)(0.2dd\ \ 2/3)$$

$$Mo = 0.0133\, \gamma\, dd^3 .(Kp - Ka)$$

a.3. Vertical Stability

A cutoff functioned as foundation may be assumed as deep strip foundation. A case of a deep strip foundation the potential surface of sliding for fairly isotropic soil mass may be assumed to be a logarithmic spiral starting under the foundation and ending with a vertical tangent, as described in chapter II.3.

The overburden pressure $\sigma o'$ is assumed to act at the base level of the footing. With the conservative assumption that the mass of soil within the logarithmic spiral is weightless, Zeevaert (1983) analyzed the limiting plastic equilibrium condition and obtain the values of the bearing capacity factor Nc and Nq which are given in Table (II.3.1). The ultimate bearing capacity for a deep footing may be written as:

$$qf >= s\ [Ct\ Nc + \sigma o']\ (Id + 0.1)$$

where s= shape factor

= 0.1 for strip and 1.2 for square and circular footing, and (Id+0.1)=1 for dense state.

The skin friction, according with the total bearing capacity of strip deep foundation , must be added to the equation (II.2.21). The skin friction is the ultimate adhesion Ca between the clay and the pile shaft (concrete, steel etc.), which is generally related to the undrained shear strength Ct as stated in chapter II.3

$$fs = Ca = \alpha\ (Ct)_{av}$$

Where α = adhesion factor

Ct_{av} = average undrained cohesion over pile shaft.

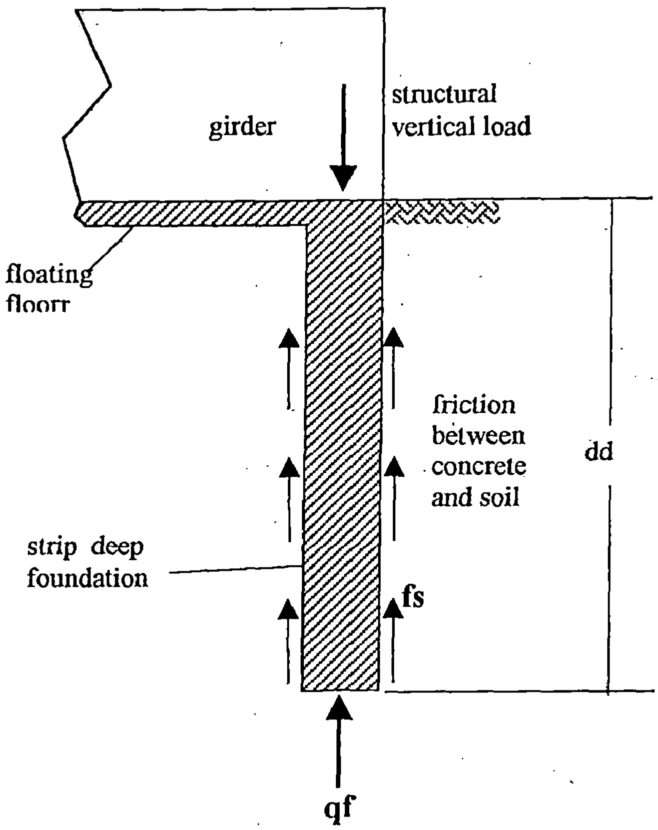


Fig. (III.2.8) Bearing and Skin Friction Resistance

The adhesion factor depend on the type of the clay, the method of installation and the pile material. The appropriate value of α is obtained from the results of load tests. For driven piles, the value of α range on the average roughly from unity for soft clay to 0.5 or less for stiff clay, while for bored piles in stiff clay α is roughly from 0.3 to 0.5. In the present study, the following values of α from chapter II.3 will be adopted for preliminary design purpose:

$$\alpha = 0.4 \text{ for } Ct = 100 \text{ kPa}$$

b. Structural Analysis for Preliminary Design

b.1. Determination of Uplift Pressure

In order to determine uplift pressure acting on the floor, the percentage pressure at upstream and downstream sheet pile lines are worked out. The pressure distribution from upstream sheet pile line to downstream sheet pile line is assumed to be linear.

(i) Percentage (%) Pressure at Upstream Sheet Pile.

For known value of b (distance between u/s and d/s sheet pile) and du (depth of u/s sheet pile), $1/\alpha = du/b$. Having known α, read out values of ϕD and ϕE from Khosla's chart

- % pressure at the bottom of sheet pile $\phi D1 = 100 - \phi D$
- % pressure at the bottom of floor $\phi C1 = 100 - \phi E$

(ii) Percentage (%) Pressure at Downstream Sheet Pile

For known value of b and dd (depth of d/s sheet pile), $1/\alpha = dd/b$, read out values of ϕE and ϕD corresponding to $1/\alpha$ from Khosla chart which would be % pressure at the bottom of floor and the bottom of sheet pile respectively. For this purpose, this chart was included in the program, so we not require read out from chart.

Correction due to floor thickness

The thickness of the floor at the location of the sheet piles are tentatively assumed for correcting the values of ϕ C1 in the upstream and ϕE in the downstream. If tf is the floor thickness ,the upstream correction due to floor thickness = $tf/du\ (\phi D1 - \phi C1)$ positive and for downstream = $tf/dd\ (\phi E - \phi D)$ negative.

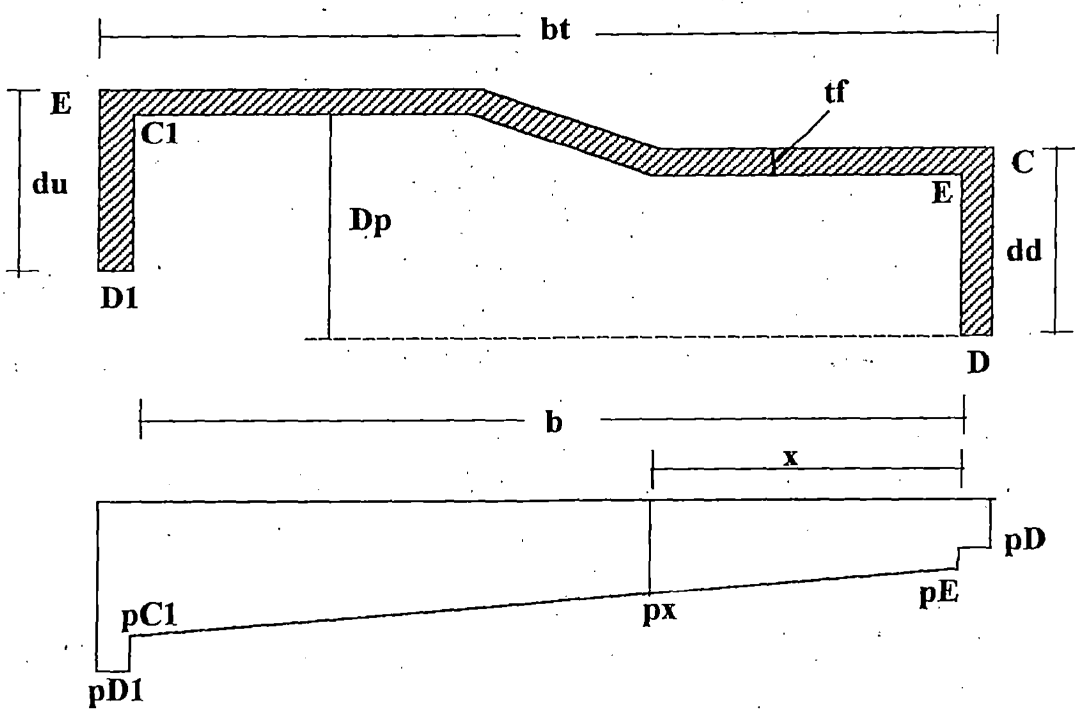


Fig.(III.2.9) Uplift Pressure at Key Points

Correction for Mutual Interference of Pile

Let C be the correction to be applied as percentage of head, b = the distance between the two pile lines, and Dp is the depth of the pile line the influence of which has to be determined on the neighboring pile of depth d. Dp is to be measured below the level at which interference is desired, d the depth of pile on which the effect is to be determined and bt = total floor length. Then

$$C = 19\left(\frac{d+Dp}{bt}\right)\sqrt{\frac{Dp}{b}} \qquad \text{(III.2.4)}$$

This correction are positive for points in the rear of back water and subtractive for points forward in the direction of flow.

Correction for the Slope of Floor

A suitable percentage correction is to be applied for a sloping floor, the correction being plus for down and minus for the up slopes following the direction of flow. The values of the correction are given in Table (III.3.2) below

Table. (III.2.2) [ref.3]

Slope Vertical : Horizontal	Correction % of pressure
1 in 1	11.2
1 in 2	6.5
1 in 3	4.5
1 in 4	3.3
1 in 5	2.8
1 in 6	2.5
1 in 7	2.3
1 in 8	1.0

b.2. Internal Forces Calculation

- **Strip Foundation**

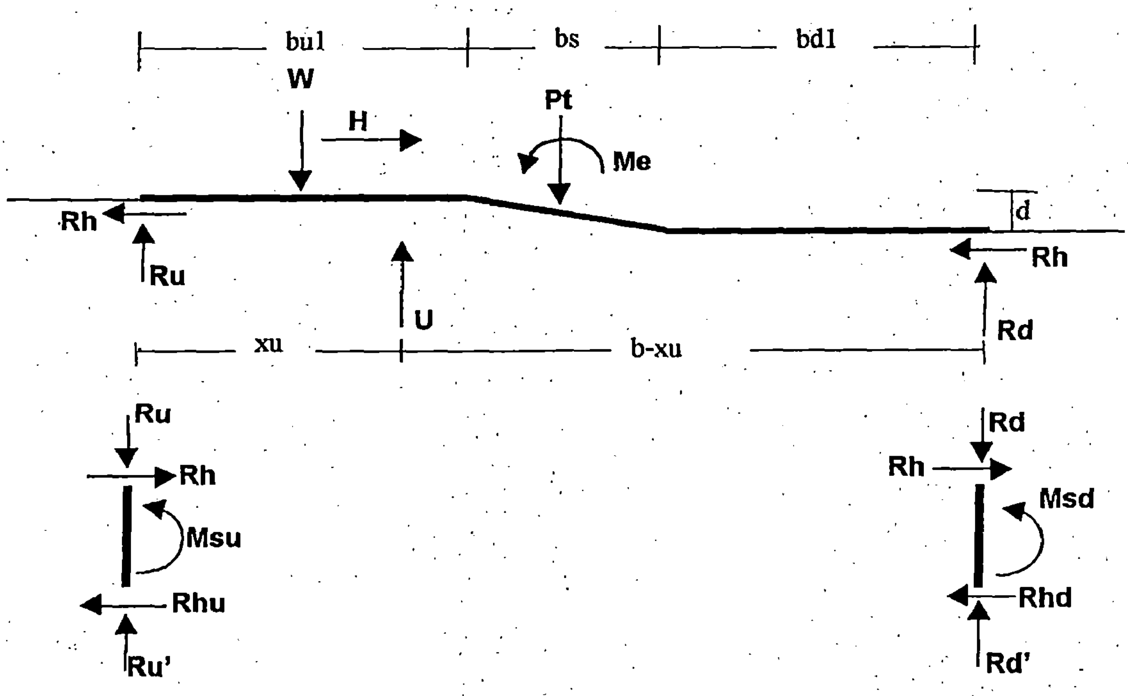


Fig. (III.2.10) Equilibrium of Forces Acting on The Barrage with Floating Floor and Ribbed Foundation

Now, let us determine the internal forces and reaction of foundation of the barrage due to external loads and uplift pressure. We assume that the floating floor and the girder are supported on two hinges and horizontal reaction of these support are equal. Thus, we can find out the problem is statically determinate. Taking equilibrium of forces in horizontal direction we get horizontal reaction of foundation at upstream and downstream (Rh) equal to 0.5 H, and equilibrium moments about ends of downstream and upstream , we get vertical reaction as follow:

$$Ru = (W(0.5bu1+bs+bd1) + Pt(0.5b+0.5td) + M + Rh\ d - U(b-ux)-H(Hw/3 + d)) / (b+td) \quad \text{(III.2.5)}$$

$$Rd = (W(0.5bu1) + Pt(0.5b+0.5td) - M + Rh\ d - U(ux)-H(Hw/3 + d)) / (b+td) \quad \text{(III.2.6)}$$

Ru = vertical reaction of upstream foundation

Rd = vertical reaction of downstream foundation

(cut off thickness at upstream and down stream are taken equal to td)

- **Girder**

To determine uplift pressure at point F, we may assume that the uplift distribution from upstream sheet pile line to downstream sheet pile line is linear. We consider point F having distance x equal to bd, as shown in fig. (III.2.11) as basic calculation for dimensioning of girder. Taking moments about point F to find out critical forces in the girder, we get

$$Mf = Rd\ x + (PF - PE)\frac{x}{2}\frac{x}{3} + PE\frac{x^2}{2} - 0.5w\ x^2$$

If we put x=bd, then

$$Mf = Rd\ x + (PF - PE)\frac{bd^2}{6} + PE\frac{bd^2}{2} - 0.5w\ bd^2 \quad \text{(III.2.7)}$$

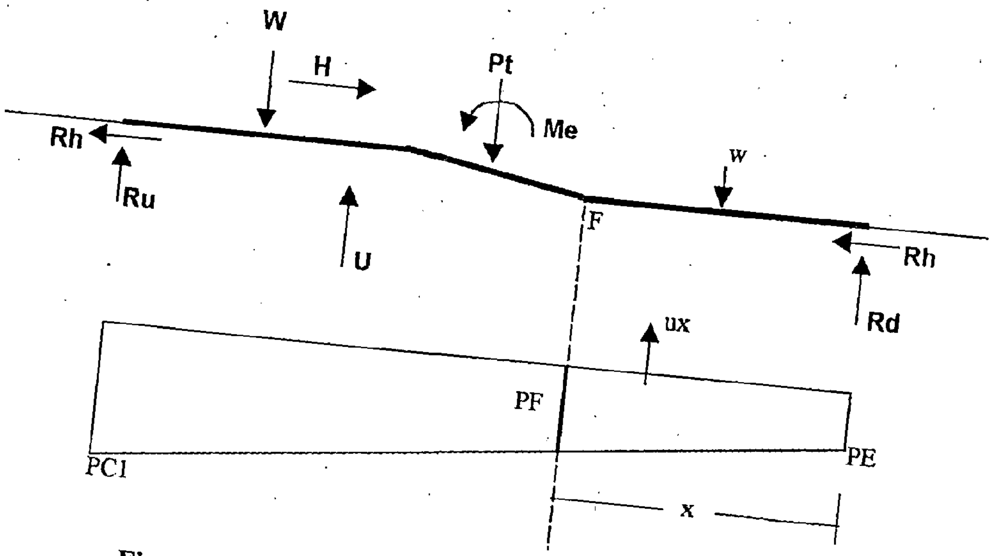


Fig. (III.2.11) Forces Acting on Girder at point F

- **Floating Floor**

There are many loads acting on the floating floor. Not only uplift pressure but also forces induced by surface flow such as deferent flow level between undersluices and other bays, external forces due to reaction of bridge, operating gate etc. To determine internal forces of the floor due to that loads, it require amount of computation and more complex structural analysis. But for preliminary design needs (also because of time constraint), we may assume that the floor act as continues beam supported on some hinges and loaded by water pressure, as shown in fig. (III.12b). For making rough estimate (preliminary design of reinforcement) we can take the bending moment and shear force at the floor equal to

$$Mmin = -0.1 \cdot PF \cdot B^2 \quad \text{(III.2.8)}$$

and

$$S\,max = 0.6 \; PF \; B \quad \text{(III.2.9)}$$

respectively, where PF is uplift pressure at point F (toe of glacis) per metre run perpendicular to the plane of paper and B is net length of bays.

