CONCEPT OF PORE PRESSURE AS APPLIED TO EARTH DAMS

DISSERTATION
SUBMITTED IN PART FULFILMENT OF THE REQUIREMENT FOR THE
M. E. (WRD) DEGREE
OF THE

UNIVERSITY OF ROORKEE

T. C. RAMACHANDRA RAO, B.E.(HONS.), P. G. DIP., A. M. I. E.

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C E R T I F I C A T E.

Certified that this Dissertation titled "CONCEPT OF PORE PRESSURE AS APPLIED TO EARTH DAMS" is prepared by Shri. T.C. RAMACHANDRA RAO under my supervision during a period of nine months from October 1966 to June 1967 for the award of M.E. (WRD) DEGREE of the UNIVERSITY OF ROORKEE.

Certified further that according to my knowledge and belief this Dissertation, in part or full, has not been submitted by him for the award of any other Degree of Diplo a of any University.

BANGALORE,
17th June 1967.

(L.T. AMINBHAVI)
CHIEF ENGINEER, PWD.,
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A C K N O W L E D G E M E N T.

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I am thankful to DR. RAGHAVENDRA RAO, Librarian of the Indian Institute of Science, Bangalore, for permitting to make use of the Institute's Library and gratefully acknowledge the various sources (listed at the end) from which the material for this work is drawn.

T.C. RAMACHANDRA RAO.

C O N T E N T S.

CONCEPT OF PORE PRESSURE AS APPLIED TO EARTH DAMS.

SYNOPSIS.

(This dissertation aims at a systematic presentation and discussion of relevant published matter from various sources. These sources are acknowledged at the end. The author does not claim to have added any new findings as no original research is carried out).

The earth dam is most commonly designed for safety against sliding by the slip circle analysis by the Standard Method of Slices. The slide failure is a shear failure and the unit shear strength of the soil at various points along the sliding circle is not of a constant value but is a function of the weight of the fill above each point as is apparent from the equation $S = \sum C' + \sum (N-U) \tan \phi'$, where N the normal component of the weight of the slice is different for each point and U the pore pressure on the base of the slice acting against N is also different from slice to slice. This pore pressure has its seat in the water and/or air in the pores of the soil skeleton.

N is called the total normal stress and N-U is the effective normal stress for each slice.

The critical stages in the life time of an earth dam during which the stability of its slopes is likely to be the least are (i) the rapid draw down state for the upstream slope, (ii) the full reservoir state for the down stream slope and (iii) the during-construction state for both the slopes. In the first two states the soil below the top seepage line is in the saturated condition and the pore pressures at different points in the fill can be found out from the flow net. Approximate methods are however available for easier application. In the third state the soil is in the unsaturated condition, and herein, the concept of pore-pressure is not very clear even to this day. Estimation of pore pressures in this case may be done by theoretical methods making use of Boyle's and Henry's Laws or by making a rough approximation based on observed pore pressures in laboratory test-samples by simulating field conditions or based on pore pressures observed in other dams in comparable instances. Different methods of controlling pore pressures by providing drainage, by controlling construction moisture, etc., are available.

Calculating the stability by taking the (N-U) forces is called the effective stress method of analysis and the corresponding shear parameters C' & ϕ' are called the effective stress parameters. Alternatively, the stability can be calculated by taking the total N forces, without deducting the pore pressure U and this method is called the total stress method of analysis.

The corresponding shear parameters C & Ø used for this method are called the total stress parameters. The effective stress parameters are found out in the laboratory by consolidated undrained test with measurements of pore pressure or by drained test. The total stress parameters are found out by unconsolidated undrained test for the during-construction state and by consolidated undrained test for the draw down and steady seepage states simulating the field conditions regarding water content, etc.

For the advancement of the knowledge about pore pressures, for obtaining advance knowledge regarding developing adverse pore pressures and thereby to take timely action for averting possible failures or damages of the dam, and for such other reasons pore pressure measuring devices called piezometers are installed in earth dams. Different types of hydraulic and electric piezometers have been developed overcoming the shortcomings of the earlier types. Systematic installation of good piezometers, efficient maintenance and reading of the piezometers, and careful analytical studies of the observed readings is leading to a clearer concept of the phenomenon of pore pressure helping evolution of safer and more and more economical designs of earth dams.

CHAPTER I.

INTRODUCTORY.

1.1. Pore Pressure and design of embankment slopes.

The phenomenon of pore pressure has an important bearing in the design of earth dam slopes against failure by slide.

The method of design of an earth dam is fundamentally different from that of any other engineering structure like a steel beam or a concrete dam. For the design of a steel beam a section is assumed and its adequacy to take up the given loads is checked by what is known as the UNIT STRESS METHOD. In this method the maximum tensile stress, compressive stress and shear stress likely to develop are computed and compared with the intrinsic strength of the material (i.e. steel in this case) in tension, in compression and in shear respectively to see that they are within limits. The intrinsic tensile strength, compressive strength and shear strength are constant for the material like steel and are independent of the stresses the material is subjected to.

In an earth dam, the most common type of structural failure will be not by tension, nor by compression but by shear due to sliding along a curved plane ~~and~~ the shape of which approximates a cylindrical arc. This sliding is similar to the sliding of a body under its own weight along an inclined plane shown in fig. 1.1(a). If W is the weight of the body, N and T the components of the weight normal and tangential to the plane and ϕ the angle of friction between the body and the plane then the force causing slide is T and the force resisting slide is

$$S = N \tan \phi \quad \ldots \quad \ldots \quad (1.1)$$

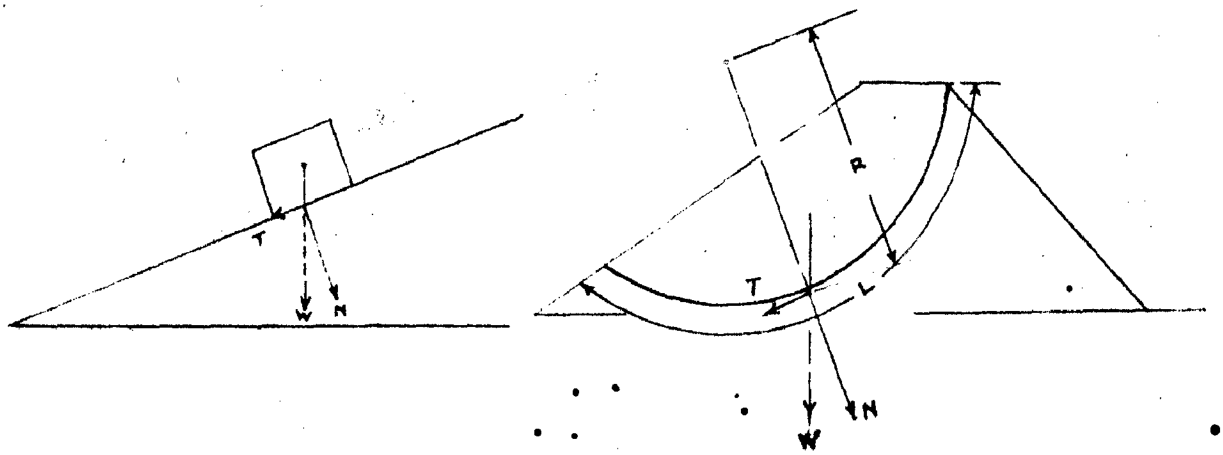


FIG. 1.1(a): BODY SLIDING ON AN INCLINED PLANE.

FIG. 1.1(b): EARTH MASS SLIDING ON A CYLINDRICAL ARC.

Similar is the case of the sliding of the earth slope in fig. 1.1(b). The body in the first case is analogous to the soil mass within the cylindrical arc and the inclined plane is analogous to the cylindrical surface. In the second case however the force resisting slide is ~~[illegible]~~

$$S = C + N \tan \phi \quad \ldots \quad \ldots \quad (1.2)$$

where C is the cohesion between the sliding mass and the mass below. This C appears only in the case of cohesive soils and not in cohesionless soils. The factor of safety (F.S.) against slide for the assumed failure surface is equal to the ratio of the moment resisting slide (C + N tan ϕ) x R divided by the moment causing slide T x R where R is the radius of the failure circle, i.e.

$$F.S. = \frac{(C + N \tan \phi) \times R}{T \times R}$$

$$= \frac{C + N \tan \phi}{T} \qquad \ldots \qquad \ldots \qquad (1.3)$$

For various assumed failure surfaces the F.S. is thus calculated and the failure surface which gives the least F.S. is called the critical surface. This F.S. should be more than a particular norm for a satisfactory design of the earth slope. This method of design is called the LIMIT DESIGN METHOD. Herein it is seen that the shear strength of the soil which is equal to C + N tan ∅ is not a constant as in the earlier example of steel but depends upon the stresses the soil is subjected to. The influence of this stress is represented in the N part of the expression C + N tan ∅. Also neither C nor ∅ are constant soil properties but depend upon various factors like amount of compaction, degree of saturation, etc.

Whatever be the amount of compaction, pores will always be present in a soil mass. When these pores are filled with water, the failure surface cuts partly the soil skeleton and partly the pore water and hence the total stress N, normal to this surface, is partly carried by the soil skeleton and partly by the pore water. The stress carried by the soil skeleton may be designated as N' and that carried by pore water as U. As can be seen from the fact that the surfaces of all bodies of water are level and cannot stand at an incline, water has no strength in shear. Hence the U component of N does not contribute towards the development of any shear strength. Only the intergranular stress N' is effective in producing the shear strength and hence it is called the effective stress. The shear strength S will now have to be expressed as

$$S = C' + N' \tan \phi' \quad \ldots \quad \ldots \quad (1.4)$$

$$= C' + (N - U) \tan \phi' \quad \ldots \quad (1.5)$$

where C' is called the effective cohesion and ϕ' is called the effective angle of intergranular friction both corresponding to the effective stress (N - U).

From equation (1.5) it is apparent that the pore pressure U affects the shear strength of the soil which in turn affects the stability of the earth dam slope.

In this limit design method of stability analysis there are two variations. One is by making use of equation (1.2) wherein the total stress parameters C & ϕ and the total stress N are used. This is called the TOTAL STRESS METHOD. The other is by making use of equations (1.3) and (1.4) wherein the effective stress parameters C' and ϕ' and the effective stress N' are used. This is called the EFFECTIVE STRESS METHOD. A knowledge of the magnitude of the pore pressures in the dam is necessary for the effective stress method only and not for the total stress method. For the total stress method, the total stress parameters C & ϕ are required to be found by triaxial shear tests in the laboratory on a specimen simulating the field conditions of the embankment regarding compaction, moisture content and drainage conditions. It is presumed here that the pore pressures developed in the laboratory sample are equal to those which would develop in the embankment at failure. Pore pressure measurements are not necessary in this case. For the effective stress method knowledge of the magnitude of the pore pressure is necessary because the effective stress N' is to be found out by deducting the pore pressure U from the total stress N.

1.2. Historical background.

The action of pore water pressure in fine grained soils was first discovered by Terzaghi in 1923 giving the equation, effective stress is equal to the total stress minus the pore water pressure, i.e., using symbols

$$\sigma' = \sigma - U_w \quad \ldots \quad (1.6)$$

This marks the beginning of the modern phase of the subject matter of pore pressure. Preceding work by such great engineers as Coulomb, Collin, Rankine, Resal, Bell and Forchemier was of limited validity owing to the absence of this fundamental unifying principle.

In 1886 Reynolds(61) had demonstrated that a rubber bag full of saturated sand was subjected to negative pore pressures when distorted. Deacon and Hawkeslay(35) had pointed out in connection with Vyrnwy dam that the pressures set up under a dam by blocking an almost imperceptible flow of water was a major factor for stability. Uplift pressures were thus vaguely recognised.

Experimental proof of the principle of effective stress was carried out by Rendulic(62) under Terzaghi's direction in Vienna in 1933. Investigations into the magnitude of pore pressure in partly saturated soils were begun by the United States

Bureau of Reclamation and included both theoretical and experimental work (Brahtz, Braggeman and Zanger[25] 1939) and field measurements (Walker and Daehn[39][59] 1948). Recent theoretical and laboratory works of Hilf[26][51] (USBR), Skempton[17] (Imperial College, London) and Bishop[29][44][49][50] on unsaturated soils have enriched the knowledge regarding pore pressure.

1.3. The structure of soil mass.

A soil mass, whatever be the amount of compaction, is a porous body, i.e. it is made up of a soil skeleton of solid soil particles and interconnected pores between the soil particles as shown in fig. 1.2. The soil particles are generally mineral grains of various sizes and shapes occuring in every conceivable arrangement~~s~~. The pores are filled with a fluid, generally air or water or both air and water. When the pores in the soil skeleton are filled with either only air or only water the soil mass is said to be in a two phase system of soil and water respectively. When the pores in the soil skeleton are filled with both air and water the mass is said to be in a three phase system of soil, ~~xxx~~ air and water. Another classification of the conditions of the soil mass will be as dry, moist and saturated according as the pores are filled with only air, air and water, and only water respectively.

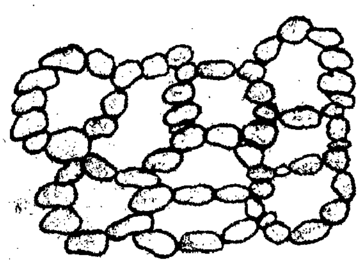


FIG. 1.2: SOIL GRAINS AND PORES

1.4. The spring analogy.

The nature of pore pressure can best be understood by the spring analogy first used by Terzaghi.

In fig. 1.3(a) to (c) are shown the various lengths assumed by a spring under various loads. In the lower row of sketches the same spring is assumed to be immersed in a water tight cylinder filled with water. In (g) a frictionless but tightly fitting piston has been placed in the cylinder and loaded

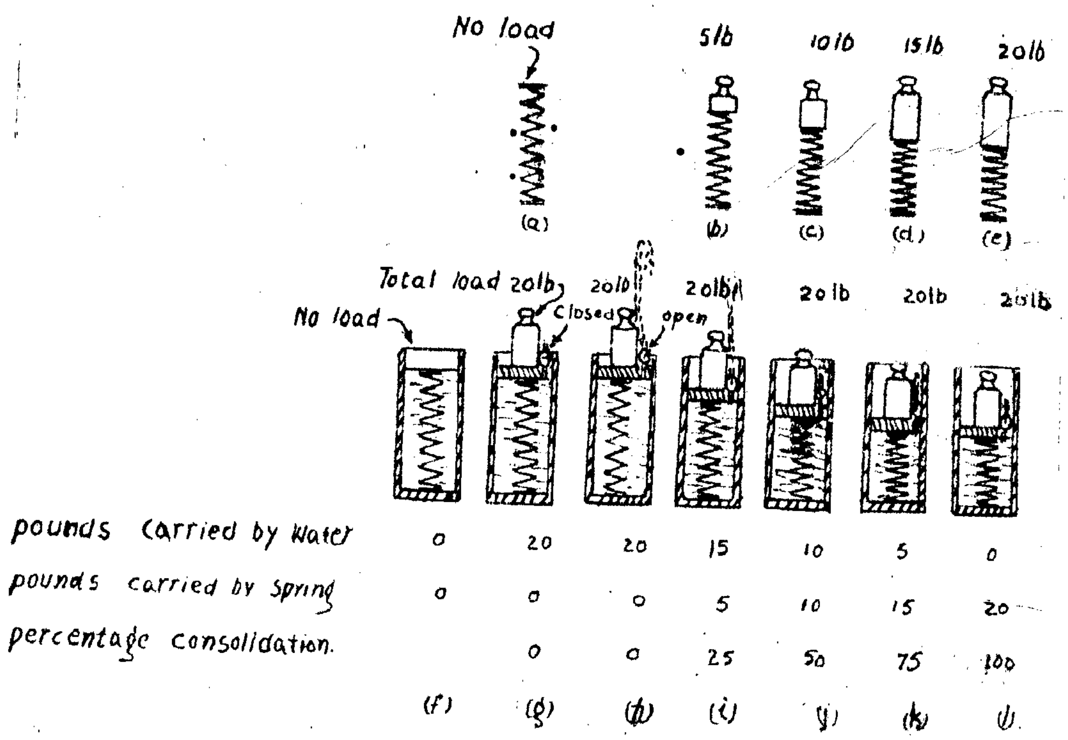


Figure 1.3 :- The spring analogy.

with a total load of 20 lbs. The piston is provided with a stop cock which is assumed to be closed so that no water can escape. Under the 20 lbs. load the spring would tend to assume the length shown at (e), but it cannot do so unless the piston descends, and the piston cannot descend because the water cannot escape. The compressibility of the spring is assumed to be so great that the strains produced in the water and in the walls of the cylinder are negligible in comparison. Consequently the spring cannot take any of the superimposed load and the water must carry it all.

Now suppose the stop cock is opened, Sketch (h) represents conditions immediately afterward. The water spurts out on account of the pressure to which it is subjected. In the first instance the pressure conditions are unchanged as noted below the figure.

As the water escapes the piston sinks lower and lower compressing the spring. At (i) the length of the spring is the same as at (b). The spring consequently must be carrying 5 lbs., and the water 15 lbs. At (j) the length is the same as at (c), at (k) the same as at (d), and at (1) the same as at (e). The resulting pressure conditions are as indicated in the sketches.

In the mechanical analogy presented above the spring represents the compressible soil skeleton of a mass of saturated soil and the water in the cylinder represents the pore water in the voids of the soil. The stop cock opening is analogous to the permeability of the soil and the compressibility of the spring to the compressibility of the soil. The pressure carried by the water in the cylinder is snalogous to the pore water pressure.

1.5. Different names for describing pore pressure.

Pore pressure is described by different names all basically referring to the same phenomenon but with slightly different shades of meaning. It is called pore water pressure when it is solely due to water; pore air pressure when due to air; hydrostatic pressure when the pore water is static; uplift pressure when it is viewed as acting vertically upwards against the gravity weight of the soil particles; hydrodynamic excess pressure, seepage pressure or viscous drag when the pore water is not static but is seeping through the pores and trying to drag the soil particles along with it. It is also called neutral pressure as it is not effective in contributing to the shearing strength etc., of the soil.

Considering atmospheric pressure as the reference datum, all positive figures of pore pressures are pressures above atmospheric pressure and negative figures of pore pressures are pressures below atmospheric pressure. This negative pressure in most of the soils is due to the surface tension exerted by the menisci of capillary water in the minute pores of the soil. It is therefore called capillary pressure, capillary suction or capillary tension. Due to this suction and its negative nature the capillary pressure increases effective stress σ' (as is apparent from equation (1.6) in contrast to the positive pore pressure which decreases the effective stress. The capillary negative pressure is reduced to zero and further on changes to positive pore pressure as the capillary menisci are broken by the addition of more and more water.

1.6. Effective pressure in two phase system and three phase system.

For saturated soils in which the voids are completely filled with water Terzaghi showed experimentally that

$$\sigma' = \sigma - U_w \quad \ldots \quad \ldots \quad (1.6)$$

as mentioned earlier. Subsequent work has confirmed this expression with a sufficiently high degree of accuracy for engineering purposes.

For partially saturated soils however tests carried out chiefly during the past decade have shown that the equation can be appreciably in error. The more general expression for effective stress will be as first suggested by Bishop in 1955,

$$\sigma' = \sigma - [U_a - x\,(U_a - U_w)] \quad \ldots \quad (1.7)$$

where x is a parameter relating to the degree of saturation and equalling unity when the soil is fully saturated. In this latte case Bishop's equation then becomes identical with Terzaghi's.

These equations can be used in all or certainly in mos practical problems. However, it would seem improbable that an expression of the form $\sigma' = \sigma - U_w$ is strictly true. And indeed when one turns to other porous materials such as concrete or lime stone, this equation is not always adequate. We may therefore anticipate that even for fully saturated materials the general expression for effective stress is more complex and that Terzaghi's equation has the status of an excellent approximation in the special case of saturated soils. The common opinion is that the effective stress σ' being the intergranular stress acting between the particles comprising the porous skeleton this stress is

$$\sigma' = \sigma - (1-a)\,U_w \quad \ldots \quad \ldots \quad (1.8)$$

where a is the area of water-tight contact between the particles per unit gross area of the material. Now there can be little doubt that a is very nearly equal to zero in soils because the contact surfaces will not be so tight as to preclude water pressure and this expression is therefore virtually identical with Terzaghi's equation.

Hilf gives the following equation for effective stress in unsaturated soils.

$$\sigma' = \sigma - U \qquad \ldots \qquad \ldots \qquad (1.9)$$

$$\text{i.e.} \quad \sigma' = \sigma - (U_a + U_c) \qquad \ldots \qquad (1.10)$$

where U_a is the pore air pressure and U_c is the pore water capillary pressure (or capillary tension) for the existing curvatures of the menisci. For unsaturated soils U_c is always negative and therefore U is less than U_a.

In routine discussions the effective stress which controls the shear strength, volume change, etc., is taken to be the intergranular stress. With this idea both the names intergranular stress and effective stress are considered as describing the same phenomenon. But recent high pressure consolidation tests on lead shot have proved (~~35~~) (17) that the stress controlling volume change is by no means equal to the intergranular stress; whilst triaxial compression tests on marble indicate that the effective stress controlling shear strength changes in rocks is also not equal to intergranular stress. Thus it is desirable to examine the physics of effective stress more closely in the hope of obtaining a theory which is reasonably consistent with all the available experimental data.

