live loads, vertical components of lateral forces, earth fill on top, etc.

- Net foundation pressure = increase in loading of the underlying soil under a foundation, if compared with the original loading condition, i.e. net foundation pressure = total foundation pressure minus the weight of the soil that used to be on top.
- Q_{ult} (ultimate bearing capacity) = soil pressure at which failure of the soil will occur, if exceeded.
- q_{ult} (safe ultimate bearing capacity) = soil pressure which safely can be applied.

$$q_{ult} = \frac{Q_{ult}}{S}$$, in which S = safety-factor.

- q_a (allowable bearing capacity) = allowable increase in net foundation pressure for an estimated maximum settlement of 2.5 cm.

To check whether a foundation pressure meets the criteria, the total foundation pressure should be compared with q_{ult} (safe ultimate bearing capacity) and/or q_a (allowable bearing capacity).

Chapter 2. Lateral Earthpressure

2.1 Rankine and Coulomb Methods.

To determine lateral earthpressure against a structure, straight sliding planes are considered, along which failure of the soil will occur. Equilibrium is then analysed either analytically or graphically, considering all driving and resisting forces acting on the structure and the plane of rupture.

Most methods commonly used to determine earth pressure can be grouped under either Rankine's or Coulomb's theory. Each theory is based on a separate set of conditions, which the foundation has to satisfy and only for the rare case when the conditions coincide can they be applied interchangeably.

The basic characteristics of the two theories are :

Coulomb	Rankine
- rupture surface is a plane surface	- do
- backfill surface is planar	- do
- friction forces are distributed uniformly along the plane ruptive surface, friction coefficient $f = \tan \varphi$	- do
- failure wedge is a rigid body	- do
- there is wall friction, the wedge moves along the back of the wall developing friction forces	- no wall friction
- resultant pressure E_a or E_p against the wall is found directly	- yields the resultant pressure against a vertical plane rising from the heel of the wall. To find the resultant pressure against the back of the wall, the weight of the soil between the vertical plane and the wall must be combined with Rankine's resultant pressure against the vertical plane.

coulomb

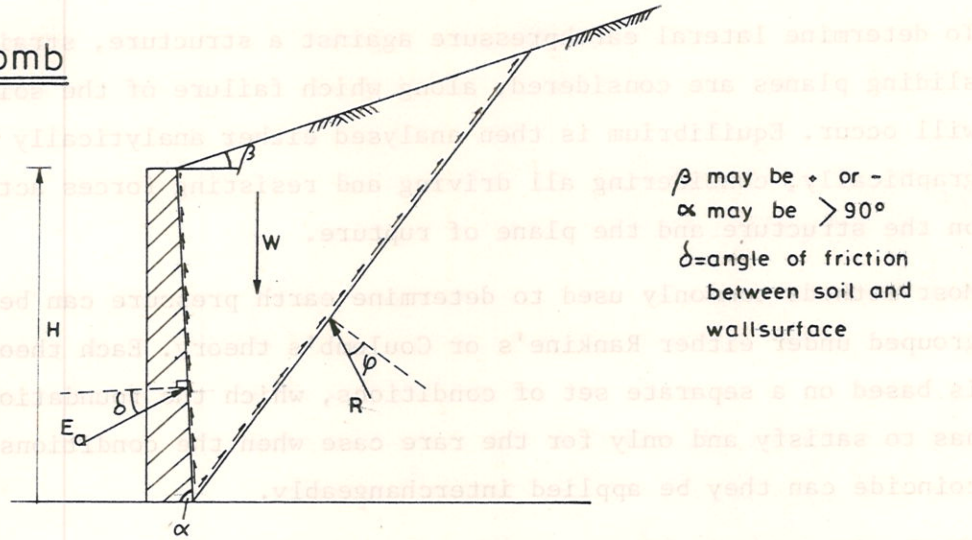


Figure VI - 2.1

rankine

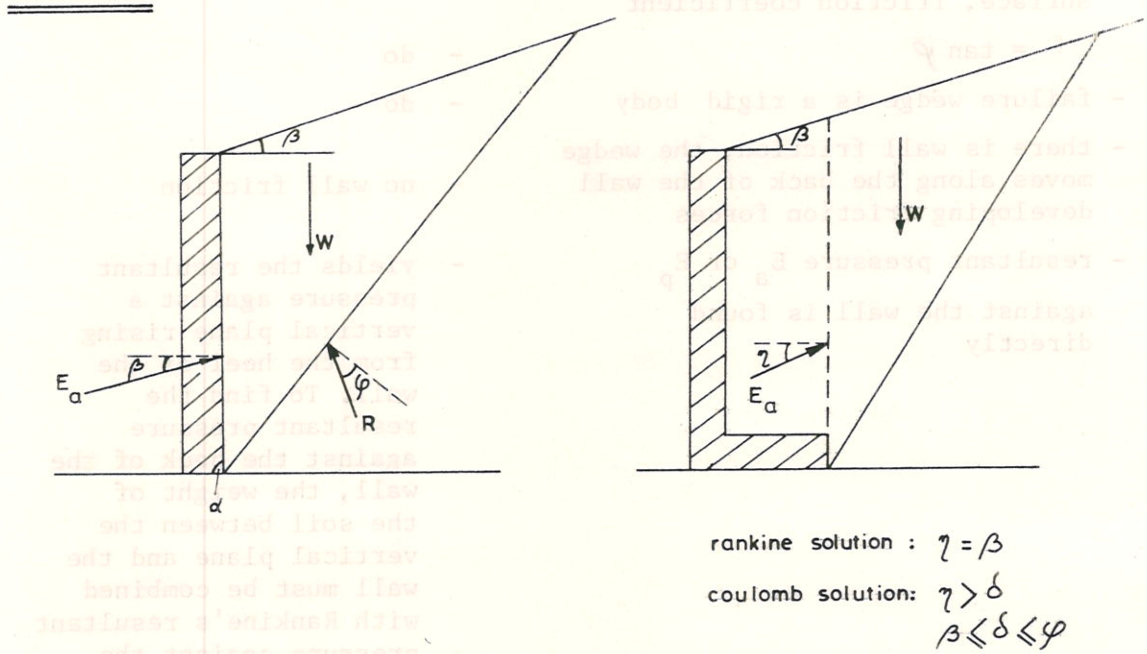


Figure VI - 2.2

Figure VI - 2.1 and VI - 2.2 illustrate the conditions that must exist before either theory may be applied to a particular situation. These conditions may be summarized as follows :

1. If the back of a wall is a plane surface, or a surface which can be assumed to be plane without introducing significant errors, either method may be applied depending upon which of the following conditions prevails :

   a) If the outer plane of rupture cannot form because of interference with the structure, Coulomb's conditions prevail.

   b) If the outer plane or rupture can form through the soil, Coulomb's conditions prevail if the obliquity of the pressure is equal to or greater than δ , and Rankine's conditions prevail if the obliquity is equal to or less than δ .

2. If the back of the wall cannot be assumed to be a plane surface, Coulomb's conditions cannot prevail, but Rankine's conditions will prevail if the outer plane of rupture can form without being obstructed by the wall.

In general, <u>solid gravity retaining walls</u> will satisfy Coulomb's conditions, especially if δ is assumed to be equal to or less than $2/3\ \varphi$.

<u>Cantilever and counterfort</u> walls do not satisfy the assumptions of either theory. However, these walls with the usual proportions can be assumed to satisfy Rankine's conditions without significant error.

The obliquity, φ , of the pressure resultant cannot be determined beforehand. The problem must be set up and solved, using either theory, and the obliquity determined. (It should be remembered that the obliquity considers only the pressure excerted by the soil on the wall and does not include the weight of the wall).The general equation for calculating the lateral earthpressure (see paragraph 1.5) is :

$E = k . \frac{1}{2} \gamma . h^2$ in which for active earthpressure the subs a and for passive earthpressure the subs b have to be added to E and k.

For Coulomb's theory:

$$k_a = \frac{\sin^2(\alpha + \varphi)}{\sin^2\alpha . \sin(\alpha - \delta) \cdot \left[1 + \sqrt{\frac{\sin(\varphi+\delta) . \sin(\varphi-\beta)}{\sin. (\alpha-\delta) . \sin(\alpha+\beta)}}\right]^2}$$

$$k_p = \frac{\sin^2(\alpha - \varphi)}{\sin^2\alpha . \sin(\alpha + \delta) \cdot \left[1 - \sqrt{\frac{\sin(\varphi+\delta) . \sin(\varphi+\beta)}{\sin(\alpha+\delta) . \sin(\alpha+\beta)}}\right]^2}$$

For Rankine's theory:

$$k_a = \cos\beta . \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi}} = \cos\beta \cdot \varepsilon$$

$$k_p = \frac{\cos\beta}{\varepsilon}$$

In the following an example will illustrate a comparison of the two theories, (see Figure VI - 2.3)

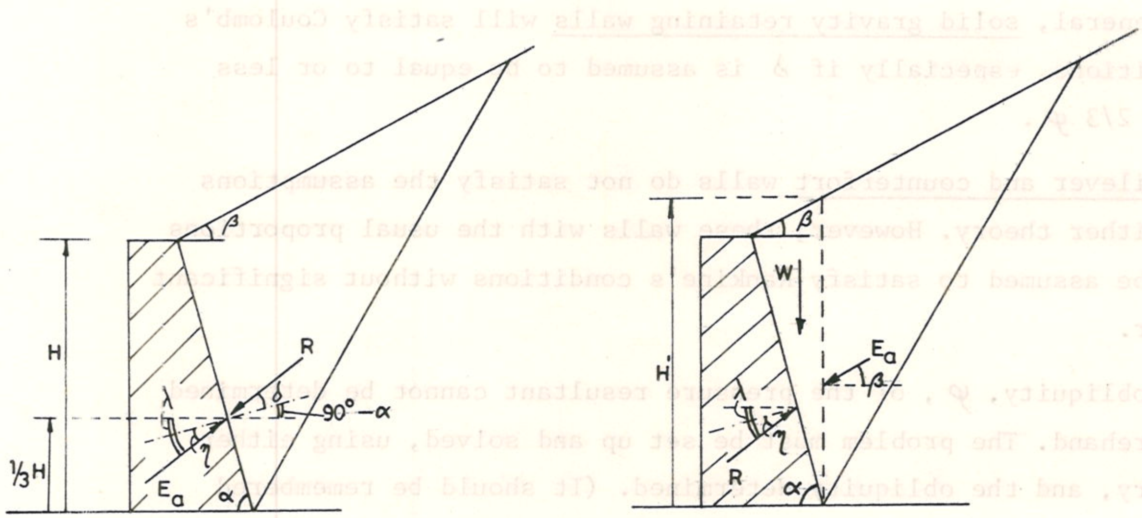


Figure VI - 2.3

assume :

$$\beta = 18.4^{\circ} \text{ (1 into 3)} \qquad \gamma = 19 \text{ KN/m}^3$$
$$\alpha = 65^{\circ} \qquad H = 5 \text{ m}$$
$$\varphi = 30^{\circ} \qquad E_a = \tfrac{1}{2} . \gamma . h^2 . k_a$$
$$\delta = 20^{\circ}$$

According to Coulomb's theory, the active earthpressure is :

$$E_a = k_a . \tfrac{1}{2} . \gamma\, h^2 = 0.793 \times \tfrac{1}{2} \times 19 \times (5)^2 = 188 \text{ KN/m}^1$$

The angle which the resultant E makes with the horizon is

determined : $\lambda = \delta + (90^{\circ} - \alpha) = 45^{\circ}$

and the angle η is determined by: $\eta = \lambda + \alpha - 90^{\circ} = 20^{\circ}$

To satisfy the Coulomb theory conditions, $\eta \geqslant \delta$, and since $\eta = \delta = 20^{\circ}$, the conditions are met.

According to Rankine's theory, the active earthpressure is :

$$E_a = k_a . \tfrac{1}{2} \gamma . h^2 = 0.398 \times \tfrac{1}{2} \times 19 \times (5.45)^2 = 122 \text{ KN/m}^1$$

horizontal component : $Ea_h = E_a \times \cos\beta = 107 \text{ KN/m}^1$

vertical component : $Ea_v = E_a \times \sin\beta = 35 \text{ KN/m}^1$

The weight of the soil wedge is : $W = 19 \times [\frac{(1.34 \times 5)}{2} + \frac{(1.34 \times 0.45)}{2}]$

$$W = 69 \text{ KN/m}^1$$

On the surface of the wall are acting:

horizontal : $R_h = Ea_h = 107 \text{ KN/m}^1$
vertical : $R_v = Ea_v + W = 104 \text{ KN/m}^1$
$R = 149 \text{ KN/m}^1$

and the angle with the horizontal is then :

$$\tan\lambda = \frac{Rv}{Rh} = \frac{104}{107} \longrightarrow \lambda = 44.2^{\circ}$$

$$\eta = 44.2 - 25^{\circ} = 19.2^{\circ}$$

This angle η is less than the friction angle $\delta = 20^{\circ}$ between the soil and surface of the structure, hence Rankine cannot be applied in this case.

In actual practice it will usually not be necessary to make preliminary computations of the angle of obliquity to determine which theory will prevail. Under most circumstances Coulomb's theory will almost always prevail for solid gravity walls, and Rankine's theory can be used for cantilever and counterfort walls. The Rankine solution is often used because the equations are simple, especially for cohesionless soils and horizontal backfill and because uncertainty of evaluating the wall friction δ. The Rankine solution gives slightly larges values of E_a.

2.2 Equivalent Fluid Pressure Method

A third method has been developed by Peck and Terzaghi, based on Rankine's theory, that is usually applied to retaining walls with a horizontal backfill. Rankine's formula for $\beta = 0$ is:

$$k_a = \frac{1 - \sin \varphi}{1 + \sin \varphi} \quad \text{and} \quad E_a = \tfrac{1}{2} \gamma \cdot k_a . H^2$$

The equivalent fluid method substitutes K for $\gamma . k_a$, where K is called the unit weight of an equivalent fluid that produces the horizontal force E_a. Values for K have been determined semi-empirically in which allowances for cohesion are made and may be applied to walls less than 6 m high.

The method consists of assigning the available backfill material to one of five categories or types of material. Depending on the type of wall and the physical conditions likely to be encountered, horizontal and vertical coefficients K_h and K_v are selected from an appropriate pressure chart. Inserting the coefficients in the general formula : $P = \tfrac{1}{2} KH^2$, the horizontal and vertical pressure components can be computed.

Figures VI - 2.4 to VI-2.5 indicate the different factors K for various cases.

The method is applicable to gravity, counterfort and cantilever walls. As with Ranine's method the computed pressures act on a vertical plane rising from the wall heel. <u>To find the resultant</u>

pressure on the wall the weight of the soil wedge between the wall and the plane must be added to the computed pressure. The five categories, or types of backfill to be used with this method are :

1. Coarse-grained soil without admixture of fine soil particles; very permeable. (clean sand or gravel).
2. Coarse-grained soil of low permeability due to admixture of particles of silt size.
3. Residual soil with stones; fine silty sand; granular materials with conspicuous clay content.
4. Very soft or soft clay; organic silts; silty clays.
5. Medium or stiff clay deposited in chunk and protected in such a way that a negligible amount of water enters the spaces between the chunks during floods or heavy rains. If this condition cannot be satisfied, the clay should not be used as backfill material. With increasing stiffness of the clay, danger to the wall increases rapidly due to infiltration of water.

The following restrictions are to made :

a. Earth pressure computed by the use of pressure charts presented in Figures VI - 2.4 to VI - 2.5 include the effects of seepage pressures and various time-conditioned changes in the backfill material. The method does not take into account hydrostatic pressure due to submergence.
b. The procedure assumes the foundation to be relatively unyielding. If the wall is constructed on a very compressible material, such as soft clay, allowances must be made for greater pressures resulting from settlement. In this case backfill pressures computed for material types 1, 2, 3 and 5 should be increased by 50%. (Backfill type 4 has already taken this factor into consideration).
c. The point of application of the pressure resultant for backfill types, 1, 2, 3 and 4 is at a point 1/3 H from the base. If the backfill is type 5 (clay chunks) the pressure resultant is computed using a height of (H-4) instead of H, and is assumed to act at a point 1/3 (H-4) above the base.