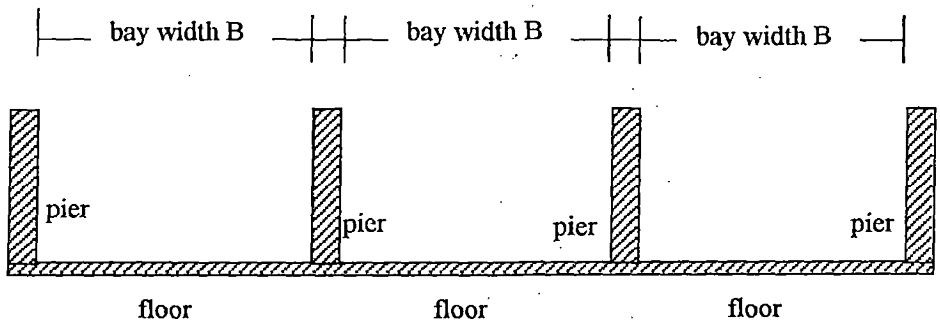


Fig. (III.2.12a) Section F

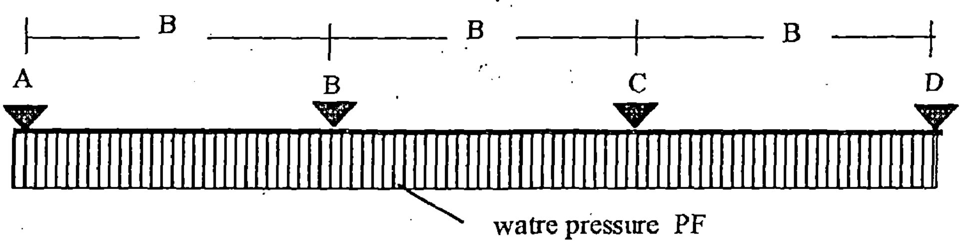


Fig. (III.2.12b) Continues Beam

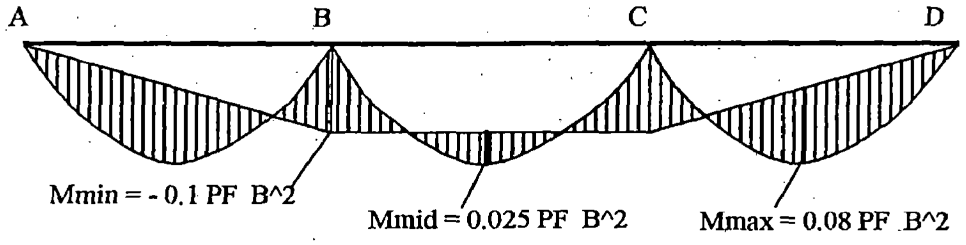


Fig. (III.2.12c) Bending Moment

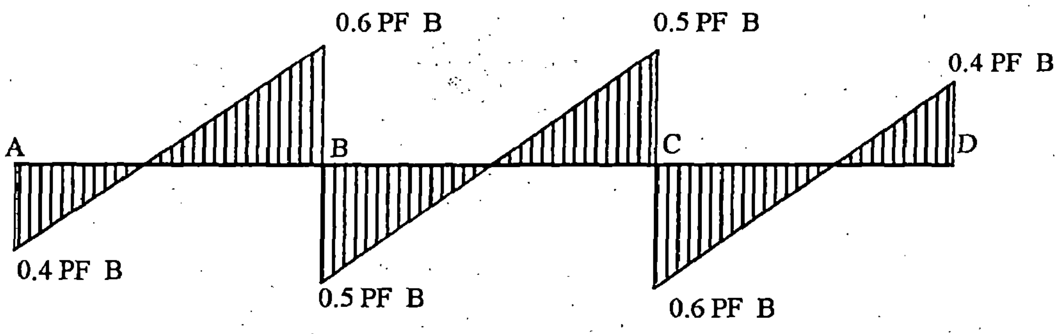


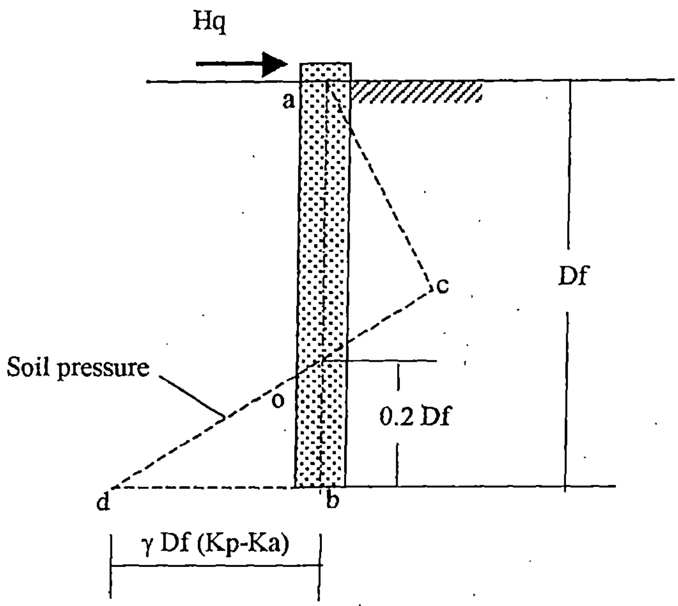
Fig. (III.2.12d) Shear Force

- **Strip Deep Foundation**

To determine forces acting on the body of strip foundation we use eq. (II.3.20). This moment is calculated based upon maximum soil resistance, thus moment ultimate of strip foundation structure per metre is equal to

$$Mu = 0.0133\,\gamma\,Df^3\,(Kp - Ka)$$

and shear ultimate force per metre is equal to maximum horizontal resistance Hq.



(III.2.13) Horizontal force acting perpendicular to the plane of strip foundation

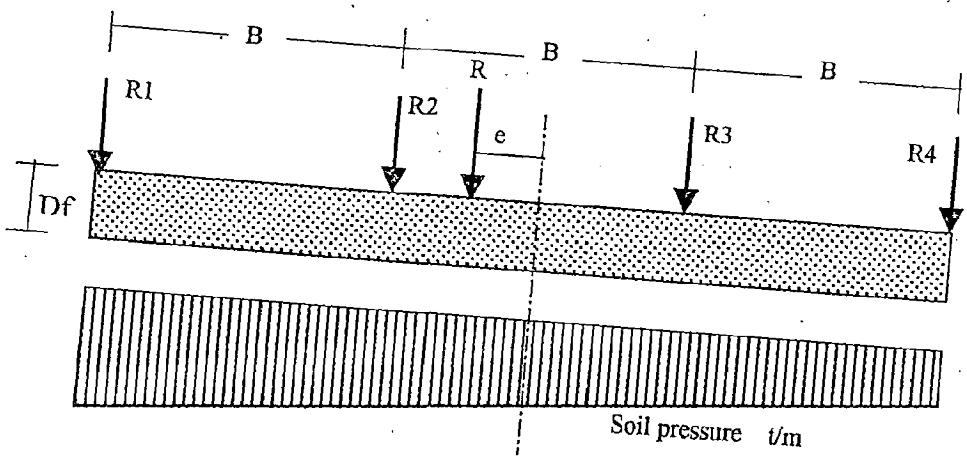


Fig. (III.2.14) Vertical force acting in the plane of strip foundation

In case of vertical forces acting on plane of strip foundation, there are various methods can be used for determine internal forces in body of foundation, such as conventional method, soil line method, simplified elastic foundation etc. In the present study, the conventional method has been chosen and discussed briefly. In this method we may assume that:

- The strip foundation, in plane direction, is infinitely rigid. Hence the deflection of the foundation does not effect to the soil pressure distribution.
- The soil pressure has a straight line or planar distribution such that the center of the soil pressure coincides with the line of action of the resultant forces of all the loads acting on the foundation.
- For this preliminary design, it is assumed that all girder loads are same and soil pressure distribution is constant along the foundation.

- The foundation is assumed as continues beam supported by some hinges supports. Thus, we can apply formulae given in eq.(III.2.8) and eq.(III.2.9) with replacing water pressure by soil pressure.

b.3. Design of Reinforcement (Ultimate Strength)

In the design of reinforcement , the section of the structure like floor, girder and strip foundation, may be assumed as rectangular with thickness or depth (d) equal to tf, D and td respectively. The width of the floor and strip foundation are taken equal to 1 m and tr for girder . Rectangular section with tension reinforcement only, can fail in one of the three principal mode. With a low percentage of reinforcement the failure will initiated by yielding of reinforcement and collapse will take place with the fracture of reinforcement. The second mode of failure, when the section is not grossly under reinforcement initiated by the yielding of reinforcement, consequently the neutral axis shifts upward and the final collapse is associated with crushing of concrete. The third mode of failure occurs when the steel ratio is high, in this case the concrete reaches the ultimate capacity before the steel start yielding. The second mode failure is gradual one and give ample warning while the third mode leads to sudden collapse. Therefore, member should be so dimensioned that failure, should it take place, will occur in the second mode and this can be ensured by setting an upper limit to depth of the compression zone. The code limit the depth of compressive stress block to 0.43 times the effective depth.

Referring to Fig. (III.2.15), and equating the longitudinal forces :

$$C = T$$

$$0.55\ \sigma cu\ b\ \ a = \sigma sy\ \ Ast$$

$$a = \frac{\sigma sy\ \ Ast}{0.55\,\sigma cu\ \ b} \qquad \text{(III.2.10)}$$

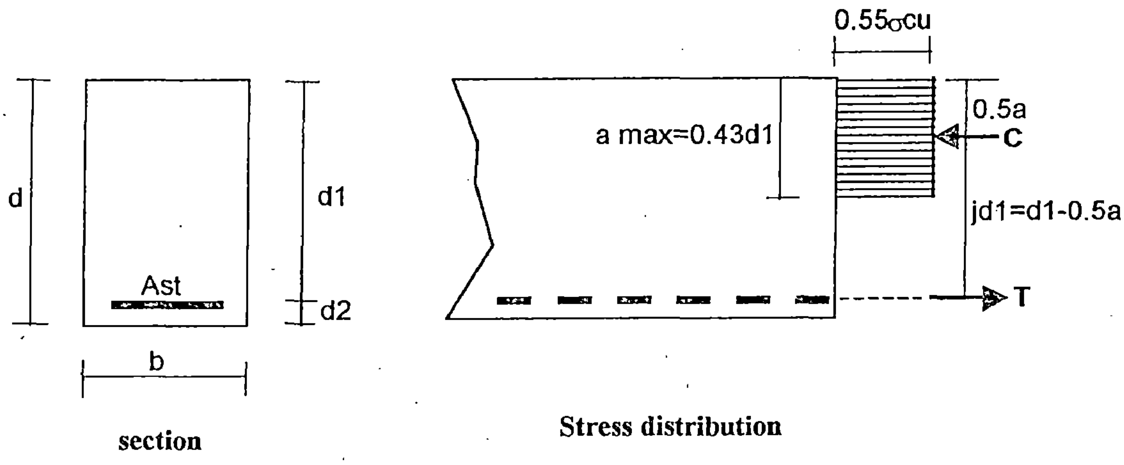


Fig. (III.2.15) Singly Reinforced Rectangular Section (Balance Design)

The lever arm of the resisting moment is given by:

$$jd1 = d1 - \frac{a}{2}$$

$$jd1 = d1 - \frac{Ast\ \ \sigma sy}{1.1\ \sigma cu\ \ b}$$

The ultimate moment of resistance of the section is given by:

- In term of tensile force in steel:

$$Mu = Ast\ \sigma sy \left(d1 - \frac{Ast\ \sigma sy}{1.1\ \sigma cu\ \ b} \right) \qquad (III.2.11)$$

- In term of compressive force in concrete:

$$Mu = 0.55\,\sigma cu\ \ b\ \ a \left(d1 - \frac{a}{2} \right) \qquad (III.2.12)$$

The maximum value of a , according to the Code, should be 0.43*d1. Substituting this value of a in eq.(III.2.10) and rearranging, we get:

$$\begin{aligned} Mu &= 0.55\ \ 0.43 \left(1 - \frac{0.43}{2} \right) \sigma cu\, b\ \ d1^2 \\ Mu &= 0.185\ \ \sigma cu\ \ b\ \ d1^2 \end{aligned} \qquad (III.2.13)$$

Equating the ultimate moments of resistance in terms of concrete and steel we get an expression for the area of steel under balance conditions:

$$\begin{aligned} 0.185\,\sigma cu\, b\ \ d1^2 &= Ast\ \ \sigma sy \left(d1 - \frac{a}{2} \right) \\ Ast &= 0.2368\ \frac{\sigma cu}{\sigma sy}\ b\ \ d1 \end{aligned} \qquad (III.2.14)$$

If the size of the section is restricted, it may happen that the concrete cannot develop the necessary compressive force required to resist the given bending moment. In this case, reinforcement is provided in the compression zone, resulting in a doubly reinforced section i.e one with the compression as well as tension reinforcement. This case occurs when it necessary to obtain a greater ultimate moment than that given by the eq. (III.2.13).

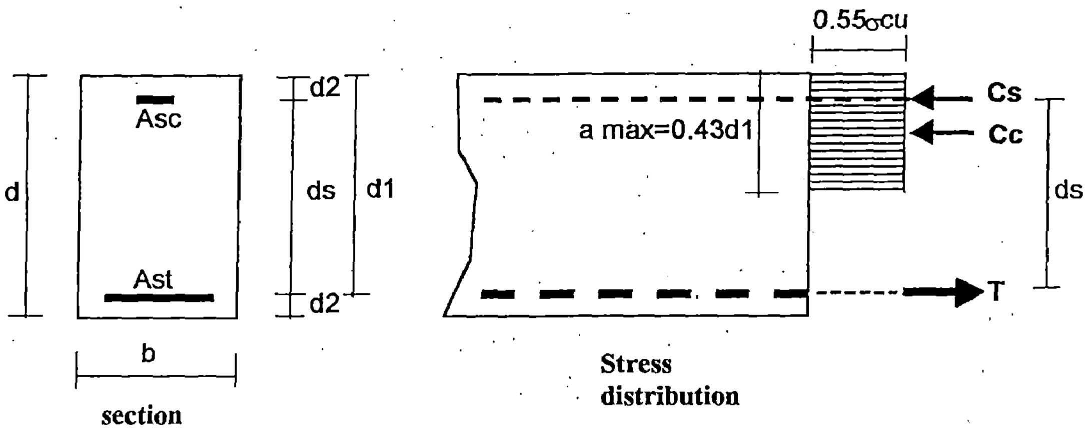


Fig. (II.2.16) Doubly Reinforcement Rectangular Section

The depth of concrete block is limited to 0.43 d1 and the maximum moment resisted by the concrete is 0.185*σcu*b*d1^2. Referring to Fig. (II.2.16), the internal moment is composed of two parts:

$$Mu = 0.185 \ \ \sigma cu \ \ b \ \ d1^2 + \sigma sc \ \ Asc \ \ ds \qquad \text{(III.2.15)}$$

where Asc= area of steel in compression,

ds= distance between centroid of tension steel and compression steel,

σsc= stress in compression steel.