1.7. Interpretation of cohesion as being an effect of capillary Pressure and intrinsic pressure.

We have seen in para 1.1 that the shear strength of soils is $S = C + N \tan \phi$ where C is the cohesion part of the shear strength and $N \tan \phi$ is the friction part. And,

$$S = C$$

when the applied pressure N is zero. In moist soils this no-load or no-pressure shear strength is partly due to the negative capillary pressure U_c. It is also partly due to the pre-load consolidation pressure which was applied and removed. This pre-load consolidation pressure is called as intrinsic pressure P_i by Taylor. Therefore, cohesion may also be interpreted as frictional strength only as in the following equation.

$$C = (U_c + p_i) \tan \phi \qquad \ldots \qquad (1.11)$$

That part of the cohesion which is due to capillary water is called "apparent cohesion". With increase in moisture content to the point of saturation, this apparent cohesion reduces to zero. The other part of the cohesion which is due to intrinsic pressure is called the "intrinsic cohesion". The total cohesion thus depends on the moisture content and on the intrinsic pressure.

1.8. Stability analysis of the embankment slope by the method of slices.

The basic principles of the stability calculations of embankment slopes by the slip circle method are briefly touched in para 1.1. Further details are described here before concluding this chapter leading to the further chapters which deal with the determination of pore pressures for stability calculations under the different conditions to which the embankment will be subjected during its life time.

Let it be assumed that the circular arc in fig. 1.4 is an arbitrarily chosen trial arc through which steady seepage is occurring.

The sliding mass of earth ABCD cannot be tested for stability in a single step considering the total weight of the mass and putting

$$F.S. = \frac{CL + (N-U)\tan\phi'}{T}$$

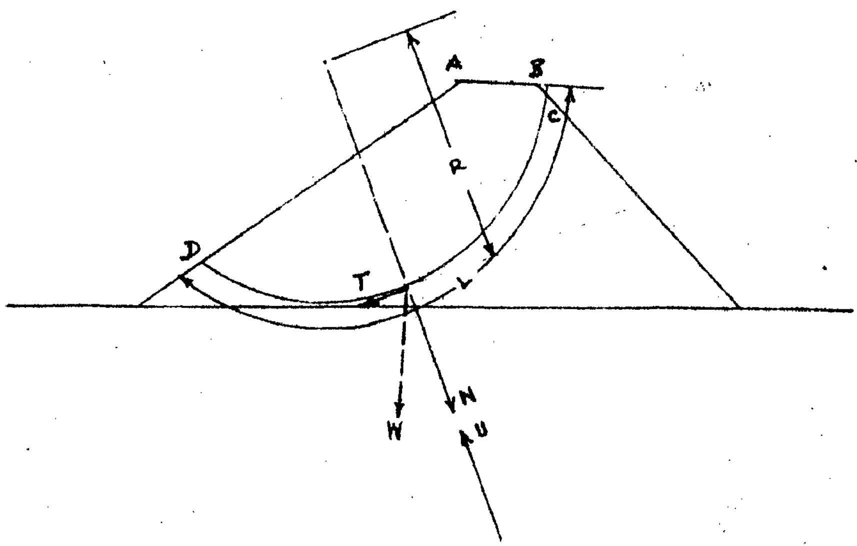


FIG. 1·4 : FORCES ON SLIDING SEGMENT.

because the shear resistance along the length of the arc DC is not constant throughout but varies with the normal pressure across the slip surface which depends on the unit weights of the materials in different zones in the segment ABCD and on the C &Ø values of the materials in the different reaches of the arc DC. The segment is therefore divided into arbitrary number of vertical slices not necessarily of equal width but chosen intelligently so as to coincide with the intersection of the surfaces with the various zones. Fig. 1.5(a) shows the slices in a homogeneous embankment.

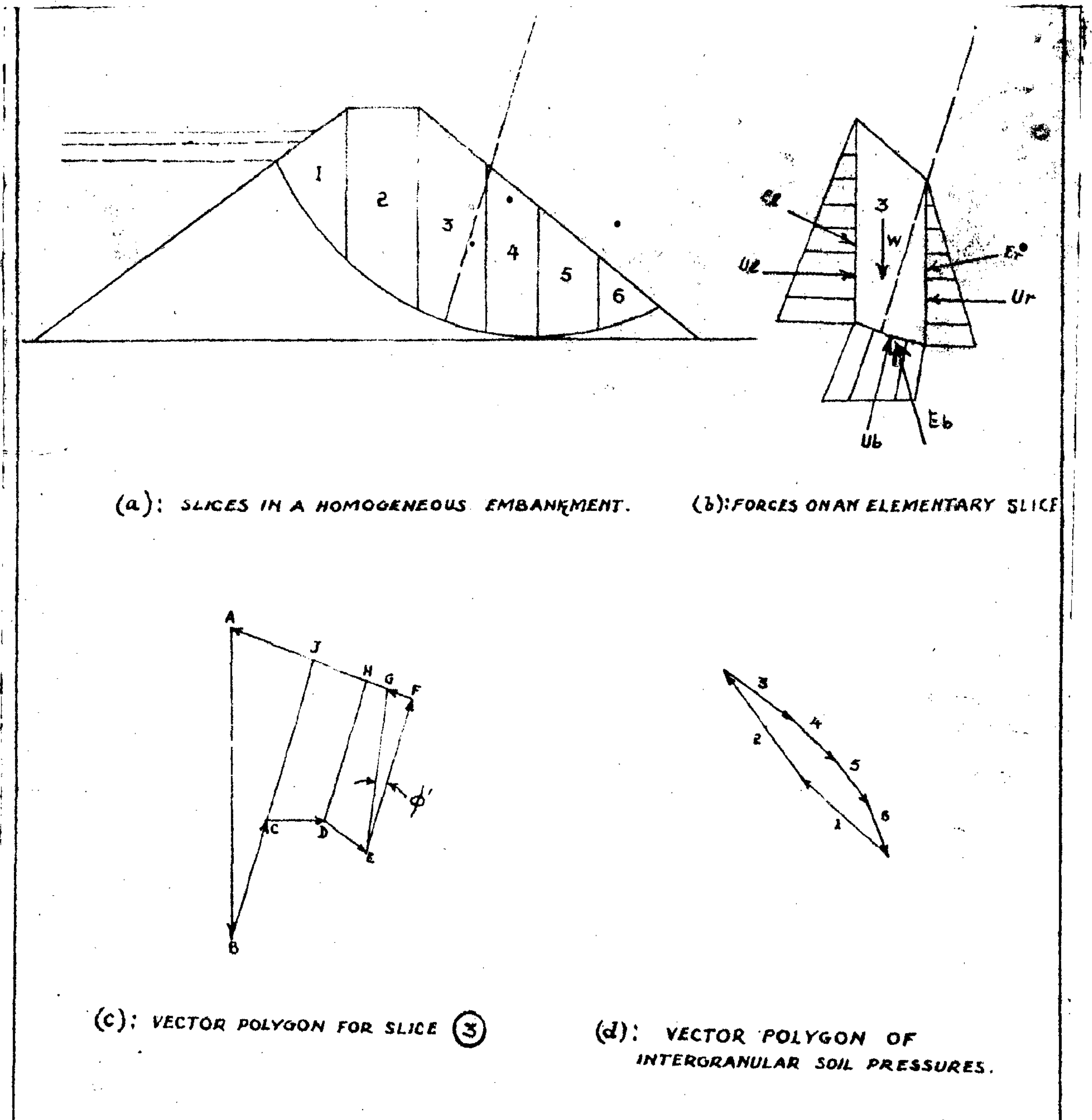


FIG. 1·5 : STABILITY ANALYSIS BY THE SLICES METHOD .

Any one of these elementary slices is in equilibrium under the action of seven forces shown in fig. 1.5(b). W is the total weight of the slice, i.e. the saturated weight. The forces U_l, U_r and U_b, are the resultant pore water pressures on the left side, right side, and bottom of the slice respectively. Their directions are normal to the surfaces on which they act. They can be determined by flow net and other methods. The forces E_l and E_r are the resultants of inter-granular soil pressures (or earth pressures) on the left side and right side of the slice respectively. E_l and E_r are indeterminate in magnitude and direction but reasonable values can be determined for them by a trial and error procedure.

In fig. 1.5(c) the vector polygon for any one slice is shown. The weight W is represented by the vector AB. U_b is represented by BC. CD is the resultant of U_l and U_r. DE is the resultant of E_l and E_r. Its magnitude and direction are arbitrarily chosen as stated above such that the combined vector diagram of DE for all the slices as shown in fig. 1.5(d) makes a closed polygon. Their vector EA is the required force E_b to keep the slice stable, in equilibrium. This force can be resolved into two components EF and FA normal and tangential to the arc respectively. The tangential component may be considered as made up of two components FG and GA. FG, the frictional part being equal to EF tan ϕ and GA the cohesion part being equal to C' x 1. The tangential component of the weight is AJ. This is the actuating force or the force causing slide.

In the above analysis the vector DE is the one which is indeterminate. If E_l and E_r are assumed to be equal and oppo-

-site DE will be zero. With this assumption the vector E_b will be represented by DA and its normal component will be DH. It can be verified by working out actual problems that with this assumption the sum of the normal components ΣDH will be smaller than ΣEF and the error will therefore be on the safer side. If however the resultants of E_l & ~~xxx~~ U_l and E_r & U_r are considered to be equal and opposite then the vector E_b will be represented by CA and its normal component by CJ. Again it is seen by working out actual examples that ΣCJ will be smaller than even ΣDH and the error will be still more on the safer side. In the most common procedure of stability analysis called the standard method of slices the pore pressures and the intergranular pressures on the sides of the slices are neglected. This is considered to be quite a satisfactory procedure as it gives conservative results along with saving considerable amount of labour.

With this simplifying assumption the normal component of E_b will be equal to CJ and

$$
\begin{aligned}
CJ &= \text{Normal component of } W - U_b \\
&= N - U
\end{aligned}
$$

by omitting the subscript to put the expression in its common form the forces resisting slide will be

$$= \Sigma (N-U) \tan \phi' + \Sigma C'l$$

the forces causing slide will be

$$
\begin{aligned}
&= \Sigma \text{ tangential components of } W \\
&= \Sigma T
\end{aligned}
$$

and the factor of safety against slide will be

$$F.S. = \frac{\Sigma (N-U) \tan \phi' + \Sigma c'l}{T} \quad \ldots \quad (1.11)$$

1.9. The critical conditions for the earth dam in its life time.

Just as the stability of a concrete dam is checked for different critical conditions like reservoir empty, reservoir full with no tail water, reservoir water upto H.F.L. with corresponding tail water down stream etc., the stability of an earth dam is checked for various critical conditions. These critical conditions are (i) the during construction or end of construction condition with reservoir empty, (ii) the steady seepage condition with reservoir full and (iii) the rapid draw-down condition, i.e. the stage immediately following a rapid draw-down of the reservoir water level from the full reservoir level to a substantially lower level. Detailed discussion regarding the pore pressures encountered in the stability analysis for these conditions is presented in the following chapters.

CHAPTER II.

PORE PRESSURE AND STABILITY OF DAM DURING RESERVOIR-FULL CONDITION.

2.1. Stability of downstream slope.

There are no records of failure of upstream slope of an earthen dam when the reservoir is full. When it has been full long enough for seepage water to percolate all the way through the embankment the pressure in the pore water in the down stream portion reaches its highest values. Under these conditions the down stream slope may have its lowest factor of safety against sliding. A shear slide when the reservoir is full causes a disastrous failure and is likely to result in considerable loss of life and property down stream due to the release of a sudden and heavy flood wave. The design of the down stream slope is therefore required to be adequately safe against slide when the reservoir is full. The stability analysis of the down-stream slope for the reservoir full condition i.e., the steady seepage condition, is most commonly done by the effective stress method. For this method an accurate knowledge of pore pressures going to be developed in the dam during this condition is necessary. The pore pressure distribution for this condition is most conveniently found out by the flow net.

2.2. Pore Pressures from the flow net.

In the steady seepage condition the soil in the dam below the phreatic line is completely saturated with water and will be in the two phase system of soil and water. There will be no air in the soil mass. The pore water will be slowly seeping through the pores with a very low velocity of a few inches to a few feet per year. The flow will be laminar gravity flow and considering it as two dimensional the flow pattern is governed by the Laplacian equation.

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \qquad \ldots \qquad (2.1)$$

where h is the total head of water at any point with coordinates x & y. This total head is made up of the pore pressure head, plus elevation head, plus velocity head, i.e.

$$h = p + z + \frac{v^2}{2g} \qquad \ldots \qquad (2.2)$$

The elevation head may be with respect to any datum. The datum

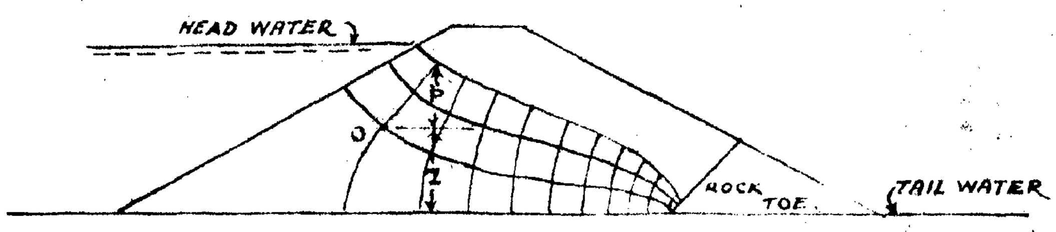


FIG. 2·1: PRESSURE HEAD AND ELEVATION HEAD AT A POINT.

normally chosen is the downstream free water surface level as shown in fig. 2.1. The velocity being not more than a few feet per year in the type of soils normally used for earth dams the velocity head $V^2/2_g$ is negligible and therefore it is not taken into account. The equation (2.2) above can, for all practical purposes, be modified as

$$h = p + z \qquad \ldots \qquad \ldots \qquad (2.3)$$

Solution of the Laplacian equation 2.1 with appropriate boundary conditions gives two sets of orthogonal curves; the curves of one set are called flow lines and the curves of the other set which are at right angles to the flow lines are called the equipotential lines. These two sets of curves together make up the flow net. The flow net for a dam of homogeneous, isotropic material is shown in fig. 2.1. From the flow net the pore pressure at any point can be found out as indicated in the figure. The pore pressure head at any point O is the height to which water will rise above that point O. This height is the vertical height between the point O and the point of intersection of the equipotential line passing through O and the top seepage line. The flow net can be obtained by any one of the following methods:

1. Graphical	5. Capillary models
2. Mathematical	6. Membrane analogy
3. Electrical analogy	7. Principal stress analogy
4. Hydraulic Models	8. Relaxation method.

The common methods are 1, 3 and 8. In the graphical method ~~the~~ the flow net is sketched with the knowledge that the net comprises of "Square" figures for isotropic soils. Normally rolled fill earth dams will not have isotropic permeability due to stratification. The permeability will be more in the horizontal direction than in the vertical direction. In that case graphical sketching for "Square" figure can be done only with a transformed section of the dam. The transformation scale is given by the following equation.

$$X_t = \sqrt{\frac{K_y}{K_x}}\, X \qquad \ldots \quad \ldots \qquad (2.4)$$

where K_x and K_y are the Darcy's coefficients of permeability in the x and y directions and X_t and X are any horizontal length to the transformed scale and natural scale respectively. After sketching the flow net on the transformed section it is retransformed to the natural scale for finding out pore pressures at required points.

2.3. Two view points of looking at pore pressures.

In a saturated soil mass under steady flow gravity seepage condition there are two ways of looking at pore pressures both giving identical results. In fig. 2.2(a) MNOP represents the cross section of an elementary volume from a soil mass within which seepage, satisfying Laplacian equation, is occurring For convenience the section chosen is a square where MP and NO represent flow lines and MN and PO represent equipotential lines. The mass MNOP is in equilibrium under the influence of three forces viz. self weight, resultant boundary intergranular force and resultant boundary neutral force, i.e. resultant of pore water pressures. Combining the water forces on opposite faces their resultants are shown in fig. 2.2(b). In both the figures (a) and (b) the thick arrows indicate the water pressures during seepage condition and the thin arrows during no seepage condition, i.e. static condition. The difference in the resultants of forces (in the two cases) on sides MN and OP is the head lost in seepage and this is called the seepage force. There is no such difference in the resultants of forces on sides MP and NO because there is no seepage across these sides in either case and there is no head loss.

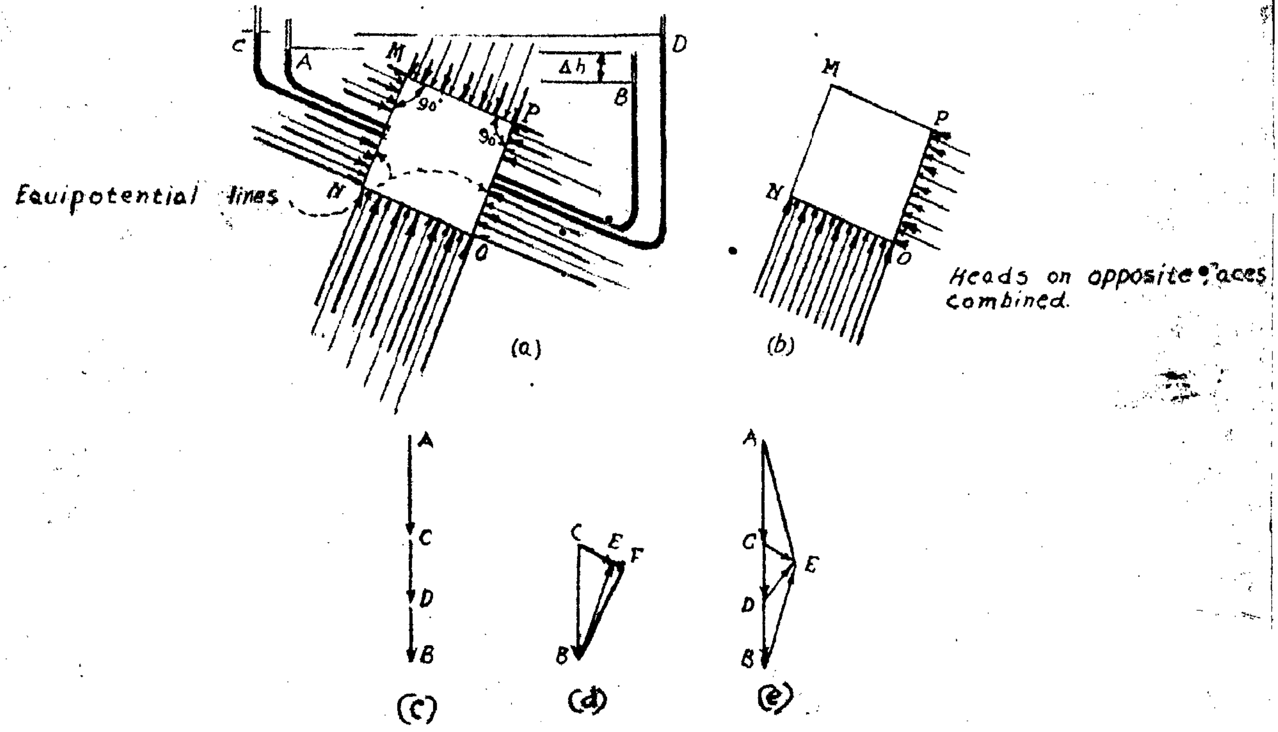


Figure 2.2 :- Diagrams of boundary neutral pressures and force polygons showing weights, buoyancy, and seepage force and their relationships.

Different aspects of the self weight of the mass and their combinations are indicated in fig.2.2(c). These combinations are

Total weight of soil-water mass AB = Dry weight AD + weight of water in pores AB = Saturated weight.

Submerged weight AC = Total weight AB + Buoyance BC

Buoyancy BC = Weight of an equal volume of water.

In fig.2.2(d) are shown combinations of the boundary water pressures. BF is the resultant of water pressures on sides NO and MP, both in seepage case and static case. FC is the resultant of water pressures on sides NM and OP in the static case and FE is the same resultant in the seepage case. The difference between the two viz. CE is the seepage force. The resultant of all water forces for the static case is the buoyancy BC. The resultant of all water forces for the seepage case is represented by BE.

The combination of self weight and boundary neutral forces for seepage case is shown in fig.(e). Here AE is the resultant of the two forces and therefore for stability the third force i.e. the resultant boundary intergranular force must be equal to EA. It is now seen that the resultant AE may be obtained by more than one way of combination of the forces. Two possible ways are, the vector sums respectively, of AC & CE or AB & BE. In other words the two possible alternative ways are (i) taking submerged weight and seepage force or (ii) taking saturated weight and pore pressures on all sides i.e. all along the boundary. The seepage force per unit volume of soil mass is given by the equation

$$\text{Seepage force} = \cancel{i\,\delta\,w}\; i\gamma_w \quad \ldots \qquad (2.5)$$

where γ_w is the unit weight of water, and i is called the hydraulic gradient which is equal to the head loss per unit flow length.