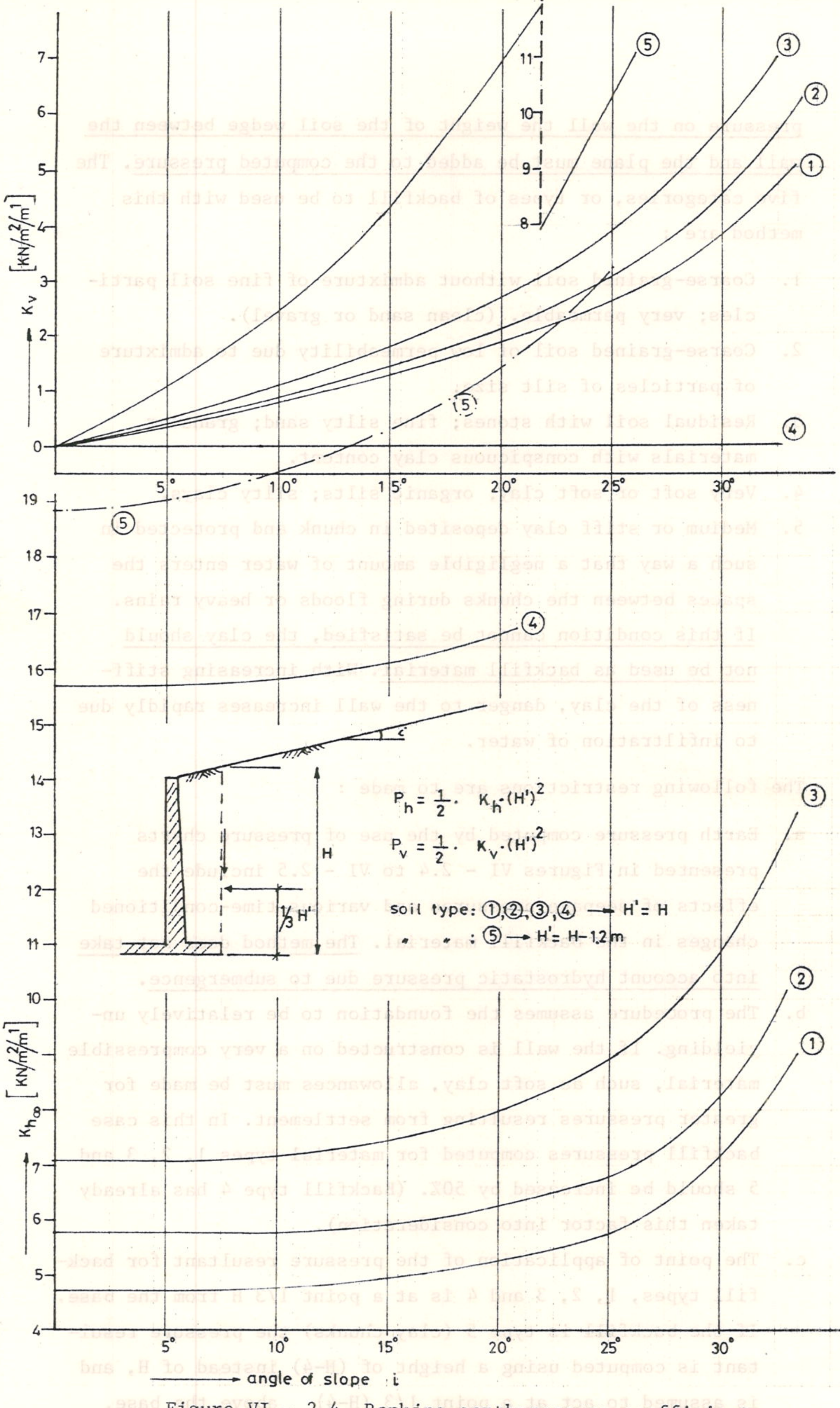


Figure VI - 2.4 Rankine earthpressure coefficients as developed by Terzaghi and Peck for plain sloping backfills.

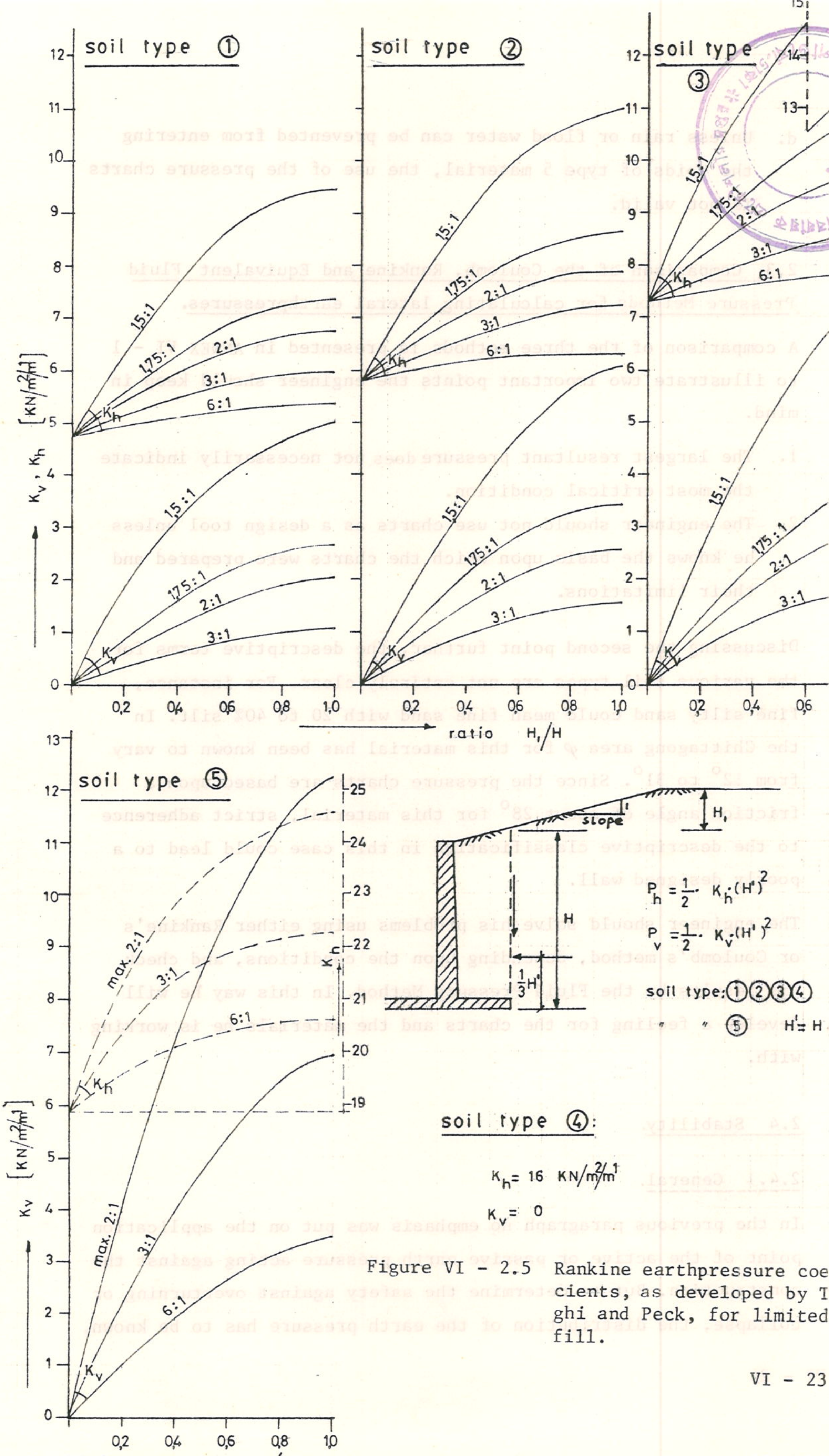


Figure VI - 2.5 Rankine earthpressure coe cients, as developed by T ghi and Peck, for limited fill.

d. Unless rain or flood water can be prevented from entering the voids of type 5 material, the use of the pressure charts is not valid.

2.3 Comparison of the Coulomb, Rankine and Equivalent Fluid Pressure Methods for calculating lateral earthpressures.

A comparison of the three methods is presented in ANNEX VI - 1 to illustrate two important points the engineer should keep in mind.

1. The largest resultant pressure does not necessarily indicate the most critical condition.
2. The engineer should not use charts as a design tool unless he knows the basis upon which the charts were prepared and their limitations.

Discussing the second point further, the descriptive terms for the various soil types are not entirely clear. For instance, fine silty sand could mean fine sand with 20 to 40% silt. In the Chittagong area φ for this material has been known to vary from 12^{o} to 31^{o}. Since the pressure charts are based upon a friction angle of about 28^{o} for this material, strict adherence to the descriptive classification in this case could lead to a poorly designed wall.

The engineer should solve his problems using either Rankine's or Coulomb's method, depending upon the conditions, and check his results by the Fluid Pressure Method. In this way he will develop a feeling for the charts and the materials he is working with.

2.4 Stability.

2.4.1 General.

In the previous paragraph no emphasis was put on the application point of the active or passive earth pressure acting against the construction. But to determine the safety against overturning or collapse, the distribution of the earth pressure has to be known.

If straight sliding planes are considered, it can be derived that the pressure distribution of lateral earth pressure equals : $P_x = k_a \cdot \gamma \cdot x$ (see Figure VI - 2.6.)

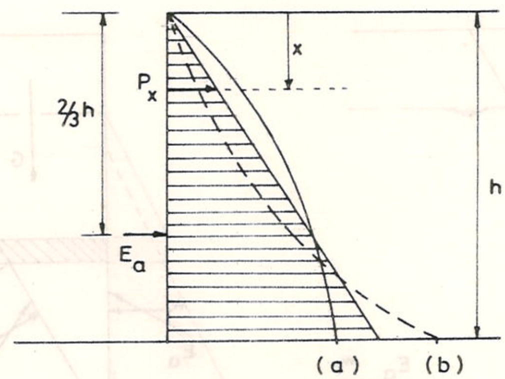


Figure VI - 2.6

The effective lateral pressure is a linear function of the depth x and the resultant earthpressure Ea is applied at a depth x = 2/3 h and equals: $E_a = k_a \cdot \frac{1}{2} \cdot \gamma \cdot h^2$.

However, this pressure distribution is only valid if the wall rotates around a point which is at the toe of the wall or deeper. If the wall rotates around a point that is higher than the toe-depth, then the pressure distribution is more complicated, and will have a form as indicated by (a) in Figure IV - 2.6. The same is applicable to the passive earth pressure distribution, which will be linear in case of a low laying pivot-point and will have a shape as indicated by (b) in case of a high laying pivot point.

If groundwater is present, it will have a very important influence on the pressure distribution against a structure.

If the groundwater is at rest, the pressure head in all points of the soil mass is the same and the water pressure is hydrostatic. So in all points of the boundary surface of the sliding wedge (along the structure and along the plane of rupture) the magnitude of the water pressure is known. The resultants w_1 and w_2 of these water pressures, placed together with the weight G of the sliding wedge, give the resultant R (Figure VI - 2.7). R must be resolved in E_a and Q. The computation should be repeated for different slide planes to find the maximum E_a.

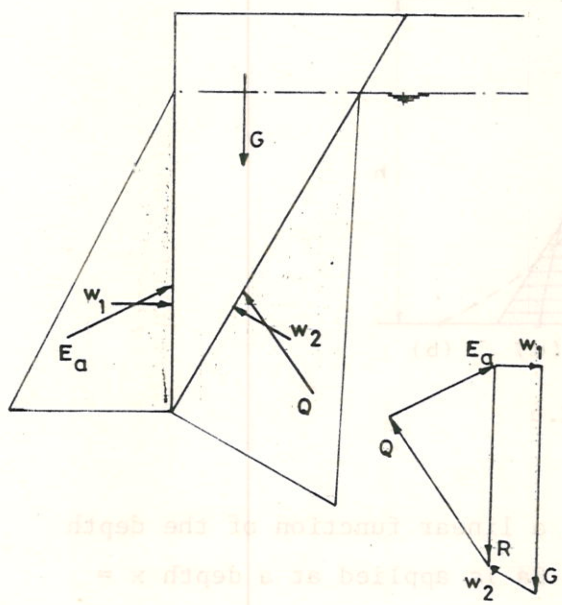


Figure VI - 2.7

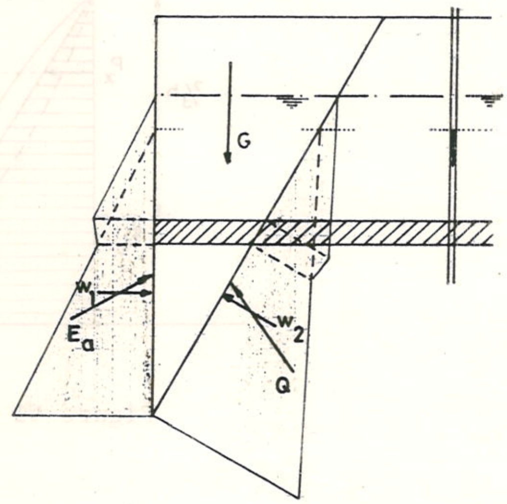


Figure VI - 2.8

In Figure VI - 2.8 a thin clay layer lies half way of the retaining structure. The pressure heads of the ground water in the water bearing strata above and below the clay layer are different.

If the groundwater is in motion, the computation becomes more difficult. The pressure head will differ from point to point. If the soil is homogeneous and permeability is the same in all directions, the water pressures along the wall and the slide plane may be determined by means of a flow net. In each point of inter-section of the slide plane and the equipotential lines the water pressure may be set out. Then the pattern of the water pressure at that plane is known.

Figure VI - 2.9 gives an example. From the force diagram of E_p it is directly seen that as the water pressures are greater, the passive earth pressure becomes very much smaller. This phenomenon may be important for lock walls, especially directly after empty-

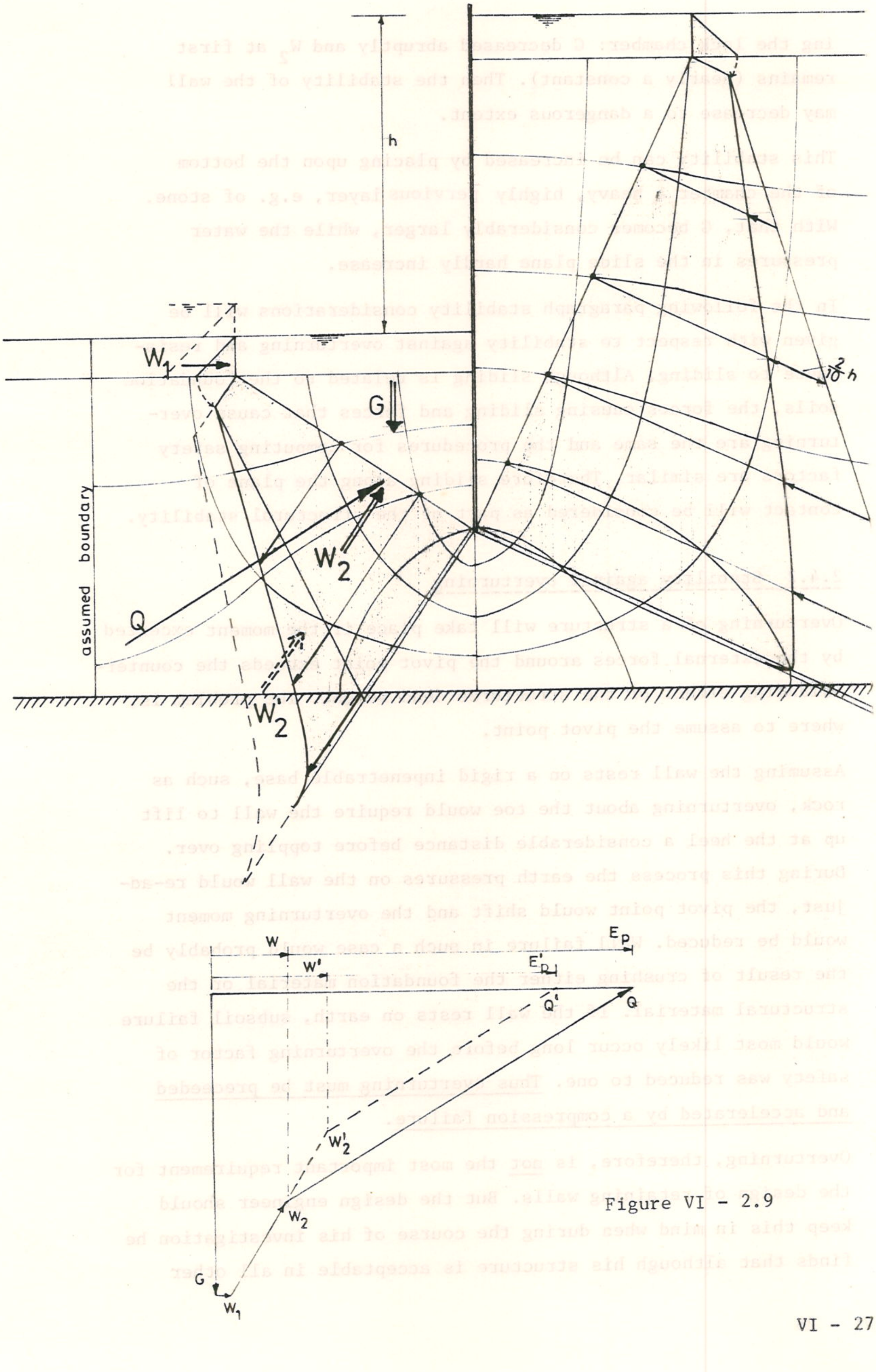


Figure VI - 2.9

ing the lock chamber: G decreased abruptly and W_2 at first remains (nearly a constant). Then the stability of the wall may decrease to a dangerous extent.

This stability can be increased by placing upon the bottom of the chamber a heavy, highly pervious layer, e.g. of stone. With that, G becomes considerably larger, while the water pressures in the slide plane hardly increase.

In the following paragraph stability considerations will be given with respect to stability against overturning and resistance to sliding. Although sliding is related to the foundation soils, the forces causing sliding and forces that cause overturning are the same and the procedures for computing safety factors are similar. Therefore sliding along the plane of contact will be considered as part of the structural stability.

2.4.2 Stability against overturning

Overturning of a structure will take place if the moment excerted by the external forces around the pivot point exceeds the counterbalancing effect of the structures dead weight. The problem is, where to assume the pivot point.

Assuming the wall rests on a rigid inpenetrable base, such as rock, overturning about the toe would require the wall to lift up at the heel a considerable distance before toppling over. During this process the earth pressures on the wall would re-adjust, the pivot point would shift and the overturning moment would be reduced. Wall failure in such a case would probably be the result of crushing either the foundation material or the structural material. If the wall rests on earth, subsoil failure would most likely occur long before the overturning factor of safety was reduced to one. Thus overturning must be preceeded and accelerated by a compression failure.

Overturning, therefore, is not the most important requirement for the design of retaining walls. But the design engineer should keep this in mind when during the course of his investigation he finds that although his structure is acceptable in all other respects his factor of safety against overturning is 1.45 and the minimum allowed is 1.50. The structure does not necessarily have to be redesigned for this reason alone.

Figure VI - 2.10 shows how stability against overturning is to be checked. The safety factor to be applied is 1.5.

For Coulomb's theory : $S_{fo} = \dfrac{W_e . a}{P_v . c - P_h . b}$

For Rankine's theory : $S_{fo} = \dfrac{W_c . a + W''.f}{P_v . c + W'.d - P_h . b}$

In both cases moments acting clockwise are positive and the pivot point is assumed at the toe of the construction.