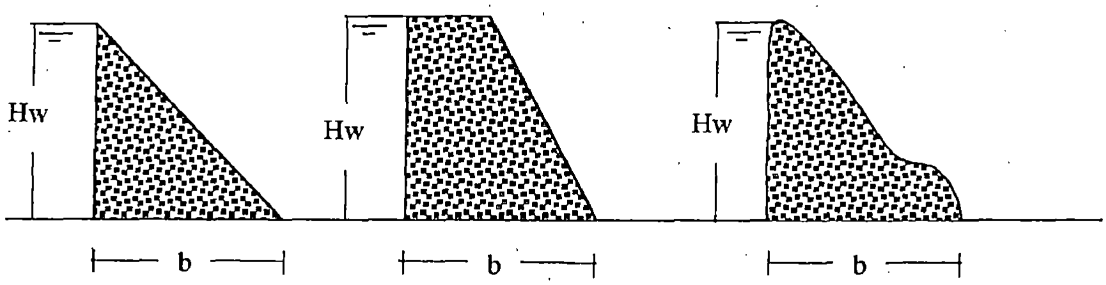
The stress in compression steel σsc is taken equal to the yield stress for mild steel reinforcement and equal to 3700 kg/cm2 for high strength with either the yield stress or 0.2 % proof stress equal to 4250 kg/cm2.[ref. 9]

III.3. DEVELOPING COMPUTER PROGRAMS

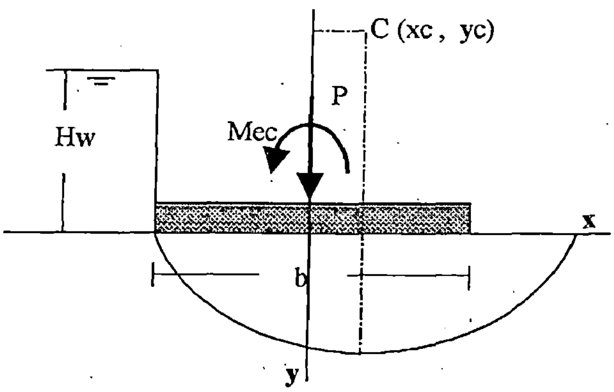
a. Circular Sliding Analysis

- **General**

The CISAN (Circular Sliding Analysis) computer program use C++ language and developed to analyze circular sliding problems in plain soil when it is subjected to strip distributed load (triangle , trapezoid or irregular shape with some modification) with or without seepage under ground surface, as shown in fig.(III.3.1a) . The loading configuration of structural weight is assumed as shown in fig. (III.3.1b). The soil below the structure is assumed as a homogeneous medium and isotropic.



(a)



(b)

Fig. (III.3.1) Input Model for CISAN

- `Input Data`

The input data of CISAN consist of structural dimension, external loads, height of water storage and soil parameters. According to fig. (III.3.1) the input data are

1. Coordinate of the centre of circle (xa , ya).
2. Width of the base of structure (b) in m.
3. Vertical load of structure weight and external loads (P) in ton, acting in the axis of the base.
4. Moment eccentricity (M) in ton-m. It is multiplying between vertical load with its eccentricity against axis of the base.
5. Height of water storage at upstream side (Hw) in m, unit weight is taken 1 ton/m3.
6. Soil parameters
   - Cohesion (ci) for clay soil (ton/m2).
   - Angle of internal friction (p) in degree for granular or cohesionless soil.
   - Unit weight of soil (Gs) in ton/m3 (use saturated condition).

To entry the input data we use (in this case) C++ editor as shown in example below

```
float xa=6,ya=-3;
float P=120,M=200,b=10,Hw=10;
float ci=0,p=30,Gs=2.1;

float L=b/2;
float r2=(L+xa)*(L+xa)+(ya*ya);
```

```
float AB=(L+xa)*2;
float R=sqrt(r2);
float t=asin(.5*AB/R);
float ch=ci*2*t*R;
float OB=R*sin(t)+xa;
float po=p*3.14159/180;
```

- **Determine Uplift Force Along Slip Circle Line**

The uplift force acting along slip circle line is determined separately with the normal force due to soil weight or external loads. The ϕ' having values ranging from $-\pi$ to 0 are divided into 20 parts with the increment (k) of 0.157, so we have 20 curves of equipotential lines along the slip circle line. To generate those curves we use ' array' and 'for' loop, as shown below

```
//cout<<"WATER PRESSURE AND FORCES ALONG THE CIRCLE
LINE"<<endl;

float Aox[10],x[10],Apr[10],Adx[9],Aax[9],Ajx[9],
      Asx[9],Afr[9],Ajpr[9];

float Aox1[11],x1[11],Apr1[11],Adx1[10],Afr1[10],
      Aax1[10],Ajx1[10],Asx1[10],Ajpr1[10];

float Tfr=0,Tfr1=0;

outfile<<"UPLIFT PRESSURE IN THE SLIDING LINE (-X
REGION)"<<endl;
outfile<<"Consindered Points of Pressure"<<endl;
```

```
outfile<<"     x              y          phi'
prs"<<endl;

for(int ko=0;ko<9; )
for(float q=-2.983;q<-1.57;q+=.157)
for(float xc=-500*b;xc<0;xc++)

   {
   float L=b/2;
   float xc1=xc/1000;
   float r2=(L+xa)*(L+xa)+(ya*ya);
   float ry=r2-(xc1-xa)*(xc1-xa);
   float y=sqrt(ry)+ya;

   float a=cos(q)*cos(q);
   float b1=sin(q)*sin(q);
   float c=(y*y)/(L*L*b1);
   float x2=(c+1)*L*L*a;
   float x=-sqrt(x2);
   float pr=y-q*Hw/3.14;
   Apr[ko]=pr;
     if(x<=xc1)
     {
     Aox[ko]=x;

cout<<setw(12)<<x<<setw(12)<<y<<setw(12)<<q<<setw(12)<<pr<<
endl;
     ko++;
outfile<<setw(10)<<x<<setw(10)<<y<<setw(10)<<q<<setw(10)<<p
r<<endl;
     break;
     }
```

```
}

outfile<<"\n   dx α  ds. pm     Fr=ds*pm.   Tot.Fr
"<<endl;

for(ko=0;ko<8;ko++)
  {
  Adx[ko]=Aox[ko+1]-Aox[ko];//dx
  Ajx[ko]=.5*(Aox[ko+1]+Aox[ko]);//(x1+x2)/2
  Aax[ko]=asin((Ajx[ko]-xa)/R);//alpha
  Asx[ko]=Adx[ko]/cos(Aax[ko]);//ds
  Ajpr[ko]=0.5*(Apr[ko+1]+Apr[ko]);//avr. pressure
  Afr[ko]=Asx[ko]*Ajpr[ko];//forces
  Tfr+=Afr[ko];//total forces


outfile<<setw(10)<<Adx[ko]<<setw(10)<<Aax[ko]<<setw(10)<<As
x[ko]
<<setw(10)<<Ajpr[ko]<<setw(10)<<Afr[ko]<<setw(10)<<Tfr<<end
l;

  }

outfile<<" UPLIFT PRESSURE IN THE SLIDING LINE(+X REGION)
"<<endl;
outfile<<"Consindered Points of Pressure"<<endl;
outfile<<"     x          y          phi'
prs"<<endl;

for(int k1=0;k1<10; )
```

```
for(float q3=-1.57;q3<-.157;q3+=0.157)
for(xc2=0;xc2<=1000*OB;xc2++)

  {

   float L1=b/2;
   float xc3=xc2/1000;
   float r12=(L1+xa)*(L1+xa)+(ya*ya);
   float ry1=r12-(xc3-xa)*(xc3-xa);
   float y1=sqrt(ry1)+ya;

   float a1=cos(q3)*cos(q3);
   float b2=sin(q3)*sin(q3);
   float c1=(y1*y1)/(L1*L1*b2);
   float x12=(c1+1)*L1*L1*a1;
   float x1=sqrt(x12);
   float pr1=y1-q3*Hw/3.14;
   Apr1[k1]=pr1;

     if(x1<=xc3)

      {
      Aox1[k1]=x1;

cout<<setw(12)<<x1<<setw(12)<<y1<<setw(12)<<q3<<setw(12)<<p
r1<<endl;
      k1++;

outfile<<setw(10)<<x1<<setw(10)<<y1<<setw(10)<<q3<<setw(10)
<<pr1<<endl;
      break;
      }
  }

outfile<<"\n  dx  α   ds    pm.   Fr=ds*pm. Tot.Fr"<<endl;
```

```
for(k1=0;k1<9;k1++)
  {
  Adx1[k1]=Aox1[k1+1]-Aox1[k1];//dx
  Ajx1[k1]=.5*(Aox1[k1+1]+Aox1[k1]);//(x1+x2)/2
  Aax1[k1]=asin((Ajx1[k1]-xa)/R);//alpha
  Asx1[k1]=Adx1[k1]/cos(Aax1[k1]);//ds
  Ajpr1[k1]=0.5*(Apr1[k1+1]+Apr1[k1]);//avr. pressure
  Afr1[k1]=Asx1[k1]*Ajpr1[k1];//forces
  Tfr1+=Afr1[k1];//total forces

cout<<setw(12)<<Adx1[k1]<<setw(12)<<Aax1[k1]<<setw(12)<<Asx
1[k1]
<<setw(12)<<Ajpr1[k1]<<setw(12)<<Afr1[k1]<<setw(12)<<Tfr1<<
endl;

outfile<<setw(10)<<Adx1[k1]<<setw(10)<<Aax1[k1]<<setw(10)
<<Asx1[k1]<<setw(10)<<Ajpr1[k1]<<setw(10)<<Afr1[k1]
<<setw(10)<<Tfr1<<endl;
  }
```

- **The Normal Force due to Soil Weight**

The mass of soil in the slip circle line is divided into many slice, every slice has width of i m. The number of slices and accuracy of computation depend on the value of i, smaller i give more number of slices and accurate result. The normal force due to total weight of soil in the circle line must be subtracted by total uplift force acting in the slip circle line to achieve effective force in that line.

```
outfile<<"SOIL IN THE SLIDING LINE"<<endl;
```

```
outfile<<" x        y       ap          wo=y*i*Gs      T=wo*cos
ap"<<endl;

for(xs1=-(L-.5*i);xs1<=(OB-.5*i);xs1+=i)

  {

  float ap=asin((xs1-xa)/R);
  float rys=r2-(xs1-xa)*(xs1-xa);
  float ys1=sqrt(rys)+ya;
  float wo=(ys1*i)*Gs;
  float T=(wo)*cos(ap);//+ch;//shear reistance

  sumo+=wo;
  sumsr+=T;

cout<<setw(12)<<xs1<<setw(12)<<ys1<<setw(12)<<ap<<setw(12)<
<wo
  <<setw(12)<<T<<endl;

outfile<<setw(10)<<xs1<<setw(10)<<ys1<<setw(10)<<ap<<setw(1
0)
  <<wo<<setw(10)<<T<<endl;   }
```

- **The Driving Load**

The driving load , in this case , net stress on the ground surface, is divided into slices for which the width of slice same as soil slice (i).

```
outfile<<"DRIVING LOADS"<<endl;
outfile<<"  x     y    qb    ap1          T=wn1*cos ap1
"<<endl;
```

```
for(float xs3=-(L-.5*i);xs3<=(L-.5*i);xs3+=i)
  {
  float rys3=r2-(xs3-xa)*(xs3-xa);
  float ys3=sqrt(rys3)+ya;

  float ap1=asin((xs3-xa)/R);
  float wo=P/b;
  float Mo=(Hw*Hw*Hw)/6;//moment due to u/s water pressure
  float I=8*L*L*L/12;//inertia of foundation
  float k1=(Mo-M)/I;
  float w1=k1*xs3+wo;//distributed load
  float w2=(w1*i);

  float u=-(0.5*Hw/L)*xs3+0.5*Hw;
  float u1=(u*i);
  float wn=w2-u1;//net distributed load on ground surface
  float n1=fabs((-k1*L+wo)-Hw);
  float wno=w1-u;
  float m1=(b*b)/((b+ys3)*(b+ys3));
  float xabs=fabs(-L-xs3);
  float m2=(xabs+0.5*ys3)/xabs;
  float nx=(wno-n1)*m1*m2+n1*(b/(b+ys3));//net load at slip
cir.
  float T1=(nx*i)*cos(ap1);//normal forces
  float Mw=(xs3-xa)*wn;//moment due to structure

  sumMw+=Mw;
  sumsr1+=T1;
  sumu1+=u1;
cout<<setw(12)<<xs3<<setw(12)<<ys3<<setw(12)<<nx<<setw(12)<
<ap1
  <<setw(12)<<T1<<endl;
```

```
outfile<<setw(10)<<xs3<<setw(10)<<ys3<<setw(10)<<nx<<setw(1
0)
  <<ap1<<setw(10)<<T1<<endl;
  }
```

- **Output Data**

Output data of CISAN is provide in text file document, it can be read out by MS-Word, MS-DOS or Excel. The following are some examples of output.

Output Explanation

```
(A). CIRCLE OF SLIDING
     Central angle (rd): 2.609088
     Radius of circle (m): 11.401754
```

- Central angle and radius of a trial slip circle are calculated by program, we only entry coordinate of a trial slip circle.

```
(B). UPLIFT PRESSURE IN THE SLIDING LINE (-X REGION)
     Considered Points of Pressure
```

(1) x	(2) y	(3) ϕ'	(4) p
-4.974594	0.0973	-2.983	9.597301
-4.891706	0.377559	-2.826	9.377559
-4.739186	0.83065	-2.669	9.33065
-4.501329	1.444097	-2.512	9.444097
-4.152554	2.190077	-2.355	9.690076
-3.670679	3.042387	-2.198	10.042386
-3.03063	3.961257	-2.041	10.461256
-2.213707	4.908643	-1.884	10.908642
-1.204251	5.837555	-1.727	11.337555

- Column no. (1) and (2) show coordinates of considered points which the pressure at that points will be determined.
- Column no. (3) contains of constants of equipotential lines, its values are given by program. There are twenty constants having values ranging from $-\pi$ to 0. For $\phi'=-\pi$, $-\pi/2$ and 0, the curves will be straight lines at upstream side, at the middle and perpendicular to the base of structure and at downstream side respectively. They are not be appeared in the output data.

(C). UPLIFT FORCES

(1) dx	(2) α	(3) ds.	(4) pm	(5) Fr=ds*pm	(6) Tot.Fr
0.082888	-1.283102	0.292125	9.48743	2.771514	2.771514
0.152519	-1.248711	0.481824	9.354105	4.507033	7.278547
0.237857	-1.1984	0.653725	9.387373	6.136764	13.41531
0.348776	-1.133105	0.822877	9.567086	7.872533	21.287844
0.481874	-1.053797	0.974914	9.866231	9.618726	30.90657
0.640049	-0.961597	1.118555	10.251822	11.467231	42.373802
0.816923	-0.857508	1.248499	10.684949	13.340144	55.713947
1.009456	-0.742487	1.370073	11.123098	15.23946	70.953407

Column no.