2.4. Action of seepage force and its control.

This seepage force which acts in the direction of the flow lines in isotropic soils assists sliding of the downstream slope in the reservoir full condition as ~~it~~ is apparent from fig. 2.3. The same seepage force opposes sliding of the upstream slope and adds to its stability as is apparent from fig. 2.4. This seepage force, which is also called viscous drag, erodes out the soil particles starting from the exit and the erosion progressing backwards along the flow lines. This is called piping and will be a great threat to the safety of the dam. When the upward hydraulic gradient at exit (called exit gradient) in sandy foundation at down stream toe of dam is nearly equal to unity, it will start the phenomenon of boiling or "quicksand". In this condition the hydraulic gradient at exit

$$\frac{h}{L} = \frac{G-1}{1+e} \qquad \dots \qquad \dots \qquad (2.6)$$

where h is the head lost in length L at exit, G is the specific gravity of solids in the soil mass and e is the void ratio.

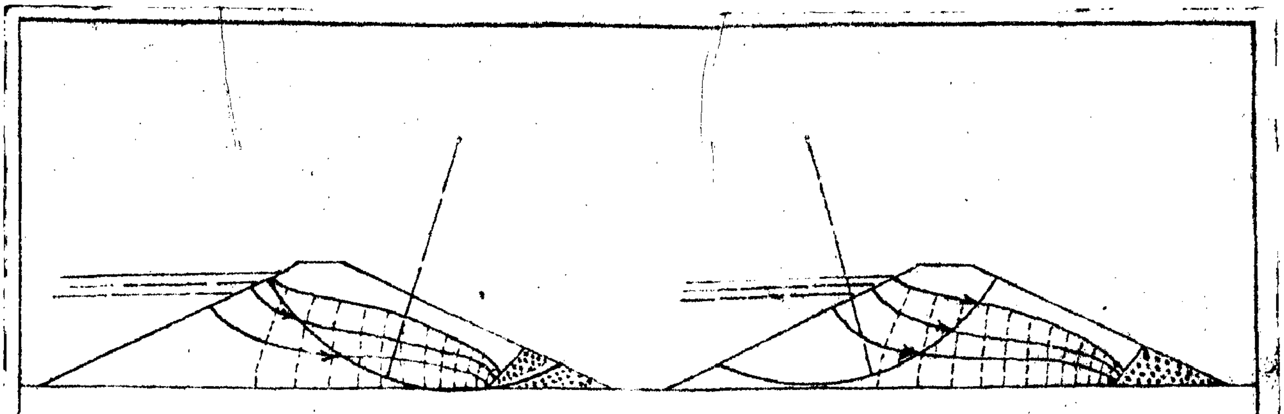


FIG. 2.3: SEEPAGE FORCE ASSISTING SLIDE.

FIG. 2.4: SEEPAGE FORCE RESISTING SLIDE.

Seepage forces, even if they are large, are not harmful if they occur near the centre of the dam and are diverted downward. If flow lines breakout on the downstream slope of the dam it is necessary that the gradients be limited to safe magnitudes or provisions be made to prevent erosion. These are achieved by providing under filters, downstream coarse material casing, rock toe, filter down stream of toe, filter drains in the body of the dam, upstream impervious blanket, cut off trench, cut off piles, curtain grouting, downstream blind drains and pressure relief wells.

2.5. Simpler approximate method of accounting for pore pressures in the slip circle stability analysis without drawing a flow net.

In the effective stress method of stability analysis we have to know the pore pressures along various points along the slip circle. These can be known by drawing the flow net as so far discussed. There is however a simpler but approximate method, the approximation being on the safer side i.e. the factor of safety obtained by this method will be slightly less than that obtained after determining the pore pressures by drawing a flow net. In this method, for the soil mass below the phreatic line, we are to take submerged unit weight for arriving at N forces and saturated unit weights for arriving at T forces. How this method takes pore pressure into account can be explained from two view points. These pertain to the two view points of looking at pore pressures discussed in section 2.3. above.

According to the conclusions therein the first method is to take saturated weight of the soil mass with pore pressures right round the boundary. The pore pressure at any point P along the boundary is $h\gamma_w$ (fig. 2.5.a) and it acts radial to the arc i.e. normal to the boundary. The equipotential lines in the sliding mass are nearly vertical and they start from the seepage line which is near the top line of the dam. If we take the equipotential lines as being truly vertical and starting from the top line of the dam itself we will be accounting slightly more pore pressure at every point as seen from a comparison of figures 2.5(a) and 2.5(b) and we will thus be erring slightly on the safer side. Now let us consider a slice of unit width. We are to take saturated weight with pore pressure.

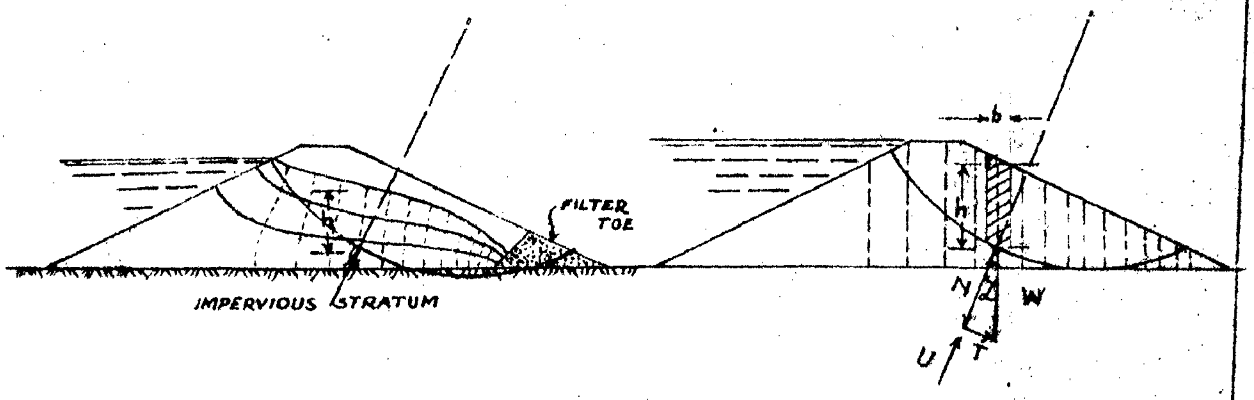


FIG. 2·5 (a) : EQUIPOTENTIAL LINES ACCORDING TO FLOW NET

FIG. 2·5(b) : APPROXIMATED VERTICAL EQUIPOTENTIAL LINES.

Let width of slice be b, height h, and let it be saturated to a height h.

$$T = b.h. \gamma_{sat} \sin\alpha$$
$$N = b.h. \gamma_{sat} \cos\alpha$$

Pore pressure on bottom surface,

$$U = w.h. \frac{h}{\cos} = w.h.b. \sec\alpha$$
$$N = bh \cos\alpha (\gamma_{sat} - \gamma_w \sec^2\alpha)$$

We are taking $N' = b.h. \gamma_{sub.} \cos\alpha$, thus neglecting $\sec^2\alpha$, which is on the unsafe side. But, we are assuming a higher value of h, which is on the safe side, in general leaving a small margin of safety.

Hence we take saturated unit weight sat for computing T forces and submerged unit weight $\gamma_{sub.}$ for computing N forces as an approximate method of accounting for pore pressures.

<u>Determination of effective stress parameters C' & Ø'.</u>

For the analysis of the stability of dam slopes by the effective stress method determination of the effective stress parameters C'&Ø' by laboratory tests is necessary. The tests available are (i) the direct shear test and (ii) the tri-axial shear test. The direct shear test has generally been in disfavour for several technical reasons and the test in vogue is the tri-axial shear test. In this test again two variations are available for determining C' & Ø'. One is the drained test and the other is the consolidated undrained test with pore pressure measurement. The values of C' & Ø' obtained from both these tests should essentially be the same. (The drained test usually takes lot of time and hence the consolidated undrained test with pore pressure measurement is often preferred.)

<u>Consolidated undrained test</u> with pore pressure measurement on saturated soil: The specimens are saturated as well as possible before shearing. ~~The~~ Two methods are used for saturating specimens (i) by percolating water through the specimen under a pressure gradient in order to wash the air out of the voids and (ii) by applying sufficient back pressure to the pore fluid to force the air into solution. Often both methods are used on clayey specimens.

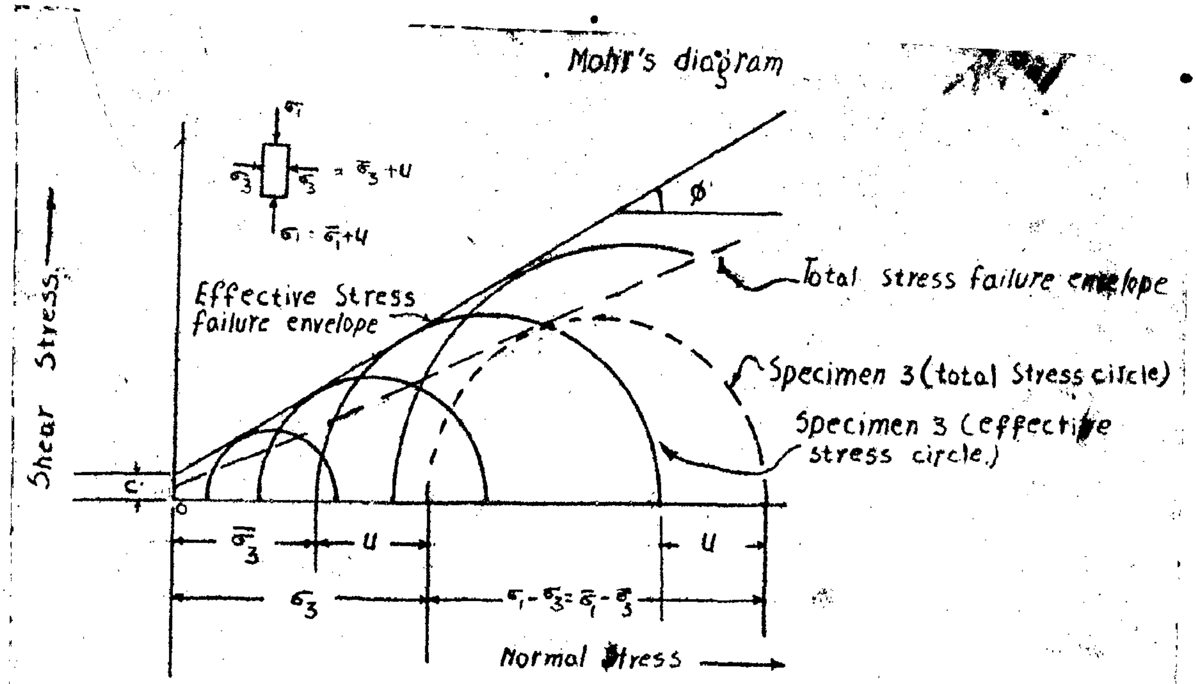


Figure 2.6 :- Typical results of consolidated Undrained tests on Saturated Specimens of compacted impervious Soil with pore pressure measurments

Figure 2.6. shows typical results of consolidated undrained tests on a saturated specimen of impervious soil in which the effective principal stresses at failure are plotted in the form of stress

circles on a Mohr's diagram. For practical purposes the rupture envelope for these circles can be approximated by a straight line which defines the effective stress strength parameters C' & Ø' as shown. The results of this type of test on saturated specimens are not influenced to any appreciable degree by the water content at which the specimens are compacted. The test results vary considerably depending primarily on the degree of saturation obtained, the pore water pressure imposed before the sample is sheared and the mode of the initial consolidation of the samples; i.e. whether they are consolidated under a hydrostatic pressure or in an an-isotropic condition i.e. $\sigma_1/\sigma_3 > 1.0$. The method of compaction used to prepare the specimen may have an important influence; however no comprehensive researches have been carried out on this point.

Drained test: The values of C' & Ø' obtained from drained test should be essentially the same as those obtained in consolidated undrained test on saturated specimens with pore pressure measurements.

Although comparisons have not been made for a great number of soil types and the direct shear test has been in disfavour for several technical reasons, there is considerable evidence to suggest that essentially the same results are obtained with direct shear and triaxial shear tests. Some engineers prefer the direct shear method for drained tests because the use of a thin sample allows more rapid pore pressure dissipation and permits the test to be performed in less time.

Drained tests should be performed on samples which are allowed to soak up sufficient water in advance to prevent any possibility that capillary pressures within the pores of the specimens may be acting to influence the results. For the direct shear test, the specimens are submerged at all times during the test in a pot of water. In triaxial compression tests, water is given access to the sample through the porous loading stones and some times through filter strips running through the sample. In addition, for tri-axial tests performed at low confining pressures, it is usually necessary to saturate the specimen in the triaxial cell or to soak it before testing.

The procedure of obtaining C' & Ø' values from the test results is the same as shown in figure 2.7 above excepting that the stress applied in the test are completely the effective stresses σ_1' & σ_3' and there are no pore pressures as free drainage is allowed.

2.7. Determination of total stress parameters C & Ø.

For the analysis of dam slopes by the total stress method the total stress parameters C & Ø are determined from consolidated undrained test. The effect of seeping water is simulated by allowing water to seep through the specimen under a small hydraulic gradient. The total principal stresses at failure are plotted in the form of stress circles on the Mohr's diagram as shown by broken lines in fig. 2.7. The total stress parameters C & Ø are then obtained by drawing the total stress failure envelope as shown by broken lines in the same figure.

2.8. Pore pressure distribution observed in dams during "full reservoir" condition.

There are a great number of measurements available of "full reservoir" pore pressures. Review of these indicates that the magnitudes and distribution of the pressures vary widely among dams of different characteristics. In dams with relatively narrow cores constructed of materials which are not excessively impervious (co-efficient of permeability greater than 50 ft/year) the measured pore water distribution through the core closely approximates the theoretical distribution of pressures for gravity flow. However measurements in dams with thick cores, constructed of very fine grained impervious materials (co-efficient of permeability less than 0.1 ft/year) frequently show pore pressures quite different from those which would be predicted on the basis of gravity flow theory. These differences result primarily from the influences of desiccation, capillarity and rainfall on the seepage pattern.

For a very impervious embankment in which a particle of water flows through the core at the rate of inches per year under the gravity head, the influence of water seeping into the crest and slopes of the dam during the rainy season, and of water being sucked out of the dam during the dry season by evaporation vastly outweighs the influence of gravity on the pore pressure distribution. In embankments of very fine soils capillary forces are much greater than the forces due to gravity, and as a result almost any distribution of pore water pressures can be obtained. In this context reference is invited to paras 1.6, 1.7, 3.8.2, 4.1 and 4.4 where the effect of capillarity under different circumstances is dealt with.

CHAPTER III.

STABILITY OF DAM DURING RESERVOIR DRAW-DOWN.

3.1. Stability of Upstream slope.

The critical stability condition for the upstream slope occurs as the reservoir is lowered after being full for some time. The upstream slope does not fail during the "reservoir full" condition because of the stabilising effect of the water load thereon. During the reservoir full condition pore pressures in the dam will have reached their maximum values. When the reservoir level is lowered rapidly the stabilising water load is removed but the pore pressures which act against the stability of the dam are not dissipated immediately if the embankment material is not free -

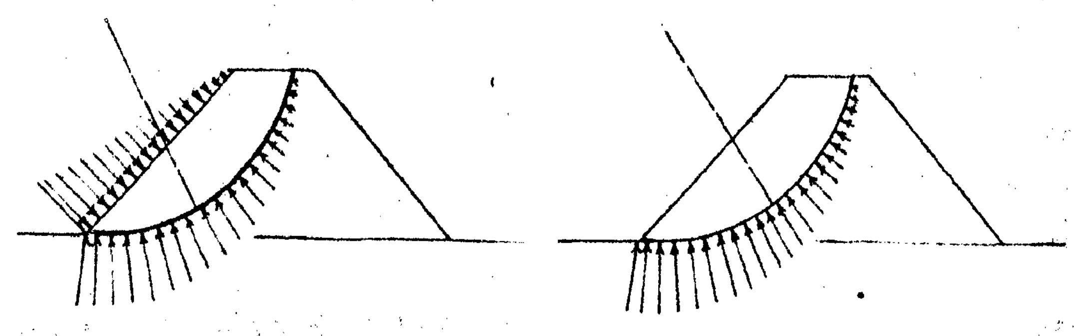


FIG. 3.1 (a): WATER PRESSURES BEFORE DRAW DOWN. FIG. 3.1 (b): WATER PRESSURES AFTER DRAW DOWN.

draining, i.e., if its permeability is low. Figure 3.1 (a) shows the pressure of reservoir water on the upstream slope which holds it back against sliding. It also shows the pore pressures on the boundary of the segment which act against the stability of the segment. Figure 3.1 (b) shows the water pressures after drawdown. In this condition the stabilising pressures of the reservoir water are no longer there. The nonstabilising pore pressures however are not dissipated due to the impermeability of the soil. As the main purpose of the dam is to retain water in the reservoir, either the core of the dam in a zoned embankment section or the whole of the dam in a homogeneous section has of necessity to be of impervious soil. The permeability of the soil will be of the order of a few inches to a few feet per year and therefore there will be considerable time lag in the slow dissipation of the pore pressure after the reservoir level is rapidly lowered. Hence the stability of the upstream slope is the least immediately following a rapid drawdown from full reservoir condition. The stability of the upstream slope can be tested either by the effective stress method or by the total stress method. However the effective stress method is more widely used at the present time. For this method, the pore pressures are to be estimated by suitably methods.

3.2. Rigid and non-rigid fills.

For the purpose of estimating pore pressures the embankments can be classified into two categories: (1) Rigid fills and (ii) Non-rigid, i.e., compressible fills. Rigid fills are those which do not undergo appreciable change in volume due to change in effective pressures imposed by drawdown. The Laplacian equation is applicable for seepage in rigid fills only and hence for rigid fills the pore pressures on drawdown can be estimated from the flow net. For seepage in nonrigid fills the Laplacian equation does not hold good and the flow net is not therefore applicable here for estimating pore pressures. The pore pressures are estimated by Bishop's method in this case.

3.3. Pore-pressures from flow net.

A discussion regarding the various methods ~~od~~ of drawing the flow net, assessing the pore-pressures therefrom, etc., has been given in the previous Chapter. All those principles hold good here also. The flow pattern existing before drawdown will be the steady seepage pattern. Immediately on drawdown the boundary conditions of the flow pattern change. The upstream slope which was an equipotential line before drawdown changes into a flow line after drawdown.

If the fill material is too impervious the flow pattern cannot adjust itself immediately to the changed boundary conditions. In that case the pore pressures will have to be estimated from the full reservoir flow net for stability analysis of the upstream slope immediately on drawdown.

If the fill material is not so impervious the flow pattern will immediately adjust itself to the changed boundary conditions without of course any water draining out from the pores. In this case the pore pressures will have to be estimated from the flow net drawn for the changed boundary conditions. The pore pressures are most commonly calculated by this method. There is however a simpler approximation on this method by which drawing of the flow net can be avoided. The presumption here, however, is that there is no drainage and the top seepage line remains in the same position as it was before drawdown.

3.4. Simpler approximate method of accounting for pore pressures in the stability analysis of Upstream slope for drawdown condition without drawing the flow net.

Figure 3.2 shows the drawdown flow net for a homogeneous embankment of isotropic material. It is seen from the figure that the equipotential lines are nearly vertical except near the top and they start from the seepage line which is near the top line of the dam. If we take the equipotential lines as being truly vertical and starting from the top line of the dam itself then we can extend the same arguments as in section 2.5 of the previous Chapter. The reasoning putforth there applies mutatis mutandis to this case also and we will be accounting for pore pressures if we take saturated unit weight for computing T forces and submerged unit weight for computing N forces. The assumption of the

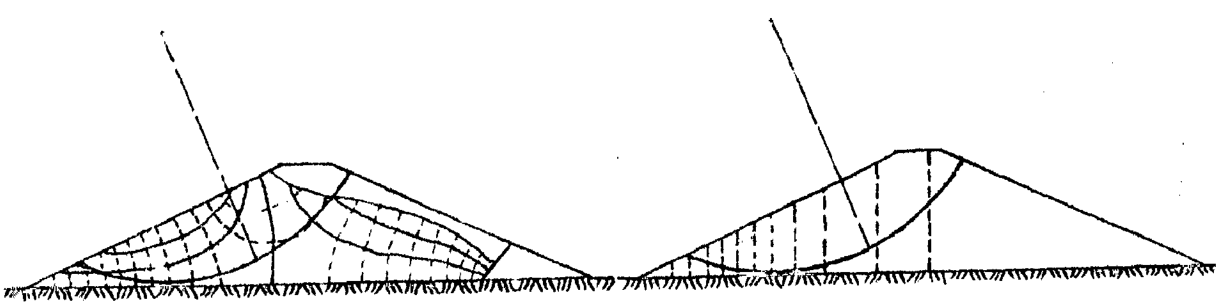


FIG. 3·2 (a): EQUIPOTENTIAL LINES ACCORDING TO DRAW DOWN FLOW NET.

FIG. 3·2 (b): APPROXIMATED VERTICAL EQUIPOTENTIAL LINES.

equipotential lines being very nearly vertical is all right for comparatively flat slopes only as can be seen from fig. 3.2. For steeper slopes this assumption will give appreciably higher pore pressures.

3.5. Pore pressures by Bishop's method for non-rigid fills, i.e. compressible fills.

This method is useful in finding the pore pressures in a compressible non-draining fill during drawdown and also during construction. In compressible fills the pore volume changes to the extent of the compressibility of the fill when there is an increase in the load on the fill. The pore volume decreases when the load is increased and the pore pressure increases as there is no drainage. During drawdown the water load on the upstream face is removed and during construction the load on any horizontal layer is increased with the addition of layer by layer of new earth work.