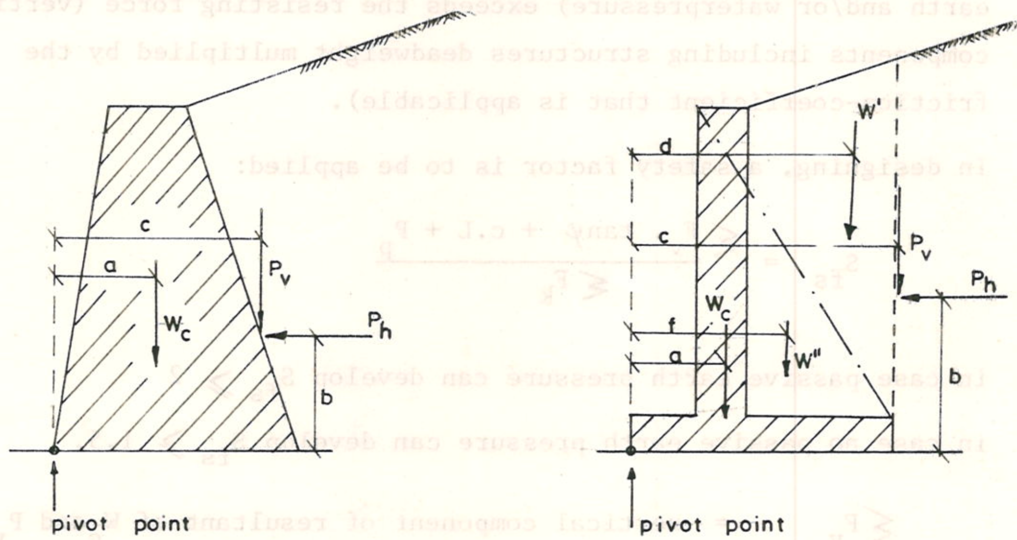


Figure VI - 2.10

2.4.3 Stability against sliding

A failure by sliding occurs when a retaining wall under load moves foreward by sliding along the contact line between soil and structure. If the under surface of the structure is smooth, the resistance to sliding is a function of frictional resistance and the adhesion between the foundation and the soil. However, foundation surfaces are almost always rough or purposely made rough so that a sliding failure almost always occurs along a shear plane through the soil adjacent to the line of contact. Thus the shearing resistance of the foundation soil, rather than the frictional resistance, is used to compute sliding resistance.

If the underlying foundation soils are cohesionless materials, such as sand, gravel or silt, the shearing resistance will increase with depth and a sliding failure will most likely occur along the line of contact. If a cohesive soil underlies the foundation with a lower shearing strength than the soil along the line of contact, failure may occur along a curved plane through the soil.

Sliding occurs when the driving force (horizontal component of earth and/or waterpressure) exceeds the resisting force (vertical components including structures deadweight multiplied by the friction-coefficient that is applicable).

In designing, a safety factor is to be applied:

$$S_{fs} = \frac{\sum F_v \cdot \tan\varphi + c.L + P_p}{\sum F_k}$$

in case passive earth pressure can develop $S_{fs} \geqslant 2$

in case no passive earth pressure can develop $S_{fs} \geqslant 1.5$.

$\sum F_v$ = vertical component of resultant of W_c and P_v (see Figure VI - 2.10).

$\tan\varphi$ = friction coefficient

c = cohesion

L = width of foundation along which friction force will develop

P_p = passive earthpressure

F_h = horizontal component of resultant of driving forces.

If it is anticipated that any of the following conditions may develop, a reduced passive resistance should be used.

- Soil in front of the wall may be removed by scour or future construction operations.
- Soil in front of the wall may be removed during construction and later replaced, but inadequately compacted. Special care must be taken if the soil is sand, gravel or silt.
- Submergence due to a high ground water table will reduce the passive resistance of sand, gravel or silt.
- In order for full passive resistance to develop, the wall must undergo a certain amount of foreward movement. The required amount of movement may be intolerable.
- If vertical cracks can develop due to disiccation or root holes, forward movement may be considerable before passive resistance can develop.

In such cases it should be considered to what extent, if at all, passive earthpressure should be taken into account.

In the absence of tests, the shearing strength of soil may be taken as the total vertical force times a coefficient of friction. Coefficients of friction commonly used are,

- Sand or gravel without silt, $\tan \varphi = 0.55$
- Sand or gravel with silt, $\tan \varphi = 0.45$
- Silt, $\tan \varphi = 0.35$
- Clay, $\tan \varphi = 1/2$ the unconfined compressive strength q_u.

Special precautions are required for foundations on silt or clay. Stiff or hard clay should have its surface well roughened by deep scoring with a rake before placing concrete. If the foundation is to rest on other clays or silts, the top 4 inches of soil should be removed and replaced with sharp grained sand or brick chips. The coefficient or friction between the sand and soil can be assumed as 0.35. However, if $1/2\ q_u$ is less than the assumed frictional resistance, the lower value should be used.

Chapter 3. Earthpressure on top of structures.

3.1 Earthpressure on top of barrel section of sluices.

The earthpressure on top of the barrel of a box sluice not only consist of the weight of the earth which is located on top of the barrel. If a certain settlement is assumed, a part of the soil next to the body of earth on top of the barrel will contribute to the surcharge.

The surcharge should therefore be calculated to consists of the trapezoidal volume of earth between two planes rising from the edges of the barrel under and angle $\alpha = 45^o + \varphi/2$ in which φ is the angle of internal friction of the soil (see Figure VI - 3.1).

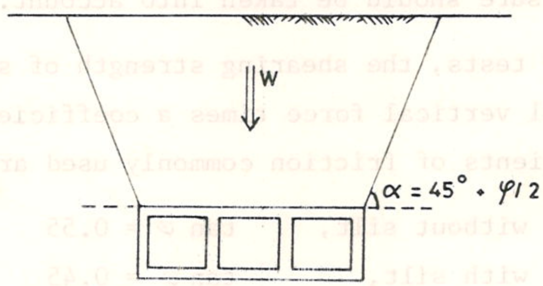


Figure VI - 3.1 Earthpressure on top of a sluice.

3.2 Earthpressure on top of burried pipes.

To calculate the earthpressure on pipes, formula's and graphs established by Roske are used. The formulas are valid for pipes which are laid in broad trenches or through a fill (embankment). The earthpressure on the pipe is then calculated as :

$$P_e = \lambda_d \cdot \gamma \cdot h. \; D \quad KN/m$$

γ = unit weight of soil (KN/m^3)

h = depth of pipe (height of fill)

D = external pipe diameter

λ_d = load factor

The load factor λ_d depends on the settlement-deflection factor $r_{s.d.}$ and a factor $a = \frac{B}{2D}$ in which B is the width of the trench at the crest of the pipe (see Figure VI - 3.2).

The settlement-deflection-factor $r_{s.d.}$ is determined experimentally by Roske.

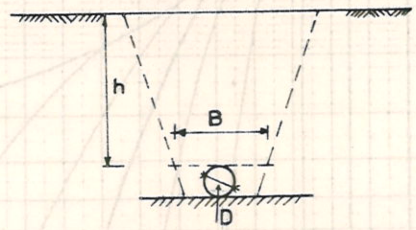


Figure VI - 3.2 Earthpressure on top of a pipe.

The values presented in Table VI - 3.1 can be maintained for $r_{s.d.}$:

Table VI - 3.1 Settlement deflection factor $r_{s.d.}$

$r_{s.d.}$	Concrete pipe
rigid foundation (piles)	1
normale foundation	0.5 - 0.8
foundation on weak soil	0.0.5

In practice it is generally adopted to apply a factor ($r_{s.d.}$ x a) = 0.35 for asbestos cement pipes. Figure VI - 3.3 shows load-factor λ_d for various values of h, D and ($r_{s.d.}$ x a).

The required value of trench-width at crestlevel of the pipe is determined with the help of Figure VI - 3.4, where B can be determined if h, D and ($r_{s.d.}$ x a) are known.

The earthload on top of the pipe calculated in this way can however be reduced for the following reasons.

- the bearing capacity of a pipe is largely dependent on the

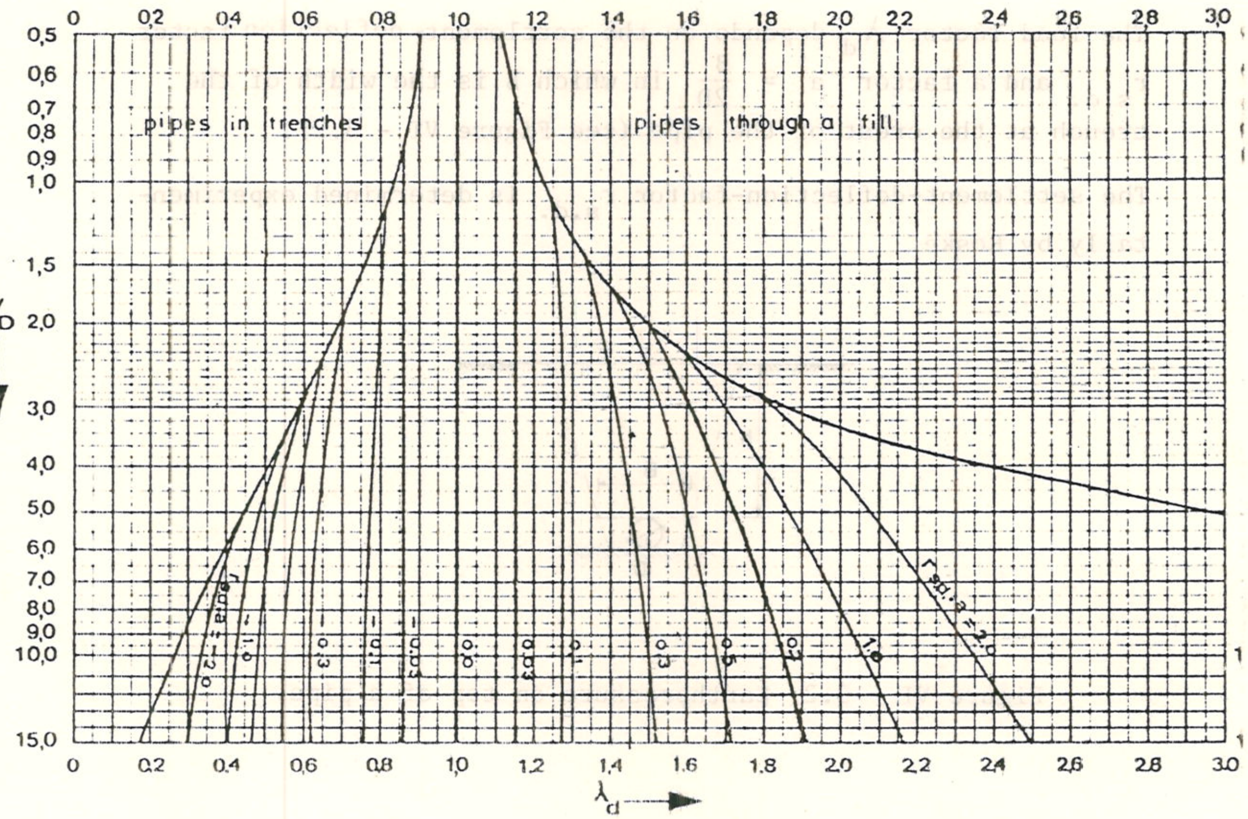


Figure VI - 3.3

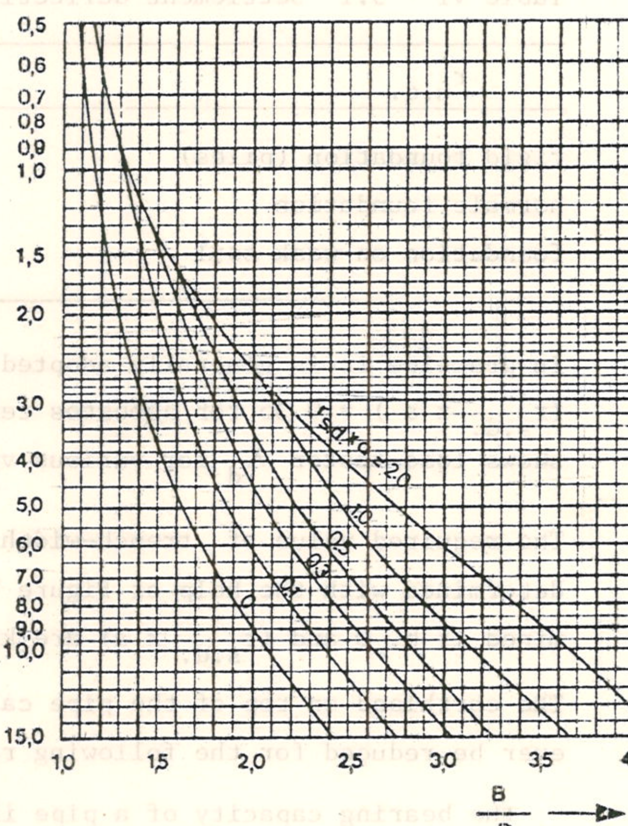


Figure VI - 3.4

bed on which the pipe is laid. The influence of support provided by the bed, is expressed in a factor K.
The following factors can be applied.

α	0^{o}	60^{o}	90^{o}
K	1	1.4	1.7

if the bed consists of concrete, then K can be taken as 1.0, 1.6 and 2.2 respectively.

- Moreover Roske advised to introduce a factor Td for pipes in wide trenches or through fills. The factor Td can increase the bearing capacity if the lateral pressure of the trenchfill can "support" the vertical pressure.

  Table VI - 3.2 gives values for Td.

Table VI - 3.2 Td factors, increasing pipe bearing capacity

	internal friction angle φ	Support angle α					
		30^{o}	60^{o}	90^{o}	120^{o}	150^{o}	180^{o}
bed is loose (loose earth, sand)	20	1.36	1.45	1.51			
	30	1.27	1.27	1.30			
	40	1.13	1.16	1.18			
bed is stiff (concrete)	20	1.43	1.58	1.35	1.37	1.37	1.27
	30	1.25	1.37	1.21	1.23	1.22	1.17
	40	1.15	1.21	1.13	1.14	1.14	1.10

example: $D = 0.15$ m, $\gamma = 18.8$ KN/m^3, $h = 4.20$ m

from Figure VI - 3.4 : $\frac{h}{D} = \frac{4.2}{0.15} = 28$

$\longrightarrow \frac{B}{D} = 3.8, \quad B = 3.8 \times 0.15 = 0.60$ m

$$\left.\begin{array}{lll} r_{s.d.} & = & 0.5 \\ a & = & \frac{B}{2D} = \frac{0.6}{0.3} = 2.9 \end{array}\right\} (r_{s.d.} \times a) = 0.95$$

from Figure VI - 3.3 : $\lambda d = 2.25$

$$P_e = \lambda d.\ \gamma.\ h.\ D.$$

$$= 2.25 \times 18.8 \times 4.2 \times 0.15 = 26.65 \text{ KN/m'}$$

the actual load $P_{ea} = \frac{P_e}{K \times Td} = \frac{26.65}{1.4 \times 1.35} = 14.1$ KN/m'

If $(r_{s.d.} \times a) = 0.35$ is used (for asbestos cement pipes) and no lateral support is assumed then :

$$P_e = \lambda d.\ \gamma\ .\ h.\ D$$

$$= 1.6 \times 18.8 \times 4.2 \times 0.15 = 18.95 \text{ KN/m'}$$

$$P_{ea} = \frac{P_e}{k} = \frac{18.85}{1.4} = 13.54 \text{ KN/m'}$$

Normally a safety factor of 1.5 is applied and thus the pipes should be able to withstand a earthload of 20.3 KN/m' to be considered as a lineload as indicated in Figure VI - 3.5.

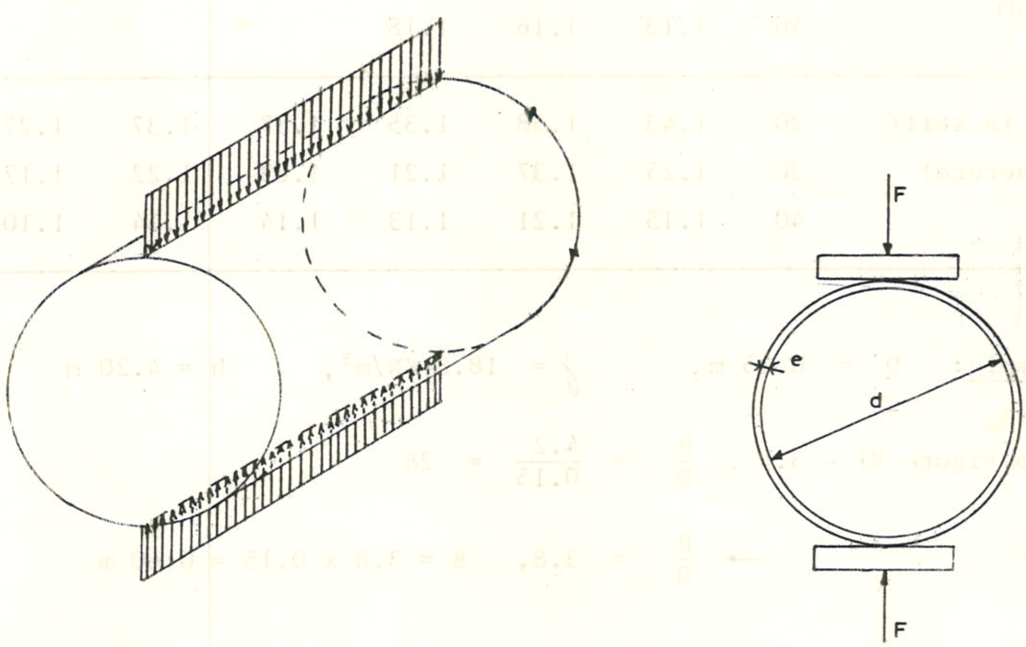


Figure VI - 3.5

To test a pipe, three rings are cut, out of three pipes, each ring 20 cm long and with parallel cuts perpendicular to the pipe-axis. The rings are loaded in radial direction between two parallel plates.