(1) is horizontal distance between two considered points.

(2) is angle of slice base inclination.

(3) is incline length of slice base.

(4) is average uplift pressure between two adjacent equipotential lines.

(5) is uplift forces at the middle of two adjacent equipotential lines.

(6) is accumulative uplift forces.

(D). SOIL IN THE SLIDING LINE

(1) x	(2) y	(3) α	(4) wo=y*i*Gs	(5) T=wo*cos α
-4.75	0.799671	-1.231044	0.839655	0.279818
-4.25	1.993746	-1.117445	2.093433	0.916883
-3.75	2.910795	-1.025805	3.056335	1.584438
-3.25	3.666146	-0.946336	3.849453	2.25062
-2.75	4.310096	-0.874816	4.5256	2.901534
-2.25	4.870038	-0.808965	5.11354	3.529611
-1.75	5.362864	-0.747381	5.631008	4.130185
-1.25	5.799858	-0.689131	6.089851	4.700139

Column no.

(1) and (2) show coordinates of the middle of slices base.

(3) is angle of slice base inclination.

(4) is weight of soil in the slice.

(5) shear force resistance in the slice base

(E). DRIVING LOADS

(1) x	(2) y	(3) qb	(4) ap1	(5) T=wn1*cos ap1
-4.75	0.799671	4.038114	-1.231044	0.672857
-4.25	1.993746	4.063694	-1.117445	0.88991
-3.75	2.910795	4.071998	-1.025805	1.055484
-3.25	3.666146	4.078046	-0.946336	1.192134
-2.75	4.310096	4.085904	-0.874816	1.309814
-2.25	4.870038	4.096919	-0.808965	1.413945
-1.75	5.362864	4.11156	-0.747381	1.507856
-1.25	5.799858	4.129982	-0.689131	1.593757
-0.75	6.188988	4.152226	-0.633556	1.673197
-0.25	6.536116	4.178303	-0.580164	1.747309

Column no.

(1) and (2) show coordinates of the middle of slices base.

(2) is angle of slice base inclination.

(3) distributed load due to external load at the slice base.

(4) additional shear force at the slice base.

```
(F). STABILITY

Uplift Force (t): 50
Weight of Soil(t): 286.996582
Normal Forces due to Weight of Soil(t): 251.432617
Normal Forces due to Weight of Soil(t): 33.867336
Total water force at the circle line(t): 244.480011
Total Normal force at the circle line(t): 285.299957
Effective force (t): 40.819946
Shear resistance due to soil cohesion (t/m2): 0
Total Shear Resistance(t): 23.567381
Resisting moment (tm): 268.709503
Driving moment (tm): 370.125
Safety factor of Circular sliding: 0.725997
```

b. Structural Design of Barrage

- **General**

The SDOB (Structural Design of Barrage) computer program use C++ language and developed to carry out structural design of barrage with floating floor and ribbed foundation. This program is useful for preliminary design only. The loads considered in this analysis are in the construction stage and ponding condition without earthquake.

- **Input Data**

The input data of SDOB consist of structural dimension, external loads, height of water storage and soil parameters. The input data are

1. Dimension of structure fig.(III.3.3) in m.
2. Vertical external loads due to bridge and appurtenant structure (P) in ton, acting in the middle of the pier.
3. Moment external loads due to bridge and appurtenant structure (M) in ton-m, acting in the middle of the pier.
4. Height of water storage at upstream side (Hw) in m, unit weight is taken 1 ton/m3.
5. Soil parameters
   - Cohesion (ci) for clay soil (ton/m2).
   - Angle of internal friction (p) in degree for granular or cohesionless soil.
   - Unit weight of soil (Gs) in ton/m3 (use submerged condition).

To entry the input data we use (in this case) C++ editor as shown in example below

```
float a=10,bs=13.5,bu=27.5,bd=28,B=15, dd=6,du=6.5,D=4,
      Hw=3.5,d=4.5,tf=0.8,tr=2.5,td=1;

float P=265,Mec=0;

long p=15;

float c=10, f=4,Gs=.9;
```

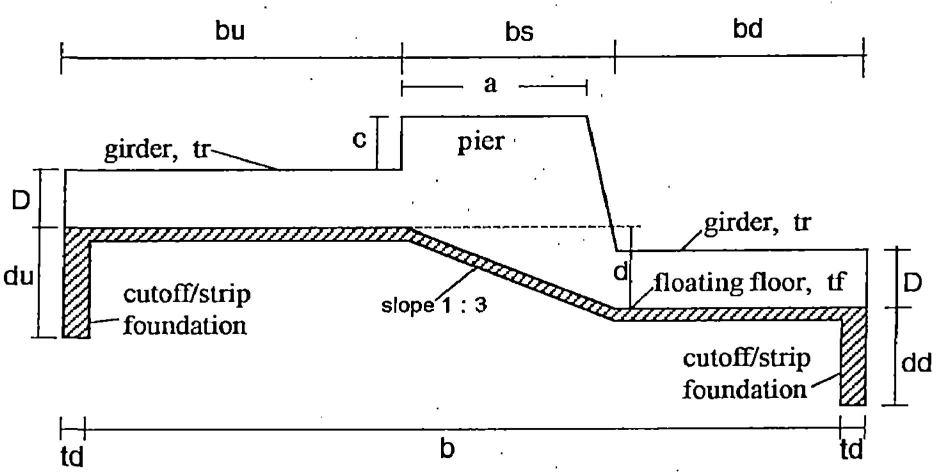


Fig. (III.3.3) Structural Dimension Input Data

- **Determine Uplift Pressure**

The uplift pressure at key points is determined by means of Khosla chart. This chart has been included in the program, thus, we only input the depth of cutoff at upstream and down stream. The uplift distribution along the base of structure is assumed to be linear. Corrections for pressure at key points due to floor thickness and interference of sheet piles are carried out.

```
outfile<<"\nPRESSURE AT THE UPSTREAM CUTOFF"<<endl;
float h=Hw+d;
float b=bu+bs+bd-2*td;
float bt=b+2*td;
float afu=du/bt;
float LD=.5*(1+sqrt(1+(bt/du)*(bt/du)));
```

```
float PD=acos((LD-1)/LD)/3.1416;
float PD1=(1-PD);
outfile<<"  -Alfa factor: "<<afu<<endl;
outfile<<"  -Pressure at bottom of cutoff (%):
"<<PD1*100<<endl;

float PDu=(1-PD)*h;
outfile<<"  -Pressure at bottom of cutoff (m):
"<<PDu<<endl;
float LE=.5*(1+sqrt(1+(bt/du)*(bt/du)));
float PE=acos((LE-2)/LE)/3.1416;
float PE1=1-PE;
outfile<<"  -Pressure at the corner  (%): "<<PE1*100<<endl;
float Cd=(PD1-PE1)*100*tf/du;
outfile<<"  -Correction depth (%): "<<Cd<<endl;
float k=du-tf;
float K=d-tf+dd;
float Ci=19*sqrt(K/b)*(k+K)/bt;
float PEc=PE1*100+Cd+Ci;
float PEu=PEc*h*.01;
outfile<<"  -Correction for interference of d/s cutoff (%):
"<<Ci<<endl;
outfile<<"  -Corrected pressure at the corner (%):
"<<PEc<<endl;
```

```
outfile<<"  -Pressure at the corner  (m): "<<PEu<<endl;

outfile<<"  -Correction depth (%): "<<Cd1<<endl;
```

- **Bearing Capacity of Foundation**

To analyze the bearing capacity of foundation use Zeevaert formula for deep foundation. All soil parameters in that formula was installed in the program.

```
//Bearing capacity

outfile<<"\nDESIGN OF FOUNDATION"<<endl;

switch (p)
  {
   case 0:

     Nc=5.7;
     Nq=1.0;
     break;

   case 5:

     Nc=7.8;
     Nq=2.7;
     break;

   case 10:

     Nc=10.5;
     Nq=2.7;
     break;

   case 15:

     Nc=15;
     Nq=4.8;
     break;
```

```
case 20:

  Nc=24;
  Nq=8.1;
  break;

case 25:

  Nc=40;
  Nq=15;
  break;

case 30:

  Nc=70;
  Nq=30;
  break;

case 35:

  Nc=140;
  Nq=65;
  break;

case 40:

  Nc=300;
  Nq=150;
  break;
}

float qf=(td*(10*Nc+(dd*Gs*Nq)*(0.4+0.1))+2*f*dd)*B;

float qf1=(td*(10*Nc+(du*Gs*Nq)*(0.4+0.1))+2*f*du)*B;
```

- **Design of Reinforcement.**

The ultimate design method of rectangular concrete section (according to IS code) is used here. In the design for preliminary purpose, the sections are doubly reinforced, the area of steel in compression zone equal to the area of steel in tension zone. The program will generate the moment ultimate of a section according to number of bar reinforcement started from 2 bars and then stopped if this moment exceed 1.5 times working moment.

```
cout<<"\nDESIGN OF REINFORCEMENT"<<endl;

for (int i=2;i<=100;i++)
  {
   float As=i*0.25*3.14*2.5*2.5;//steel area
tension=compression
   float ac=As*3700/(0.55*300*1*100);//concrete block
diagram
   float Mc=.55*300*1*100*ac*(100*tf-10-ac*0.5))*0.00001;
   //concrete
   float Ms=3700*As*(100*tf-20)*0.00001;//steel
   float Mu=Mc+Ms;//ultimate floor
   int Amax=(.236*300*1*100*(100*tf-10)/3700)/4.9;
       if(Mu>=1.5*Mx)
       {

       cout<<"\nFloor:  no. of bars per m: "<<i<<endl;
       cout<<"         Moment ultimate per m: "<<Mu<<endl;
       cout<<"         Max. no. of bars per m:
"<<Amax<<endl;

       outfile<<"\nFloor:  no. of bars per m: "<<i<<endl;
       outfile<<"         Moment ultimate per m:
"<<Mu<<endl;
       outfile<<"         Max. no. of bars per m:
"<<Amax<<endl;

       break;
```

```
    }
}
```

- **Output Data**

The following are sample of output data of SDOB. For detail program we can see appendix.

```
PRESSURE AT THE UPSTREAM CUTOFF
  -Alfa factor: 0.099237
  -Pressure at bottom of cutoff (%): 80.616486
  -Pressure at bottom of cutoff (m): 6.449319
  -Pressure at the corner  (%): 72.127159
  -Correction depth (%): 1.306051
  -Correction for interference of d/s cutoff (%): 1.676391
  -Corrected pressure at the corner (%): 75.109596
  -Pressure at the corner  (m): 6.008768


PRESSURE AT THE DOWNSTREAM CUTOFF
  -Alfa factor: 0.091603
  -Pressure at bottom of cutoff (%): 18.673073
  -Pressure at bottom of cutoff (m): 1.493846
  -Pressure at the corner (%): 26.816732
  -Correction depth (%): -1.357276
  -Correction for interference of u/s cutoff: -0.217557
  -Corrected pressure at the corner: 25.241898
  -Pressure at the corner (m): 2.019352
```

Chapter IV

APLICATION OF PROGRAM

IV.1. PRELIMINARY STRUCTURAL DESIGN OF BARRAGE (AN ILLUSTRATIVE EXAMPLE)

An example of design of barrage described in 'Theory and Design of Irrigation Structures' vol. II by Varshney-Gupta-Gupta was taken for the purpose of making comparative study between Gravity Floor system and Floating Floor and Ribbed Foundation system. The example of a barrage with gravity floor system has structural data described in the fig. (IV.1a) and in fig. (IV.1b) is the barrage with floating floor and ribbed foundation , they have same dimension except in floor and cutoff thickness. The example in that figure shows typical cross section of undersluice bays.

Loading Condition

For preliminary design purpose, some assumption for external loading have been considered as follows:

- Total Vertical loads due to 15 m span of bridge is taken equal to 180 ton.
- Gate loads (inclusive supporting structures) is 85 ton.
- Moment and horizontal loads due to live loads not considered.
- Hydrodynamic loads and Hydrostatic loads due to differential level of flow between adjacent bays not considered.
- Earthquake not considered.

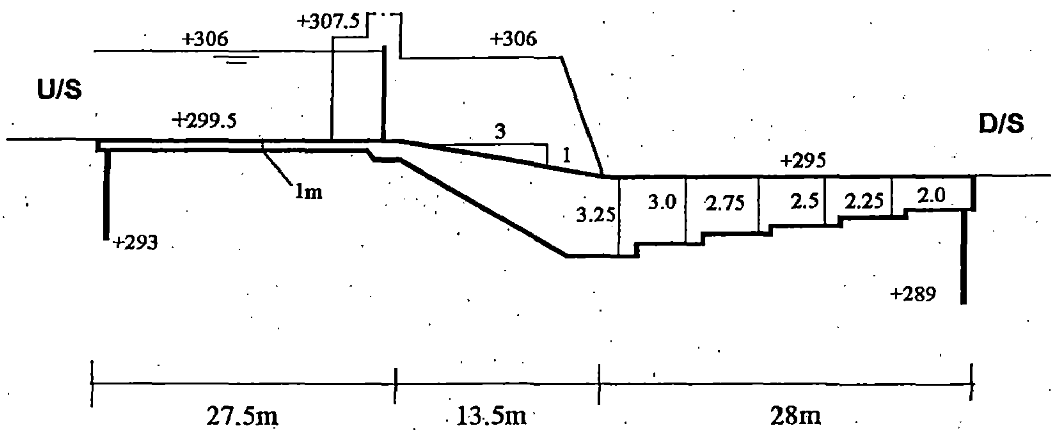


Fig. (IV.1a) Barrage with Gravity Floor
(taken from: Theory and Design of Irrigation Structure, book II by Varshney-Gupta-Gupta)

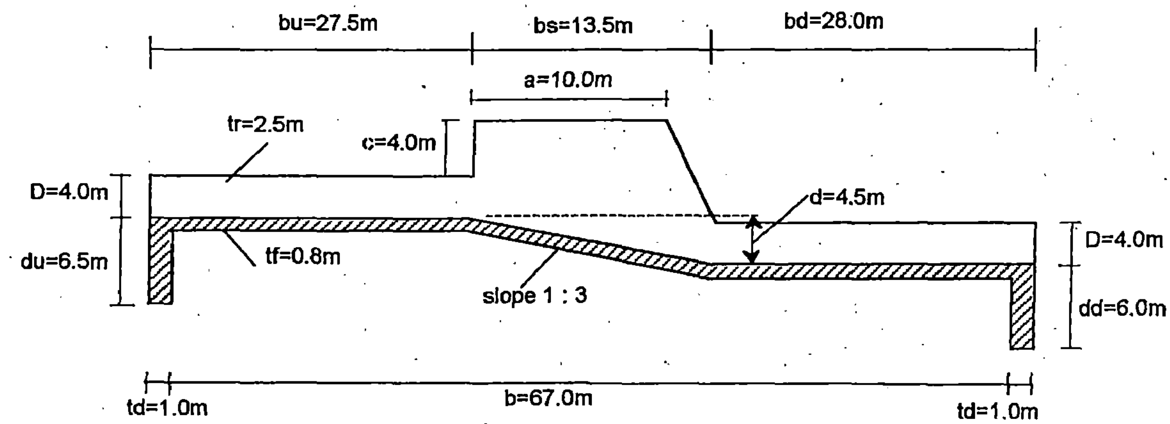


Fig. (IV.1b) Barrage with Floating Floor and Ribbed Foundation

- **Input Data**

structure dimension

To entry input data we refer to fig. (IV.1b)

- Length of upstream floor bu = 27.5 m.
- Length of downstream floor bd = 28 m.
- Horizontal length of glacis bs = 13.5 m.
- Cutoff thickness upstream and downstream was taken equal to td = 1 m.
- The net length between cutoff b = 67 m.
- Depth of upstream cutoff du = 6.5 m.
- Depth of downstream cutoff dd = 6 m.
- Depth of girder D = 4 m.
- Thickness of girder and pier tr = 2 .5 m.
- Width top of pier a = 10 m.
- Difference level between upstream and downstream floor d = 4.5 m.
- Floating floor thickness tf = 0.8 m.