In 1954 Skempton[28] co-related the changes in pore pressures with the changes in load on a soil specimen by two co-efficients A & B called pore pressure co-efficients. Bishop[9] has applied these co-efficients for evaluation of pore pressures in an earth dam during construction and during rapid drawdown. The co-relation is derived as follows.

Consider first a sample of soil subjected to an alround pressure σ. If this pressure is increased by an amount $\Delta\sigma$ and drainage is prevented, there will, in general, be an increase of pore pressure Δu. The ratio $\Delta u/\Delta\sigma$ denoted by B, depends on the compressibility of the soil structure as a whole relative to that of the fluid contained in the voids, which is expressed by the following equation.

$$\frac{\Delta u}{\Delta \sigma} = B \qquad \ldots \qquad \ldots \qquad (3.1)$$

$$\text{and } B = \frac{1}{1 + n(C/C_c)} \qquad \ldots \qquad (3.2)$$

where n = the porosity, i.e., the volume of voids per unit volume of soil.

C_c = the average compressibility of the soil structure over the pressure range considered.

C = The compressibility of the pore water or pore air.

In perfectly saturated soils the voids are filled with water which is relatively incompressible and therefore B = 1. In perfectly dry soils the air is highly compressible and B = 0. For partially saturated soils B has some intermediate value between 0 & 1.

Now consider an increment of pressure $\Delta\sigma$ in one direction only. If the soil structure is regarded as an elastic solid, the volumetric strain produced by a pressure increment in one direction only is 1/3rd of that due to an alround increment of pressure of the same magnitude. Hence the theoretical increase of pore pressure $\Delta u = 1/3\, B\, \Delta\sigma$. As the stress-strain characteristics of the soil structure are not linear the factor 1/3rd must be replaced by a co-efficient A which can be determined experimentally. This co-efficient varies considerably with the type of soil from plus 1 or even more for normally consolidated clays of high plasticity to minus 0.5 for over consolidated clays.

If an element of soil is originally in equilibrium under an alround horizontal pressure σ_3(minor principal stress) and a vertical pressure 1(major principal stress) and due to external loading these principal stresses are increased by $\Delta\sigma_3$ & $\Delta\sigma_1$ respectively, the stress changes may be considered as taking place in two stages namely an equal alround increment $\Delta\sigma_3$ and a deviator stress ($\Delta\sigma_1 - \Delta\sigma_3$) in the direction of σ_1. The resultant change in pore pressure is thus the combined effect of these two stages and may be expressed as

$$\Delta u = B\left[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)\right] \quad \ldots \quad (3.3)$$

In the above equation the two unknowns A & B can be found out from two sets of observations from laboratory experiments.

Bishop has given the following method for the application of these co-efficients A & B for the evaluation of pore pressures in earth dams. To express Δu as a function of $\Delta\sigma_1$ only we divide both the sides of equation 3.3 by $\Delta\sigma_1$.

Then

$$\frac{\Delta u}{\Delta\sigma_1} = B\left[\frac{\Delta\sigma_3}{\Delta\sigma_1} + A\left(1 - \frac{\Delta\sigma_3}{\Delta\sigma_1}\right)\right] \quad \ldots \quad (3.4)$$

We can now put

$$\Delta u = \bar{B}\,\Delta\sigma_1 \quad \ldots \quad \ldots \quad (3.5)$$

Where

$$\bar{B} = B\left[\frac{\Delta\sigma_3}{\Delta\sigma_1} + A\left(1 - \frac{\Delta\sigma_3}{\Delta\sigma_1}\right)\right] \quad \ldots \quad (3.5.1)$$

The initial pore pressure in an element of soil beneath the upstream slope of an earth dam in which steady seepage is established (fig. 3.3) is U_o where

$$U_o = \gamma_w \left[h_1 + h_2 + h_w - h' \right] \quad \ldots \quad (3.6)$$

In this equation

h_1 = height of impervious fill

h_2 = height of free draining fill

h' = drop in head under steady seepage condition at this point.

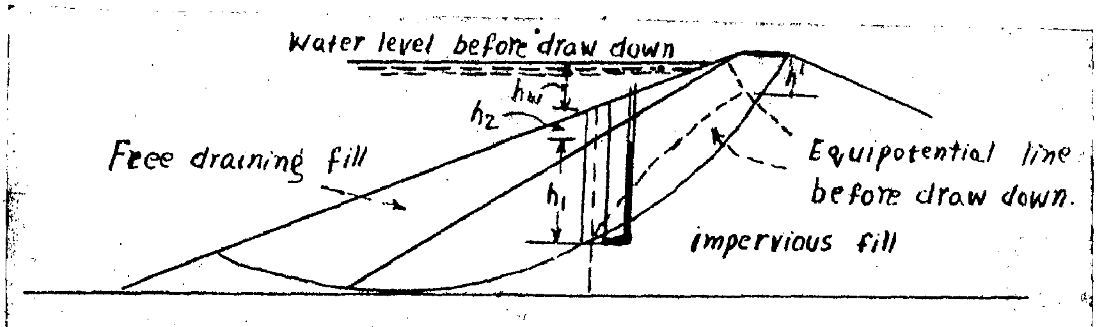


Figure 3.3 :- pore pressure after draw down.

After rapid drawdown the pore pressure will change to U where

$$U = U_o + \Delta u \quad \text{and} \quad \Delta u = \bar{B} \Delta \sigma_1$$

and thus $U = U_o + \bar{B} \Delta \sigma_1 \quad \ldots \quad \ldots \quad (3.6.1)$

If the major principal stress is taken as being equal to the vertical head of material (soil and water) above the element, then, before draw down,

$$(\sigma_1)_0 = \gamma_1 h_1 + \gamma_2 h_2 + \gamma_w h_w \quad \ldots \quad (3.7)$$

and after drawdown

$$\sigma_1 = \gamma_1 h_1 + \gamma_{2d} h_2 \quad \ldots \quad (3.8)$$

where

	γ_1	=	Saturated density of the impervious fill.
	γ_2	=	Saturated density of the free draining fill.
	γ_{2d}	=	drained density of the free draining fill
and	γ_w	=	density of water.

Thus $\Delta\sigma_1 = (\sigma_1) - (\sigma_1)_0$

$$= -\left[(\gamma_2 - \gamma_{2d}) h_2 + \gamma_w h_w \right] \quad \ldots \quad (3.9)$$

Now $\gamma_{2d} = \gamma_2 - n_s \gamma_w \quad \ldots \quad \ldots \quad (3.10)$

Where n_s denotes the specific proposity of the free draining fill i.e., the volume of water draining out of the free draining fill per unit volume. Hence from equations 3.6 and 3.6.1.

$$U = \gamma_w \left[h_1 + h_2(1 - \bar{\bar{B}}.n_s) + h_w (1 - \bar{\bar{B}}) - h' \right] \ldots \quad (3.11)$$

It can be seen from equation 3.11 that, on drawdown, the lower the value of $\bar{\bar{B}}$ the greater will be the residual pore pressure and therefore the lower will be the factor of safety. It is thus of interest to examine the range of $\bar{\bar{B}}$ values which may be expected in practice.

For fully saturated soils B will be almost equal to unity and the upper limit of A is also unity. The change in total stress on drawdown are represented by a decrease in major principal stress, and as the shear stress is increased by an even larger drop in minor principal stress. It follows immediately from equation 3.5.1 that with $A = 1$, $\bar{B} = B = 1$ and for lower values of A, $\bar{B} > B$ and hence $\bar{B} > 1$. As a safe working rule therefore $\bar{B}$ may be taken as unity.

The expression for U then simplifies to the form

$$U = \gamma_w \left[h_1 + h_2 (1-n_s) - h' \right]$$

This equation gives values of pore pressure which are in substantial agreement with the existing field measurements under rapid drawdown conditions (e.g. Alcova dam[29]).

3.6. Determination of effective stress parameters C' & Ø'

For the effective stress method of analysis the effective stress parameters C' & Ø' are determined by laboratory tests as explained in section 2.6 of Chapter II.

3.7. Determination of total stress parameters C & Ø.

In the total stress method of analysis, the results of consolidated undrained tests on saturated specimens usually are used to determine the shear strength. The test procedure is explained in section 2.7 of Chapter II. The samples are tested under conditions which simulate the stresses and pore pressures which would exist in the embankment as well as the designer is able to predict them. There is no widely adopted standard method of testing. Recently methods of analysis have been employed in which the laboratory samples are consolidated an isotropically before testing in undrained shear.

3.8. Factors affecting drawdown pore-pressures.

The two important factors which influence the drawdown pore pressures are (i) the reservoir level and (ii) drainage. Drainage depends upon the permeability of the fill, length of drainage path and the rate of drawdown. The influence of all these factors is discussed below.

3.8.1. Influence of reservoir level.

For a homogeneous dam, it may appear that the most critical drawdown condition exists after the reservoir has been completely emptied. However there is only a minor decrease in the factor of safety after the reservoir has been lowered to mid-height of the dam. For a dam with a large upstream zone of pervious soil or rock, calculations often indicate that the safety factor is actually lower when the reservoir is partially empty than when the reservoir is completely empty. This is due to the fact that, when the reservoir is partially full the shear strengths of the upstream pervious zone at the upstream tow which provides the primary resistance to movement is dependant on the buoyant weight of the material; whereas when the reservoir is completely empty the strength is dependant on the full weight. For this reason it is advisable to calculate the stability of the upstream slope for several assumed reservoir levels, including at and below mid-height of the dam, as well as for complete draw down.

3.8.2. Influence of drainage.

The rate of pore pressure dissipation due to drainage following reservoir drawdown is dependant primarily on the permeability, degree of saturation, and capillarity of the material comprising the upstream slope (and on the pore volume change if any). Our present ability to evaluate these factors even in a rough manner in advance of construction for embankments of fine grained soils by theoretical procedures is practically nil. Also we have very few measurements of pore water pressures in operating dams to guide the developments of theories. Consequently, for embankments constructed of fine grained soils, we can only assume that the influence of drainage is negligible and can therefore be neglected. In well rolled embankments of silt or silty sand any slight tendency for drainage will immediately cause the development of menisci at the boundary and create negative pore pressures i.e., capillary tension forces in the zone of the embankment above reservoir level.

In the computations of pore water pressures due to gravity seepage, the influence of capillary forces is neglected since we have no way to evaluate them. Capillary forces tend to prevent the water from flowing out of the pores of fine grained embankments following reservoir drawdown by forming menisci at the seepage boundary, and also within the dam, with resulting tension in pore water. Therefore ignoring capillary action results in a conservative pore pressure estimate. On the other hand, it is quite conceivable, and even probable, that for many soils capillarity will completely dominate gravity flow pattern, making it impossible for any appreciable positive drawdown pore pressure to exist within the embankment above the reservoir elevation. In future years additional laboratory researches and measurements of drawdown pore pressures in dams may demonstrate that our present procedures are excessively conservative except for certain soil types.

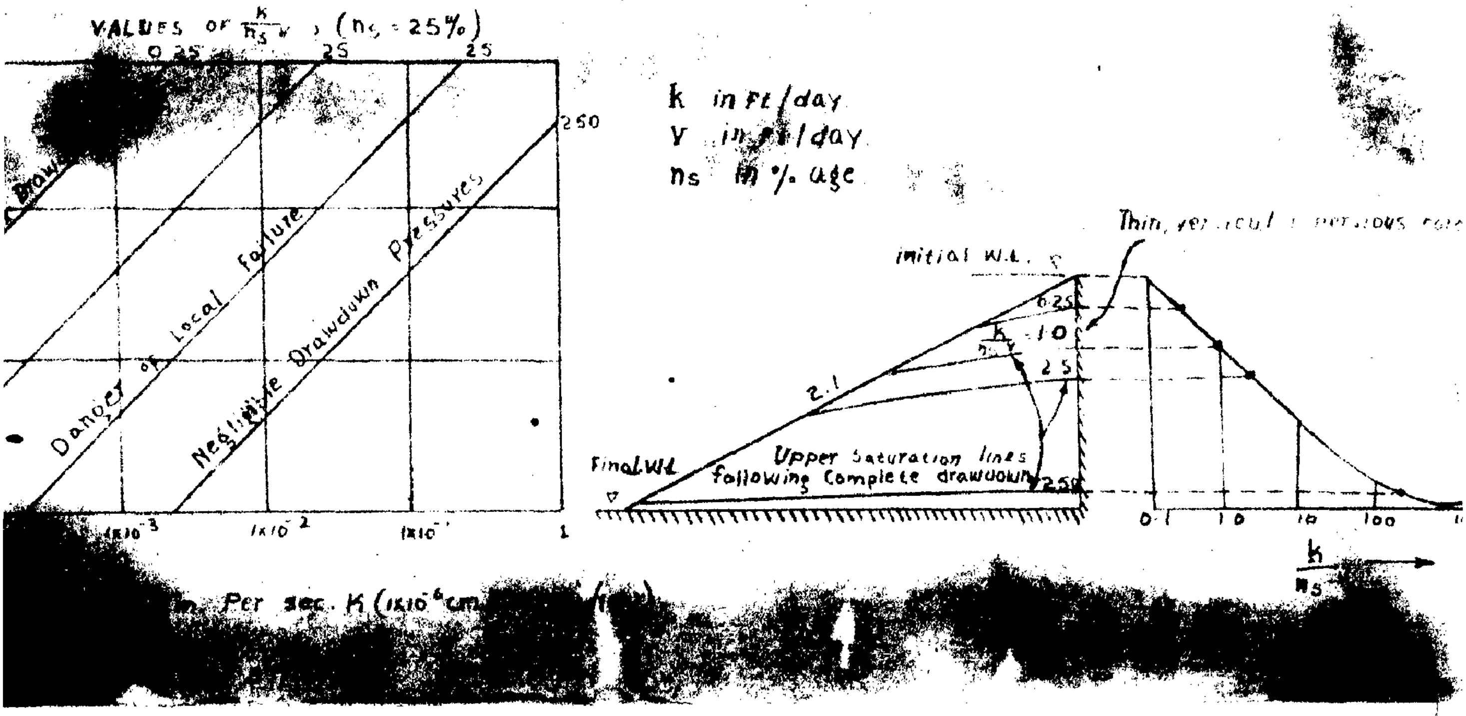
3.8.3. Effect of rate of draw-down.

In para 3.8.2 above the influence of drainage on the drawdown pore pressures is discussed. The drainage, however, is itself influenced by the permeability of the fill, the length of the drainage path and the time interval available for drainage. The interval available for drainage from the time of beginning of drawdown from the full reservoir level till the time the reservoir is lowered to the drawdown level is obviously dependent on the rate of drawdown. If the lowering is by say 40' and the rate of lowering is 1' per day 40 days will be available for drainage. With the rate of lowering of 2' per day only 20 days will be available for drainage. In para 3.3 wherein the aspect of estimating pore pressures from alternative flow nets is discussed the main presumption is that there is no drainage and the top seepage line remains in the same position as it was before drawdown. When we take drainage into consideration the top seepage line cannot be assumed to be in the same position but it will have to be lower depending on the amount of drainage. The flow net will have to be drawn with the lowered top seepage line and with the changed boundary conditions. There is no easy way of predicting the location of the top seepage line accurately taking into consideration the drainage characteristics of the fill and the rate of drawdown. Approximate methods[58] [64] with simplifying assumptions have however been developed. These methods do not take into consideration the influence of capillary forces which develop above the lowered top seepage line. These capillary forces (which may be looked upon as negative pore pressures) not only reduce the amount of drainage but will reduce the time rate of drainage also. It is not practicable to estimate these effects of the capillary forces and the design engineer will have to ignore the capillary forces.

3.8.4. Effect of elasticity of grain structure.

In this connection an extract from Glover Cornwall's[19] description of the pore pressure phenomenon is reproduced below:

" There is some disagreement in regard to what happens to the pore pressures following a rapid drawdown. One view is that the pore pressures remain unchanged until they are relieved by a process of diffussion instigated by the change of pressures at the upstream face. This is regarded as being too severe an assumption, since if this were true, almost any slope could be rendered unstable by a sufficiently rapid draw-down. Experiments on model dams under conditions of extremely rapid drawdown failed to produce unstability in a slope of 3 horizontal to 1 vertical and therefore it is probable that other factors beside diffussion are present to modify the pressures. One of these factors can be identified in the elasticity of the grain structure. Since the individual grains in this structure are in contact with their neighbours at a few points only, a pressure applied to the grain structure will produce high local stresses at the points of contact and deformations much higher than would be possible in a solid body of the same material. It seems probable that the grain structure is much more compressible than the pore water and that an instantaneous drawdown would produce an immediate readjustment controlled by the requirement of a constant volume strain at each point, after which the pressures would revert



The positions of the top seepage line after drawdown for a simple dam section with an impervious central core worked out by Reinius [64] for different soil properties and rates of drawdown is given in fig. 3.7 (a) . The extent of drawdown pore pressures is indicated in fig. 3.7 (b).

The position of the top seepage line and the drawdown pore pressures depend on the parameter $K/n_s V$ where K is the co-efficient of permeability, n_s is the volume of water draining out of unit volume of the soil in percentage and V is the rate of drawdown.

to the gravity flow system by a process of diffussion. The removal of the water load from the upstream face, with the consequent tendency for expansion, might be expected to reduce temporarily the pore pressures and enhance the stability. But this benefit would be lost as soon as the gravity flow pattern becomes established."

3.9. Control of drawdown pore-pressures.

While drains have not been widely used to control drawdown pore pressures they can perform this function very reliably. Upstream sloping chimney drains in an otherwise homogeneous dam will change the direction of pore water flow following rapid drawdown and thereby can have an important beneficial

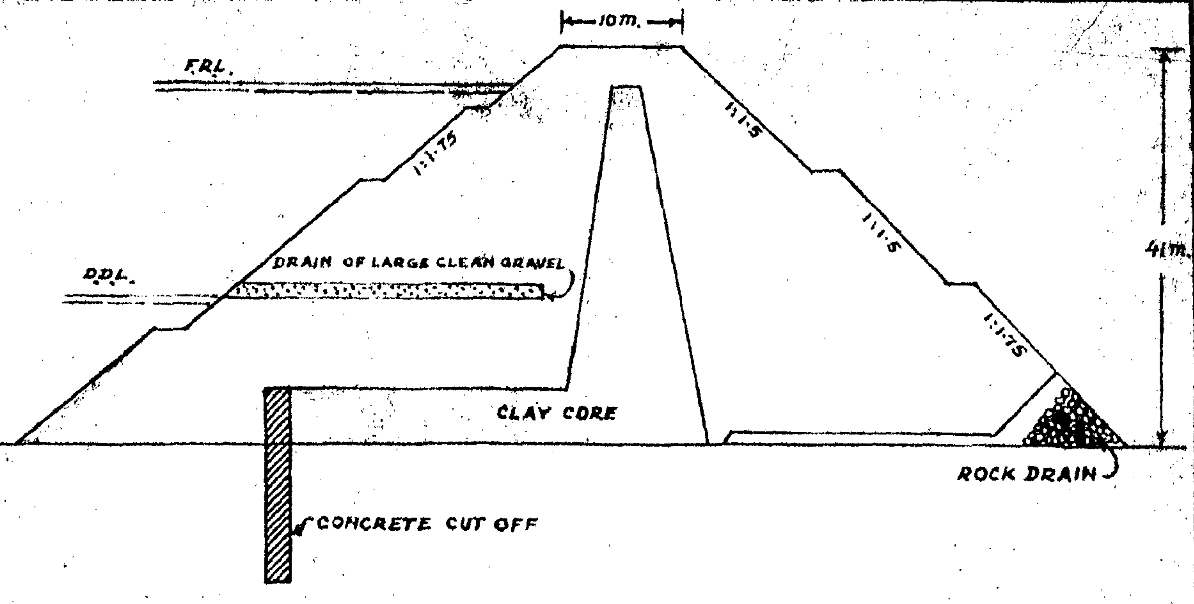


FIG.3·4 : ROSSHAUPTEN DAM, DRAINAGE BLANKET IN UPSTREAM SLOPE.

influence on the magnitude of the pore pressures. At a few dams where the designer has doubted whether the upstream slope was pervious enough to prevent high residual drawdown pressures, one or more horizontal layers of pervious material have been installed. Fig. 3.4 shows a cross section of the 130' Roßhaupten dam[11] (completed in 1953 in Germany) in which a horizontal drainage blanket was placed in the semi-pervious upstream slope at the low water level.