The radial compression strength σ_s is then :

$$\sigma_s = \frac{F.\ (d + e)}{a.\ e^2} \times 0.955\ (N/mm^2)$$

in which: F = load (N)

d = internal diameter (mm)

e = wall thickness at crack (mm)

a = length of the ring (200 mm)

In our example , the radial compression strength should therefore be $\sigma_s \gg 9.34\ N/mm^2$.

Chapter 4. Design of shallow foundations.

4.1 General.

As already indicated in paragraph 1.7 of Chapter 1, it should be investigated if the soil underlying the structure is capable of supporting the loads imposed on it via the foundation. In this respect the behaviour of the soil should be analysed concerning:

- the bearing capacity, or strength or the soil
- the settlements to be expected under certain load conditions.

Normally, the bearing capacity will not be the limiting factor in foundation design, more often the settlements that are calculated to occur under the imposed load, will determine the type and size of foundation.

4.2 Bearing capacity of soil.

According to Hansen, the ultimate bearing capacity or the ultimate soil pressure that a soil can sustain without rupture due to shear failure can be expressed as (ref. 1) :

$$Q_{ult} = c.N_c.S_c.\,d_c + \bar{q}.\,N_q.\,s_q.\,d_q + \tfrac{1}{2}.\gamma.\,B.N_\gamma.\,s_\gamma$$

in which factors influencing the bearing capacity, like depth and shape of foundation, have been taken care of by means of coefficient s and d, and c = cohesion and N_c, N_q and N_γ are bearing capacity factors (see Table VI - 4.1 and Figure VI - 4.1). In case $\varphi = 0$, the formula is changed for :

$$Q_{ult} = 5.14 \times c\,(1 + s'_c + d'_c) + \bar{q}$$

Table VI - 4.1 Depth and shape factors for Hanson's formula for $Q_{ult.}$

Shape factor	depth factor	
	$D \leqslant B$	$D > B$
$s_c = 1 + \frac{N_q}{N_c} \cdot \frac{B}{L}$	$d_c = 1 + 0.4 \frac{D}{B}$	$d_c = 1 + \frac{0.4}{\tan D/B}$
$s_q = 1 + \frac{B}{L} \cdot \tan \varphi$	$d_q = 1 + 2 \tan \varphi \cdot (1-\sin \varphi)\ 2.\frac{D}{B}$	$d_q = 1 + \frac{2 \tan \varphi . (1-\sin \varphi)^2}{\tan D/B}$
$s_\gamma = 1 - 0.4 \left(\frac{B}{L} \right)$		
$s'_c = 0.2 \frac{B}{L}$	$d'_c = 0.4 \frac{D}{B}$	$d'_c = \frac{0.4}{\tan D/B}$

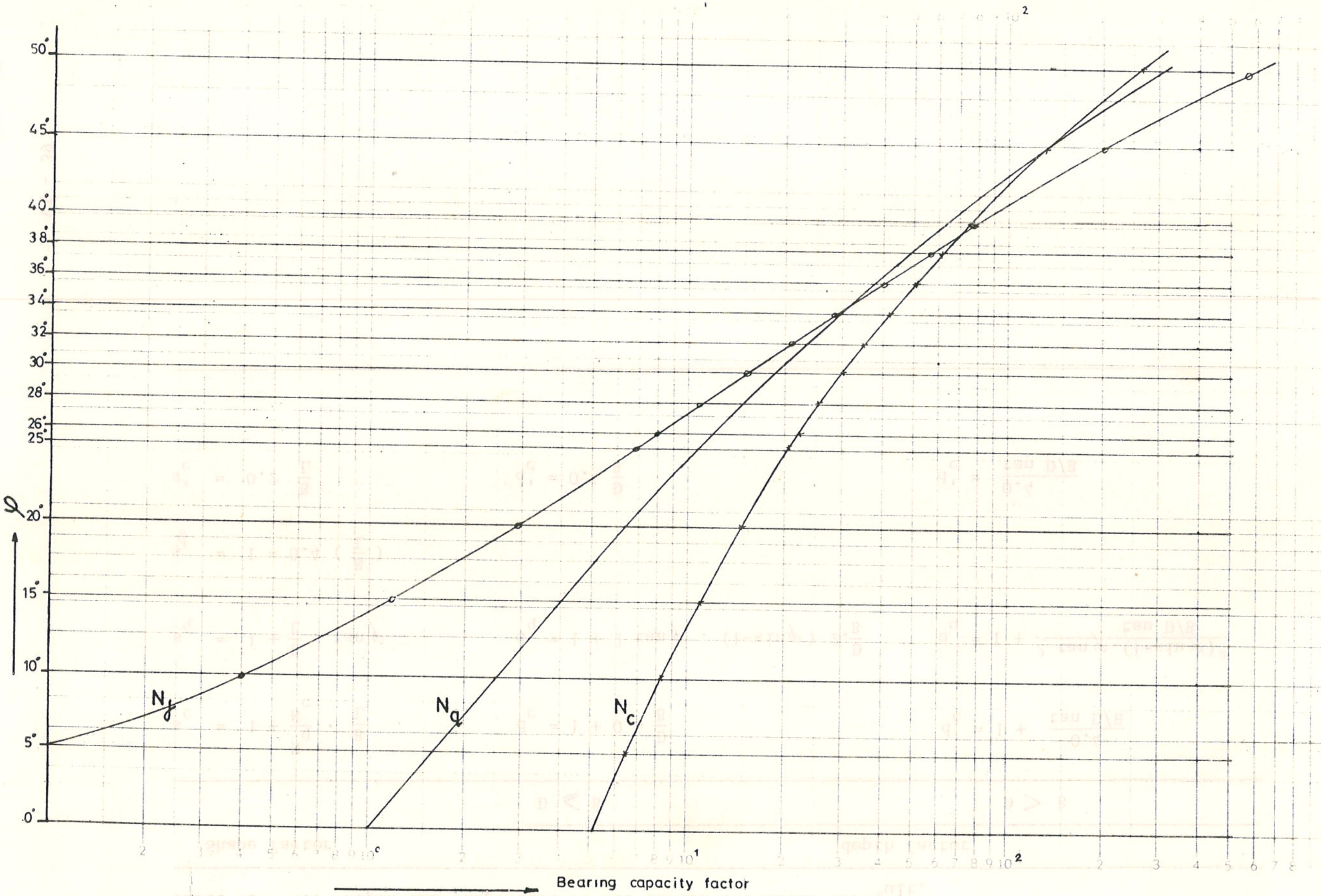


Figure VI - 4.1 Bearing capacity factors.

Furthermore :

B = width of the foundation (least-dimension)

D = depth of foundation below original groundlevel

$\bar{q}$ = effective surcharge = $D \times \gamma$ (KN/m^2)

γ = effective unit weight of soil

L = length of foundation (largest-dimension)

In the Hansen equation the effective unit weight of soil should be used. However, when the water table is level with the base of the foundation, then the saturated unit weight should be taken. When the water table is at some distance below the foundation then a reduction factor F_w is to be applied.

$$\gamma = F_w . \gamma_{sat}$$

$$F_w = 0.5 + \frac{d_w}{B . \tan (45 + \varphi/2)}$$

d_w = distance between foundation and watertable.

To calculate the effective surcharge pressure the unit weight as determined from the soil samples should be applied or a division has to be made to include partly γ_{sat} and the natural unit weight.

In case a foundation slab is eccentrically loaded, the dimensions of the footing should be reduced in the calculations for ultimate bearing capacity. If eccentricity of load is e_x and e_y then, the dimensions will be : (see Figure VI - 4.2)

$$B' = B - 2 . e_x$$

$$L' = L - 2 . e_y$$

and area $A_f' = B' . L'$

e_x and e_y are the offset of the forces (vertical) from the centre of the footing.

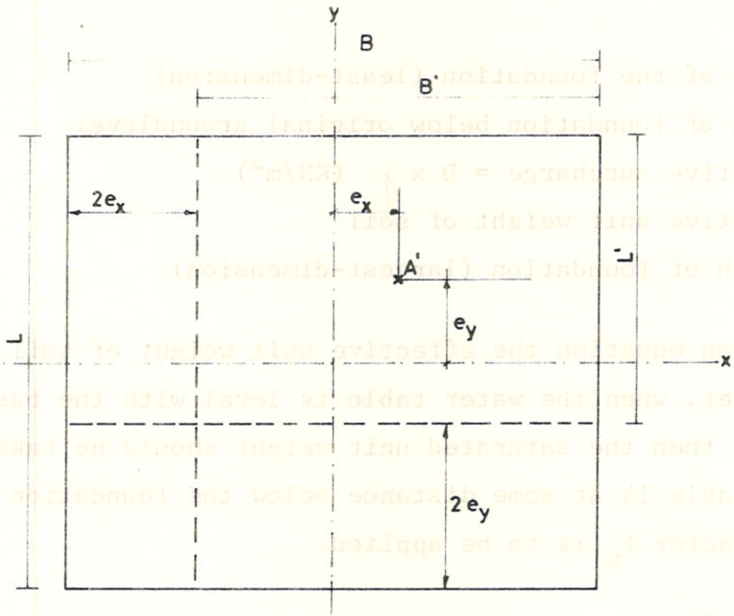


Figure VI - 4.2 Eccentrically loaded foundation.

It is suggested to use the above presented method both for cohesive and non-cohesive soils as a simple means to estimate the soils bearing capacity.

To establish the bearing capacity of cohesion less soils such as sand, sand-gravels, and sand with some silt, SPT values can be used. The method however is largely empirical. The allowable bearing capacity can be determined from the following equations. (see Figure VI-4.3).

$$q_a = \frac{N}{2.5} \cdot k_d \qquad (B \leqslant 4 \text{ ft})$$

$$q_a = \frac{N}{4} \frac{(1 + B)^2}{D} \cdot k_d \qquad (B > 4 \text{ ft})$$

in which q_a is the allowable increase in soil pressure, expressed in kips/sqft,*) for an estimated maximum settlement of 1 inch and $k_d = 1 + \frac{D}{3B}$ with a maximum of $k_d = 1.33$.

*) 1 kips/sq.ft. = 1 ksf = 47.9 KN/m^2

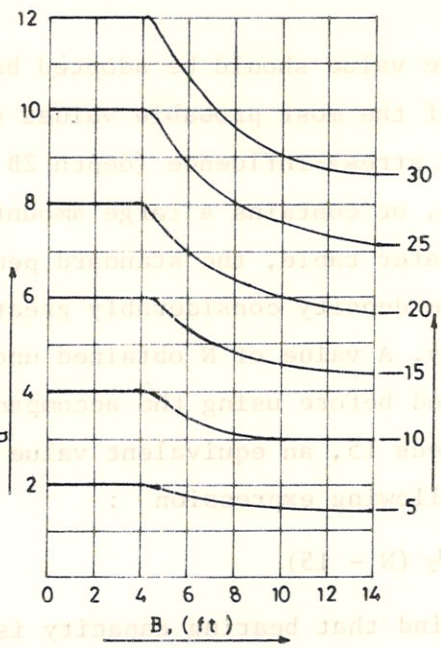


Figure VI - 4.3 Allowable bearing capacity for surface loaded footings with maximum 2.5 cm settlement.(ref.1).

The equations and Figure VI - 4.3 are based on the empirical relation between SPT values and angle of internal friction φ, shown in Table VI - 4.2. The relation may also be used directly to determine φ from SPT values of cohesion less soils; φ can then be used in the Hansen equation.

Table VI - 4.2 Empirical values for φ, based on SPT values for cohesionless soils

Description	very loose	loose	medium	dense	very dense	
Relative density	0	0.15	0.35	0.65	0.85	1.0
SPT-Value N		4	10	30	50	
Approximate angle of internal friction	$25-30^o$	$27-32^o$	$30-35^o$	$25-40^o$	$38-43^o$	

*) 1 kips/sq.ft. = 1 ksf = 47.9 KN/m^2

For N, a representative value should be adopted based on a rough statistical analysis of the most probable values determined by borings in the zone of stress-influence (depth 2B below footing). When sand is very fine, or contains a large amount of silt, and is located below the water table, the standard penetration tests may indicate a relative density considerably greater than the actual relative density. A value of N obtained under these conditions must be modified before using the accompanying charts. Therefore, when N exceeds 15, an equivalent value N' should be determined from the following expression :

$$N' = 15 + \tfrac{1}{2}(N - 15)$$

It should be kept in mind that bearing capacity is not likely to control the foundation design. Settlements will be much more determinant and bearing pressures to limit settlements are likely to be less than the allowable value obtained from any bearing capacity calculation.

An example of a shallow foundation design is presented in ANNEX VI - 2.

4.3 Settlements.

4.3.1 General.

All structures, except those on solid rock, will experience a certain amount of settlements, which are of two general types :

- immediate settlements, taking place during construction and when applying the loading,
- consolidation; settlements taking place over a long period.

Settlements may be uniform, with the foundation remaining plane, or differential, with a difference in settlement between two points of the foundation.

Uniform settlements will not cause significant problems as long as the amount of settlement remains within the tolerance.

Differential settlements however may be a problem as was illustrated in Chapter 1, paragraph 1.7.

It is generally accepted that for hydraulic structures the tolerable total settlement is 5 cm and that the maximum differential settlement may not exceed 3/4 of the computed total settlement. However, it should be kept in mind that depending on the size, type and importance of a structure, these figures should be adopted.

4.3.2 Uniform immediate settlements

Uniform immediate settlements of soils such as non saturated clays and silts, and saturated or non saturated sands and clayey sands can be computed from :

$$S = q \cdot B \cdot \frac{1 - \mu^2}{E_s} \cdot I_w$$

in which : S = settlement

q = intensity of contact pressure

B = least lateral dimension of foundation

I_w = shape and rigidity factor of foundation (see Table VI - 4.3)

E_s, μ = elastic properties of the soil (see Table VI-4.4 and 4.5)

The factor E_s can be obtained from stress-strain-plots from triaxial tests, or if to be based on SPT values :

$$E_s = 4.9\ (N + 15) \text{ for sand } (kg/m^2)$$

$$E_s = 2.9\ (N + 5) \text{ for clayey sands } (kg/m^2)$$

However, laboratory test have indicated that E_s determined by unconsolidated undrained triaxial tests can be 0.7 - 1 times the actual E_s.

Because the settlement formula for S is meant for foundations on groundlevel, a correction factor F_3 has to be applied if the foundation is at depth D into the ground; Figure VI - 4.4 indicates the correction factor. The corrected settlement becomes then :

$S_f = S \times F_3$.

Table VI - 4.3 I_w-factor for rigid foundation.

Shape L/B	I_w	I_m
0.2		2.29
0.5		3.33
1.0	0.88	3.7
1.5	1.06	4.12
2	1.20	4.38
5	1.7	4.82
10	2.1	4.93
100	3.4	5.06

Table VI - 4.4 Typical of E_s (= $\frac{\sigma}{\varepsilon}$ = $\frac{\text{stress}}{\text{strain}}$), stress/strain modulus.

Soil		E_s (kg/m^2)
Clay :	very soft	3-30
	soft	20-40
	medium	45-90
	hard	70-200
	sandy	300-425
Sand :	silty	50-200
	loose	100-250
	dense	500-1000
Silt :		20-200

Table VI - 4.5 Typical ranges of μ ($\mu = \frac{\varepsilon_3}{\varepsilon_1} = \frac{\text{lateral strain}}{\text{longitudinal strain}}$), μ = Poisson's ratio.

Soil	μ
saturated clay	0.4-0.5
unsaturated clay	0.1-0.3
sandy clay	0.2-0.3
silt	0.3-0.35
sand : dense coarse	0.15
dense fine	0.25

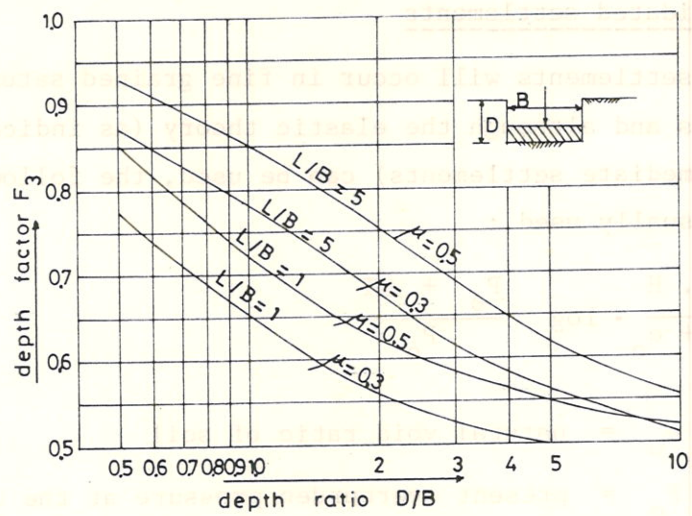


Figure VI - 4.4 Correction factor F_3 for footing at depth D.

4.3.3 Differential settlements

For differential settlements, the influence of rotation due to excentric loads is expressed as :

$$\tan \varphi = \frac{V.e}{B.L^2} \times \frac{(1 - \mu^2)}{E_s} \times I_m \quad \text{(see Figure VI - 4.5)}$$

The differential settlement S_d = L. tan φ . For factor I_m, Table VI - 4.3 should be consulted.