Loads

- Vertical load P = 265 ton
- Moment M = 0

Soil Parameters

- Unit weight Gs = 0.9 t/m3 (submerged).
- Cohesion Ct = 10 t/m2.
- Angle of internal friction $\phi = 15$

- **Output Data**

DESIGN OF FLOATING FLOOR AND RIBBED FOUNDATION OF THE BARRAGE

by JUNAEDI 25072000

DIMENSION AND PARAMETERS

-External Vertical Load (P in ton): 265

-External Moment (Mec in t-m): 0

-Floor thickness (tf in m): 0.8

-Depth of u/s cutoff (du in m): 6.5

-Cutoff thickness (td in m): 1

-Depth of d/s cutoff (dd in m): 6

-Depth of girder (D in m): 4

-Girder/ Pier thickness (tr in m): 2.5

-Length top of pier (a in m): 10

-Pier height (c in m): 4

-Deference level between u/s and d/s (d in m): 4.5

-Depth of water an u/s: 3.5

-Width of bays (B in m): 15

-Length of u/s floor (bu in m): 27.5

-Length of glacis (horizontal) (bs in m): 13.5

-Length of d/s floor (bd in m): 28

-Unit weight of soil (Gs t/m3): 0.9

-Skin friction of soil (f t/m2): 4

-Internal friction angle of soil (p deg): 15

PRESSURE AT THE UPSTREAM CUTOFF

-Alfa factor: 0.094203

-Pressure at bottom of cutoff (%): 81.081055

-Pressure at bottom of cutoff (m): 6.486485

-Pressure at the corner (%): 72.818268

-Correction depth (%): 1.016959

-Correction for interference of d/s cutoff (%): 1.613518

-Corrected pressure at the corner (%): 75.448746

-Pressure at the corner (m): 6.0359

PRESSURE AT THE DOWNSTREAM CUTOFF

-Alfa factor: 0.086957

-Pressure at bottom of cutoff (%): 18.222948

-Pressure at bottom of cutoff (m): 1.457836

-Pressure at the corner (%): 26.149818

-Correction depth (%): -1.056916

-Correction for interference of u/s cutoff: -0.235851

-Corrected pressure at the corner: 24.857052

-Pressure at the corner (m): 1.988564

The Loads

-External Vertical Load (ton): 265

-External Moment (ton-m): 0

-Weight of water at u/s side (ton): 1443.75

-Weight of structure at ponding condition (ton): 3232.198

-Weight of structure at no water condition (ton): 4060.19

-Uplift force (ton): 4203.427246

Upstream foundation

-Reaction at ponding condition (ton): 460.696228

-Reaction at no water condition (ton): 2175.675293

-Bearing capacity at u/s foundation (ton): 3205.5

Downstream foundation

-Reaction at ponding condition (ton): 282.905334

-Reaction at no water condition (ton): 2149.523438

-Bearing capacity at d/s foundation (ton): 3132

STABILITY ANALYSIS

-Horizontal movement safety factor: 1.599294

-Vertical movement safety factor: 1.396229

-Overturning moment safety factor against u/s: 1.424717

-Overturning moment safety factor against d/s: 1.206756

STRUCTURAL ANALYSIS

-Moment at Girder (t-m): 6935.697266

-Moment at Floor (t-m): 57.599728

-Deflection at Floor (cm): 0.004708

-Horizontal moment at Foundation/cutoff (t-m): 27.502998

-Vertical moment at Foundation/cutoff (t-m): 3263.512695

Floor: no. of bars per m: 4

Moment ultimate per m: 92.798508

Max. no. of bars per m: 27

Girder: no. of bars: 77

Moment ultimate: 10526.162109

Max. no. of bars: 380

Cutoff

Reinforcement due to Horizontal Load

-no. of bars per m: 2

-Moment ultimate per m: 61.32119

-Max. no. of bars per m: 35

Reinforcement due to Vertical Load

-no. of bars : 22

-Moment ultimate per m: 5023.651367

-Max. no. of bars per m: 249

- **Comparison of the results**

	Floating Floor	Gravity Floor*
PRESSURE AT THE U/S CUTOFF		
Alfa factor:	0.094203	0.095
Pressure at D1 (%):	81.081055	81
Pressure at D1 (m):	6.486485	6.48
Pressure at C1 (%):	72.818268	73
Correction depth (%):	1.024698	1.23
Correction for interference(%):	1.637246	1.55
Corrected pressure at C1(%):	75.287994	75.8
Pressure at C1 (m):	6.023039	6.06
PRESSURE AT THE D/S CUTOFF		
Alfa factor:	0.086957	0.087
Pressure at D(%):	18.222948	18.000
Pressure at D(m):	1.457836	1.440
Pressure at C(%):	26.149818	27.000
Correction depth (%):	-1.056916	-2.250
Correction for interference(%):	-0.235851	-0.120
Corrected pressure at C:	24.857052	24.600
Pressure at the corner (m):	1.988564	1.970
FLOOR THICKNESS (in m)		
Upstream floor	0.8	1.0
Downstream floor	0.8	2.0 up to 3.25
CUT OFF / SHEET PILE	reinforced Concrete of 100 cm thickness	steel sheet pile

*) These data are taken from 'Theory and Design of Irrigation Structure' Vol. II by Varshney-Gupta-Gupta.

- **Discussing of the Results**

  - The pressure at key points calculated by means of SDOB are practically same with conventional method. The slightly difference in determination of pressure correction is caused by the floor thickness that had been assumed.
  - By putting the floor thickness equal to 0.8 m having weight 1.12 t/m2 , this weight is less than minimum uplift pressure at downstream (2 t/m2), therefore, the floor is floating.
  - The upward deflection of the floor is 0.004713 cm or less than permissible deflection 1/250 B or 1500/250 or 6 cm. The number of reinforcement bars are 4ϕ 25 mm per m length, meanwhile maximum number of reinforcement are 27ϕ 25.
  - By the given dimension of structure, the stability factors of the structure are greater than 1.2 , the structure is safe. Those factors can be increased by enlarge dimension of structure, lengthen of upstream floor, for example. It will produce counter weight in upstream side.

Chapter V

CONCLUSIONS

In the design of hydraulic structure, stability of soil base and the structure are influenced by uplift pressure. An analysis of a simple structure resting on clayey soil shows how the uplift pressure acts below the foundation and influences the stability of the structure. The stability against circular sliding of structure founded on granular soil (ϕ soil) is strongly influenced by pore pressure or uplift pressure along the sliding line, but in the soil having cohesion and frictional characteristics (C-ϕ soil), the contribution of friction in the stability is very small compared to cohesion. Thus, in the stability analysis of structure against circular sliding, the frictional part may be ignored. The soil may be assumed as clayey soil. By putting width of the base equal to height of the structure (or b= 0.952h respect to the horizontal sliding requirement SF = 0.75) , the safety factor against circular sliding of the structure founded on clayey soil is very high. The structure is safe.

An analytical method by using conformal mapping is adopted to carry out determination of uplift pressure at any location below the structure. If the structure has simple shape (flat bottom) this method is very useful.

In the design of barrages with gravity floor concept, stability of the barrage is governed by the weight of the floor. The thickness of gravity floor required, therefore, is to be large. The floating floor and ribbed foundation barrage system is a barrage having a quite thin of floor but structurally it has high rigidity , because the floor is stiffed by two or more ribs of reinforced concrete wall below the floor and girders in upper side. These ribs are arranged perpendicular to each other. The stability of the structure is dependent on the rigidity of the structure to transfer the loads (weight of water at upstream side, weight of the structure and external loads) as counter balance against the uplift force. By equating equilibrium of that forces, the weight of structure required can be determined.

The floor is designed as thin as possible to resist uplift pressure , but structurally the floor must strong enough to resist the pressure without cracking and permitted camber/deflection in the permissible range . The total weight of the barrage, therefore, can be designed as small as possible. Consequently , the problems in the construction stage can be significantly reduced and time of construction can be speeded up.

Due to the features described above, the barrage with floating floor and ribbed foundation has some advantages :

1. The system suitable for the alluvial zones which usually have low bearing capacity of soil base.
2. The floor is designed to resist the uplift pressure by its structural strength, so the floor thickness will be quite thin, total dimension of structure will be less and cost of construction can be significantly reduced.
3. Due to light weight and high rigidity of structure, this system is superior in seismic zones compared to the gravity floor system.
4. The size being very small, the problems encountered in construction stage will be less and construction can be speeded up .

APPENDIX

Computer Programs

- **Pmin (Weight of Barrage Required Analysis)**

```
#include <iostream.h>
#include <process.h>
#include <conio.h>
#include <math.h>
#include <iomanip.h>
#include <fstream.h>
void main()
{clrscr();
ofstream outfile("Pmin.txt");

float b=10,M=0,Hw=5;
outfile<<"Pmin.cpp:Minimum of Structure Weight"<<endl;
outfile<<"Length of floor: "<<b<<endl;
outfile<<"The External moment: "<<M<<endl;
outfile<<"Height of waterat U/S floor: "<<Hw<<endl;

for(float bu=0;bu<=10;bu++)

{

float W=Hw*bu;
float bd=b-bu;
float P=-bu*Hw+0.5*b*Hw;
float Pu=(M+0.5*b*Hw*3*b/8-W*0.5*bu-Hw*Hw*Hw/6)/(0.5*b);
float Pd=(-M+0.5*b*Hw*5*b/8-
W*(0.5*bu+bd)+Hw*Hw*Hw/6)/(0.5*b);
cout<<setw(16)<<bu<<setw(15)<<Pu<<setw(15)<<Pd<<setw(15)<<P<
<endl;
outfile<<setw(16)<<bu<<setw(15)<<Pu<<setw(15)<<Pd<<setw(15)<
<P<<endl;

}

getch();
}
```