Horizontal drainage blankets of the type frequently used under the downstream slope of homogeneous dams have also been placed under the upstream slopes of a number of dams. Examples of these[11] are the Arkabutla Dam in Mississippi (1943) and the Pomme de Terre Dam in Missouri (1960) where the blankets served the dual purpose of controlling drawdown pore pressures in the embankment and accelerating the consolidation of clay

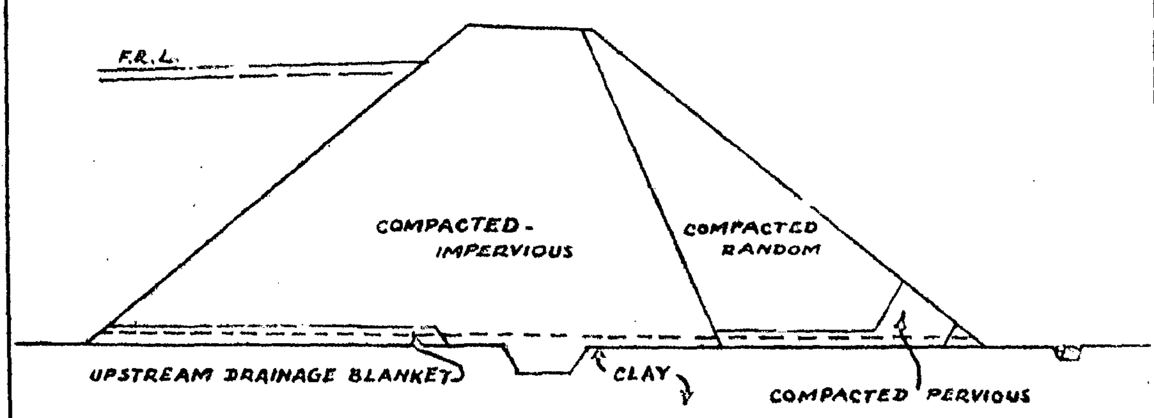


FIG. 3.5 : UPSTEAM DRAINAGE BLANKET AT FOUNDATION LEVEL.

foundations, fig. 3.5. Drains placed in the upstream slope primarily for the control of drawdown pore pressures may also be of value in controlling construction pore pressures.

3.10. Pore pressure distribution observed in dams on rapid drawdown.

Not much data is available regarding measurements of actual pore pressures in dams following rapid drawdown.

Figure 3.6 shows the effect of rapid drawdown on the pore water pressures measured in Alcova Dam,[7] Wyo. It will be noted that the reservoir water surface was lowered 120' in 40 days, which is an extremely rapid drawdown for a dam of this height. Fig. 3.6(a) shows the phreatic line indicates that virtually steady state conditions were present prior to drawdown. Fig. 3.6(b) shows the pressures under drawdown conditions.

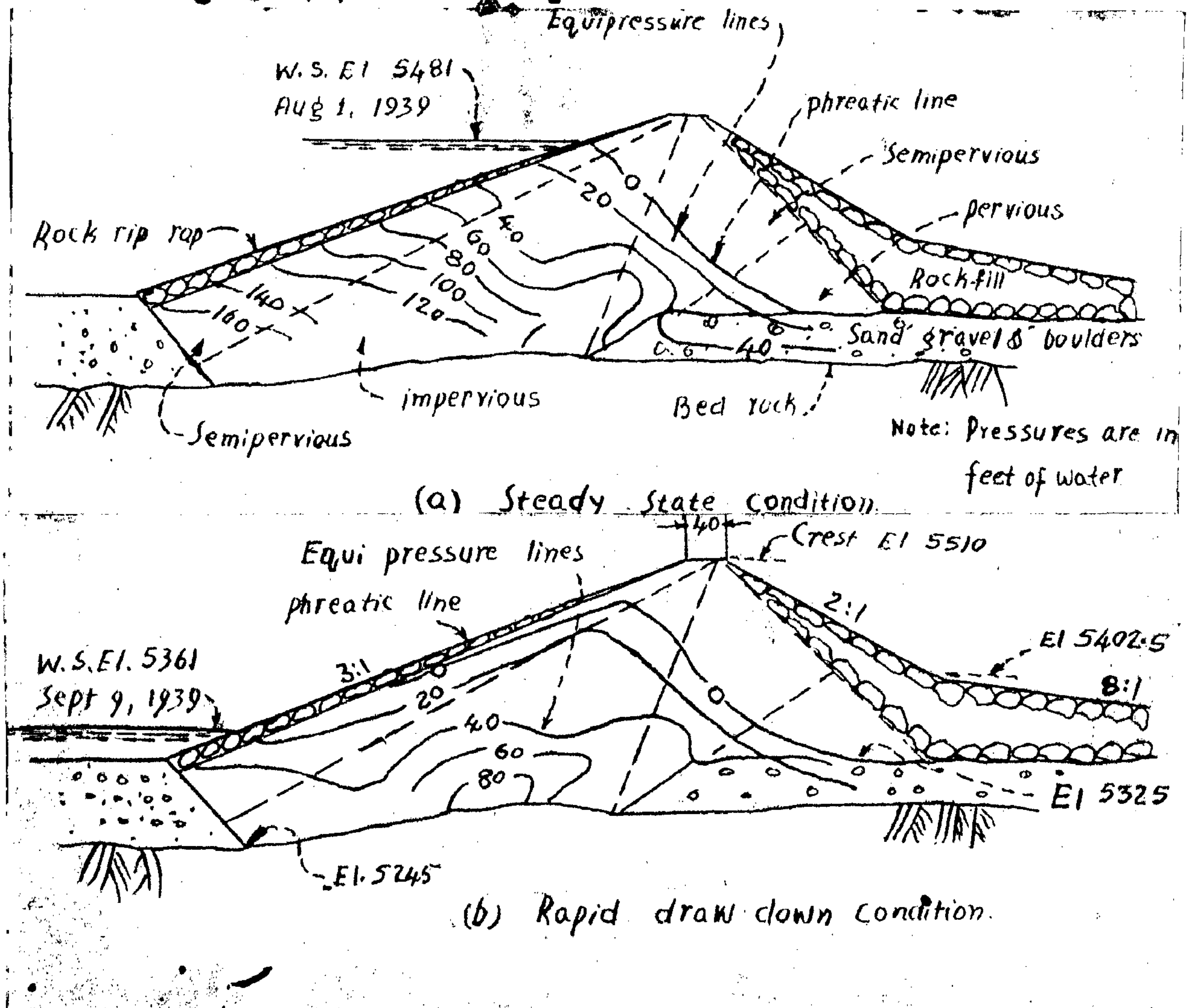


Figure 3.6 :- observed pore pressure in Alcova Dam, Wyo

noted that the reservoir water surface was lowered 120' in 40 days, which is an extremely rapid drawdown for a dam of this height. Fig. 3.6(a) shows the phreatic line indicates that virtually steady state conditions were present prior to drawdown. Fig. 3.6(b) shows the pressures under drawdown conditions.

Fig. 3.6 demonstrates that appreciable pore water pressures remain in an embankment after drawdown.

CHAPTER IV.

PORE PRESSURE AND STABILITY OF DAM DURING AND AT THE END OF CONSTRUCTION.

4.1. Pore pressures by compaction moisture.

In the drawdown condition or in the full reservoir condition the pore water that exerts pore pressures is the reservoir water that has seeped into the pores of the embankment soil. But during construction and at the end of construction the pore water that exerts pressure is not the reservoir water as the reservoir will be empty during that period. It is the water that is used with the soil for its compaction during construction. Some pore pressure is also exerted by the pore air that is entrapped in the soil which is in the three phase system having all the three constituents viz. soil grains, water and air. It is a known fact that soils cannot be compacted without the addition of some water to make them moist. This water lubricates the soil grains for compaction and also holds them together by negative capillary pressures. But the same water which assisted compaction acts adversely with increasing compaction due to the negative capillary pressures being converted into positive pore pressures. This conversion is brought about by the compactive effort of the heavy compacting machinery and also by the increasing over-weight of the layer by layer of the soil mass being deposited with progressing construction. In fills of low permeability these pore pressures do not get dissipated even over two to four years and there will be no substantial drop in the pore pressures even by the end of construction period.

4.2. Optimum moisture content and pore pressures.

One way of controlling the high development of these pore pressures is by using less water for compaction. In 1933 Proctor showed that for a given soil and for a given compactive effort the compaction is maximum for one particular moisture content is less or more than this optimum then maximum compaction will not be achieved. Fig. 4.1 shows the compaction curve for a soil sample wherein the dry density of the compacted soil is taken as the index of compaction. From the figure it is seen that for a given range of compaction the moisture content can be either on the dry side of optimum or on the wet side of optimum. From the discussions in para 4.1 it is obvious that if the construction pore pressures are to be less then the compaction moisture content should be on the dry side of optimum. But this again leads to the following trouble. On subsequent saturation by the reservoir water the embankment gets excessively softened and there will be considerable settlement. This may result in cracking of the fill which will be dangerous in a storage embankment. The construction

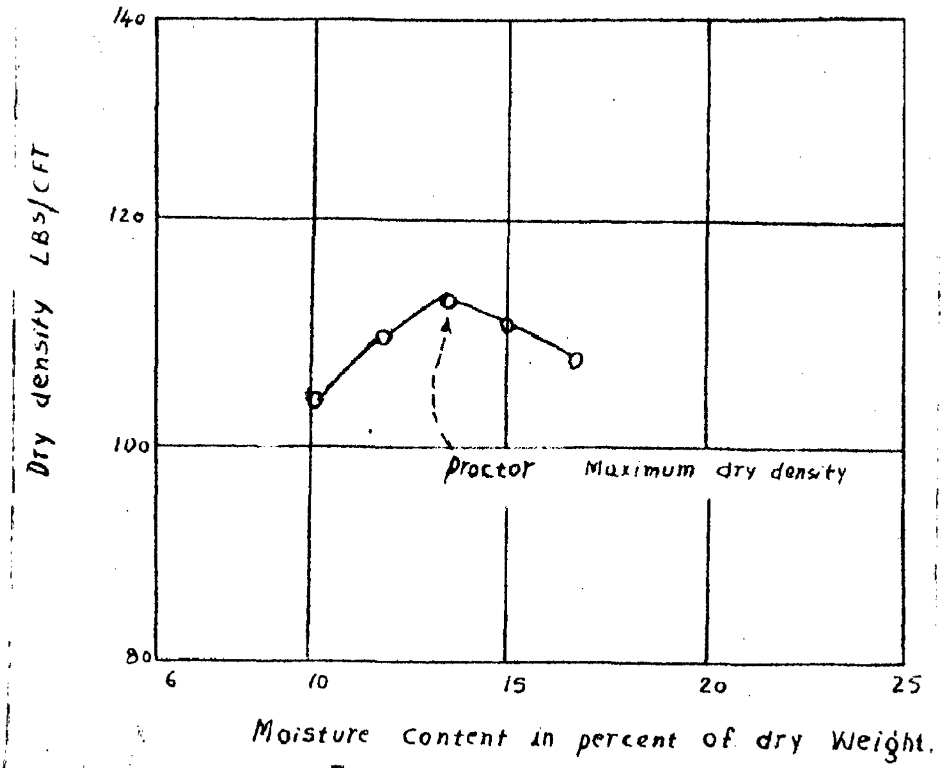


Figure 4.1 :- Compaction Curve.

pore pressures can be controlled by other methods described in Section 4.8, but settlement and cracking cannot be controlled. This suggests that it is preferable to have the compaction moisture content on the wet side of optimum inspite of possible increase in pore pressures. But here too there is a limit. The moisture content should not be so excessive as to affect the convenience of operation of construction machinery. By these standards there will be a very narrow range between the upper and lower limits of acceptable moisture content.

4.3. End of construction condition compared with full reservoir condition and drawdown condition.

Depending upon several factors going to be discussed later the pore pressures in the dam at the end of construction are likely to be more than during rapid drawdown condition for the upstream slope and during steady seepage condition for the downstream slope. The unit weight of the soil is also higher due to saturation in the latter two cases. For the drawdown state the unit weight of the soil at the upstream toe below drawdown level will be less than for the construction state due to buoyancy. On the whole the factor of safety of the slopes for the end of construction state is mainly affected by the pore pressures than by the unit weights of soil and if the pore ~~pressures than by the unit weights of soil and if the pore~~ pressures are more than in the other two states the factor of safety against sliding will be less. However slides of rolled fill dams during construction have not occurred as frequently as during operation of the reservoir and have not resulted in failures of the catastrophic type. Nevertheless as there is a chance of the pore pressures in the embankment and foundation during construction being higher than at any subsequent time it is usually advisable to analyse the stability of the embankment for this condition. The construction condition is especially likely to be critical for dams on soft foundations.

4.4. Divergent views regarding construction pore pressures.

During the construction stage the soil in the embankment is in the moist condition i.e. the unsaturated condition. When the moisture content is low much of it is likely to be held in the capillary pores of the soil and is likely to exert capillary tension, i.e. negative pressure instead of positive pore pressure and thereby increase the stability of the embankment instead of decreasing it. With increasing moisture the capillary tension gradually decreases, becomes zero and then becomes positive pore pressure much depending on the extent of moisture content. The reliability of pore pressure measurement obtained during laboratory shear tests on unsaturated cohesive soils and in embankments in construction stage has been doubted. The criterion of failure in laboratory shear tests of compacted cohesive soils in which pore pressures are measured has been the subject of diverse opinions. The large magnitude of negative pore water pressure in soils compacted near the optimum moisture content has come as a surprise to investigators.[26] It has been seriously questioned as to whether pore pressure is a reality in so far as strength characteristics of unsaturated soils are concerned. The subject of pore pressures in unsaturated soils is still a matter of much conjecture and controversy.

4.5. Shear parameters for the total stress and effective stress analysis.

In the total stress method of stability analysis, the total stress parameters C & Ø obtained by undrained shear tests on unsaturated samples are used to determine the shear strength. The results of the laboratory tests are greatly influenced by the water contents at which the samples are compacted and it is difficult to estimate exactly the average water content which may be obtained in the field during construction. Therefore several sets of laboratory tests on samples compacted at different water contents should be performed and average as well as conservative estimates of the shear parameters should be used in the analysis. The undrained shear test is explained in section 2.7 of Chapter II.

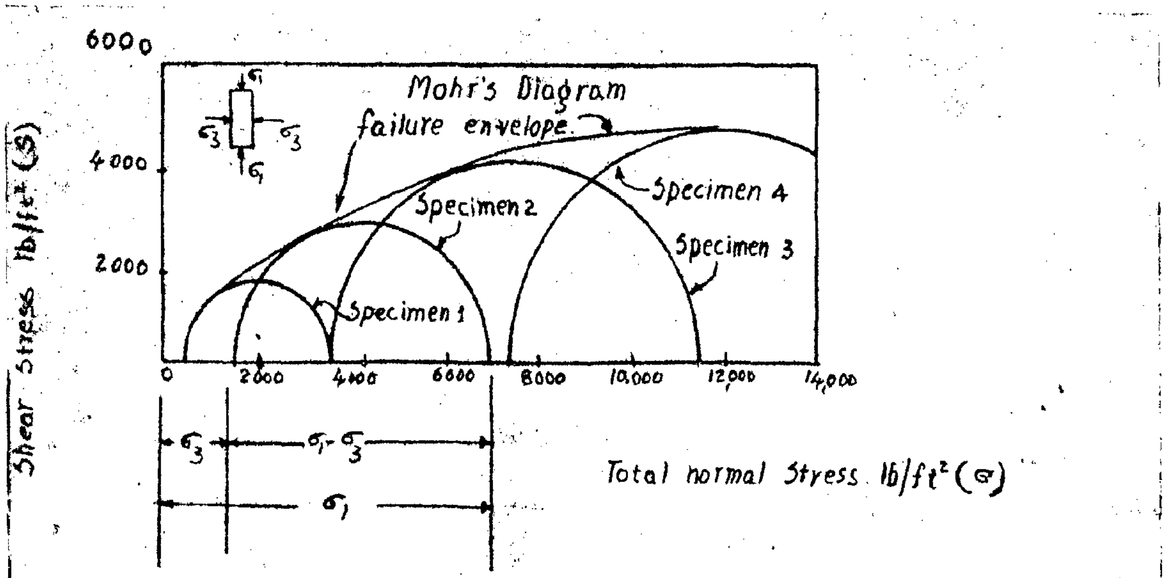


Figure 4.2 :- Typical results of undrained triaxial tests on Semisaturated Compacted Soil (all Samples compacted to the same original Water content & density)

In the effective stress method of analysis the effective stress parameters C' & Ø', determined by laboratory tests as explained in section 2.6 of Chapter II are used.

4.6. Estimation of pore pressures for effective stress analysis.

The different methods of pore pressure estimation are explained below.

4.6.1. Based on experience: Though we have theory and laboratory test procedures which can be used to predict construction pore pressures, as well as considerable number of measurements with piezometers in dams under various conditions we still are not able to predict reliably or precisely in the design stage what these pressures might be in a given dam. The primary reason for this is that the construction pore pressures depend on factors which the designer cannot evaluate well in advance, such as the amount of construction which will be done in wet weather, the influence of drying in dry weather, and the effectiveness of moisture and density control during construction. Because of this inability to estimate the pore pressures reliably, and also due to the fact that during-construction stability usually should not govern the design section (except in the case of dams on soft foundation), the construction pore pressures are usually estimated conservatively based on past experience.

For a preliminary design of earth dam the USBR practice is to assume the pore pressure in feet head of water to be equal to 2/3rd the depth of the fill on the upstream side and 1/3rd the depth on the downstream side at all points along the slip circle. Alternatively the pore pressure is assumed as equal to 1¼ times the depth of the impervious fill at any point along the slip circle.

<u>4.6.2. By triaxial test:</u> An estimation of the pore pressures to be expected in field can also be obtained in the laboratory by direct measurement of the pore pressures in sealed laboratory specimens which have been compacted to water contents and densities expected during construction and which are then subjected to increasing stresses with a test procedure designed to simulate the stresses anticipated on the various elements of the embankment by the added weight of the overlying material during construction.

4.6.3. By Bishop's method: Further again the pore pressures can be computed from theory by Bishop's method or by Hilf's method.

Bishop's method of calculating pore pressures from the superimposed loads is explained at length in section 3.5 of Chapter III. In drawdown condition the change in load is due to the removal of the reservoir water load; the change is a decrease. At the end of construction the change in load at any point is due to the addition of the weight of the earth work above that point; the change is an increase. The rest of the procedure of calculating the pore pressures remains the same as explained in section 3.5 of Chapter III excepting that C in equation 3.2 is the compressibility of the air-and-water mixture in the pores in this case. By using Boyle's law for compressibility of air and Henry's law for solubility of air in water, both at constant temperature, Bishop has given the following equation for the compressibility of the air-water mixture.

$$C = \frac{1}{P_o}\left(\frac{1}{n}\cdot\frac{\Delta V}{V} + 1 - S + SH\right) \qquad \ldots \qquad (4.1)$$

where

- C = Compressibility of the air-water mixture,
- P_o = the initial absolute pressure in the voids before the stress is applied,
- n = the porosity of the soil i.e. the volume of voids per unit volume of soil mass,
- V = the initial volume of an element of soil,
- ΔV = the volume change of the element of soil after loading (in this case the change is a decrease and hence negative),
- S = the degree of saturation i.e. the ~~xxxxx~~ ratio of volume of pore water to the volume of pores (voids),

H = Henry's co-efficient of solubility of air in water by volume.

= 0.0198 Vols. of air per unit vol. of water, approximately at room temperature.

The expression 4.1 is based on the fact that the volume change in the pore water is negligible compared with the volume decrease in the entrapped air due to its compressibility and solubility in water. This expression is valid until all the air has passed into solution, i.e., until the volume decrease ($-\Delta V/V$) is equal to the initial volume of the air in the voids, $(1-S)n$. From this point onwards the material is fully saturated and the pore pressure co-efficient B is equal to unity with respect to further increases in stress.

The volume change is measured in an Oedometer test. The Oedometer test requires less skill than for the triaxial test referred in para 4.6.2 but is more time consuming.

<u>4.6.4. By Hilf's method:</u> The method developed by Hilf and being used by the USBR is based on the following primary assumptions.

1. Only vertical embankment strain (compression) takes place during construction; there is no lateral bulging.

2. The relationship between embankment compression and effective stress is known.

3. The pressures in the pore water and pore air are always equal and directly after compaction of the embankment material they are equal to atmospheric pressure.

4. The decrease in the embankment volume at any given elevation under the weight of the fill placed above is caused by compression of the air in the voids and solution of the air in the pore water.

5. Boyle's and Henry's laws are valid for this compression and solution.

6. No dissipation of pore water pressures from drainage occurs during construction.

From these assumptions the following equation relating pore pressure and embankment compression was derived by Hilf.

$$U = \frac{P_a \Delta e}{C_{ao} + he_w - \Delta e} + U_c \qquad \ldots \qquad \ldots \qquad (4.2)$$

where

U = induced porepressure (p.s.i)

P_a = absolute atmospheric pressure (p.s.i), i.e., 14.7 lbs. per square inch at sea level

Δe = embankment compression in percent of original total embankment volume

e_{ao} = volume of free air in the voids of the soil directly after compaction in per cent of original total embankment volume

e_w = volume of pore water, in per cent of original total embankment volume

U_c = Capillary tension, usually of small order and therefore ignored.

The numerical values for the initial air and water volume for use in equation 4.2 can be estimated from laboratory compaction tests and from previous experience with average embankment densities. The embankment compressibility can be derived from laboratory consolidation tests or from the average of field measurements for similar embankment materials.

[Equation 4.2 can also be used to compute the theoretical pore pressure at which all the air is forced into solution; i.e. when the embankment becomes completely saturated. This state which occurs when the compression of the specimen Δe becomes equal to the initial volume of air in the voids e_{ao} is described by the equation,

$$U = \frac{P_a \; e_{ao}}{h \; e_w} \quad \ldots\ldots \quad \ldots \quad (4.3)$$]

The derivation of equation 4.2 is given separately at the end of this chapter as an appendix.

4.6.5. By Bishop's modification of Hilf's method accounting for some pore pressure dissipation during construction.

One of the basic assumptions of Hilf's method as already stated is that no dissipation of pore water pressures from drainage occurs during construction. This is not strictly true in actual practice and Hilf's method normally gives higher values of pore pressures than would actually develop in the embankment.