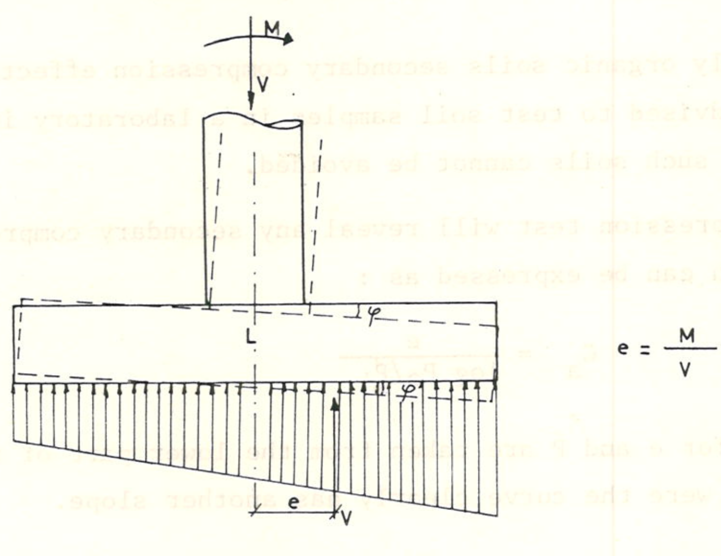


Figure VI - 4.5 Rotation of a rigid footing

4.3.4 Consolidated settlements

Consolidated settlements will occur in fine grained saturated cohesive soils and although the elastic theory (as indicated to calculated immediate settlements) can be used, the following equation is usually used :

$$S = \frac{C_c \cdot H}{1 + e_o} \cdot \log \frac{P_o + \Delta P}{P_o}$$

in which : e_o = natural void ratio of soil

P_o = present overburden pressure at the centre of the considered soil layer

ΔP = increase in soil pressure at centre of soil water

H = height of soil layer(s) subject to compression

C_c = compression index.

To be able to use this equation, it is necessary to establish the required soil properties by a consolidation test.

$$C_c \text{ (= compression index)} = \frac{\Delta e}{\log P_2/P_1}$$

(see Volume II, Chapter 6, paragraph 6.3.3).

In highly organic soils secondary compression effects may develop. It is advised to test soil samples in a laboratory in case foundation on such soils cannot be avoided.

The compression test will reveal any secondary compression index C_a which can be expressed as :

$$C_a = \frac{e}{\log P_2/P_1}$$

Values for e and P are taken from the lower part of the compression diagram were the curve clearly has another slope.

The secondary compression is then :

$$S_c = \frac{C_a \cdot H}{1 + e_o} \log \frac{t_{total}}{t_{prim}}$$

in which : S_c = secondary settlement

t_{total} = service life of construction

t_{prim} = time up to the end of the primary settlement.

Consolidation test are expensive and time consuming. If for these reasons it is not justifiable to do these test for smaller structures, an empirical method as presented below, can be used.

<u>The allowable bearing capacity for non-cohesive soils</u> as derived from the Meyerhoff equations for surface loaded footings, resulting in a maximum settlement of 2.5 cm, are plotted in Figure VI-4.3. The figures from the graph should be corrected for the depth of the foundation with factor F_3 as indicated in paragraph 4.3.2. (Figure VI - 4.4).

The effect of water is already included as an effect on N and no additional adjustments are required.

<u>For foundations on clay the</u> Table VI - 4.6 can be used. Values shown are sufficiently safe to keep differential settlements within 2 cm (3/4 times the maximum settlement of 2.5 cm), provided the soil is not influenced by loadings from an adjacent structure, and provided the soil beneath the foundation is not a soft or very soft normally loaded clay. If such is the case then consolidation tests have to be executed in order to acquire the required information.

Table VI - 4.6 q_a for clay.

description	N	q_a	q_a(ksf)*
very soft	<2	<0.6	<0.44
soft	2-4	0.6-1.2	0.44-0.9
medium	4-8	1.2-2.4	0.9 -1.8
stiff	8-15	2.4-4.8	1.8 -3.6
very stiff	15-30	4.8-9.6	3.6 -7.2
hard	>30	>9.6	>7.2
		Square footings	Continuous footings

* inclusive safety factor 3

1 ksf = 47.9 KN/m^2

Chapter 5. Design of Deep Foundation.

5.1 Introduction.

If settlements of a foundation are expected to be too big for the construction to allow, or bearing capacity is very poor and there are no other possibilities to reduce the settlements, a pile foundation has to be designed.

Piles can be made of wood, steel or concrete and are usually driven with a succession of blows on top of the pile. Also it is possible to lower piles with a water jet or with the help of vibrators.

- Wooden piles, preferably should be of first quality sundari bullah. A straight line between top and butt should be within the pile shaft.

Table VI - 5.1 Specification for wooden piles

length	diam. at top of of the tree	diameter at butt of the tree
$<$ 7.5 m	0.15 m	0.25 m
$>$ 7.5 m	0.15 m	0.30 m

Permissable load on wooden piles varies from 100-200 KN per pile, depending on the allowable design stress in the type of wood, the tip cross-section of the pile and the structure of the soil. For point bearing piles the allowable design load is the tip cross-section of the pile times the allowable stress in the type of wood. For combined friction end-bearing piles the thus calculated pile capacity may be increased. Timber piles should be placed below the lowest watertable. Piles will rot above water level.

- Concrete piles are made of reinforced concrete, cast in a square form. Reinforcement is to be designed on picking up forces, forces during transport and any tension forces during driving. The allowable load on a concrete pile can be calculated from the cross-section area of steel and concrete multiplied by respective allowable material stresses. Minimum dimension of pile cross-section is 20 x 20 cm^2.

- Steel piles are usually round tubes or standard H or I profiles. Hollow piles behave vitually the same as closed-end piles during driving. Allowable loads are calculated in the same way as for concrete piles. Hollow steel piles are usually filled up with concrete after driving.

When designing a pile foundation, it should carefully be considered which method of execution of the work is the best for the specific conditions at the construction site. It might be better to design a foundation on many short piles, which are easier to handle and less vulnerable during transport and driving, than the application of long piles. Of course the soil conditions will influence the possibilities greatly.

5.2 Bearing capacity of piles.

5.2.1 General.

Normally the pile reaction force to resist the pile load is delivered partly by skin friction, partly by toe resistance. If the toe resistance is predominant, piles are called and bearing piles. If skin friction is predominant they are called friction piles. Experience teaches that if a pile is surrounded entirely by very soft soil, lateral support will be sufficient to prevent buckling. If the pile or part of the pile is surrounded by water, the possibility of buckling must be considered.

To determine the bearing capacity of a pile a loading test can be performed during which settlement of the pile is recorded as the load is increased in steps. Loading tests are very costly and consume a lot of time and can only be justified for highly expensive structures. Moreover, it is not possible to distinguish skin friction and toe resistance separately. Sometimes these two have to be known, for instance in case where the soil is expected to settle and thus will contribute to the pile load with a downward force (negative skin friction). Toe resistance alone has to withstand all forces including the negative skin friction.

Negative skin friction may develop in compressible soils on which additional load is being applied. The pile may remain stable and the surrounding soil will move downward due to compression, causing downward friction forces on the pile.

Furthermore, if a pile is driven as part of a group, the soil may be compacted. In this case future settlement of the soil may be less and bearing capacity of the subsoil may increase, resulting in an improved load settlement response of the piles.

Because of all the uncertainties and the rather high costs, loading test is seldom executed.

Another means of determining the bearing capacity of a pile is to deduce it from the penetration during driving. Several formulas have been developed, some empiric, some based on Newton's impact theories, both for plastic and elastic impact.

One of those formulas expresses the admissible pile load as :

$$P_a = \frac{W^2 . H}{F.s\,(P+W)}$$

in which :
- P_a = admissible pile load
- W = weight of piling hammer
- H = drop of hammer
- F = safetyfactor (5-6)
- s = penetration of pile by one hammer blow
- P = weight of the pile.

The formula is known as Eytelwein's formula or the Dutch pile formula.

A modern technique to design a pile foundation is based on soil borings in which S.P.T. values are recorded.

The ultimate pile resistance is then computed as :

P_u = point bearing + skin resistance = $P_{pu} + P_s$

For displacement piles, Meyerhof proposed the following approximations based on SPT - values.

P_{pu} = $383 \times N.A_p$ (KN)

P_s = $1.92 \times N.A_s$ (KN)

A_p = pile's cross-section in m^2

A_s = pile's surface in m^2

N = S.P.T. value

For piles in layered deposits, consisting of both cohesive and non-cohesive soils, one must compute the contribution of seperate layers for skin resistance and evaluate the point bearing capacity for that layer in which the point terminates. The total pile capacity is the sum of resistances from all layers. Friction piles in silt and silty sands require additional considerations :

- if the dry density is more than 12.7 KN/m^3, the bearing capacity might be sufficient to avoid a pile foundation.
- if dry density is lower than 12.7 KN/m^3 then it may be necessary to use point bearing piles which penetrate through the silt layer to a firm under lying stratum.

Static pile bearing capacity formulas as given in handbooks are only applicable if soil properties are carefully determined. Common practice is to use unconfined undrained ($\varphi = 0$) compression test on cohesive soil samples to determine cohesion of the soil and penetration test (SPT) for cohesionless soils.

5.2.2 Pile capacity in cohesive soils

For cohesive soils, the ultimate pile capacity becomes :

$$P_u = P_{up} + P_s$$

$$P_u = c.N_c.A_p + \alpha.c.p.\ L$$

in which c = average cohesion for the soil layer of interest

N_c = bearing capacity factor, (see Figure VI-4.1) take N_c minimum 9.

A_p = area of pile point

α = adhesion factors (see Figure VI - 5.1 and Table VI - 5.2)

p = pile perimeter

L = pile length

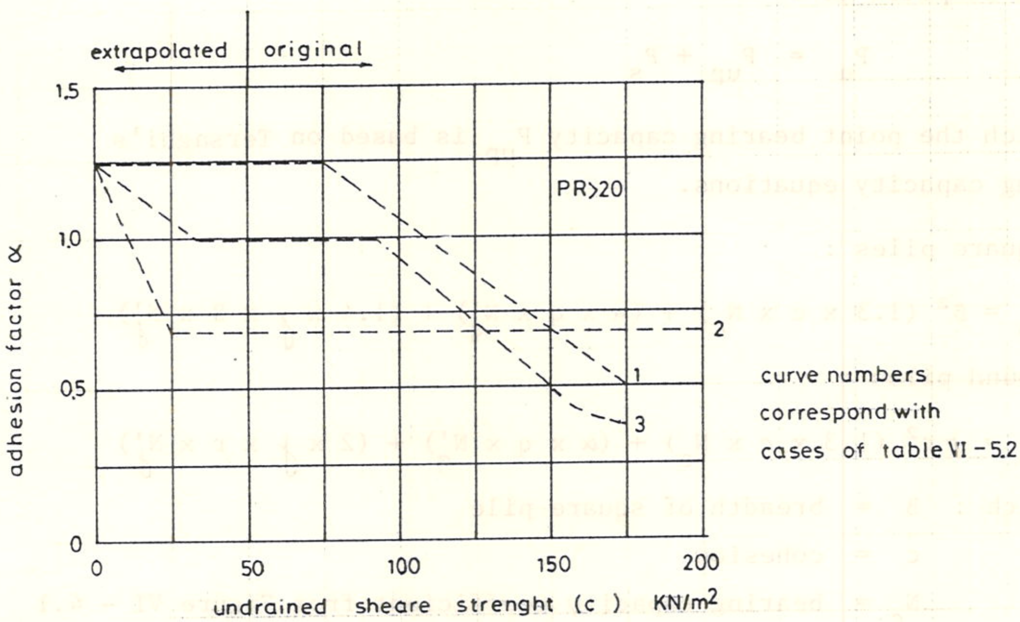


Figure VI - 5.1 Relationship between soil and adhesion factor.

The allowable pile capacity is obtained as :

$$P_a = \frac{P_u}{F}$$

in which P_a should be compared against the pile-material capacity and F is a safety factor as 4 for cohesive soils because of the larger uncertainties in pile design as compared with shallow foundation design.

Table VI - 5.2 Values of adhesion factor α for piles driven into stiff cohesive soils (ref.1).

Case	Soil condition	PR=Penetration Ration *)	α
1	Sand/sandy gravel overlying stiff to very stiff cohesive soils	<20	1.25
2	Soft clays or silt overlying stiff to very stiff cohesive soils	8<PR<20	0.40
3	Stiff to very stiff cohesive soils, no overlying strata	8<PR<20	0.40

*) PR = Penetration Ratio = $\frac{\text{depth of penetration into cohesive soil}}{\text{diameter of pile}}$

5.2.3 Pile capacity in cohesion-less soils

For cohesion-less soils the ultimate bearing capacity of piles can be computed as :

$$P_u = P_{up} + P_s$$

in which the point bearing capacity P_{up} is based on Tersaghi's bearing capacity equations.

For square piles :

$$P_{up} = B^2\ (1.3 \times c \times N_c) + (\alpha \times q \times N'_q) + (1.4 \times \gamma \times B \times N'_\gamma)$$

For round piles :

$$P_{up} = \pi r^2\ (1.3 \times c \times N_c) + (\alpha \times q \times N'_q) + (2 \times \gamma \times r \times N'_\gamma)$$

in which : B = breadth of square pile

c = cohesion

N_c = bearing capacity coefficient from Figure VI - 4.1

N'_q, N'_γ = bearing capacity coefficient.
if $\varphi < 20°$ use Figure VI - 4.1, if $20° < \varphi < 45°$
then use Figure VI - 5.2

q = over burden pressure ($\gamma \times D$)

α = adhesion factor (see Figure VI - 5.1)

r = radius of round pile.

The skin friction is calculated with :

$$P_s = \int_p^L . \bar{q}. K. z. (\tan \delta). dz$$

which is in principle calculating the lateral earth pressure against the pile perimeter p using friction coefficient $\tan \delta$. (for δ see Table VI - 5.3).

Furthermore is :

$\bar{q}$ = $\gamma'. z$ (effective overburden pressure on element z)

K = earth pressure coefficient

K = 0.6 for silty sands - fine silty sands; 1.25 for other deposits.

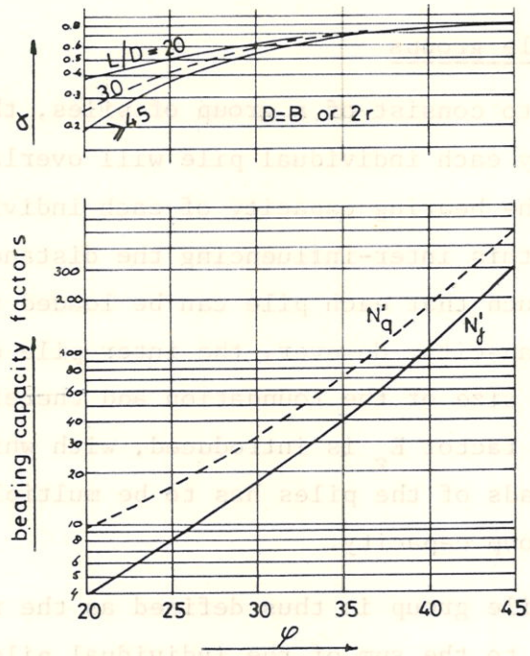


Figure VI - 5.2 Bearing capacity factors for deep foundations (ref.1).

Table VI - 5.3 Friction angle δ.*)

interface material	δ (in degrees)
mass concrete or masonry on :	
- gravel-sand mixture, coarse sand	29-31
- clean fine to medium sand, silty medium to coarse sand,	24-29
- clean fine sand, silty or clayey fine to medium sand	19-24
- fine sandy silt, non plastic silt	17-19
- medium stiff and stiff clay and silty clay	17-19
- very stiff and hard preconsolidated clay	22-26
formed concrete or concrete sheetpile on :	
- gravel sand mixtures	22-26
- clean sand, silty sand-gravel misture	17-22
- silty sand or sand mix with silt or clay	17
- fine sandy silt, non plastic silt	14

*) based on NAFAC (1971)

5.2.4 Capacity of pile groups

When a foundation is to consist of a group of piles, the soil pressures developed by each individual pile will overlap each other and influence the bearing capacity of each individual pile negatively. To avoid this inter-influencing the distances between the piles should be such that each pile can be loaded to its individual calculated capacity. However, the inter-pile distance may be limited by the size of the foundation and therefore a pile group efficiency factor E_g is introduced, with which the sum of permissable loads of the piles has to be multiplied to arrive at the pile group capacity.

The efficiency of a pile group is thus defined as the ratio of actual group capacity to the sum of the individual pile capacities.

The Converse-Labarre equation has been widely used to compute E_g:

$$E_g = 1 - \theta \frac{(n - 1)\,m + (m - 1)\,n}{90.\,m.n.}$$

in which the symbols are defined as indicated in Figure VI - 5.3.

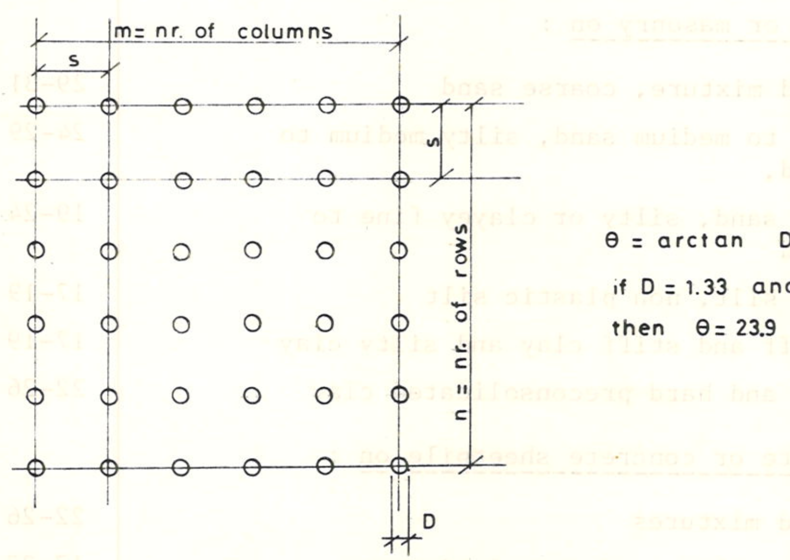


Figure VI - 5.3 Pile group

The equation is only based on comparing the sum of skin resistances of all piles with the skin resistance of the earth block designated by the circumference $2 [(n-1) s + (m-1) s]$ of the pile group.