- **CISAN (Circular Sliding Analysis)**

```
#include <iostream.h>
#include <process.h>
#include <conio.h>
#include <math.h>
#include <iomanip.h>
#include <fstream.h>
void starline();
void main()
{clrscr();
start:

ofstream outfile("CISAN.txt");//checked 10/10-2000

float xa=6,ya=-3;
float P=120,M=200,b=10,Hw=10;
float Gs=2.1,ci=0,p=30;

float q1,s,h1,h2,y,xn,yn,Gt,xi,xq,xq1,xq2,xs,xs1,xs2,
     q2,H,xc2;

float sumsr=0,sumsr1=0,sumu1=0,sumMw=0,
      sumo=0;

float i=.5;
//starline();
outfile<<"DATA"<<endl;
//cout<<"Centre of Circle (xc,yc)"<<endl;
//cout<<"Xa : ";
//cin>>xa;
//cout<<"Ya : ";
//cin>>ya;
outfile<<"   Centre of Circle (xa,ya):"<<xa<<" ,"<<ya<<"
"<<endl;
//cout<<"Vertical Load of structure (+ downward, t): ";
//cin>>P;
outfile<<"Vertical Load of structure (+ downward, t):
"<<P<<endl;
//cout<<"Moment Load of structure (+ counter clockwise, tm):
";
//cin>>M;
outfile<<"   Moment Load of structure (+ counter clockwise,
tm): "<<M<<endl;
//cout<<"Length of base of structure (m): ";
//cin>>b;
```

```
outfile<<"   Length of base of structure (m): "<<b<<endl;
//cout<<"Length of u/s floor of structure (t/m3): ";
//cout<<"Unit weight of soil (t/m3): ";
//cin>>Gs;
outfile<<"   Unit weight of soil (t/m3): "<<Gs<<endl;
//cout<<"Angle of internal friction of soil (deg): ";
//cin>>p;
outfile<<"   Angle of internal friction of soil (deg):
"<<p<<endl;
//cout<<"Cohesion of soil (t/m2): ";
//cin>>ci;
outfile<<"   Cohesion of soil (t/m2): "<<ci<<endl;
//cout<<"Height of water (m): ";
//cin>>Hw;
outfile<<"   Height of water(m): "<<Hw<<endl;

float L=b/2;
float r2=(L+xa)*(L+xa)+(ya*ya);
float AB=(L+xa)*2;
float R=sqrt(r2);
float t=asin(.5*AB/R);
float ch=ci*2*t*R;
float OB=R*sin(t)+xa;
float po=p*3.14159/180;


//I. WATER PRESSURE
starline();
//cout<<"WATER PRESSURE AND FORCES ALONG THE CIRCLE
LINE"<<endl;
cout<<"CIRCLE OF SLIDING"<<endl;
cout<<"  Central angle (rd): "<<2*t<<endl;
cout<<"  Radius of circle (m): "<<R<<endl;

//outfile<<"WATER PRESSURE AND FORCES ALONG THE CIRCLE
LINE"<<endl;
outfile<<"CIRCLE OF SLIDING"<<endl;
outfile<<"   Central angle (rd): "<<2*t<<endl;
outfile<<"   Radius of circle (m): "<<R<<endl;

float Aox[10],x[10],Apr[10],Adx[9],Aax[9],Ajx[9],
      Asx[9],Afr[9],Ajpr[9];

float Aox1[11],x1[11],Apr1[11],Adx1[10],Afr1[10],
      Aax1[10],Ajx1[10],Asx1[10],Ajpr1[10];

float Tfr=0,Tfr1=0;
```

```
starline();
cout<<"UPLIFT PRESSURE IN THE SLIDING LINE (NEGATIVE REGION)
"<<endl;

outfile<<"UPLIFT PRESSURE IN THE SLIDING LINE (NEGATIVE
REGION)"<<endl;
outfile<<"Consindered Points of Pressure"<<endl;
cout<<"      x          y          phi'          prs"<<endl;
outfile<<"      x          y          phi'
prs"<<endl;

for(int ko=0;ko<9; )
for(float q=-2.983;q<-1.57;q+=.157)
for(float xc=-500*b;xc<0;xc++)

   {

   float L=b/2;
   float xc1=xc/1000;
   float r2=(L+xa)*(L+xa)+(ya*ya);
   float ry=r2-(xc1-xa)*(xc1-xa);
   float y=sqrt(ry)+ya;

   float a=cos(q)*cos(q);
   float b1=sin(q)*sin(q);
   float c=(y*y)/(L*L*b1);
   float x2=(c+1)*L*L*a;
   float x=-sqrt(x2);
   float pr=y-q*Hw/3.14;
   Apr[ko]=pr;
     if(x<=xc1)
     {
     Aox[ko]=x;

cout<<setw(12)<<x<<setw(12)<<y<<setw(12)<<q<<setw(12)<<pr<<en
dl;
     ko++;

outfile<<setw(10)<<x<<setw(10)<<y<<setw(10)<<q<<setw(10)<<pr<
<endl;
     break;
     }

   }

//starline();
cout<<"\nAverage Pressure and Total Force"<<endl;
```

```
cout<<"        dx         Alpha        ds.        avprs.
Fr=ds*avprs.   Tot.Fr   "<<endl;
outfile<<"\n        dx         Alpha        ds.        avprs.
Fr=ds*avprs.   Tot.Fr   "<<endl;
for(ko=0;ko<8;ko++)
  {
  Adx[ko]=Aox[ko+1]-Aox[ko];//dx
  Ajx[ko]=.5*(Aox[ko+1]+Aox[ko]);//(x1+x2)/2
  Aax[ko]=asin((Ajx[ko]-xa)/R);//alpha
  Asx[ko]=Adx[ko]/cos(Aax[ko]);//ds
  Ajpr[ko]=0.5*(Apr[ko+1]+Apr[ko]);//avr. pressure
  Afr[ko]=Asx[ko]*Ajpr[ko];//forces
  Tfr+=Afr[ko];//total forces


cout<<setw(12)<<Adx[ko]<<setw(12)<<Aax[ko]<<setw(12)<<Asx[ko]

<<setw(12)<<Ajpr[ko]<<setw(12)<<Afr[ko]<<setw(12)<<Tfr<<endl;

outfile<<setw(10)<<Adx[ko]<<setw(10)<<Aax[ko]<<setw(10)<<Asx[
ko]

<<setw(10)<<Ajpr[ko]<<setw(10)<<Afr[ko]<<setw(10)<<Tfr<<endl;

  }

starline();

cout<<" UPLIFT PRESSURE IN THE SLIDING LINE (POSITIVE REGION)
"<<endl;
cout<<"      x           y           phi'          prs"<<endl;
outfile<<" UPLIFT PRESSURE IN THE SLIDING LINE(POSITIVE
REGION) "<<endl;
outfile<<"Consindered Points of Pressure"<<endl;
outfile<<"      x           y           phi'
prs"<<endl;

for(int k1=0;k1<10; )

for(float q3=-1.57;q3<-.157;q3+=0.157)
for(xc2=0;xc2<=1000*OB;xc2++)

  {


  float L1=b/2;
  float xc3=xc2/1000;
```

```
    float r12=(L1+xa)*(L1+xa)+(ya*ya);
    float ry1=r12-(xc3-xa)*(xc3-xa);
    float y1=sqrt(ry1)+ya;

    float a1=cos(q3)*cos(q3);
    float b2=sin(q3)*sin(q3);
    float c1=(y1*y1)/(L1*L1*b2);
    float x12=(c1+1)*L1*L1*a1;
    float x1=sqrt(x12);
    float pr1=y1-q3*Hw/3.14;
    Apr1[k1]=pr1;

        if(x1<=xc3)

         {
         Aox1[k1]=x1;

 cout<<setw(12)<<x1<<setw(12)<<y1<<setw(12)<<q3<<setw(12)<<pr1
 <<endl;
         k1++;

 outfile<<setw(10)<<x1<<setw(10)<<y1<<setw(10)<<q3<<setw(10)<<
 pr1<<endl;
         break;
         }
    }

 //starline();
 cout<<"\nAverage Pressure and Total Force"<<endl;
 cout<<"          dx      Alpha          ds           avprs.
 Fr=ds*avprs.  Tot.Fr"<<endl;
 outfile<<"\n          dx      Alpha          ds           avprs.
 Fr=ds*avprs. Tot.Fr"<<endl;

 for(k1=0;k1<9;k1++)
   {
   Adx1[k1]=Aox1[k1+1]-Aox1[k1];//dx
   Ajx1[k1]=.5*(Aox1[k1+1]+Aox1[k1]);//(x1+x2)/2
   Aax1[k1]=asin((Ajx1[k1]-xa)/R);//alpha
   Asx1[k1]=Adx1[k1]/cos(Aax1[k1]);//ds
   Ajpr1[k1]=0.5*(Apr1[k1+1]+Apr1[k1]);//avr. pressure
   Afr1[k1]=Asx1[k1]*Ajpr1[k1];//forces
   Tfr1+=Afr1[k1];//total forces


 cout<<setw(12)<<Adx1[k1]<<setw(12)<<Aax1[k1]<<setw(12)<<Asx1[
 k1]
```

```
<<setw(12)<<Ajpr1[k1]<<setw(12)<<Afr1[k1]<<setw(12)<<Tfr1<<en
dl;

outfile<<setw(10)<<Adx1[k1]<<setw(10)<<Aax1[k1]<<setw(10)<<As
x1[k1]

<<setw(10)<<Ajpr1[k1]<<setw(10)<<Afr1[k1]<<setw(10)<<Tfr1<<en
dl;
  }
starline();
float Frs=Tfr+Tfr1;

cout<<"SOIL IN THE SLIDING LINE"<<endl;
outfile<<"SOIL IN THE SLIDING LINE"<<endl;

cout<<"     x              y          ap         wo=y*i*Gs
T=wo*cos ap"<<endl;
outfile<<"     x              y          ap         wo=y*i*Gs
T=wo*cos a"<<endl;
for(xs1=-(L-.5*i);xs1<=(OB-.5*i);xs1+=i)

   {

   float ap=asin((xs1-xa)/R);
   float rys=r2-(xs1-xa)*(xs1-xa);
   float ys1=sqrt(rys)+ya;
   float wo=(ys1*i)*Gs;
   float T=(wo)*cos(ap);//+ch;//shear reistance

   sumo+=wo;
   sumsr+=T;

cout<<setw(12)<<xs1<<setw(12)<<ys1<<setw(12)<<ap<<setw(12)<<w
o
  <<setw(12)<<T<<endl;

outfile<<setw(10)<<xs1<<setw(10)<<ys1<<setw(10)<<ap<<setw(10)
<<wo
  <<setw(10)<<T<<endl;
  }

starline();
cout<<"DRIVING LOADS"<<endl;
outfile<<"DRIVING LOADS"<<endl;
```

```
cout<<"          x          y        qb          ap1        T=wn1*cos
ap1 "<<endl;
outfile<<"         x          y        qb          ap1
T=wn1*cos ap1 "<<endl;

for(float xs3=-(L-.5*i);xs3<=(L-.5*i);xs3+=i)
  {
   float rys3=r2-(xs3-xa)*(xs3-xa);
   float ys3=sqrt(rys3)+ya;

   float ap1=asin((xs3-xa)/R);
   float wo=P/b;
   float Mo=(Hw*Hw*Hw)/6;//moment due to u/s water pressure
   float I=8*L*L*L/12;//inertia of foundation
   float k1=(Mo-M)/I;
   float w1=k1*xs3+wo;//distributed load
   float w2=(w1*i);

   float u=-(0.5*Hw/L)*xs3+0.5*Hw;
   float u1=(u*i);
   float wn=w2-u1;//net distributed load on ground surface
   float n1=fabs((-k1*L+wo)-Hw);
   float wno=w1-u;
   float m1=(b*b)/((b+ys3)*(b+ys3));
   float xabs=fabs(-L-xs3);
   float m2=(xabs+0.5*ys3)/xabs;
   float nx=(wno-n1)*m1*m2+n1*(b/(b+ys3));//net distributed on
   slip circle
   float T1=(nx*i)*cos(ap1);//normal forces
   float Mw=(xs3-xa)*wn;//moment due to structure

   sumMw+=Mw;
   sumsr1+=T1;
   sumu1+=u1;


cout<<setw(12)<<xs3<<setw(12)<<ys3<<setw(12)<<nx<<setw(12)<<a
p1
  <<setw(12)<<T1<<endl;

outfile<<setw(10)<<xs3<<setw(10)<<ys3<<setw(10)<<nx<<setw(10)
<<ap1
  <<setw(10)<<T1<<endl;

  }
starline();
```

```
float TN=sumsr+sumsr1;//normal force
float TS=(TN-Frs)*tan(po)+ch;//shear resistance
float Md=fabs(sumMw);//driving moment against xa
float Mts=TS*R;//resisting monent
float SFs=Mts/Md;

cout<<"STABILITY"<<endl;
outfile<<"STABILITY"<<endl;

cout<<"Uplif Force : "<<sumu1<<endl;
cout<<"Weight of Soil: "<<sumo<<endl;
cout<<"Normal Forces due to Weight of Soil: "<<sumsr<<endl;
cout<<"Normal Forces due to external load: "<<sumsr1<<endl;

cout<<"Total water force at the circle plane: "<<Frs<<endl;
cout<<"Total Normal force at the circle plane: "<<TN<<endl;
cout<<"Efective force (t): "<<(TN-Frs)<<endl;
cout<<"Shear resistance due to soil cohesion (t/m2):
"<<ch<<endl;
cout<<"Total Shear Resistance: "<<TS<<endl;
cout<<"Resisting moment (tm): "<<Mts<<endl;
cout<<"Driving moment (tm): "<<Md<<endl;
cout<<"Safety factor of Circular sliding: "<<SFs<<endl;

outfile<<"Uplif Force (t): "<<sumu1<<endl;
outfile<<"Weight of Soil(t): "<<sumo<<endl;
outfile<<"Normal Forces due to Weight of Soil(t):
"<<sumsr<<endl;
outfile<<"Normal Forces due to Weight of Soil(t):
"<<sumsr1<<endl;
outfile<<"Total water force at the circle line(t):
"<<Frs<<endl;
outfile<<"Total Normal force at the circle line(t):
"<<TN<<endl;
outfile<<"Efective force (t): "<<(TN-Frs)<<endl;
outfile<<"Shear resistance due to soil cohesion (t/m2):
"<<ch<<endl;
outfile<<"Total Shear Resistance(t): "<<TS<<endl;
outfile<<"Resisting moment (tm): "<<Mts<<endl;
outfile<<"Driving moment (tm): "<<Md<<endl;
outfile<<"Safety factor of Circular sliding: "<<SFs<<endl;

starline();
char cr;
cout<<"Try once more:(n/y)";
cin>>cr;
```