An approximate method of determination of pore pressures by including the effect of pore pressure dissipation has been developed by Bishop and Li as a modification on the Hilf's method. The pore pressures developed in the fill due to the first load increment is computed with the help of Hilf's equation by assuming no drainage. Thereafter, a certain assumed percentage of pore pressure is allowed to dissipate. The ratio of this pressure relief to the total pore pressure is known as the dissipation factor. The conditions existing at the end of the first stage of assumed dissipation becomes the initial condition for the second load increment and the computation is successively repeated to obtain the construction pore pressures. The step computations by Bishop's method are based on the following assumptions:-

(i) Hilf's equation continues to be valid during step computations.

(ii) The degree of saturation remains unchanged before and after dissipation.

Figure 4.3 summarises the various steps(33) of the method. From consolidation test data on soil samples from the borrow area for the embankment material an effective stress ($\bar{\sigma}$) versus compression (Δ) curve is plotted. Corresponding to an arbitrary assumed stress increment $\bar{\sigma}_1$, $\Delta 1$ is read off from the curve, and by Hilf's equation, pore pressure U_1 indicated by the point 1a in the figure is computed. An assumed dissipation factor of say $\frac{1}{2}$ is then applied thus bringing down the pore pressure. (The dissipation factor may be anything like 1/3, 1/5, 1/6, 1/7, etc.). As the pore pressure decreases effective stress increases by the same amount resulting in additional compression, shown by point 1_b in the figure. The first stress increment allowing dissipation ends here and the second stress increment $\bar{\sigma}_2$ begins.

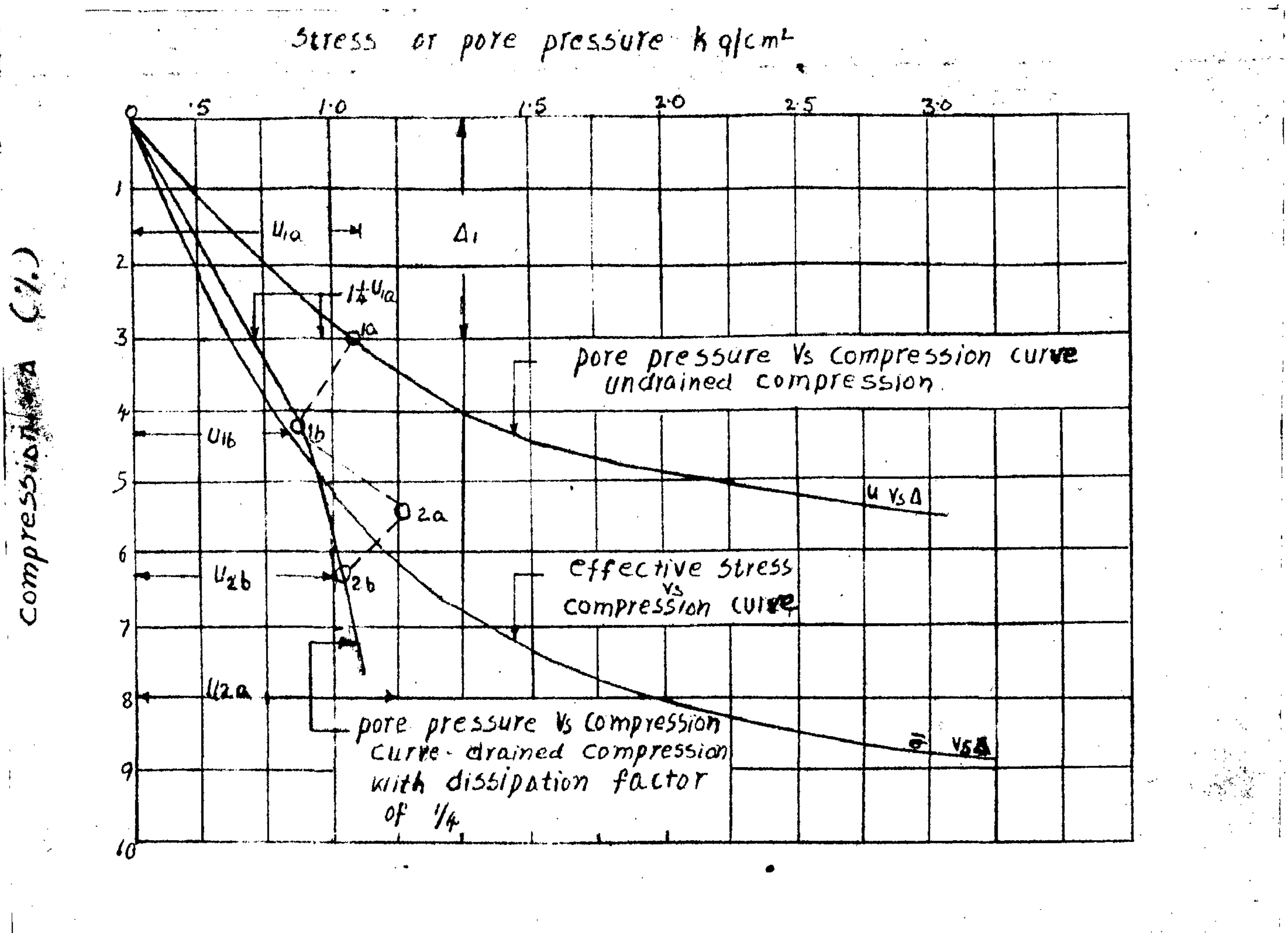


Figure 4·3 :- Computation of pore pressures allowing drainage.

From $\bar{\sigma}$ Vs. Δ curve, $\Delta 2$ corresponding to further increase in effective stress is obtained and Hilf's equation again applied for computing the resulting pore pressure. The modified values of the various constants to be applied in Hilf's equation will be as follows:

Initial air pressure P'_a = atmospheric pressure + Undissipated pore pressure.

$$= P_a + U_{1b}.$$

Total air volume $e_{a1} + h_{ew1}$.

$$= (e_{ao} + he w_o) \left(\frac{P_a}{P_a + U_{1b}}\right) \left[\frac{100n - \Delta 1b}{100n - (e_{ao} + hew_o) U_{1b}/(P_a + U_{1b})} \right]$$

where	$e_a + h_{ewo}$	=	Initial air volume
	P_a	=	Atmospheric pressure
	U_{1b}	=	Undissipated pore pressure
	n	=	Porosity
	Δ_{1b}	=	Compression for second stress increment.

By substituting values of the various constants, the pore pressure at the second stage of stress increment, allowing no dissipation is obtained by Hilf's equation and is shown by point 2(a). The pore pressure dissipation is again allowed for in the same ratio and the procedure repeated. A smooth curve passing through the origin and the points 1_b, 2_b, 3_b, etc., thus obtained gives the pore pressures allowing for dissipation according to the assumed dissipation factor.

4.7. Factors influencing construction pore pressures.

The different factors which influence the construction pore pressures are narrated below.

4.7.1. Construction water content: The water content at which the embankment is constructed has the largest influence on the magnitude of the pore pressures which develop. A high dam constructed of almost any type of soil with an average water content on the wet side of standard Proctor optimum water content will develop higher construction pore water pressures than with a water content on the dry side of optimum. Embankments constructed during wet seasons are likely to have more pore pressures than those constructed during drier seasons.

4.7.2. Grading and fineness of soils: The magnitude of construction pore pressures also depends on the type of embankment soil. Embankments of well graded clayey sands and sand-gravel-clay mixtures have the highest construction pore pressures because of their relatively low initial air content and relatively high compressibility. Embankments of uniform silts and fine silty sands are the least susceptible to the development of pore pressures because of their high initial air content and relatively low compressibility. Also, even though less information is available to support the conclusion, there is evidence that embankments of moderate height of very fine grained clayey soils compacted near optimum water content, may not develop appreciable construction pore pressures because of high capillary forces and relatively large initial air content.

4.7.3. Permeability of soils: The relationship between the coefficient of permeability of the impervious core and the development and dissipation of construction pore pressures is not well known. On the basis of the limited information available Sherard gives a rough estimate of the relationship between the coefficient of permeability of relatively thick impervious cores and the dissipation of pore pressures in the central part of the cores as follows:

Coefficient of permeability.	Dissipation of pore pressures during construction.
Less than 0.5 ft./year (0.5×10^{6} cm/sec.).	No dissipation of pore pressure.
0.5 to 5 ft./year (0.5 to 5.0×10^{-6}) cm/Sec).	Some dissipation.
5 to 50 feet/year(5 to 50×10^{-6} cm/sec).	Appreciable dissipation.
More than 50 ft./year (50×10^{-6} cm/sec).	Complete dissipation.

4.7.4. Drainage layers and length of drainage path: Drainage layers or the pervious shell surrounding the core assist dissipation of pore pressures in their vicinity. With the increase of length of drainage path towards the centre of the core the influence of the drainage layer will be less and less. At the outside edges of the impervious core, construction pore pressures are always reduced by drainage of pore water to the adjacent more pervious zones. Consequently the pore pressures will be more in thick cores than in thin cores.

4.7.5. Speed of construction and periods of work stoppage:

If the embankment construction proceeds very fast high pore pressures remain in the embankment due to lack of sufficient time for any appreciable dissipation.

Fairly high pore pressures develop within the core of a dam during the early part of construction and then are later dissipated to some degree by drainage during a period of work stoppage. They subsequently increase at a rate which is appreciably smaller when construction commences again. Hence delays in construction may have a doubly beneficial influence on the stability of earth dams during construction.

The reason for the decrease in the rate of pore pressure development after periods of work stoppage is that the embankment becomes denser and less compressible as the result of the consolidation which occurs. When work starts again the soil structure is stiffer than before, and a smaller percentage of the added weight of the new embankment is thrown to the pore fluid.

4.8. Control of construction pore pressures.

High construction pore pressures exist only during the first few years of the life of the dam. Hence it is not very necessary to use more conservative and expensive designs than would otherwise be required. A number of design and construction procedures are available to reduce the construction pore pressures and thus eliminate the necessity for making the slopes flatter for the "during-construction" stability condition. These procedures are described below.

4.8.1. Control of construction water content: The maximum construction pore pressures can be limited by compacting the impervious section at an average water content a few per cent below the standard Proctor optimum.

The experience of the USBR indicates that by placing impervious materials at an average water content between 1 and 3 percent below Proctor's optimum, the pore pressures can be kept within reasonable values. These values are usually limited to a maximum of about 30 per cent of the weight of the overlying embankment.

However, although this practice is accepted by many experienced engineers, many others including Middlebrooks of the Army Corps of Engineers argue that the dam built with low water content will settle on subsequent saturation and the embankment will be too brittle to follow differential settlement without cracking. They argue that post construction settlement on saturation should be the over-riding consideration, and not pore pressures, in determining the construction water content.

The following methods are available for controlling construction pore pressures without reducing construction water content below Proctor's optimum.

<u>4.8.2. Core width and drainage:</u> The impervious section can be made thinner so that high construction pore pressures will have less influence on the stability of the dam and will dissipage more rapidly.

Measurements have demonstrated that during construction periods of normal length, little pore pressure dissipation from drainage occurs in the central portions of thick core dams composed of fine soils. On the other hand, in the peripheral portions of the cores adjacent to the pervious zones, the pore pressures are completely dissipated. The rate of dissipation in the centre is approximately inversely proportional to the square of the core width, so that the construction pore pressures can be considerably reduced by reducing the thickness of the core.

In homogeneous dams, or other dams at sites where only small quantities of pervious embankment material are available, dissipation of construction pore pressures can be accelerated by judicious installation of an internal system of downstream drains. In special cases the drains are provided on the upstreamside also. Different arrangements of drains are shown in fig. 4.4.

<u>4.8.3. Delayed construction:</u> ~~Delayed const~~ Longer construction periods and periods of work stoppage at intervals will reduce construction pore pressures appreciably as explained earlier in para 4.7.5.

~~xxxx~~

4.8.4. Lower safety factors: Another reasonable procedure when high construction pore pressures are anticipated is to accept a lower safety factor for the during construction condition.

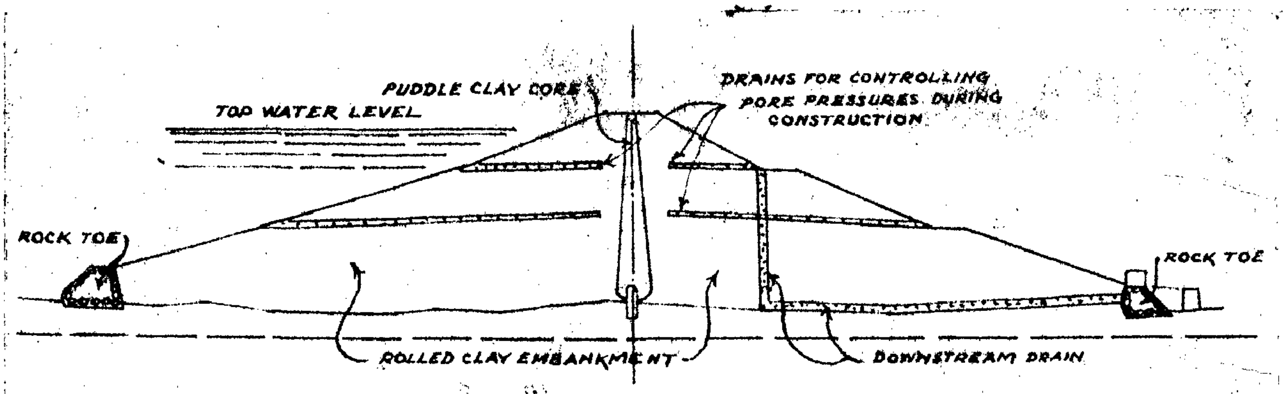


FIG. 4·4 : DRAINS FOR CONTROLLING CONSTRUCTION PORE PRESSURES.

Since a construction slide in a compacted earth dam does not result in a failure of the catastrophic type which involves loss of life and great property damage, it is within the realm of acticity where a calculated risk is permissible. Except when the dam is founded on a deposit of sensitive clay or silt (where the movement can be larger) the worst results of a construction slide due to high pore pressures can only be: (1) a few feet of relatively slow movement in a portion of the embankment; (2) repair costs which are a fraction of the original cost of the dam; (3) some delay in the completion of the construction; and (4) embarrassment for the engineer if the client has not been properly warned in advance about the calculated risk.

The selection of minimum allowable safety factors against shear failure at all times in the life of an earth dam is a difficult problem. A computed safety factor of 1.5 is usually specified for the full reservoir condition, and a somewhat lower value is often considered acceptable for the condition of reservoir drawdown. For the during-construction condition if realistic estimates of the embankment strength and pore pressure are made, a computed safety factor of slightly greater than unity can reasonably be used provided that the construction pore pressures and embankment strains are carefully observed by piezometer and cross-arm installations. Study of the measured pore water pressures and the movements of the cross-arm or other monuments during construction will enable the engineer to anticipate an imminent slide in time to take preventive action before any large damage can occur.

4.9. Construction pore pressures measured in dams.

Pore pressures during construction are being measured by the USBR by Piezometer installations since 1935. A detailed analysis of these measurements from twenty six dams constructed between 1936 and 1952 which was published by Gould[63] constitutes one of the largest and most valuable concentrations of experience available to the profession. The dams included in Gould's study cover the whole range of soil types encountered by the USBR and a fairly wide range of average construction water contents.

Measured construction pore pressures in some USBR dams are given in figure 4.5. Contours of equal pressure have an oval configuration as seen therein, conforming to the shape of the central impervious zone of the dam. The highest pressures develop in the central portion of the impervious core where the overburden pressures are the highest and the effect of drainage to the adjacent pervious zones is the least.

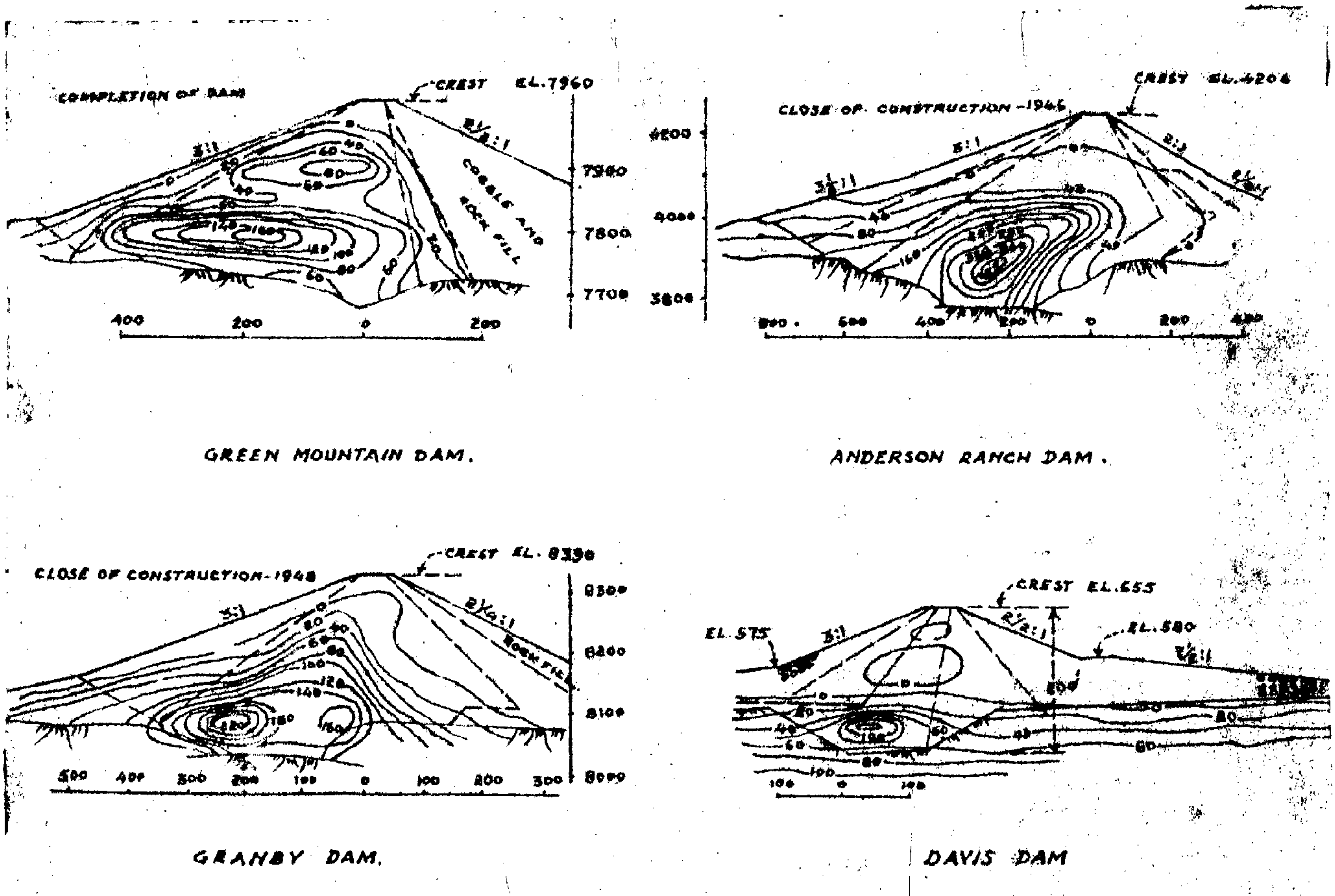


FIG. 4.5 : MEASURED CONSTRUCTION PORE PRESSURES IN SOME USBR DAMS.

As a major part of his study Gould compared the measured pore pressures with those predicted on the basis of Hilf's method. He found that the two agreed roughly and that the main difference between the predicted values and the measured values at a given dam could be explained reasonably from the construction history. For example the measured pressure was higher than predicted when concentrations of wet and compressible soil had been placed in the embankments. Drainage relief and surface tensions in the pore water caused the pore pressures in the impervious zone to be lower than predicted and differential settlements lead to different pressures than predicted because they caused transfers of total stress within the embankments.

Though the study indicated that the measured and calculated pore pressures were roughly equal and hence that the theory is probably satisfactory for the type of soils analysed it does not by any means follow automatically that the measured pore pressures could have been estimated reliably in advance of construction. In the theoretical prediction, small differences in the assumed construction water content and density or in the average soil type make large difference in the computed pore water pressures. In order to obtain an accurate prediction of the construction pore pressures in the design stage the engineer would have to be sure that he was testing the average soil from the borrow pit and that the moisture density control during construction would result in an embankment in which the average water content and density approximated closely the values assumed on the basis of the preliminary laboratory tests. In addition, he would have to estimate the compressibility of the embankment either from previous measurements or from laboratory consolidation tests. Since all these elements, required for the theoretical prediction, are difficult to estimate reliably, except perhaps under ideal conditions, the theory should only be considered as a general guide for design purposes.

In India, methodical observations and study of construction pore pressures have been carried out at Nanaksagar Dam[32] and Moosakhand Dam[33] about 100 ft. high both in the State of Uttar Pradesh. The type of soil and the construction equipment used in both cases are similar but the placement moistures are very much different. At Nanaksagar Dam the placement moisture is one per cent wet of optimum while at Moosakhand Dam it is three per cent dry of optimum. The value of Skempton's pore pressure ~~xxtixx~~ ratio $\bar{B}$ has been found to be 1.1 for Nanaksagar Dam and 0.8 for Moosakhand Dam.[33]. This clearly brings out the fact that when the placement moisture is more the pore pressures are more. The pore pressures computed by Hilf's equation are found to be higher than the observed values in both the dams. Computations of pore pressures by Bishop's modification of Hilf's method, with dissipation factor of 1/7, yield pore pressure values in close agreement with the observed data for Nanaksagar Dam. The value of dissipation factor for Moosakhand Dam has been found to be $\frac{1}{4}$. This indicates that the rate of dissipation of pore pressure is slow when the dam is compacted wet of optimum. There may however be other factors also like permeability of adjacent zones, etc.