In case of the foundation slab, which connects all the piles, is cast directly on the soil surface, as is mostly the case, the group capacity is the block capacity based on perimeter shear and bearing capacity of the block at the pile points.

It should be checked whether the block perimeter shear exceeds the pile group efficiency shear.

<u>Block perimeter shear</u> = $L \times [(n-1) s + (m-1) s] \times 2 \times c$

in which : L = hight of block (= pile length)

n, m, s = see Figure VI - 5.3

c = cohesion of soil

= 0.5 x unconfined compression stength

<u>Pile group efficiency shear</u> = $E_g \times m \times n \times P_s$

in which : E_g = pile group efficiency

m, x, n = number of piles

P_s = skin resistance for each individual pile.

The smallest of the two should be used for design purposes and the bearing capacity of the block-area at depth L should be added to arrive at the total group bearing capacity.

5.3 Design of pile foundations.

5.3.1 General.

Pile systems as a rule are designed assuming that a pile can only transmit an axial load. Thus, it is assumed that the pile ends are hinges and lateral support of the soil is neglected.

If, for a 2 dimensional problem (see Figure VI - 5.4) the loads I, II, and III which result from force R are far greater than can be carried by the corresponding piles (pile rows) the lines I, II and III have to be considered as centre lines for 3 pilegroups. The piles of each group can be placed symmetrically round about these lines.

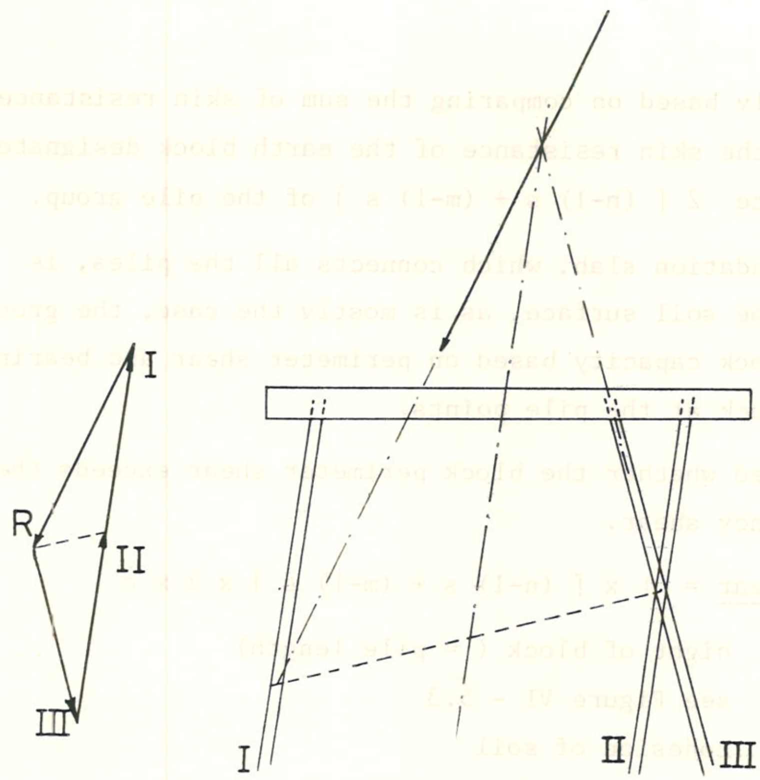


Figure VI - 5.4

But the resultant of the forces of the piles of one group only coincides with the group's centre if all piles of this group are equally loaded. To fulfil this condition the movement of the superstructure should be a parallel one. Generally this is not so. The error which is introduced by assuming parallel displacement may however be neglected if the distances between the extreme piles "a" of each group are small in proportion to the distances "b" between one centre line and the point of intersection with the 2 others (see Figure VI - 5.5).

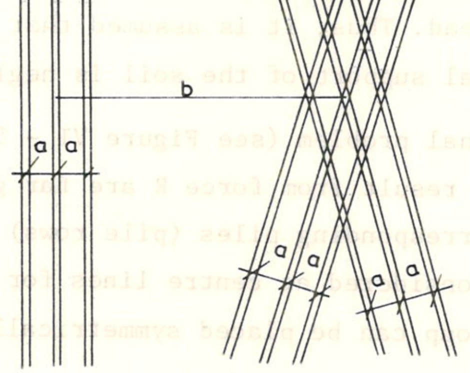


Figure VI - 5.5

For preliminary designs this method will give sufficient results, for final design a control computation has to be made.

5.3.2 Control computation by means of the elastic centre of a pile system

The elastic centre of a pile system is defined as follows : Every external force, acting at the rigid foundation block, supported by a pile system, intersecting the elastic centre, gives this block a rotation - free (parallel) displacement. A moment, acting on the block, will give it a rotation about this point.

The determination of the elastic centre can be done graphically in the following way (see Figure VI - 5.6).

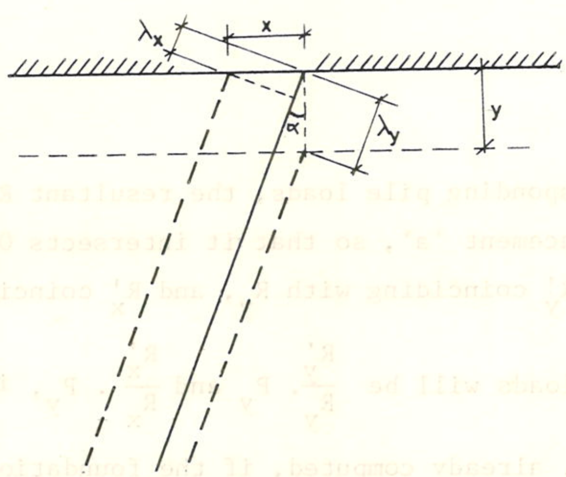


Figure VI - 5.6

a: Give the superstructure a vertical displacement = y. If the inclination of a pile is α , the displacement λ_y of the head of the pile in the direction of the pile axis will be $\lambda_y = y.\cos\alpha$, and since the displacement is proportional to force P_y it follows:

$$\lambda_y = c.P_y \text{ and } P_y = \frac{y.\cos\alpha}{c}$$

So the load of every pile can be determined and with a force diagram and link polygon, the resultant force R_y of the pile loads can be found in magnitude, direction and position.

b: Give the superstructure a horizontal displacement = x, then

$$\lambda_x = x.\sin\alpha \text{ and } P_x = \frac{x.\sin\alpha}{c}$$

Again the resultant force R_x of all pile loads can be determined with a force diagram and a link polygon.

The point of intersection of R_y and R_x is the elastic centre of the pile system.

Generally the resultant force R' acting on the pile system will not intersect the elastic centre 0 (see Figure VI - 5.7).

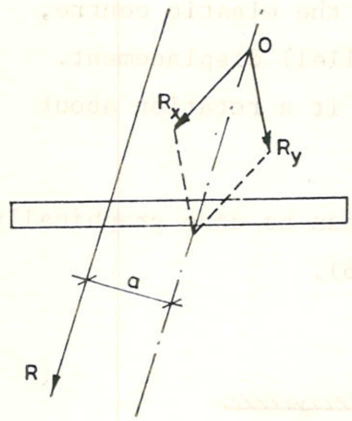


Figure VI - 5.7

To calculate the corresponding pile loads, the resultant R' is given a parallel displacement 'a', so that it intersects 0. Resolve R' into 2 components, R'_y coinciding with R_y, and R'_x coinciding with R_x.

The corresponding pileloads will be $\frac{R'_y}{R_y} . P_y$ and $\frac{R'_x}{R_x} . P_y$, in which P_x and P_y are the loads, already computed, if the foundation undergoes a vertical displacement y, and a horizontal displacement x respectively.

By giving R' a parallel displacement 'a', a moment M = R'.a has been introduced. This moment M gives the foundation block a rotation on 0. If the rotation is φ, the corresponding pile loads can easily be expressed in φ and r (Figure VI - 5.8)

$$\lambda_t = r . \varphi . \sin\beta$$

since $r . \sin\beta = a \rightarrow \lambda_t = a . \varphi \, ; \; P_\varphi = \frac{a - \varphi}{c}$ and the moment of

P_φ about $0 = P_\varphi . \; a = \frac{a^2 . \varphi}{c}$

Conditions of equilibrium : $\sum P . \; a = \frac{\sum a^2 . \varphi}{c} = \varphi . \sum \frac{a^2}{c} = M.$

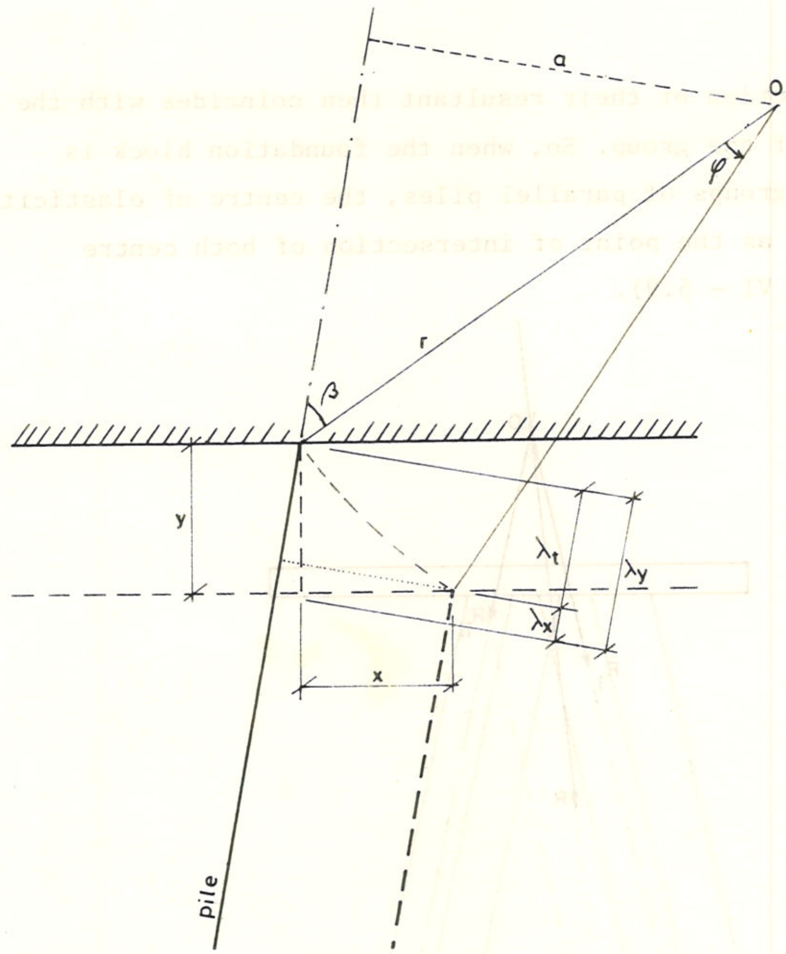


Figure VI - 5.8

Thus, the pile load, due to the moment M, can be computed. The resulting pile loads, caused by a resultant force R' are found by adding up the 3 forces.

$$P_{total} = P'_x + P'_y + P_\varphi$$

ANNEX VI - 3 gives an example of the computation of a given pile system of 4 piles, loaded by a force R'.

5.4 Pile systems

It is obvious that a pile system should be as simple as possible; often it is possible to limit the number of pile directions to two; then the pile loads can be computed very easily.

Parallel piles, which have the same dimensions, will be equally loaded if the superstructure undergoes a parallel displacement.

The line of action of their resultant then coincides with the centre line of the group. So, when the foundation block is carried by 2 groups of parallel piles, the centre of elasticity will be found as the point of intersection of both centre lines (Figure VI - 5.9).

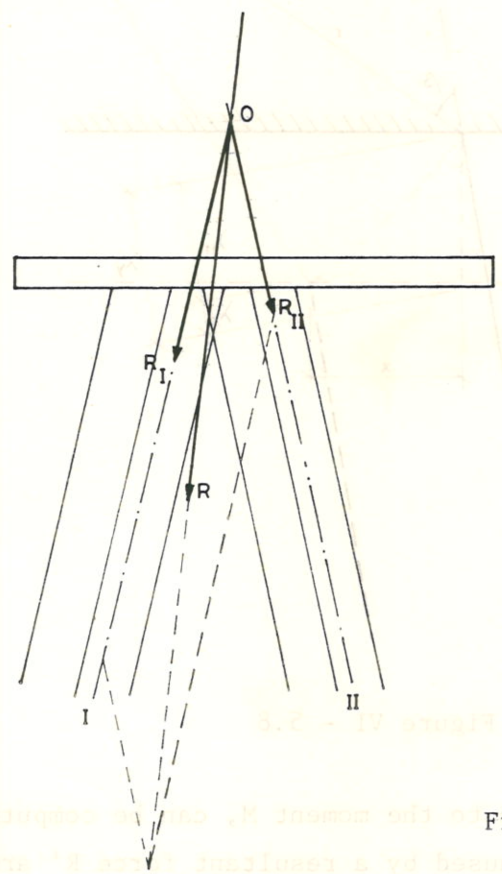


Figure VI - 5.9

So, if R is the resultant force acting on the foundation block, the pile system can be designed as follows :

Draw two lines, I and II, parallel to the intended inclination of the pile groups, whose point of intersection lies on R. Resolve R in the directions I, and II. Determine the number of piles of each direction and group them symmetrically with respect to I and II.

Every point of R can be chosen as point of intersection : it will be chosen so that a logical disposal of the piles under the structure is obtained.

Often R is not constant in direction, magnitude or position, owing to variations in water level, alterations of the loads, etc. (see

Figure VI - 5.10). Then, all resultants R have to be determined.

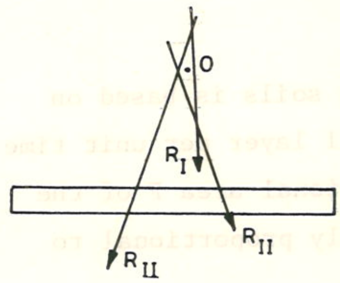


Figure VI - 5.10

Generally the points of intersection of these resultants will cover a limited area. Now the elastic centre must be chosen so that it is in this area and the pile loads have to be determined for each resultant separately. Each resultant R has to be displaced to the elastic centre, whereby a moment M is introduced. But because the distance of displacement is very short, this moment will be small.

Chapter 6. Seepage.

6.1 General.

The theory on the seepage of water through soils is based on Darcy's law. The discharge Q through a soil layer per unit time is directly proportional to the cross-sectional area F of the layer and the hydraulic head h and inversely proportional to the length of the flow-path:

$$Q = k.\ F.\ \frac{h}{l}$$

wherein k is the coefficient of permeability which dependents on the nature of the soil, the viscosity and the unit weight of the liquid and the temperature. It can vary considerably for different soils.

The discharge velocity v is defined as :

$$v = \frac{Q}{F} = k.\ \frac{h}{l}$$

The actual velocity of the water in the voids, however, is much greater because only a part of a cross-section is used for the flow of water, the rest, e.g. 60%, if the porosity is 40%, is occupied by grains. Hence the average velocity of the water is equal to the discharge velocity divided by the porosity :

$$v_s = \frac{v}{0,4}$$

The symbol v_s stands for seepage velocity.

The discharge velocity is generally used in computations. If, however, the coefficient k has to be determined in situ, for instance by injecting salt in the ground water and then measuring the time which elapses till the salt concentration in a point downstream reaches a maximum, then the seepage velocity v_s is determined.

The term $\frac{h}{l}$ is called the hydraulic gradient.

If the hydraulic gradient becomes great, Darcy's law does not hold any more, the discharge velocity is not any longer proportional to the hydraulic gradient. For coarse sands the boundary is lower than for fine soils.

Table VI - 6.1 gives an impression of some values of k.

Table VI - 6.1 Coefficient of permeability.

material	Permeability (m/s)		
Gravel	1.0	-	10^{-2}
River sand	10^{-2}	-	10^{-3}
Fine sand			10^{-4}
Sandy silt	10^{-4}	-	10^{-6}
Sandy clay	10^{-6}	-	10^{-7}
Silty clay	10^{-8}	-	10^{-9}
Peat	10^{-7}	-	10^{-9}
Clay	10^{-9}	-	10^{-12}

6.2 Seepage flow and uplift pressure under a water retaining structure.

The flow pattern of a seepage flow can be sketched as is indicated in Figure VI - 6.1. The flow pattern consists of a grid of flow lines, along which the flow is progressing and equipotential lines, along which the piezometric head is constant.

In a flow pattern, the flow lines and equipotential lines are perpendicular to each other.

The lines p in Figure VI - 6.1 represent a number of equipotential lines at distances Δs, chosen in such a way, that the potential drop between two adjacent equipotential lines is constant.