- **CISANx (Critical Centre Analysis)**

```
#include <iostream.h>
#include <process.h>
#include <conio.h>
#include <math.h>
#include <iomanip.h>
#include <fstream.h>
void starline();
void main()
{clrscr();
start:
ofstream outfile("hydrx.txt");//checked 28/10-2000

for(float ya=-5;ya<=-2;ya+=.5)
for(float xa=4;xa<=7;xa+=.5)

{
//float xa=5;
float P=120,M=200,b=10,Hw=10;
float Gs=2.1,ci=10,p=30;

float sumsr=0,sumsr1=0,sumu1=0,sumMw=0,sumo=0;
float Tfr=0,Tfr1=0;
float i=0.5;

float L=b/2;
float r2=(L+xa)*(L+xa)+(ya*ya);
float AB=(L+xa)*2;
float R=sqrt(r2);
float t=asin(.5*AB/R);
float ch=ci*2*t*R;
float OB=R*sin(t)+xa;
float po=p*3.14159/180;

//I. WATER PRESSURE

   float Aox[20],x[10],Apr[10],Adx[9],Aax[9],Ajx[9],
      Asx[9],Afr[9],Ajpr[9];

   float Aox1[11],x1[11],Apr1[11],Adx1[10],Afr1[10],
      Aax1[10],Ajx1[10],Asx1[10],Ajpr1[10];

   for(int ko=0;ko<9; )
   for(float q=-2.983;q<-1.57;q+=.157)
   for(float xc=-500*b;xc<0;xc++)
```

```
{

float L=b/2;
float xc1=xc/1000;
float r2=(L+xa)*(L+xa)+(ya*ya);
float ry=r2-(xc1-xa)*(xc1-xa);
float y=sqrt(ry)+ya;

float a=cos(q)*cos(q);
float b1=sin(q)*sin(q);
float c=(y*y)/(L*L*b1);
float x2=(c+1)*L*L*a;
float x=-sqrt(x2);
float pr=y-q*Hw/3.14;
Apr[ko]=pr;

  if(x<=xc1)
  {
  Aox[ko]=x;

//cout<<setw(12)<<x<<setw(12)<<y<<setw(12)<<q<<setw(12)<<pr<<
endl;
  ko++;

//outfile<<setw(12)<<x<<setw(12)<<y<<setw(12)<<q<<setw(12)<<p
r<<endl;
  break;
  }

}


for(ko=0;ko<8;ko++)
{
Adx[ko]=Aox[ko+1]-Aox[ko];//dx
Ajx[ko]=.5*(Aox[ko+1]+Aox[ko]);//(x1+x2)/2
Aax[ko]=asin((Ajx[ko]-xa)/R);//alpha
Asx[ko]=Adx[ko]/cos(Aax[ko]);//ds
Ajpr[ko]=0.5*(Apr[ko+1]+Apr[ko]);//avr. pressure
Afr[ko]=Asx[ko]*Ajpr[ko];//forces
Tfr+=Afr[ko];//total forces


//cout<<setw(12)<<Adx[ko]<<setw(12)<<Aax[ko]<<setw(12)<<Asx[k
o]
```

```
//<<setw(12)<<Ajpr[ko]<<setw(12)<<Afr[ko]<<setw(12)<<Tfr<<end
l;

//outfile<<setw(12)<<Adx[ko]<<setw(12)<<Aax[ko]<<setw(12)<<As
x[ko]

//<<setw(12)<<Ajpr[ko]<<setw(12)<<Afr[ko]<<setw(12)<<Tfr<<end
l;

  }



   for(int k1=0;k1<10; )

   for(float q3=-1.57;q3<-.157;q3+=0.157)
   for(float xc2=0;xc2<=1000*OB;xc2++)

   {

   float L1=b/2;
   float xc3=xc2/1000;
   float r12=(L1+xa)*(L1+xa)+(ya*ya);
   float ry1=r12-(xc3-xa)*(xc3-xa);
   float y1=sqrt(ry1)+ya;

   float a1=cos(q3)*cos(q3);
   float b2=sin(q3)*sin(q3);
   float c1=(y1*y1)/(L1*L1*b2);
   float x12=(c1+1)*L1*L1*a1;
   float x1=sqrt(x12);
   float pr1=y1-q3*Hw/3.14;
   Apr1[k1]=pr1;

     if(x1<=xc3)

     {
     Aox1[k1]=x1;

//outfile<<setw(12)<<x1<<setw(12)<<y1<<setw(12)<<q3<<setw(12)
<<pr1<<endl;
     k1++;

//outfile<<setw(12)<<x1<<setw(12)<<y1<<setw(12)<<q3<<setw(12)
<<pr1<<endl;
     break;
```

```
    }
  }

  for(k1=0;k1<9;k1++)
  {
  Adx1[k1]=Aox1[k1+1]-Aox1[k1];//dx
  Ajx1[k1]=.5*(Aox1[k1+1]+Aox1[k1]);//(x1+x2)/2
  Aax1[k1]=asin((Ajx1[k1]-xa)/R);//alpha
  Asx1[k1]=Adx1[k1]/cos(Aax1[k1]);//ds
  Ajpr1[k1]=0.5*(Apr1[k1+1]+Apr1[k1]);//avr. pressure
  Afr1[k1]=Asx1[k1]*Ajpr1[k1];//forces
  Tfr1+=Afr1[k1];//total forces


//cout<<setw(12)<<Adx1[k1]<<setw(12)<<Aax1[k1]<<setw(12)<<Asx
1[k1]

//<<setw(12)<<Ajpr1[k1]<<setw(12)<<Afr1[k1]<<setw(12)<<Tfr1<<
endl;


//outfile<<setw(12)<<Adx1[k1]<<setw(12)<<Aax1[k1]<<setw(12)<<
Asx1[k1]

//<<setw(12)<<Ajpr1[k1]<<setw(12)<<Afr1[k1]<<setw(12)<<Tfr1<<
endl;
  }


  //"SOIL IN THE SLIDING LINE"
  for(float xs1=-(L-.5*i);xs1<=(OB-.5*i);xs1+=i)

  {
  float r2=(L+xa)*(L+xa)+(ya*ya);
  float ap=asin((xs1-xa)/R);
  float rys=r2-(xs1-xa)*(xs1-xa);
  float ys1=sqrt(rys)+ya;
  float wo=(ys1*i)*Gs;
  float T=(wo)*cos(ap);//+ch;//shear reistance

  sumo+=wo;
  sumsr+=T;

//cout<<setw(12)<<xs1<<setw(12)<<ys1<<setw(12)<<ap<<setw(12)<
<wo
  //<<setw(12)<<T<<endl;
```

```
}

//"DRIVING LOADS"
for(float xs3=-(L-.5*i);xs3<=(L-.5*i);xs3+=i)
{
float r2=(L+xa)*(L+xa)+(ya*ya);//radius
float rys3=r2-(xs3-xa)*(xs3-xa);
float ys3=sqrt(rys3)+ya;
float ap1=asin((xs3-xa)/R);//slice base inclination
float wo=P/b;//vertical load
float Mo=(Hw*Hw*Hw)/6;//moment due to u/s water pressure
float I=8*L*L*L/12;//inertia of foundation
float k1=(Mo-M)/I;
float w1=k1*xs3+wo;//distributed load
float w2=(w1*i);

float u=-(0.5*Hw/L)*xs3+0.5*Hw; //uplift
float u1=(u*i);
float wn=w2-u1;//net distributed load on ground surface
float n1=fabs((-k1*L+wo)-Hw);
float wno=w1-u;
float m1=(b*b)/((b+ys3)*(b+ys3));
float xabs=fabs(-L-xs3);
float m2=(xabs+0.5*ys3)/xabs;
float nx=(wno-n1)*m1*m2+n1*(b/(b+ys3));//net distributed
on slip circle
float T1=(nx*i)*cos(ap1);//normal forces
float Mw=(xs3-xa)*wn;//moment due to structure

sumMw+=Mw;
sumsr1+=T1;
sumu1+=u1;


//cout<<setw(12)<<xs3<<setw(12)<<w1<<setw(12)<<u<<setw(12)<<a
p1
//<<setw(12)<<T1<<endl;

//outfile<<setw(12)<<xs3<<setw(12)<<w1<<setw(12)<<u<<setw(12)
<<ap1
//<<setw(12)<<T1<<endl;

}

float Frs=Tfr+Tfr1;
float TN=sumsr+sumsr1;//normal force
```

```
float TS=(TN-Frs)*tan(po)+ch;//shear resistance
float Md=fabs(sumMw);//driving moment against xa
float Mts=TS*R;
float SFs=Mts/Md;
cout<<setw(12)<<ya<<setw(12)<<xa<<setw(12)<<SFs<<endl;
outfile<<setw(12)<<ya<<setw(12)<<xa<<setw(12)<<SFs<<endl;

}//forxaya

char cr;
cout<<"Try once more:(n/y)";
cin>>cr;
if(cr=='y')
goto start;

}
void starline()
{
for(int j=0;j<65;j++)
cout<<'=';
cout<<endl;

getch();
}
```

```
#include<iostream.h>
#include<conio.h>
#include<math.h>
#include<iomanip.h>
#include<fstream.h>
void starline();
void main(){clrscr();
ofstream outfile("SDOB.TXT");//checked 18/10-200
///Input
float a=10,bs=13.5,bu=27.5,bd=28,B=15,c=4,dd=6,du=6.5,D=4,Hw=3.5,
      tf=0.8,tr=2.5,td=1,d=4.5;
float P=265,Mec=0;

float f=4,Gs=0.9,Ct=10;
long p=15,Nc,Nq;
///
char ch;
starline();
cout<<"STABILITY OF THE BARRAGE"<<endl;
cout<<"by JUNAEDI 25072000"<<endl;
outfile<<"DESIGN OF FLOATING FLOOR AND RIBBED FOUNDATION OF THE
BARRAGE"<<endl;
outfile<<"by JUNAEDI 25072000"<<endl;
start:
starline();
//cout<<"\nDimension and Parameter"<<endl;
//cout<<"   -External Vertical Load (P in ton): ";
//cin>>P;
//cout<<"   -External Moment (M in t-m): ";
//cin>>Mec;
//cout<<"   -Floor thickness (tf in m): ";
//cin>>tf;
//cout<<"   -Depth of u/s cutoff (du in m): ";
//cin>>du;
//cout<<"   -Cutoff thickness (td in m): ";
//cin>>td;
//cout<<"   -Depth of d/s cutoff (dd in m): ";
//cin>>dd;
//cout<<"   -Depth of girder (D in m): ";
//cin>>D;
//cout<<"   -Girder/ Pier thickness (tr in m): ";
//cin>>tr;
//cout<<"   -Length top of pier (a in m): ";
//cin>>a;
```

```
//cout<<"    -Pier height (c in m): ";
//cin>>c;
//cout<<"    -Deference level between u/s and d/s (d in m): ";
//cin>>d;
//cout<<"    -Depth of water an u/s: ";
//cin>>Hw;
//cout<<"    -Width of bays (B in m): ";
//cin>>B;
//cout<<"    -Length of u/s floor (bu in m): ";
//cin>>bu;
//cout<<"    -Length of glacis (horizontal)  (bs in m): ";
//cin>>bs;
//cout<<"    -Length of d/s floor (bd in m): ";
//cin>>bd;
//cout<<"    -Unit weight of soil (Gs t/m3): ";
//cin>>Gs;
//cout<<"    -Skin friction of soil (f t/m2): ";
//cin>>f;
//cout<<"    -Internal friction  angle of soil (p deg): ";
//cin>>p;

outfile<<"\nDIMENSION AND PARAMETERS"<<endl;
outfile<<"    -External Vertical Load (P in ton): "<<P<<endl;
outfile<<"    -External Moment (Mec in t-m): "<<Mec<<endl;
outfile<<"    -Floor thickness (tf in m): "<<tf<<endl;
outfile<<"    -Depth of u/s cutoff (du in m): "<<du<<endl;
outfile<<"    -Cutoff thickness (td in m): "<<td<<endl;
outfile<<"    -Depth of d/s cutoff (dd in m): "<<dd<<endl;
outfile<<"    -Depth of girder (D in m): "<<D<<endl;
outfile<<"    -Girder/ Pier thickness (tr in m): "<<tr<<endl;
outfile<<"    -Length top of pier (a in m): "<<a<<endl;
outfile<<"    -Pier height (c in m): "<<c<<endl;
outfile<<"    -Deference level between u/s and d/s (d in m):
"<<d<<endl;
outfile<<"    -Depth of water an u/s: "<<Hw<<endl;
outfile<<"    -Width of bays (B in m): "<<B<<endl;
outfile<<"    -Length of u/s floor (bu in m): "<<bu<<endl;
outfile<<"    -Length of glacis (horizontal)  (bs in m):
"<<bs<<endl;
outfile<<"    -Length of d/s floor (bd in m): "<<bd<<endl;
outfile<<"    -Unit weight of soil (Gs t/m3): "<<Gs<<endl;
outfile<<"    -Skin friction of soil (f t/m2): "<<f<<endl;
outfile<<"    -Internal friction  angle of soil (p deg):
"<<p<<endl;
starline();
//cout<<"Do you want to change the input data? (y/n)";
//cin>>ch;
```

```
//if(ch=='y')
//goto start;
//starline();

//uplift
//upstream
cout<<"\nPRESSURE AT THE UPSTREAM CUTOFF"<<endl;
outfile<<"\nPRESSURE AT THE UPSTREAM CUTOFF"<<endl;
float h=Hw+d;
float b=bu+bs+bd-2*td;
float bt=b+2*td;
float afu=du/bt;
float LD=.5*(1+sqrt(1+(bt/du)*(bt/du)));
float PD=acos((LD-1)/LD)/3.1416;
float PD1=(1-PD);
cout<<"  -Alfa factor: "<<afu<<endl;
cout<<"  -Pressure at bottom of cutoff (%): "<<PD1*100<<endl;
outfile<<"  -Alfa factor: "<<afu<<endl;
outfile<<"  -Pressure at bottom of cutoff (%): "<<PD1*100<<endl;

float PDu=(1-PD)*h;
cout<<"  -Pressure at bottom of cutoff (m): "<<PDu<<endl;
outfile<<"  -Pressure at bottom of cutoff (m): "<<PDu<<endl;

float LE=.5*(1+sqrt(1+(bt/du)*(bt/du)));
float PE=acos((LE-2)/LE)/3.1416;
float PE1=1-PE;
cout<<"  -Pressure at the corner  (%): "<<PE1*100<<endl;
outfile<<"  -Pressure at the corner  (%): "<<PE1*100<<endl;

float Cd=(PD1-PE1)*100*tf/du;
cout<<"  -Correction depth (%): "<<Cd<<endl;
outfile<<"  -Correction depth (%): "<<Cd<<endl;

float k=du-tf;
float K=d-tf+dd;
float Ci=19*sqrt(K/b)*(k+K)/bt;
float PEc=PE1*100+Cd+Ci;
float PEu=PEc*h*.01;

cout<<"  -Correction for interference of d/s cutoff (%):
"<<Ci<<endl;
cout<<"  -Corrected pressure at the corner (%): "<<PEc<<endl;
cout<<"  -Pressure at the corner  (m): "<<PEu<<endl;
outfile<<"  -Correction for interference of d/s cutoff (%):
"<<Ci<<endl;
outfile<<"  -Corrected pressure at the corner (%): "<<PEc<<endl;
```

```
outfile<<"  -Pressure at the corner  (m): "<<PEu<<endl;

//down stream
cout<<"\nPRESSURE AT THE DOWNSTREAM CUTOFF"<<endl;
outfile<<"\nPRESSURE AT THE DOWNSTREAM CUTOFF"<<endl;

float afd=dd/bt;
float Ld=.5*(1+sqrt(1+(bt/dd)*(bt/dd)));
float PD2=acos((Ld-1)/Ld)/3.1416;
cout<<"  -Alfa factor: "<<afd<<endl;
cout<<"  -Pressure at bottom of cutoff (%): "<<PD2*100<<endl;
outfile<<"  -Alfa factor: "<<afd<<endl;
outfile<<"  -Pressure at bottom of cutoff (%): "<<PD2*100<<endl;

float PDd=PD2*h;
cout<<"  -Pressure at bottom of cutoff (m): "<<PDd<<endl;
outfile<<"  -Pressure at bottom of cutoff (m): "<<PDd<<endl;

float LE2=.5*(1+sqrt(1+(bt/dd)*(bt/dd)));
float PE2=acos((LE2-2)/LE2)/3.1416;
float PEd=PE2*h;
cout<<"  -Pressure at the corner (%): "<<PE2*100<<endl;
outfile<<"  -Pressure at the corner (%): "<<PE2*100<<endl;

float Cd1=(PD2-PE2)*100*tf/dd;
cout<<"  -Correction depth (%): "<<Cd1<<endl;
outfile<<"  -Correction depth (%): "<<Cd1<<endl;

float k1=dd-tf;
float K1=du-d-tf;
  if(K1<0)
    {
    cout<<"  -No interference of u/s cutoff."<<endl;
    outfile<<"  -No interference of u/s cutoff."<<endl;
    K1=0;
    }
float Ci1=19*sqrt(K1/b)*(k1+K1)/bt;
cout<<"  -Correction for interference of u/s cutoff: "<<-
Ci1<<endl;
outfile<<"  -Correction for interference of u/s cutoff: "<<-
Ci1<<endl;

float PEc1=PE2*100+Cd1-Ci1;
float PEd1=PEc1*h*.01;
cout<<"  -Corrected pressure at the corner: "<<PEc1<<endl;
cout<<"  -Pressure at the corner (m): "<<PEd1<<endl;
outfile<<"  -Corrected pressure at the corner: "<<PEc1<<endl;
```

```
outfile<<"  -Pressure at the corner (m): "<<PEd1<<endl;

//starline();
//starline();
//uplift force beneath the floor
float UL1=(b)*PEd*B;//rectangel
float UL2=(b)*(PEu-PEd)*.5*B;//triangle
//uplift force beneath cutoff
float UL3=PDu*td*B;
float UL4=PDd*td*B;
float UL=UL1+UL2+UL3+UL4;

//Vertical Forces
//weight of structure
float Wr=(bu+bd+bs)*tr*D*2.4;//girder
float W1=(.33333*a+c+c)*.5*a*tr*2.4;
float W2=.5*(.33333*a+c)*(bs-a)*tr*2.4;
float Wp=W1+W2;//weight of peir+girder
float Wf=(bu+bs+bd)*tf*2.4*B;//floor dry
float Wfs=(bu+bs+bd)*tf*1.4*B;//floor submerged
//float Wcd=(dd)*td*1.4;//ds cutoff
//float Wcu=(du)*td*1.4;//us cutoff
float Ws=Wr+Wf+Wp;//totaldry
float Wss=Wr+Wfs+Wp;//totalsub.

//weight of water
float Ww=Hw*bu*B;

//Horisontal Forces
float p1=p*3.14/180;
float ka=tan(0.785-0.5*p1)*tan(0.785-0.5*p1);
float kp=1/ka;
float H=.5*(Hw*Hw)*B;//water
float Sp=.125*Gs*(kp-ka)*(du*du+dd*dd)*B;//soil lateral
float fsh=Sp/H;

//foundation analysis
//overtuning moment against end of d/s
//float Mc=(du*td*2.4)*(b+td);//cutoff
float Mf=Wf*0.5*(b+td);//floor dry
float Mfs=Wfs*0.5*(b+td);//floor submerged
float Mr=Wr*0.5*(b+td);//girder
float M1=W1*(bd-.5*a+bs-.5*td);
float M2=W2*(bd+.66667*(bs-a)-.5*td);
float Mp=M1+M2;//pier
float Mst=Mf+Mr+Mp;//m structure
float Msts=Mfs+Mr+Mp;//m structure
```

```
float Mbg=(P)*(bd+(bs-.5*a)-.5*td);//bridge+gate
float Mwh=H*Hw/3;//water
float Mwv=Ww*(.5*bu+bs+bd-.5*td);//water
float MUL=UL1*(.5*td+.5*b)+UL2*(.5*td+2*b/3)+UL3*(b+td);//uplift
float Mfr=(f*2*du)*(td+b);//friction
float Msp=0.125*Gs*dd*dd*(kp-ka)*d;;//soil lat. prs.
float dM=Msts+Mec+Mbg+0.5*H*d-Mwh+Mwv-MUL;
float Ru=dM/(b+td);//reaction of u/s found. at ponding

float dMn=Mst+Mec+Mbg;
float Run=dMn/(b+td);//reaction of u/s found. no water

float fsm=(Msts+Mec+Mwv+Mfr+Mbg+Msp)/(MUL+Mwh);//at ponding

//Overtuning moment against end of u/s
//float Mc1=(dd*td*2.4)*(b+td);//cutoff
float Mf1=Wf*.5*(b+td);//floor dry
float Mfs1=Wfs*.5*(b+td);//floor sub
float Mr1=Wr*.5*(b+td);//girder
float M11=W1*(bu+.5*a-.5*td);
float M21=W2*(bu+a+.33333*(bs-a)-.5*td);
float Mp1=M11+M21;//pier
float Mst1=Mf1+Mr1+Mp1;//m structure dry
float Msts1=Mfs1+Mr1+Mp1;//m structure sub
float Mbg1=(P)*(bu+(a*.5)-.5*td);//bridge+gate
float Mwh1=H*Hw/3;//water
float Mwv1=Ww*(.5*bu-.5*td);//water
float MUL1=UL1*(.5*td+.5*b)+UL2*(.5*td+b/3)+UL4*(b+td);//uplift
float Mfr1=(f*2*dd)*(td+b);//friction
float Msp1=.125*Gs*du*du*(kp-ka)*d;//soil lat. prs.
float dM1=Msts1+Mec+Mbg1+Mwh1+Mwv1+0.5*H*d-MUL1;
float Rd=dM1/(b+td);//reaction of d/s found. at ponding

float dMn1=Mst1+Mec+Mbg1;
float Rdn=dMn1/(b+td);//reaction of ds found. no water

float fsm1=(Mst1+Mec+Mwv1+Mfr1+Mbg1+Msp1)/(MUL1+Mwh1);
float Fr=f*(2*dd+2*du);//skin friction
float fsv=(P+Ws+Ww+Fr)/UL;


//girder
float px=bd*(PEu-PEd1)/b+PEd1;//uplift at bd down stream
float Mg1=Rd*bd;//m foundation reaction
float Mg2=PEd1*.5*bd*bd*B;//m uplift rect.
float Mg3=(px-PEd1)*bd*bd*B/6;//m uplift trian.
float Mg4=bd*tr*D*2.4*.5*bd;//m girder
```

```
float Mg5=tf*1.4*B*.5*bd*bd;//m floor
float Mg=Mg1+Mg2+Mg3-Mg4-Mg5;//m tot at girder F

//floor
float Mx=(px-1.4*tf)*B*B/10;//moment tm
float tb=100*tf;
float I=100*pow(tb,4)/12;
float y=(10*(px-
1.4*tf)*pow((100*B),4))/(384*210000*I);//deflection
//cutoff
float Mchu=0.1*Gs*du*du*du*(kp-ka);
float qq=(Run)/B;
float Mcvu=0.1*qq*B*B;


//Bearing capacity

switch (p)
  {
   case 0:

     Nc=5.7;
     Nq=1.0;
     break;
   case 5:

     Nc=7.8;
     Nq=2.7;
     break;
   case 10:

     Nc=10.5;
     Nq=2.7;
     break;
   case 15:

     Nc=15;
     Nq=4.8;
     break;
   case 20:

     Nc=24;
     Nq=8.1;
     break;
   case 25:
```

```
      Nc=40;
      Nq=15;
      break;
    case 30:

      Nc=70;
      Nq=30;
      break;
    case 35:

      Nc=140;
      Nq=65;
      break;
    case 40:

      Nc=300;
      Nq=150;
      break;
    }

    float qf=(td*(Ct*Nc+(dd*Gs*Nq)*(0.4+0.1))+2*f*dd)*B;
    float qf1=(td*(Ct*Nc+(du*Gs*Nq)*(0.4+0.1))+2*f*du)*B;


starline();
cout<<"\nDESIGN OF FOUNDATION"<<endl;
cout<<"\nThe Loads"<<endl;
cout<<"  -External Vertical Load (ton): "<<P<<endl;
cout<<"  -External Moment (ton-m): "<<Mec<<endl;
cout<<"  -Weight of water at u/s side (ton): "<<Ww<<endl;
cout<<"  -Weight of structure at ponding condition (ton):
"<<Wss<<endl;
cout<<"  -Weight of structure at no water condition (ton):
"<<Ws<<endl;
cout<<"  -Uplift force (ton): "<<UL<<endl;
//starline();
cout<<"\nUpstream foundation"<<endl;
cout<<"  -Reaction of u/s foundation at ponding condition (ton):
"<<Ru<<endl;
cout<<"  -Reaction of u/s foundation at no water condition (ton):
"<<Run<<endl;
cout<<"  -Bearing capacity at u/s foundation (ton): "<<qf1<<endl;
cout<<"\nDownstream foundation"<<endl;
cout<<"  -Reaction of d/s foundation at ponding condition (ton):
"<<Rd<<endl;
cout<<"  -Reaction of d/s foundation at no water condition (ton):
"<<Rdn<<endl;
```

```
cout<<"  -Bearing capacity at d/s foundation (ton): "<<qf<<endl;

cout<<"\nSTABILITY ANALYSIS"<<endl;
cout<<"  -Horizontal movement safety factor: "<<fsh<<endl;
cout<<"  -Vertical movement safety factor: "<<fsv<<endl;
cout<<"  -Overturning moment safety factor against u/s:
"<<fsm1<<endl;
cout<<"  -Overturning moment  safety factor against d/s:
"<<fsm<<endl;
starline();
cout<<"\nSTRUCTURAL ANALYSIS"<<endl;
cout<<"   -Moment at Girder (t-m): "<<Mg<<endl;
cout<<"   -Moment at Floor (t-m): "<<Mx<<endl;
cout<<"   -Deflection at Floor (cm): "<<y<<endl;
cout<<"   -Horizontal moment at Foundation/cutoff (t-m):
"<<Mchu<<endl;
cout<<"   -Vertical moment at Foundation/cutoff (t-m):
"<<Mcvu<<endl;

outfile<<"\nThe Loads"<<endl;
outfile<<"  -External Vertical Load (ton): "<<P<<endl;
outfile<<"  -External Moment (ton-m): "<<Mec<<endl;
outfile<<"  -Weight of water at u/s side (ton): "<<Ww<<endl;
outfile<<"  -Weight of structure at ponding condition (ton):
"<<Wss<<endl;
outfile<<"  -Weight of structure at no water condition (ton):
"<<Ws<<endl;

outfile<<"  -Uplift force (ton): "<<UL<<endl;
outfile<<"\nUpstream foundation"<<endl;
outfile<<"  -Reaction  at ponding condition (ton): "<<Ru<<endl;
outfile<<"  -Reaction at no water condition (ton): "<<Run<<endl;
outfile<<"  -Bearing capacity at u/s foundation (ton):
"<<qf1<<endl;
outfile<<"\nDownstream foundation"<<endl;
outfile<<"  -Reaction at ponding condition (ton): "<<Rd<<endl;
outfile<<"  -Reaction  at no water condition (ton): "<<Rdn<<endl;
outfile<<"  -Bearing capacity at d/s foundation (ton):
"<<qf<<endl;

outfile<<"\nSTABILITY ANALYSIS"<<endl;
outfile<<"  -Horizontal movement safety factor: "<<fsh<<endl;
outfile<<"  -Vertical movement safety factor: "<<fsv<<endl;
outfile<<"  -Overturning moment safety factor against u/s:
"<<fsm1<<endl;
outfile<<"  -Overturning moment safety factor against d/s:
"<<fsm<<endl;
```

```
outfile<<"\nSTRUCTURAL ANALYSIS"<<endl;
outfile<<"   -Moment at Girder (t-m): "<<Mg<<endl;
outfile<<"   -Moment at Floor (t-m): "<<Mx<<endl;
outfile<<"   -Deflection at Floor (cm): "<<y<<endl;
outfile<<"   -Horizontal moment at Foundation/cutoff (t-m):
"<<Mchu<<endl;
outfile<<"   -Vertical moment at Foundation/cutoff (t-m):
"<<Mcvu<<endl;

cout<<"\nDESIGN OF REINFORCEMENT"<<endl;

for (int i=2;i<=100;i++)
  {
   float As=i*0.25*3.14*2.5*2.5;//steel area tension=compression
   float ac=As*3700/(0.55*300*1*100);//concrete block diagram
   float Mc=.55*300*1*100*ac*(100*tf-10-
(ac*0.5))*0.00001;//concrete
   float Ms=3700*As*(100*tf-20)*0.00001;//steel
   float Mu=Mc+Ms;//ultimate floor
   int Amax=(.236*300*1*100*(100*tf-10)/3700)/4.9;
       if(Mu>=1.5*Mx)
       {

       cout<<"\nFloor:  no. of bars per m: "<<i<<endl;
       cout<<"          Moment ultimate per m: "<<Mu<<endl;
       cout<<"          Max. no. of bars per m: "<<Amax<<endl;

       outfile<<"\nFloor:  no. of bars per m: "<<i<<endl;
       outfile<<"          Moment ultimate per m: "<<Mu<<endl;
       outfile<<"          Max. no. of bars per m: "<<Amax<<endl;

       break;
       }
  }

  for (int i1=2;i1<=500;i1++)
  {
   float As=i1*0.25*3.14*2.5*2.5;//steel area tension=compression
   float ac1=As*3700/(0.55*300*tr*100);//concrete block diagram
   float Mc1=.55*300*tr*100*ac1*(100*D-10-
   (ac1*0.5))*0.00001;//concrete
   float Ms1=3700*As*(100*D-20)*0.00001;//steel
   float Mu1=Mc1+Ms1;//ultimate  girder
   int Amax=(.236*300*tr*100*(100*D-10)/3700)/4.9;
     if(Mu1>=1.5*Mg)
       {
```

```
    cout<<"\nGirder:  no. of bars: "<<i1<<endl;
    cout<<"         Moment ultimate: "<<Mu1<<endl;
    cout<<"         Max. no. of bars: "<<Amax<<endl;

    outfile<<"\nGirder:  no. of bars: "<<i1<<endl;
    outfile<<"         Moment ultimate: "<<Mu1<<endl;
    outfile<<"         Max. no. of bars: "<<Amax<<endl;

    break;
    }
}

for (int i2=2;i2<=500;i2++)
{
  float As=i2*0.25*3.14*2.5*2.5;//steel area tension=compression
  float ac2=As*3700/(0.55*300*1*100);//concrete block diagram
  float Mc2=.55*300*1*100*ac2*(100*td-10-
  (ac2*0.5))*0.00001;//concrete
  float Ms2=3700*As*(100*td-20)*0.00001;//steel
  float Mu2=Mc2+Ms2;//ultimate  cutoff
  int Amax=(.236*300*1*100*(100*td-10)/3700)/4.9;


   if(Mu2>=1.5*Mchu)
    {
    cout<<"Cutoff"<<endl;
    cout<<"\nReinforcement due to Horizontal Load"<<endl;
    cout<<"   -no. of bars per m: "<<i2<<endl;
    cout<<"   -Moment ultimate per m: "<<Mu2<<endl;
    cout<<"   -Max. no. of bars: "<<Amax<<endl;
    outfile<<"Cutoff"<<endl;
    outfile<<"\nReinforcement due to Horizontal Load"<<endl;
    outfile<<"  -no. of bars per m: "<<i2<<endl;
    outfile<<"  -Moment ultimate per m: "<<Mu2<<endl;
    outfile<<"  -Max. no. of bars per m: "<<Amax<<endl;

    break;
    }
}

for (int i3=2;i3<=500;i3++)
{
  float As=i3*0.25*3.14*2.5*2.5;//steel area tension=compression
  float ac21=As*3700/(0.55*300*td*100);//concrete block diagram
  float Mc21=.55*300*1*100*ac21*(100*du-10-
  (ac21*0.5))*0.00001;//concrete
  float Ms21=3700*As*(100*du-20)*0.00001;//steel
```

```
    float Mu21=Mc21+Ms21;//ultimate  cutoff
    int Amax1=(.236*300*td*100*(100*du-10)/3700)/4.9;
       if(Mu21>=1.5*Mcvu)
        {

        cout<<"\nReinforcement due to Vertical Load"<<endl;
        cout<<"   -no. of bars : "<<i3<<endl;
        cout<<"   -Moment ultimate per m: "<<Mu21<<endl;
        cout<<"   -Max. no. of bars: "<<Amax1<<endl;

        outfile<<"\nReinforcement due to Vertical Load"<<endl;
        outfile<<"  -no. of bars : "<<i3<<endl;
        outfile<<"  -Moment ultimate per m: "<<Mu21<<endl;
        outfile<<"  -Max. no. of bars per m: "<<Amax1<<endl;

        break;
        }
  }
starline();
cout<<"Try once more? (y/n)";
cin>>ch;
if(ch=='y')
goto start;
}
void starline()
{
for(int j=0;j<75;j++)
cout<<'*';
cout<<endl;
getch();
}
```