APPENDIX TO CHAPTER IV.

DERIVATION OF HILF'S[25] EQUATION FOR CONSTRUCTION PORE PRESSURES BASED ON BOYLE'S AND HENRY'S LAWS.

4.A.1. Work done by different researchers.

The problem of fluid pressures in unsaturated soil masses was first attached theoretically by Brahtz and experimentally by Hamilton both of the USBR. They correctly combined Boyle's and Henry's laws of ideal gases to determine air pressure in a sealed compressed specimen of soil. They incorrectly assumed however that this air pressure was identical with the pore ~~pressure~~ water pressure. The vapour pressure of water and the effects of surface tension were neglected. Hilf showed how Brahtz's equation could be combined with field or laboratory compression data to estimate the magnitude of pore pressures during construction of dams and he provided a limited amount of field evidence indicating that the method gave results that were on the safe side.

4.A.2. Compressibility of air and pore air pressure in dry soils.

Figure 4.A.1 represents a soil mass which is devoid of water such as very fine sand or silt which has been oven dried, disintegrated and compacted. If a sample of this soil were

sealed with a thin rubber membrane and subjected to an external ambient compressive stress σ, it is clear that a portion of the applied load would act on the soil skeleton and cause it to undergo deformation resulting in a decrease in volume of the soil mass. In the sealed sample the air in the voids could be compressed. Since the compression of gases follows Boyle's law, if the initial void volume of the compacted soil is known and the amount of volume change is measured, the pressure in the compressed air

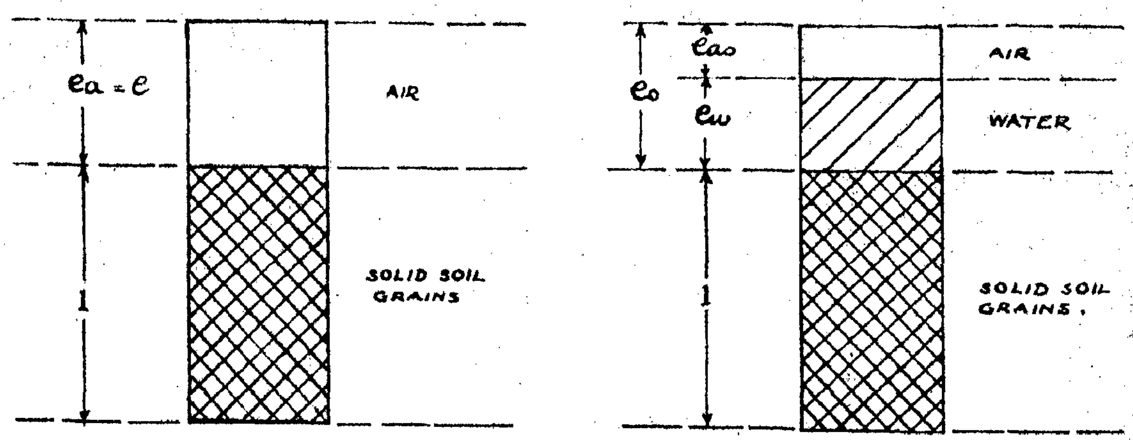


FIG. 4A-1: DRY SOIL

FIG. 4A-2: PARTIALLY SATURATED SOIL.

can be determined. It will be assumed that temperature remains constant and that compressibility of the individual soil particles is negligible compared to the compressibility of the soil skeleton.

Let $\theta_a = \theta$ = Volume of air in the soil mass of volume $1 + \theta$; let P_a equal atmospheric pressure; let the subscripts 0 & 1 denote initial and final conditions respectively. Boyle's and Charle's law for an ideal gas or mixture of ideal gases is:

$$PV = nRT \qquad \ldots \qquad \ldots \qquad (4.A.1)$$

where P = absolute pressure in atmoshperes

V = Volume in cubic centimeters

n = number of molecules

T = absolute temperature in $^{\circ}K$, i.e. $(273 + ^{\circ}C)$

R = Universal gas constant = 82.06 $\frac{cm^3 . \ atmos.}{^{\circ}K.mols.}$

For the pressures and temperatures ordinarily encountered in soil mechanics the soil air can be considered to obey the gas laws. The density of air is 1.2928 gms. per litre and its molecular weight in gms. is 1.2928 x 22.4 = 28.960. Hence equation 4.A.1 can be written as

$$P\theta_a = \frac{W_a}{28.960} RT \qquad \ldots \qquad \ldots \qquad (4.A.2)$$

where W_a = weight of air in gms.

For the condition of compression with no escape of air it follows that

$$\frac{W_a}{28.960} = \frac{P_a \theta_{ao}}{RT} = \frac{(P_a + U_a)\ \theta_{a1}}{RT}$$

where P_a = initial air pressure (atmospheric) and

U_a = final air pressure (above atmospheric)

from which $U_a = P_a \left(\frac{\theta_{ao} - \theta_{a1}}{\theta_{a1}} \right)$

Designating $\Delta e = e_{ao} - e_{al}$,

then $$U_a = \frac{P_a \Delta e}{e_{ao} - e} \quad \ldots \quad \ldots \quad (4.A.3)$$

which gives pore air pressure in terms of initial air void ratio and change in void ratio for a perfectly dry sealed soil mass.

By neglecting the small specific area of contact between particles of the soil skeleton, the average contact stress or effective stress normal to every plane in the compressed soil mass can be found from Terzaghi's equation

$$\bar{\sigma} = \sigma - U = \sigma - U_a$$

for the dry soil mass. The compressibility of a soil mass has been defined in terms of volume change by the coefficient

$$m_v = \frac{\Delta e}{(1+e_o)\Delta \bar{\sigma}}$$

Bratz defined compressibility of the soil skeleton by the term "consolidation modulus" which is the reciprocal of m_v when $e_o = 0$. He recognised that the consolidation modulus, E, was not constant but assumed it stationary in the range of stresses dealt with using an average value.

4.A.3. Solubility of air.

Air is soluble in water to an extent that must be considered in soils. According to Henry's law

$$P_y = \bar{p} = Hx \quad \ldots \quad \ldots \quad (4.A.4)$$

where P = total absolute pressure (air plus water vapour) in atmospheres.

p = partial pressure of air in atmospheres

y = Molecular fraction of air in the air-water vapour mixture.

H = Henry's constant in atmospheres per mol. of air

X = mol fraction of dissolved air in the liquid phase

= ratio of the number of gram formula weights of the gas in solution to the sum of that number and the number of gram formula weights of the water in which the air is dissolved.

The molecular weight of air = 28.960

The molecular weight of water = 18.015

$$\therefore \frac{p}{H} = \frac{W_aD/28.960}{W_aD/28.960+W_w/18.015} \quad \ldots(4.A.5)$$

where W_aD = weight of air dissolved

and W_w = weight of water

For pressures encountered in soil mechanics $\frac{W_aD}{W_w}$ for compacted soils is very small (approximately 0.0001 for 100 p.s.i. air pressure); hence equation 4.A.5 can be written approximately

$$\frac{p}{H} = \frac{W_aD/28.960}{W_w/18.015}$$

$$\therefore W_aD = \frac{1.609}{H} .p.W_w \quad \ldots \quad (4.A.6)$$

4.A.4. Pore pressure in unsaturated soils.

Consider a compacted soil mass of volume $1 + e_o$ as in figure 4.A.2 with air volume e_{ao} at atmospheric pressure P_a. It is then compressed at constant temperature T without drainage to a void ratio e_1. For pressures used in soil mechanics the entire change in void ratio $e_0 - e_1$ can be considered as occurring in air since the bulk modulus of water is about 300,000 p.s.i. and the bulk modulus of mineral grains is about 10 times higher than that of water (5,000,000 p.s.i. for quartz.). The soil skeleton is considered to be reduced in volume without change in solid volume by means of rearrangement of particles or elastic bending of the grains. The soil water is of volume e_w for conditions e_0 & e_1 but it contains more air in solution in the latter condition because of Henry's law. Consider the weight of air, free (designated by F) and dissolved (designated by D) prior to and subsequent to compression.

From equation 4·A·2

$$W_aF_o = 28.960\,\frac{P_a\,e_{ao}}{RT},\quad W_aF_1 = 28.960\,\frac{P_a+U_a}{RT}\,e_{a1}$$

From equation 4.A.6

$$W_aD = \frac{28.960}{18.015H}\,P_a\rho e_w,\quad W_aD_1 = \frac{28.960}{18.015H}\,(P_a+U_a)\rho e_w$$

Since no drainage occurred

$$W_aF_o + W_aD_o = W_aF_1 + W_aD_1,\quad \text{and}$$

$$\frac{P_a\,e_{ao}}{RT} + \frac{P_a\rho e_w}{18.015H} = \frac{(P_a+U_a)e_{a1}}{RT} + \frac{(P_a+U_a)\rho e_w}{18.015\,H}$$

Multiplying both sides by RT and putting $h = \dfrac{RTS}{18.015\,H}$

$$P_a(e_{ao} + h e_w) = (P_a+U_a)(e_{a1}+h e_w)$$

Solving for U_a,

$$U_a = P_a\left(\frac{e_{ao} + h e_w}{e_{a1} + h e_w} - 1\right)$$

Since $e_{ao} - e_{a1} = \Delta e$

$$U_a = P_a \Delta e / e_{a1} + h e_w$$

$$= \frac{P_a \Delta e}{e_{ao} + h e_w - \Delta e} \quad \ldots \quad \ldots \quad (4.A.7)$$

The quantity $\theta_{ao} + h\theta_w$ is the total volume of air (free and dissolved) in the compacted soil prior to compression. When $\Delta\theta$ equals θ_{ao} the air pressure becomes

$$U_{as} = \frac{P_a\,\theta_{ao}}{h\theta_w}$$

which gives the pressure required to dissolve the air completely in accordance with Henry's law.

4.A.5. Pore water pressure.

The pressure in the pore water of an unsaturated soil which is the pressure in the fluid in contact with the soil skeleton is given by the formula

$$U = U_a + U_c$$

where U_c is the capillary pressure for the existing curvature of the menisci. For an unsaturated soil (U_a less than U_{as}) U_c will always be negative; hence U is less than U_a. If the soil is compressed to saturation without drainage $U_a = U_{as}$ and $U_c = 0$; since no free air is present and the curvature of the menisci is zero. Thus in the compression of a compacted cohesive soil without drainage both U_a & U_c vary with change in void ratio. The value of U_c must be measured or estimated. The complete equation for pore water pressure for the condition of no drainage is

$$U = \frac{P_a\,\Delta\theta}{\theta_{ao} + h\theta_w - \Delta\theta} + U_c \quad \ldots \quad (4.A.8)$$

CHAPTER V.

FIELD MEASUREMENTS OF PORE PRESSURES.

5.1. General.

Field observations of pore pressures and deformation are made during construction, impounding and drawdown to check the performance of the dam. As discussed in the previous Chapters the prediction of pore pressures within necessary narrow limits during the design stage is difficult. Their direct measurement during construction and during during operation of the dam provides an early indication of any unsafe departure from the design assumptions and permits, if necessary, modification of the construction programme or modification of the cross section of the dam at an early stage before large damages can occur. The devices used for the measurement of pore pressures are called Piezometers. They are used both in embankments and foundations. Foundation Piezometers have to be installed in drill holes in the foundation. Embankment

Piezometers can be installed by drilling holes into the embankment after its construction or they are more commonly placed in the embankment-fill as it is being constructed. Piezometers should be located in such a way that they will provide information about the magnitude and distribution of pore pressures in critical areas. Generally a battery of Piezometers is set into one or more transverse sections of the dam to give a general picture of the pore water distribution and additional Piezometers are placed at spots where specific information is needed such as along abutment contacts. In a typical installation these Piezometers are installed at a critical cross section starting right from the bottom most elevation of the fill upto a few feet below the maximum reservoir level. The Piezometers are located mostly in the central impervious zone of the fill in two or more tiers, with suitable horizontal distances between the Piezometers in a tier and with suitable vertical intervals between the tiers. It is desirable to include in each tier one or two Piezometers in the outer shell both in the upstream and downstream portions beyond the impervious zone to study the magnitude of pore pressures developed in the upstream during sudden drawdown condition and the effect of inclined filters on the dissipation of pore pressures in the downstream. The foundation Piezometers are also normally installed at the same cross section as the one chosen for the installation of embankment Piezometers. The foundation Piezometers are installed at different elevations with suitable horizontal and vertical intervals and also in particular spots in the foundation considered critical. Fig.5.1 shows typical arrangement of installation of Piezometers in a Dam. The arrangement is shown both in

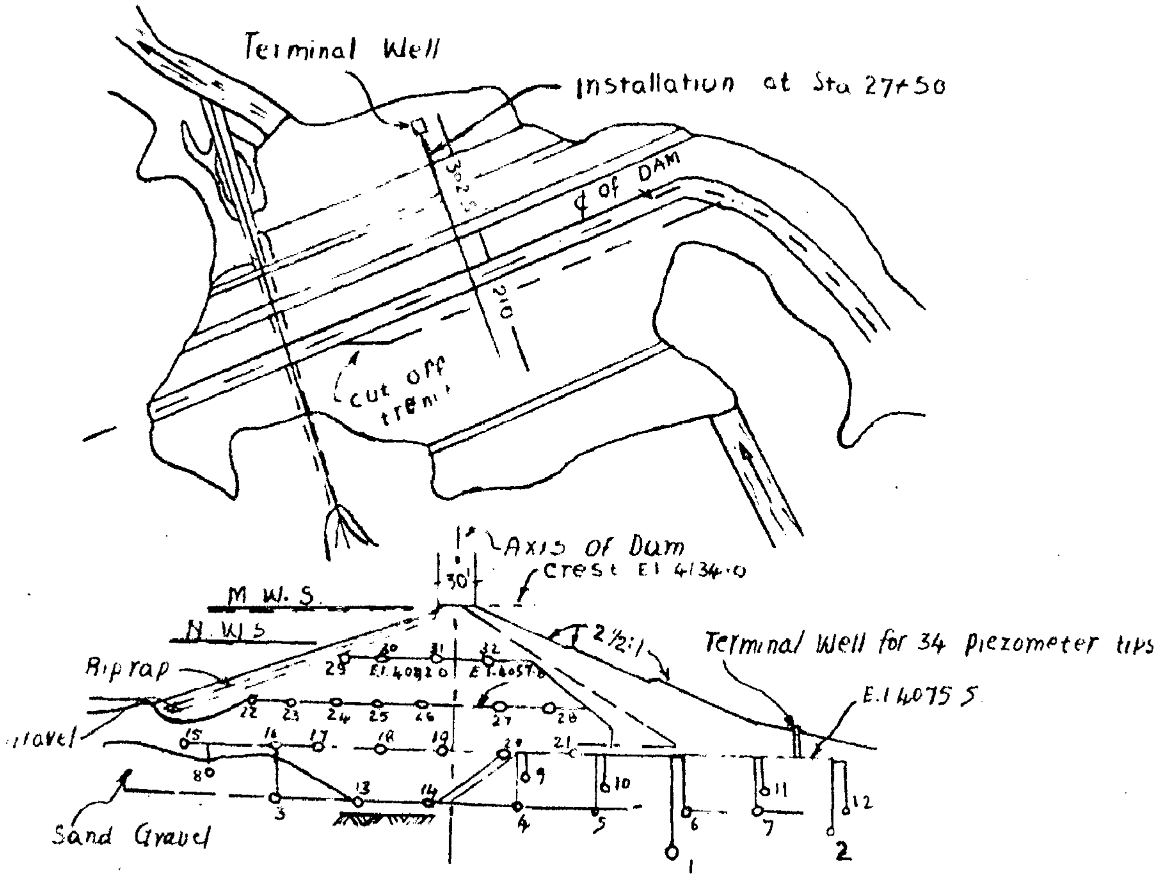


Figure 5.1 :- Typical USBR piezometer installation key hole Dam

the plan of the dam and in the instrumentation cross section. The various types of Piezometers available for use are described in the following paragraphs.

5.2. Simple vertical open pipe or "Well point" Piezometer.

In pervious sands and gravels pore pressure at any point can be measured by sinking a simple vertical open pipe or a "well point" to the required point and measuring the height to which the water rises in it. This type is not suitable for fine grained soils as the water takes too much time to seep from the impervious soil into the open pipe and reach equilibrium. Owing to the quantity of water required and the time lag this type cannot register rapid changes in pore pressures.

5.3. Porous tube Piezometer.

The porous tube Piezometer is a modification on the open pipe and is designed to overcome the above disadvantages. This was first developed by Casagrande and is therefore known as Casagrande type Piezometer also. Here the perforated or open end of the pipe is replaced by a porous cylinder connected to a small bore tube of rigid polythene or similar material. The porous cylinder is usually of vitrified bauxite in Great Britain and the Continent and of alundum in the United States. The response in this type of Piezometer is greatly accelerated by the reduced volume of water required to indicate a change in pressure. Where the Piezometer is embeded in the impervious section of the dam the porous tip is ~~often, but not always,~~ surrounded by a small pocket of sand which acts as a collection reservoir for the pore water and increases the effective area of the porous tip. Fig.5.2 shows details of the porous tube Piezometers. The size of the polythene tube (usually of 1 c.m. or 0.5 inch bore) has to be sufficiently large to permit a sounding device to be lowered into it to measure the water level accurately. The sounding device most commonly used is an electrical gadget which short circuits when the water surface is touched and registers on an ohm meter.

In the perforated tube Piezometer as well as in the porous tube Piezometer if the pressure head is high enough to cause the water to rise above the embankment surface it is measured either by extending the tube above the embankment surface or more conveniently by using a Bourdon gauge.

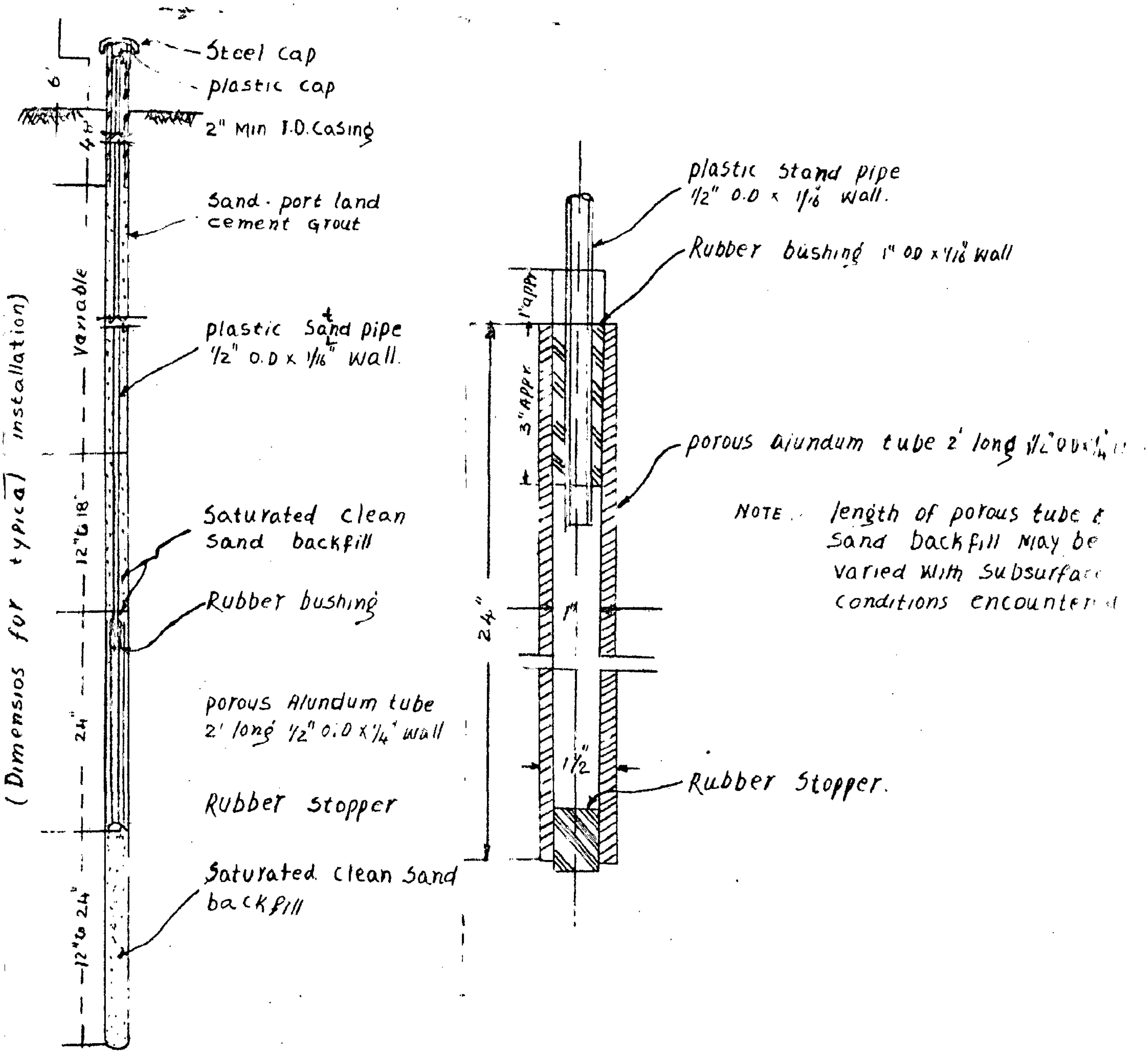


Figure 5.2 :- porous tube piezometer.