Perpendicular to these lines a number of flow lines is drawn, at distance Δn, in such a way that the discharge between each two lines per unit time and per unit width is constant = Δq. The strip between two flow lines is called a flow channel, the parts of the flow channel between two adjacent equipotential lines are

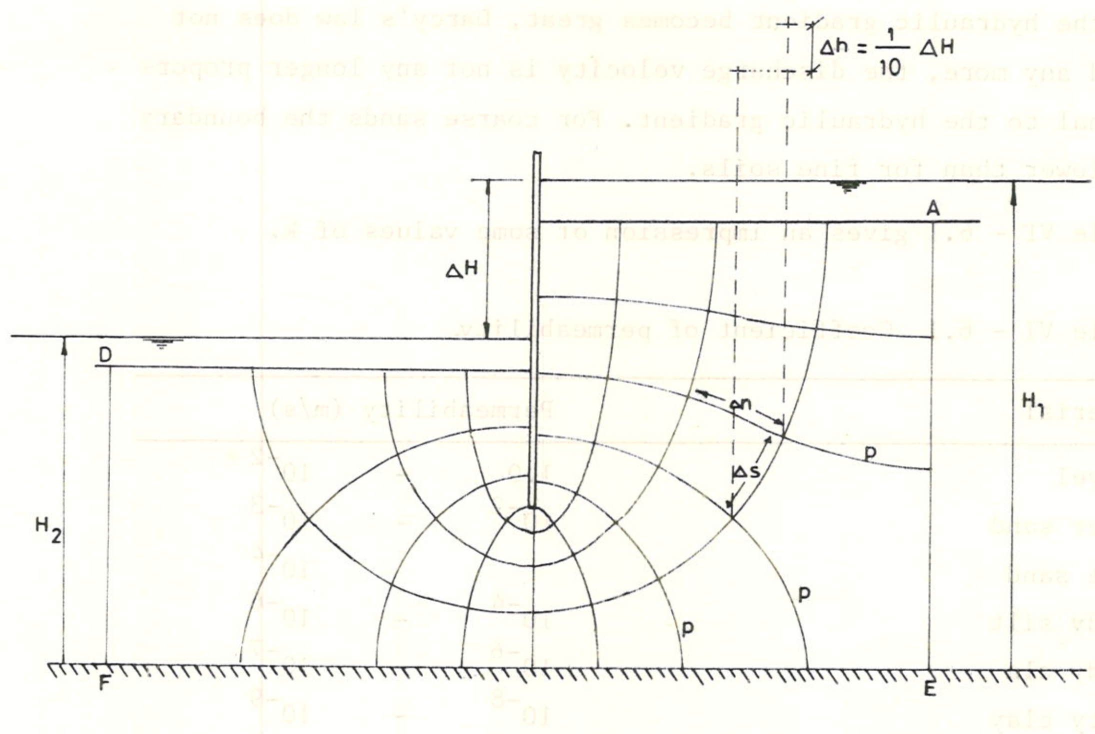


Figure VI - 6.1

called fields. The smaller Δq and Δh, the more the fields approach rectangles.

Darcy's law : $\quad q = k.\ F.\ \frac{h}{l}$ becomes :

$$\Delta q = k.\ \Delta n.\frac{\Delta h}{\Delta s} \quad \text{and} \quad \frac{\Delta s}{\Delta n} = \frac{\Delta h}{\Delta q}$$

If, in drawing the system of flow lines, the ratio $\Delta s/\Delta n$ is chosen constant, then $\Delta h/\Delta q$ will be constant as well. This means that with a constant q, the head difference over a square will be constant too.

The system of flow lines, and equipotential lines is in that case called a flow net.

In Figure VI - 6.1 the flow pattern should have been extended beyond the lines AE and DF, but the flow lines would become very long and the hydraulic gradient and thus the velocity of flow would be very small compared with the hydraulic gradient and the velocity along flow lines near the sheet piling. The fault which is made by neglecting the seepage outside the area AEFD may be neglected. The line AEFD is then a boundary flow line. Now the hydraulic boundary conditions are known and the flow net can be constructed, by trial and error, until the pattern obtained looks satisfactory. If this net consists of n squares measured along a flow line, the hydraulic head between two adjacent equipotential lines is :

$$\Delta h = \frac{H_1 - H_2}{n}$$

In each arbitrary point the piezometric level, the direction of flow and also, if the value of k is known, the velocity of the seepage may be determined. The discharge in each flow channel per unit time is :

$$q = k. \Delta h \quad , \quad (\frac{\Delta n}{\Delta s} \text{ being } 1)$$

The total discharge per unit time :

$$Q = m \times q$$

in which m is the number of squares measured along an equipotential line.

Although the leakage under the construction determined according to the above method gives a good insight in the pressure distribution under the construction, is does not give any details as to whether the leakage will cause any danger due to wash-out of soils. Moreover, the method is a two-dimensional one and any seepage along the sides of a structure are not considered.

A practical way to check whether a certain construction is safe with respect to uplift pressure and seepage-flow both under and along the sides of a structure is the empirical method of Lane.

The Lane - method is based on the following :

- The weighted creep distance is the sum of the vertical creep distances plus one - third of the horizontal creep distances.
- The weighted creep-head ratio is the weighted creep distance

or, the length of the percolation path, divided by the effective head. Lane's recommendations for the weighted-creep ratio are for fine sand or silt 8.5.

- The upward pressure can be estimated by assuming that the reduction in water head between upstream and downstream head is proportional to the weighted creep distance.

The weighted creep distance or percolation path L is defined as follows :

for flow passing under a construction : $L_u = V_v + \frac{h}{3} + 2.\ S$

for flow passing along the sides : $L_s = 0.75 \times V_h + 2.S$

in which V_v = vertical path along vertical surface

h = horizontal path along horizontal surface

S = path through earth or embankment

V_h = horizontal path along vertical surface

For Figure VI - 6.2 the following calculation has been made :

$$V_v = 3.1 + 4 + (5 \times 0.3) + 2 + 2.3 = 12.9 \text{ m}$$

$$h = 20.7 \text{ m}$$

$$L_a = 12.9 + \frac{20.7}{3} = 19.8 \text{ m}$$

head difference $H = 2$ m

creep ratio $= \frac{19.8}{2} = 9.9$

Minimum for silt is 8.5, i.e. with the same percolation path, the maximum head difference could have been 19.8 : 8.5 = 2.30 m.

The uplift pressure under the construction can now be determined as indicated in Figure VI - 6.2.

6.3 Seepage through a dam or embankment

If we assume the dam consists of pervious sand, which rests on an impermeable base, the water will seep through the dam and will flow out of the downstream slope. Let the water level at the upstream

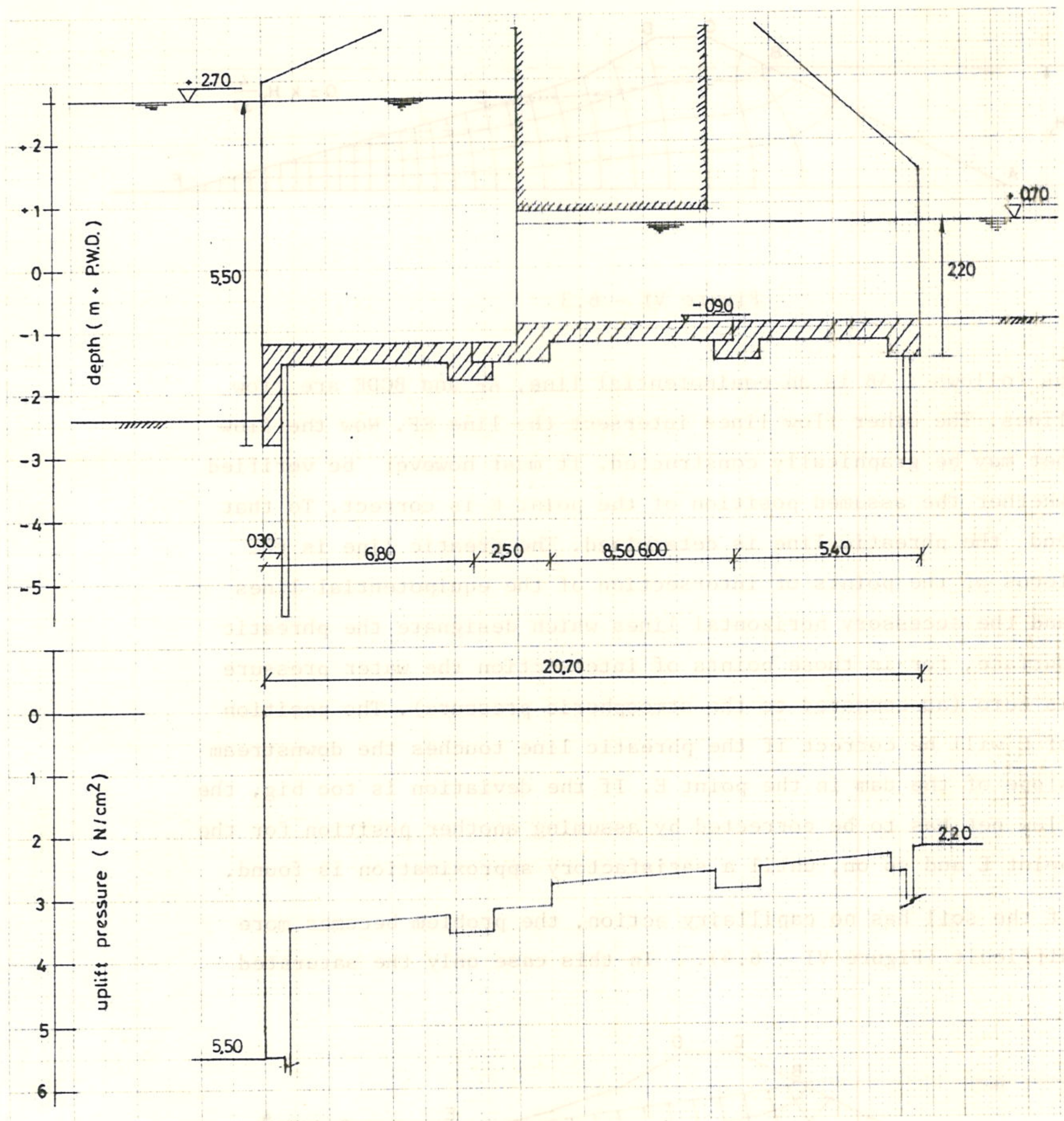


Figure VI - 6.2

side be H_1, while at the downstream side the seepage water is immediately carried off (Figure VI - 6.3).

The capillairy rise of the water in the dam is assumed to be very great, so the dam is completely saturated.

We assume the water flows out of the lower part EF of the downstream side of the dike. The hydraulic boundary conditions are

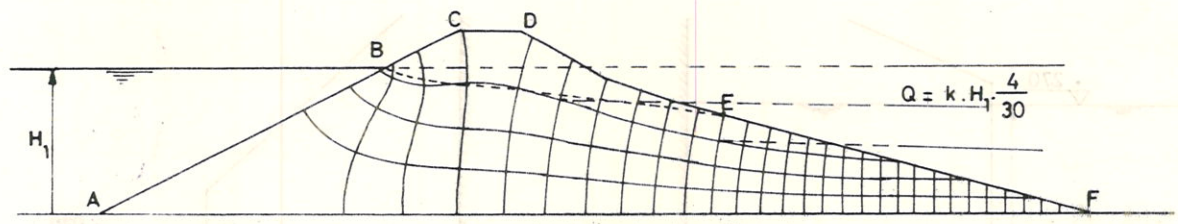


Figure VI - 6.3

as follows : AB is an equipotential line, AF and BCDE are flow lines. The other flow lines intersect the line EF. Now the flow net may be graphically constructed. It must however be verified whether the assumed position of the point E is correct. To that end the phreatic line is determined. The preatic line is the locus of the points of intersection of the equipotential lines and the accessory horizontal lines which designate the phreatic surface, for in those points of intersection the water pressure is zero (as compared to the atmospheric pressure). The position of E will be correct if the phreatic line touches the downstream slope of the dam in the point E. If the deviation is too big, the flow net has to be corrected by assuming another position for the point E and so on, until a satisfactory approximation is found.

If the soil has no capillairy action, the problem becomes more difficult (Figure VI - 6.4). In this case only the saturated

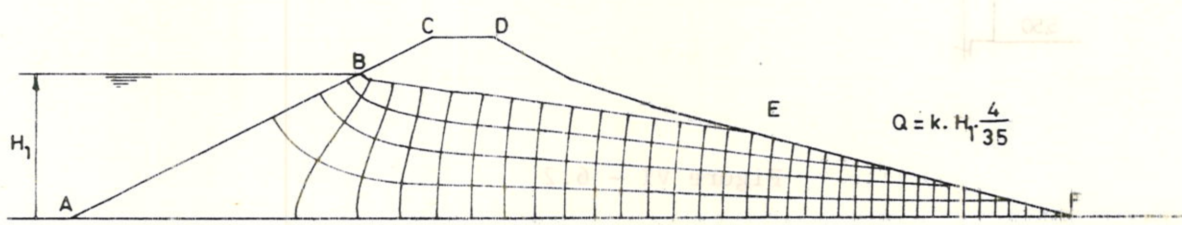


Figure VI - 6.4

lower part of the dam will take part in the seepage. The flow net is constructed by assuming an arbitrary but reasonable phreatic line BE. Except for the part of the phreatic line near B, this line will have a slightly convex upward curved shape because to the right the cross-section of the zone of seepage decreases, the velocity of the water and thus the hydraulic gradient must increase. With the hydraulic boundary conditions known, the flow net can be drawn. The position of the assumed phreatic line will be correct if the points of intersection of the equipotential lines with the phreatic line have a constant difference in height. The differences have to be equal to the potential drop Δh.

Figure VI - 6.5 gives the flow pattern if at the downstream side the water has risen to a height H_2. In this case the line GF is an equipotential line, so the flow must intersect the inner slope GF at right angles.

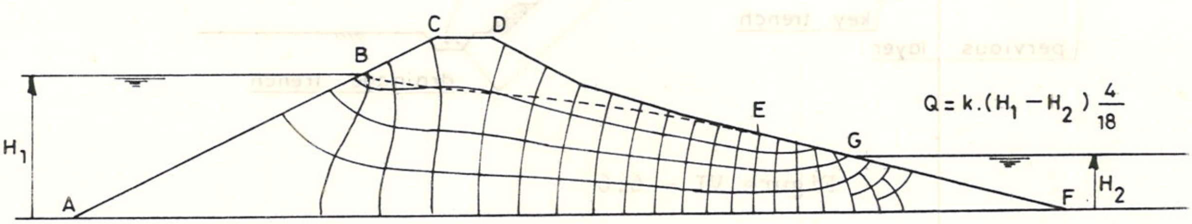


Figure VI - 6.5

6.4 Drainage of excavations.

6.4.1 Open drainage.

In general, building pits have to be drained from groundwater that is seeping in through the slopes and the floor. If the removal of this water is done via pumping from trenches in which the water is caught, the drainage system is called open. Drainage by filter wells is applied for deep excavations with the purpose to lower the groundwater head around the pit and thus reducing or stopping the seepage.

For an open drainage, the building pit has to be protected against flooding by a ring bund upto a height of the highest water to be expected during the lifetime of the open drainage. The berm between the toe of this bund and the shoulder of the excavation should be sufficiently wide to locate a drainage trench or ditch.

If a pervious layer exists at limited depth below ground level a cutoff wall or clay trench can solve a lot of problems. The slope of the building pit shall also be not too steep and the bottom width of the building pit sufficient wide to accomodate the sluice to be constructed with an additional width for a drainage trench and working space (see Figure VI - 6.6).

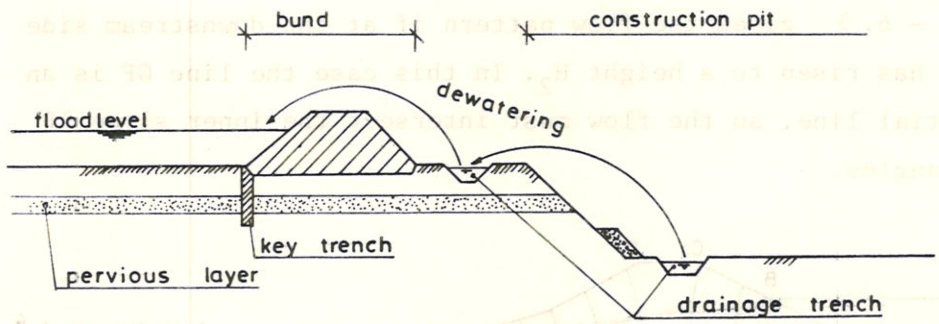


Figure VI - 6.6

Slopes of the building pit should be checked with respect to their stability against sliding. If after excavation still serious seepage is taking place so that the slope of the excavation is damaged, a filterlayer could be constructed against the toe to protect the slope against "boiling".

The excavation has to be made in layers of 0,3 - 0,5 m depth each starting with a drainage trench. Before excavating the next layer, the drainage trench has to be deepened and pumped as dry as possible.

If during the excavation of the building pit uncontrollable "boiling" effects occur additional drainage by sinking filter wells has to be applied.

6.4.2 Filter well drainage.

With a drainage by filter wells, excavations can be carried out under far more favourable circumstances than with an open drain. By placing a number of filter wells along the perimeter of the building pit it is possible to lower the surrounding phreatic level below the bottom and far within the slopes. Seepage forces do not occur and the slopes can be made steeper. The soil is first excavated to ground water level. In this position wells are installed along the slope at regular distances. These wells are connected by a conduit leading to the pump. Due to the limited suction head of the pump, a lowering of the ground water table of max. 5 m, can be affected. Then the digging can be continued down to that depth. If excavations must go deeper, then a second drainage system is placed 5 m below the first, on a berm in the excavation's slope.

In Figure VI - 6.7 the phreatic level in a cross-section through the wells (I-I), and through a cross-section between 2 wells (II-II) is demonstrated.