LIST OF SYMBOLS

a	= width of top of pier, depth of concrete stress block.
b	= clear distance between upstream and downstream cutoff, width of rectangular beams.
c	= height of pier measured from girder face.
d	= difference level between upstream and down stream, depth of beams.
e	= eccentricity.
fs	= skin friction.
h	= height of structure (elementary profile of gravity dam).
i	= increment, width of slices.
j,k	= increment of 'for loops'.
p	= angle of internal friction (ϕ) in computer program.
q	= equipotential constants in computer program.
u	= velocity in x-direction.
v	= velocity in y-direction.
w	= velocity in z-direction.
x	= horizontal distance measured from y-axis.
y	= vertical distance measured from x-axis.
B	= width of bay of barrage.
C	= correction for mutual interference of sheet piles.
D	= girder depth.
H	= horizontal water thrust.

L = distance from the middle of the structure to outer side.

M = external moment.

P = external vertical load.

R = radius of slip circle.

T = shear force at the base of slice.

U = uplift force below the base of structure.

W = weight of water at upstream side.

bd = length of downstream floor.

bs = horizontal length of glacis.

bu = length of upstream floor.

dd = depth of downstream cutoff.

du = depth of upstream cutoff.

Hq = horizontal resistance of strip foundation.

qf = bearing capacity of soil base.

td = thickness of cutoff / strip foundation.

tr = thickness of girder or pier.

tf = thickness of floating floor.

Ct = cohesion of soil.

Df = depth of foundation.

Fr = uplift forces along slip circle line.

Hw = height of water storage.

Ka = coefficient of active pressure of soil.

Kp = coefficient of passive pressure of soil.

Mu = moment ultimate of reinforced concrete members.

Nq, Nc, Nγ = bearing capacity factors.

Ru = reaction of upstream foundation.

Rd = reaction of downstream foundation.

SF = safety factor.

α = angle of slice base inclination, adhesion factor.

ϕ' = constants of equipotential lines.

ψ' = constants of stream / flow lines.

θ = central angle of slip circle.

σ = stresses on ground surface.

γw = unit weight of water.

γc = unit weight of concrete.

γsat = unit weight of saturated soil.

γ' = unit weight of submerged soil.

γ = unit weight of dry soil.

σcu = compressive strength of concrete.

σau = tensile strength of steel.