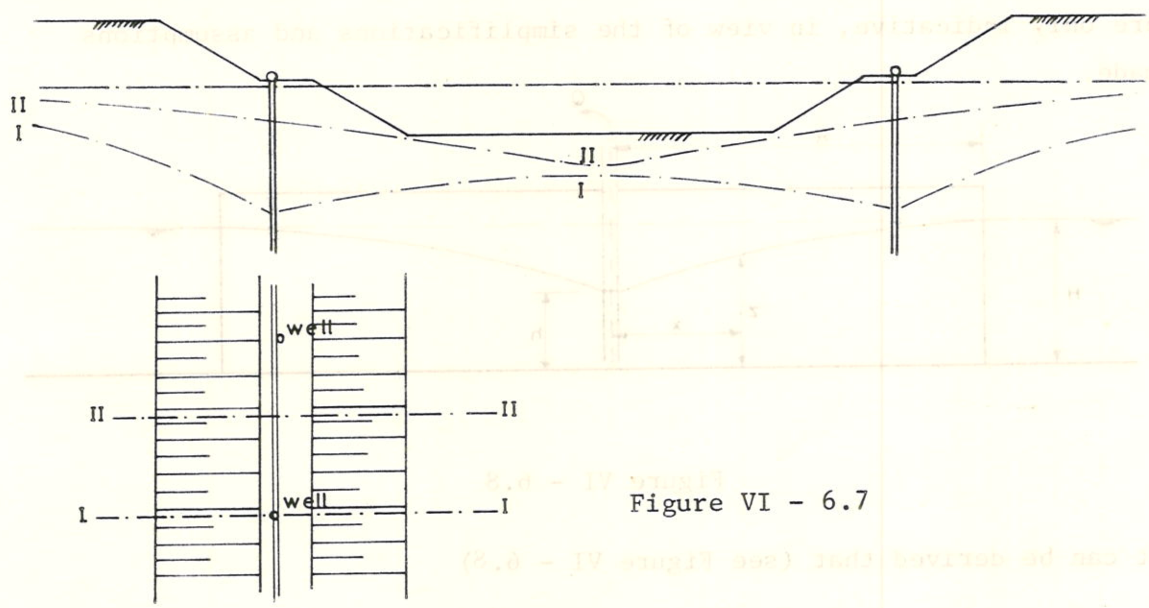


Figure VI - 6.7

6.4.3 Operation and application of filter well drainage

The operation of a filter well system is practically restricted to soils will a permeability between $k = 1 \times 10^{-4}$ m/s to $k = 1 \times 10^{-2}$ m/s. If $k < 1 \times 10^{-4}$ m/s the method is often not effective because the lowering of the ground-water table is only felt upto a short distance from the well point. In such cases drainage in the open is more effective.

A filter well itself is made by driving a borehole tube into the ground up to the required depth of the well. In this bore hole tube a so-called well point is placed. Then the borehole tube is removed while filtermaterial is dumped around the well point. In all well points a suction pipe is lowered and all suction pipes are interconnected to a headerpipe which leads to the pump.

The well point is a steel, perforated pipe surrounded by a mesh-wire filter screen. The filter material consists of coarse sand which will prevent the soil to wash into the pipe. In this way silting up of the well is avoided.

In view of pressure losses in the pipe-system and to prevent cavitation, the maximum lowering of the watertable at the well point is 3 to 5 m.

The amount of water which has to be pumped out can be calculated with formulas which are based on Darcy's law. However, the results are only indicative, in view of the simplifications and assumptions made.

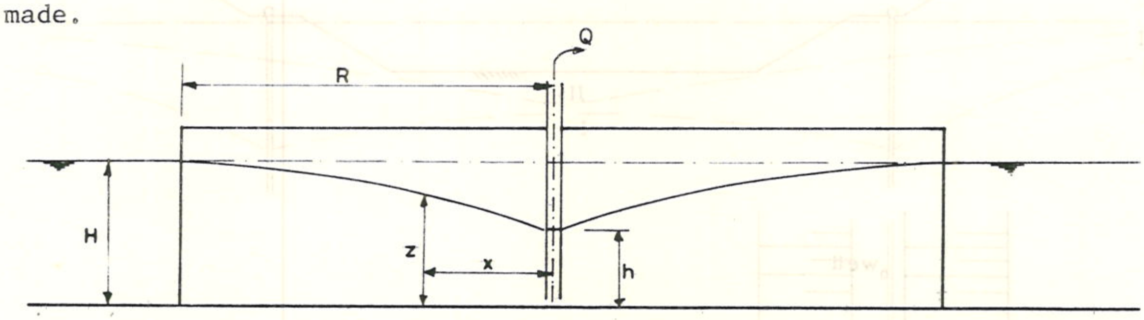


Figure VI - 6.8

It can be derived that (see Figure VI - 6.8)

$$H^2 - z^2 = \frac{Q}{\pi \cdot k} \cdot \ln \frac{R}{x}$$

R can be estimated for small drainages as :

$$R = 3000\ (H - h) \cdot \sqrt{k}$$

in which: R = distance over which lowering of groundwater table is noticable (in meter).

In deriving the formula it is assumed that the impervious layer will be at the same level as the bottom of the well. Thus it is assumed that no water will flow to from beneath this level. In fact this will not be true and therefore the calculated Q has to be increased by 20% to balance this simplification.

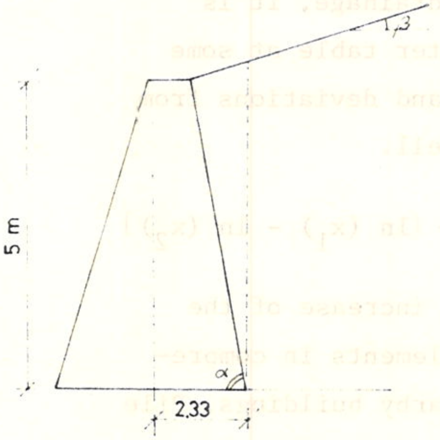
For important drainages k, R and H are mostly determined with the aid of a trial drainage.

To calculate the permeability k from a trial drainage, it is better to do the measurements of the groundwater table at some distance from the well to avoid disturbances and deviations from the formula which are greatest close to the well.

The formula is then : $(z_1)^2 - (z_2)^2 = \frac{Q}{\pi k} \cdot [\ln (x_1) - \ln (x_2)]$

Lowering of the water table with a consequent increase of the effective granular stresses will lead to settlements in compressible soils. These settlements can affect nearby buildings. Pile foundations can be exposed to negative skin friction. Sometimes it is possible to feed back the pumped groundwater at a suitable place, thus reducing any negative effects of the drainage.

 COMPARISON BETWEEN RANKINE, COULOMB AND EQUIVALENT FLUID PRESSURE METHODS



Soil

fine silty sand,

permanently above groundwater table.

$\beta = 18.4°$ (1:3)

$\alpha = 65°$

$\gamma = 19.1$ KN/m³

$\varphi = 30°$

$\delta = 2/3\ \varphi = 20°$

I : Coulomb's condition $\quad \beta \leqslant \delta \leqslant \varphi$ is met:

$18.4 \leqslant 20 \leqslant 30$

according to Coulomb:

$$k_a = 0.79$$
$$P_a = \tfrac{1}{2} \cdot \gamma . k_a . H^2$$
$$= \tfrac{1}{2} \times 19.1 \times 0.79 \times 5^2$$
$$= 188.5 \text{ KN/m'}$$
$$P_h = P_a . \cos(\alpha - \delta) = 133.3 \text{ KN/m'}$$
$$P_v = P_a . \sin(\alpha - \delta) = 133.3 \text{ KN/m'}$$

angle with horizontal:

$$\lambda = \delta + (90 - \alpha) = 45°$$
$$\eta = \lambda + \alpha - 90 = 20°$$

Coulomb condition is herewith satisfied. $(\eta \geqslant \delta)$

II: Rankine :

$$k_a = 0.398$$
$$P_a = \tfrac{1}{2} . \gamma . k_a . H^2 = \tfrac{1}{2} \times 19.1 \times 0.398 \times (5.77)^2$$
$$= 126.8 \text{ KM/m'}$$

Forces acting on vertical plane through the heel of the wall:

$$P_h = P_a . \cos\beta = 120.3 \text{ KN/m'}$$
$$P_v = P_a . \sin\beta = 40.0 \text{ KN/m'}$$

Weight of the soil wedge:

$$W = \left(\frac{2.33 \times 5}{2} + 0.903\right) \times 19.1 = 128.5 \text{ KN/m'}$$

Resultant on wall :

$$P_v = 40.0 + 128.5 = 168.5 \text{ KN/m'}$$

$$P_a = \sqrt{P_v^2 + P_h^2} = 207.0 \text{ KN/m'}$$

This force should be compared with the P_a determined by Coulomb's method.

However, Rankine's conditions are not met since :

- Angle of resultant force P_a with the horizontal is :

$$\tan \lambda = \frac{R_v}{R_h} = \frac{168.5}{120.3} = 1.6899 \longrightarrow \lambda = 54.5°$$

$$\eta = 29.5°$$

$\eta > \delta$, Rankine does not prevail.

III: Equivalent pressure method

The backfill material is fine silty sand and comes under type 3 - soil. From Figure VI - 2.4 it follows that :

$K_h = 7.87 \text{ KN/m}^2$, $P_h = 131.2 \text{ KN/m'}$

$K_v = 2.39 \text{ KN/m}^2$, $P_v = 40.0 \text{ KN/m'}$

Following the above procedure under Rankine :

$$\Sigma P_v = 168.5 \text{ KN/m'}$$

$$P_a = \sqrt{P_v^2 + P_h^2} = 213.6 \text{ KN/m}^1$$

$$\tan \lambda = \frac{16.85}{131.2} = 1.2843 \longrightarrow \lambda = 52.1° \text{ , } \eta = 27.1°$$

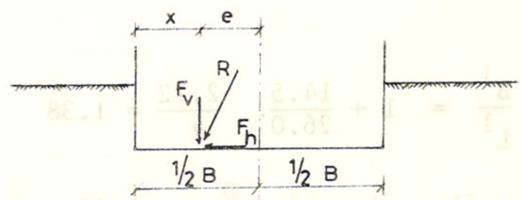
IV. SUMMARY

Method	P_h(KN/m')	P_v	P_a	λ
Coulomb	133.3	133.3	188.5	45°
Rankine	120.3	168.5	207.0	54.5°
Eq. Fl. Pr.	132.1	68.5	213.6	52.1°

Although Coulomb's method yields the lowest resultant force, the over-turning force P_h is the highest and the stabilizing force P_v is the smallest, thus giving a more critical conditions than the Equivalent Fluid Pressure method.

If comparing Rankine's method with the Equivalent Fluid Pressure method, it could be shown that the two methods would give almost the same result, if φ were 28° instead of 30°. If field tests had indicated that $\varphi = 22°$ and the Equivalent Fluid Pressure method had been used, the design would have been in error on the danger side.

BEARING CAPACITY CALCULATION



$F_v = 78000\ N = 78\ KN$

$F_h = 34\ KN$

$M = 106\ KNm$

$B = 4\ m$

Eccentricity of resulting :

$$x = \frac{M}{F_v} = \frac{106}{78} = 1.36\ m$$

$$e = \tfrac{1}{2} B - x = 2.00 - 1.36 = 0.64\ m$$

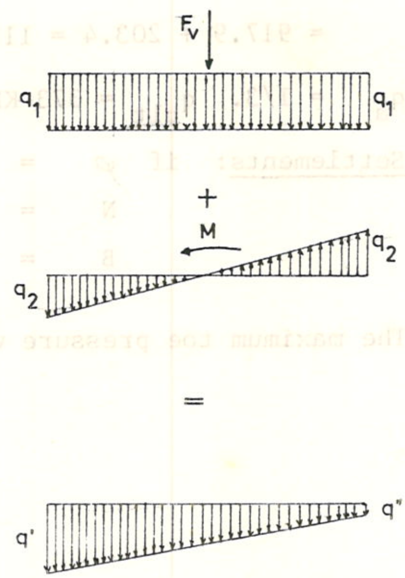
Foundation reaction is :

$$q_1 = \frac{Fv}{B} = \frac{78}{4} = 19.5\ KN/m^1$$

$$M = 78 \times 0.64 = 49.92\ KN/m$$

$$= 2\left(\frac{q_2 \times \tfrac{1}{2}B}{2} \times 2/3 \times \tfrac{1}{2} B\right)$$

$$q_2 = 18.72\ KN/m^1$$



$q' = q_1 + q_2 = 38.22\ KN/m'$ || $q'' = q_1 - q_2 = 0.78\ KN/m'$

effective base width:

$$B^1 = B - 2.e = 4 - (2 \times 0.64) = 2.72\ m$$

The ultimate bearing capacity can then be calculated:

$$q_{ult} = c.\ N_c .\ s_c .\ d_c + \bar{q}.\ N_q .\ s_q .\ d_q + \tfrac{1}{2}\gamma .\ B'.N_\gamma .s_\gamma$$

let us assume that c = 0

$$\bar{q} = D \times \gamma = 2 \times 18.8 \text{ KN/m}^2$$

$$\varphi = 28° : N_q = 14.5$$

$$s_q = 1 + \frac{N_q}{N_c} \cdot \frac{B^1}{L^1} = 1 + \frac{14.5}{26.0} \cdot \frac{2.72}{4} = 1.38$$

$$d_q = 1 + 2 \tan\varphi \, (1 - \sin\varphi)^2 \cdot \frac{D}{B^1} = 1.22$$

$$N_\gamma = 10.9$$

$$s_\gamma = 1 - 0.4 \frac{B^1}{L^1} = 1 - 0.4 \frac{2.72}{4} = 0.73$$

$$q_{ult} = (2 \times 18.8 \times 14.5 \times 1.38 \times 1.22) + (\tfrac{1}{2} \times 18.8 \times 2.72 \times 10.9 \times 0.73)$$

$$= 917.9 + 203.4 = 1121 \text{ KN/m}^2$$

$$q_a = 1/3. \; q_{ult} = 373 \text{ KN/m}^2$$

<u>Settlements</u>: if $\varphi = 28°$

$N = 10$ $\quad$ $q_a = 3 \text{ ksf} = 143 \text{ KN/m}^2$

$B = 4$ m

The maximum toe pressure was calculated to be 38.2 KM/m^2.

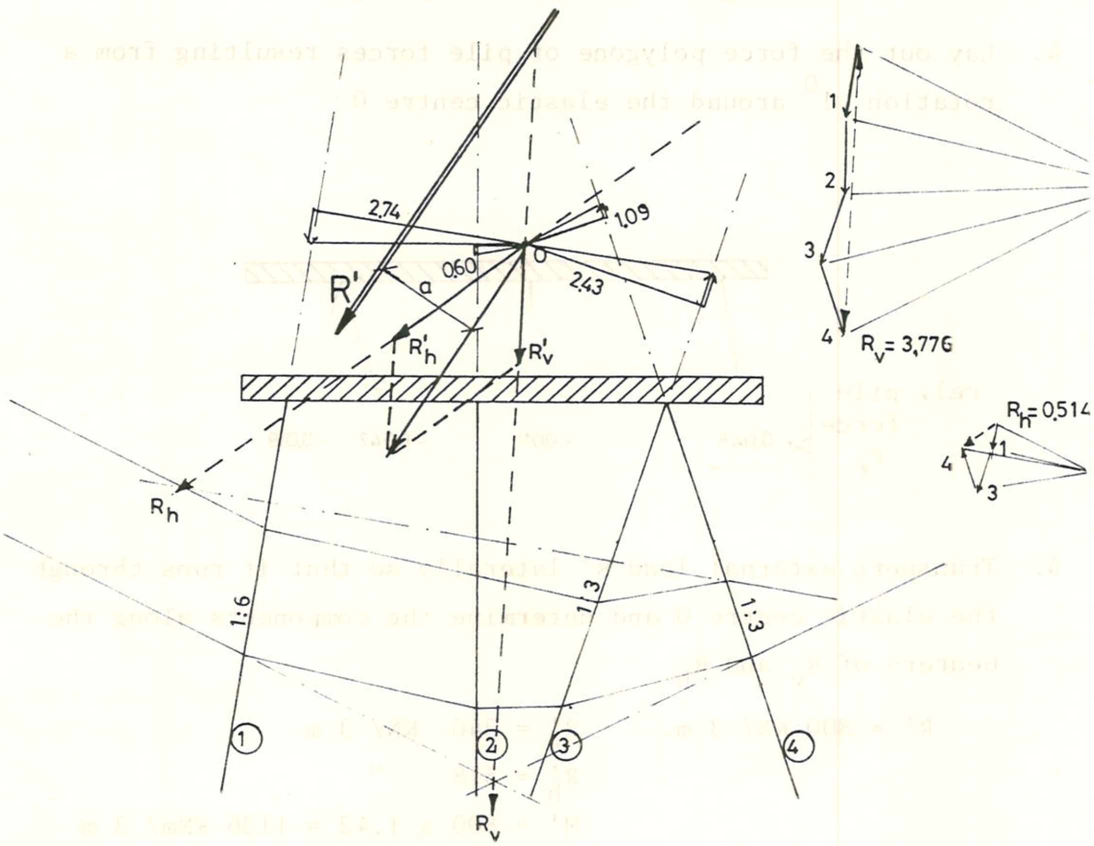
---

1 ksf = 1 Kip/sq. ft. = 47.9 KM/m^2

γ_w = 62.6 lbs./cuft. = 9,807 KN/m^2

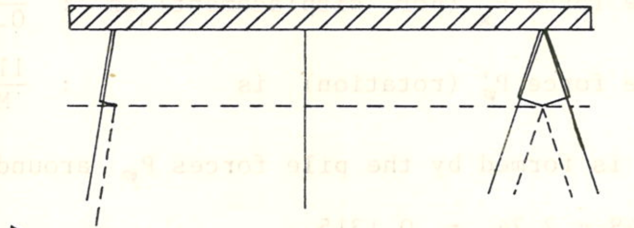
γ_{soil} = 90 - 130 lbs/cuft. = 14 - 21 KN/m^3

N.B. The sketched pile system is not necessarily the most suitable for the given external load.

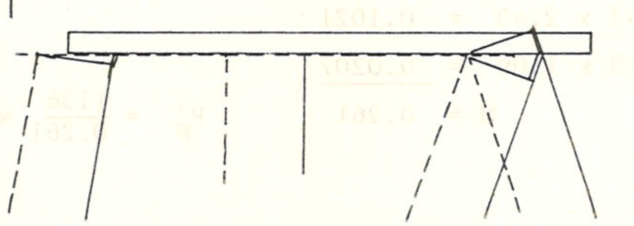


1. Determine relative pile forces

assume vertical displacement of 1 cm and pile elasticity c = 1.



rel. pile force P_y + 0,986 + 1,000 + 0,949 + 0,949



rel. pile force P_x + 0,164 0 + 0,316 - 0,316