Both the perforated or open tube Piezometer and the porous tube Piezometer come under the category of direct reading Piezometers. The vertical tubes of these direct reading Piezometers are an obstacle to construction operations for the embankment and hence remote reading Piezometers are generally used for measuring pore pressures. The Piezometers described in the succeeding paragraphs are remote reading Piezometers.

5.4. USBR Piezometers with single tube.

The earliest type of Piezometer developed by the USBR consists of a porous disc of alundum connected to a pressure gauge by a small bore plastic tubing filled with water. The flexible plastic tubing~~s~~ from the porous disc (called the Piezometer tip or the pore pressure cell) ~~are~~ is led horizontally, burried in trenches in the fill and thus got out of way for the construction operations for the embankment to be continued without obstruction. The tubes are led to a central measuring ~~xx~~ point, called the terminal well at the downstream toe of the dam or in the foundation gallery where such a gallery is provided. The terminal well is a reinforced concrete room completely equipped with pressure gauges, pumps, air trap and water supply.

Where the dam foundation is of relatively soft soil there is a tendency for the base of the dam to spread and consequently to stretch horizontal Piezometer lines. Hence at some dams extra lengths of tubing are placed in the trenches in the form of snake like undulations in order to allow the base of the dam to spread without causing tension in the tubing. The trenches are carefully back filled with compacted fine grained impervious soil before raising the embankment further so that any water will not seep along the lines and alter the pore pressure at the Piezometer tip.

5.5. USBR Piezometers with double tubes.

The double tube Piezometer was developed by the USBR around 1940 as an improvement on the single tube system which caused difficulty with air bubbles. This difficulty with air bubbles was practically not present in the vertical tube Piezometer system. In recent years there have been a number of experiences particularly with foundation Piezometers in which gas from some unknown source enters the system causing very high pressure readings in the Bourdon pressure gauges. In some of these cases when the gauge was removed and the pressure measured in a vertical stand pipe, the gas simply bubbled to the surface and the water pressure in the stand pipe was much less than was previously measured with the Bourdon gauge. Thus, in vertical tubes, the air bubbles rise rapidly to the water surface without influencing the measurements. (However if a Bourdon gauge is necessary to measure the pressure some difficulty with air bubbles may develop.)

The USBR double tube Piezometer overcomes the difficulty with air bubbles present in the single tube system which is not however present in the open vertical tube Piezometers.

The double tube system allows circulation of water through the system to remove air. When the pore water pressure is high, the air is dissolved in the water and causes no trouble. But under low and moderate pressures it remains in the form of bubbles which cause air locks in the line resulting in erratic readings. Even though with the double tube system flushing out of air by circulating fresh water is possible, in some cases more and more bubbles enter into the lines to replace those which are flushed out and this becomes a continuous process. Several approaches have been made to the solution of thisproblem.

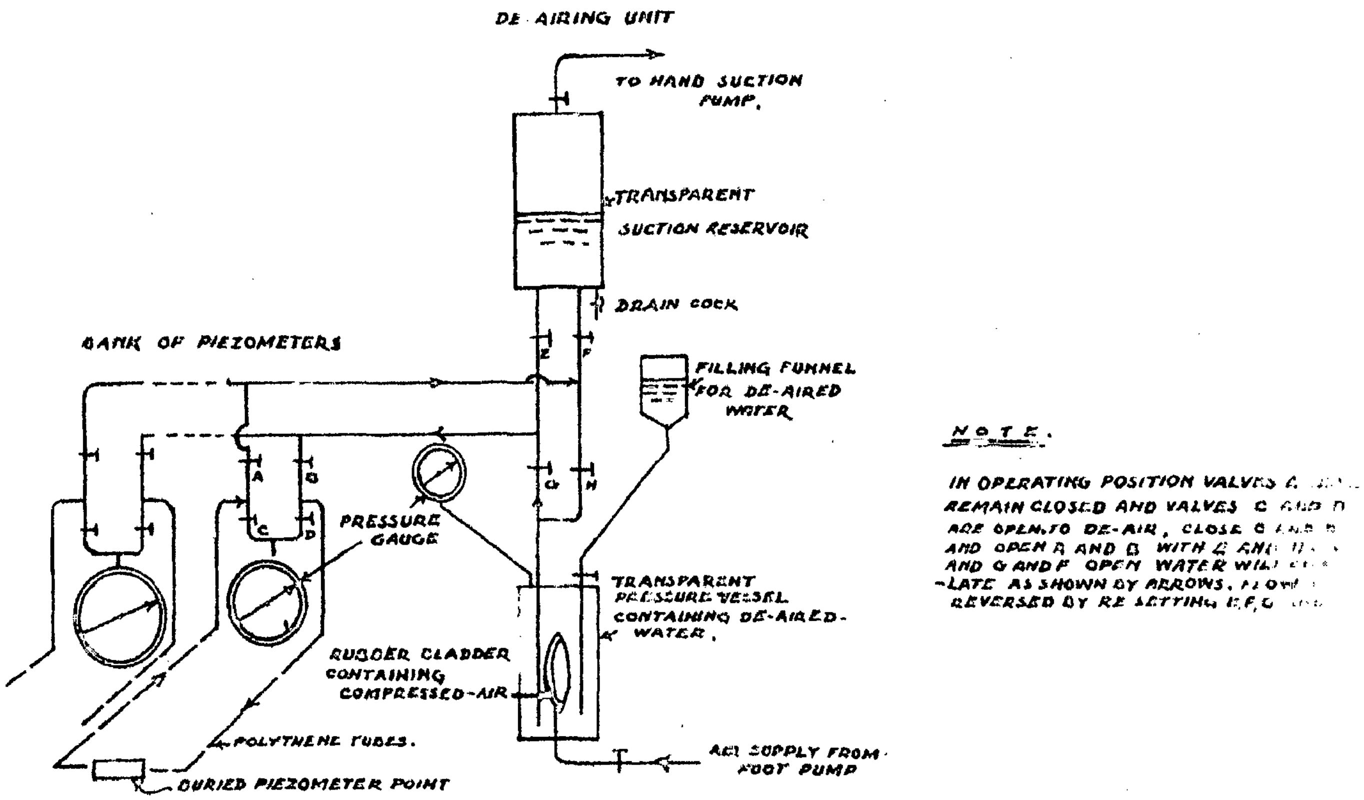


FIG. 5.3 : LAYOUT OF DE-AIRING UNIT.

At the Building Research Station in England, experiments have lead to the development of methods of de-airing the water. These are described in Ref.52. The general lay out of the de-airing system is shown in figure 5.3. It is designed to circulate de-aired water under a steady pressure through the pressure measuring system in either direction, the return water being collected in a transparent suction tank where the presence of air bubbles can be observed. One de-airing apparatus serves a bank of a dozen or more Piezometers and if the pressure is positive through the system it may require only periodic use.

In the USBR, the Engineers have recently begun to use porous stone tips which have much finer pores and higher "air entry" (or "bubbling") pressure; because laboratory researches

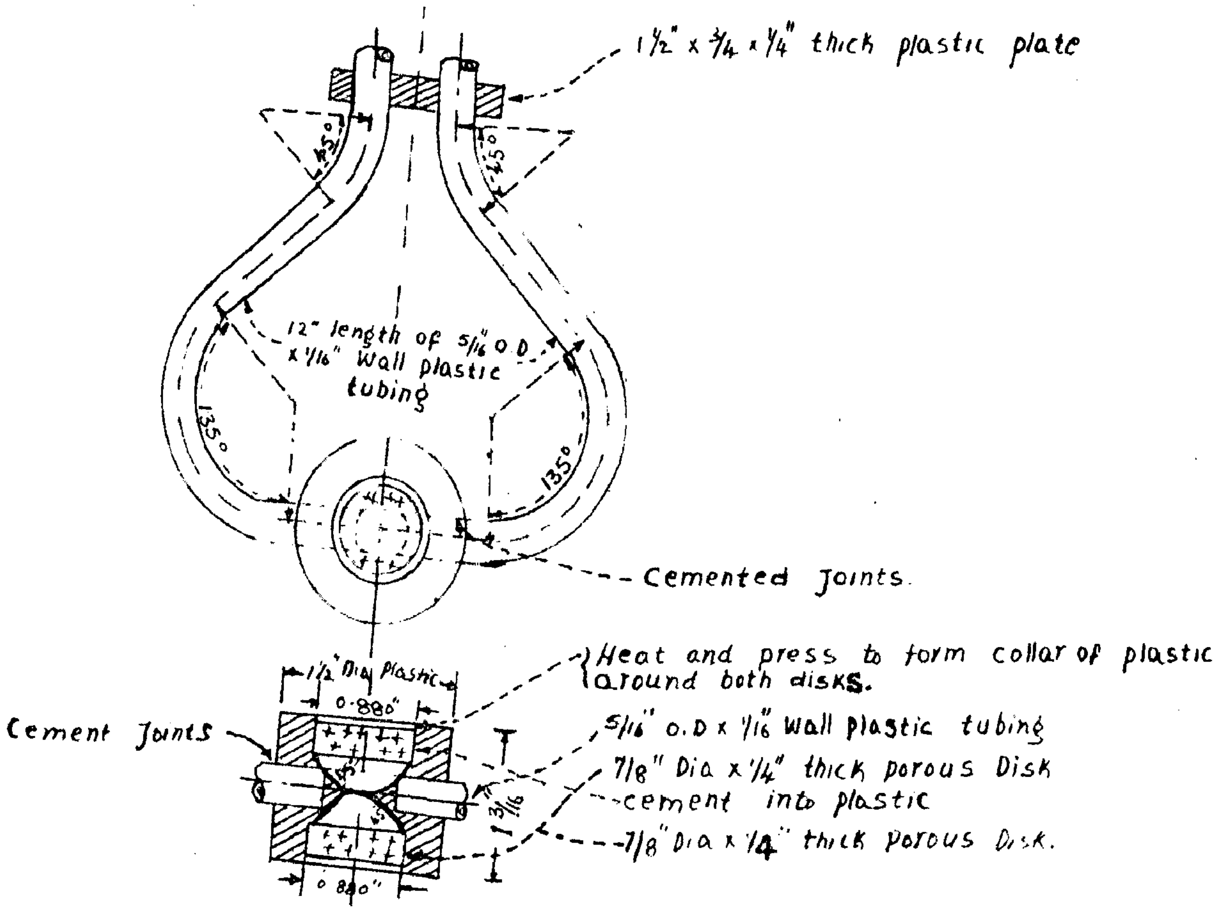


Figure 5.4 :- Embankment piezometer tip

indicate that air bubbles enter fine grained stones less easily than coarse grained ones. This innovation seems very promising although there is not enough experience available now to indicate the degree of improvement. But since the fineness of the stone should have little, if any, influence on the measured pore water pressure in a no flow system, while it probably has a considerable influence on the penetration of air it seems advisable to use fine stones for future installations of Piezometers. Such Piezometers with high air entry pressures were used at Selset Dam in England.

In these Piezometers there are two types; one for being installed in the embankment at different elevations as the embankment work comes up to those elevations and the other for being installed in drill holes in the foundation.

<u>5.5.1. Embankment type Piezometer tip</u>. This is illustrated in fig.5.4. This tip has two 7/8 inch diameter and ¼ inch thick porous stone discs sealed and crimped into the tip. When these tips are installed in the dam embankment the flat sides of the discs should be placed horizontally.

<u>5.5.2. Foundation type Piezometer tip.</u> The earlier and recent types of foundation Piezometers are described below:

(i) Prior to 1955, in the USBR, all foundation Piezometers were of the type shown in figure 5.5. ~~Th~~ In this type of Piezometer assembly the porous stone tip contacted the foundation pore fluid pressures through a pipe extending vertically downward or laterally into the dam foundation. This tip contained a single porous stone disc, and the section of the tip below the disc was extended to enclose a 4-inch length of 7/8 inch OD plastic pipe. Plastic pipe in 20 foot sections having 7/8 inch OD and 1/8th inch wall thickness formed the extensions from these tips to a

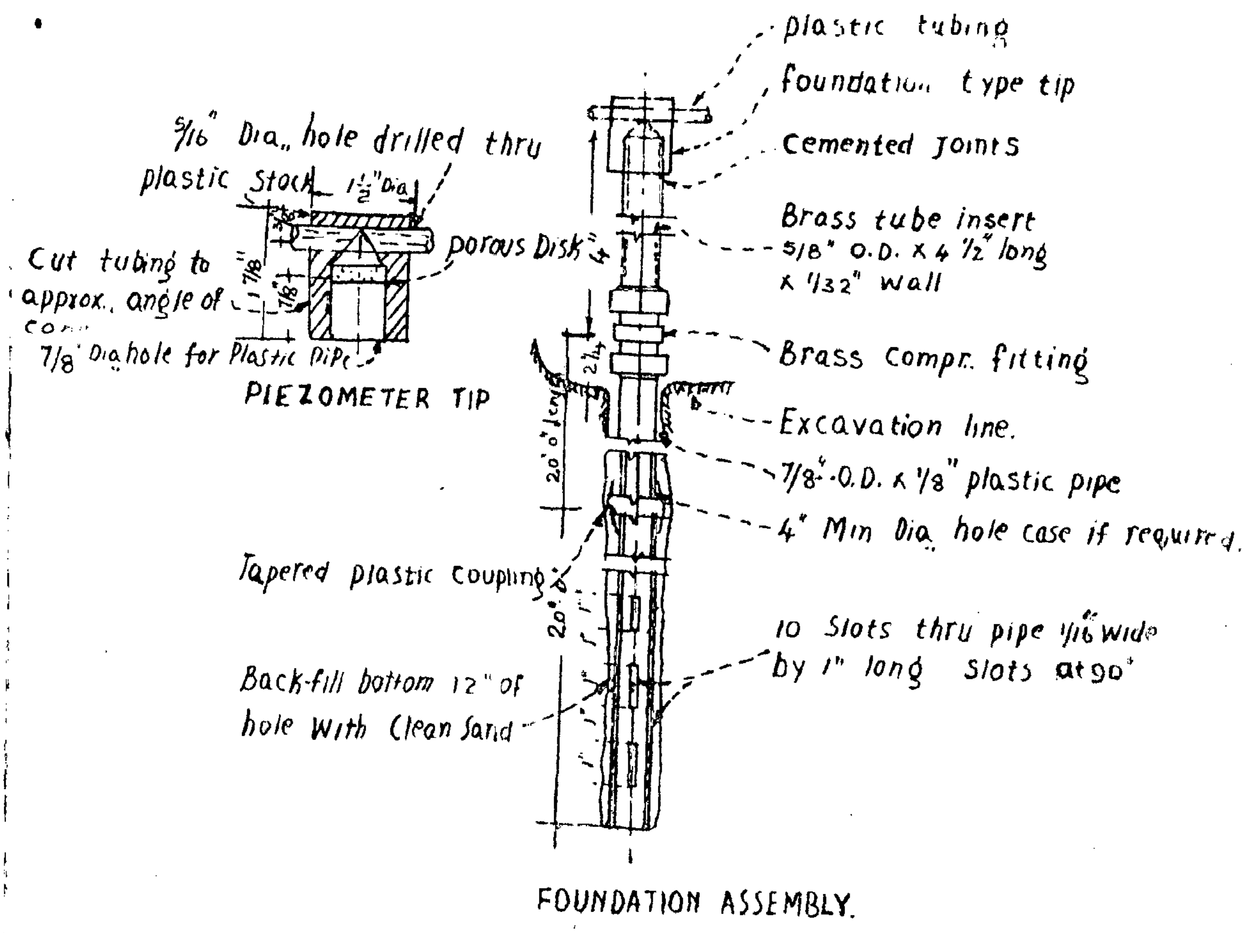


Figure 5.5 ; FOUNDATION PIEZOMETER TIP (with pipe Extn)

predetermined elevation in the foundation. Sections of the pipe were cut to required lengths with a hacksaw. Lengths of pipe were coupled together by special plastic connections which were cemented to the pipe sections.

(ii) The type of foundation Piezometer tip currently being used also contains a single porous disc but contacts the foundation pore fluid pressures directly instead of through an extended pipe.. This type is illustrated in figure 5.6. The Piezometer tip itself is placed in the foundation drill hole at

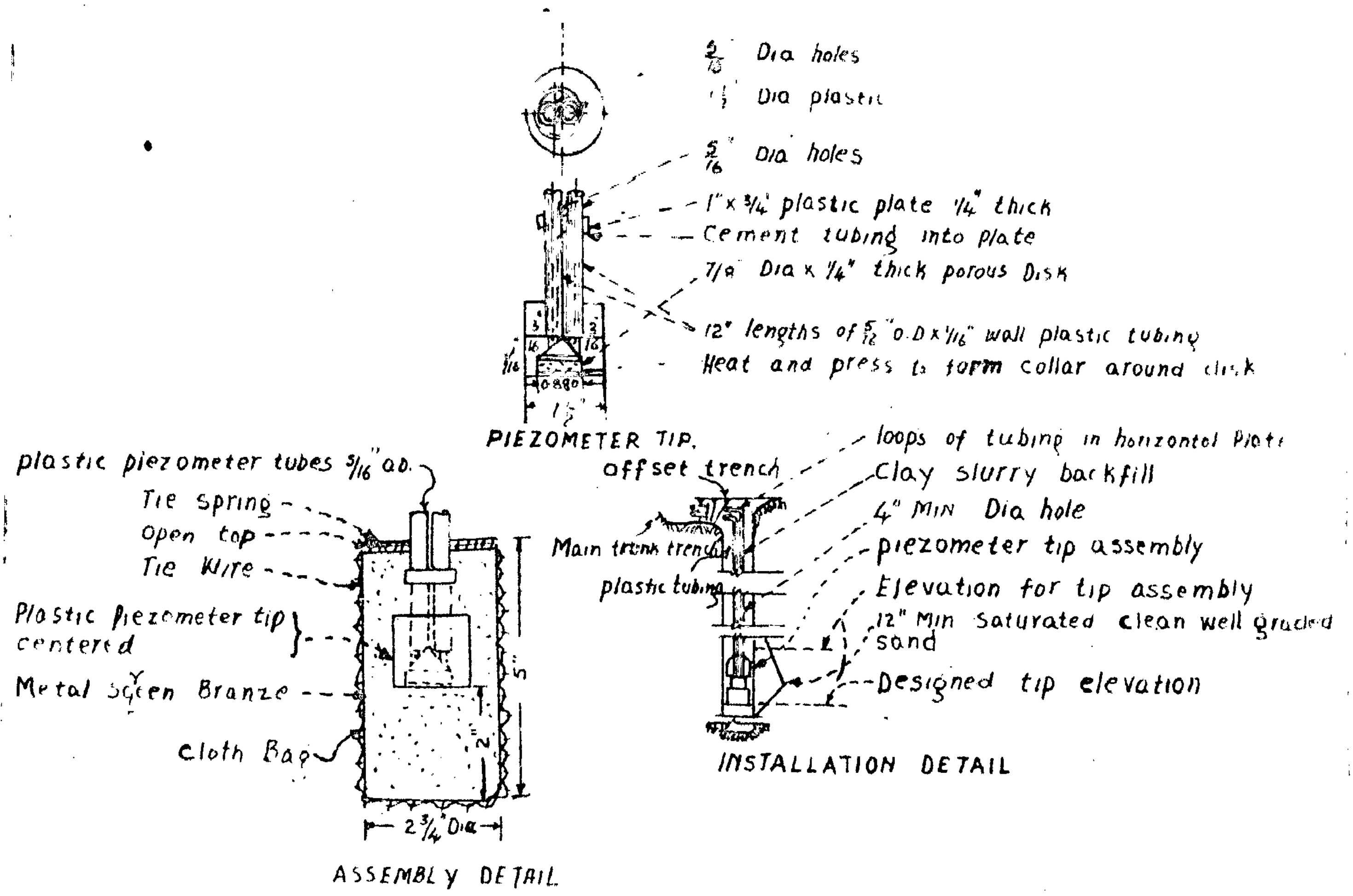


Figure 5.6 :- Foundation piezometer tip and assembly

the required elevation and the 5/16 inch OD by 1/16 inch wall thickness plastic tubings are taken out therefrom.

5.6. Electric Piezometers.

The Piezometers discussed so far are all of the hydraulic type. Therein the pore pressures at the tip are communicated to the pressure measuring device at the other end by water in the lead pipes. Electric Piezometers which are discussed in the following paragraphs are, in principle, much superior to the hydraulic Piezometers particularly for measuring pore pressures in fine grained soils. Because in electric Piezometers no appreciable flow is required in and out of the Piezometer tip in order to register a change in pressure. Also they eliminate all troubles associated with leaks, air locks, and volume change in the tubings of the hydraulic Piezometer. In fact, in the core of a large dam constructed of highly impervious clay it is quite doubtful if it would be practically possible to measure the pore pressures reliably with hydraulic Piezometers especially if the hydraulic lines are long. In addition it is much easier to instal an electric cable and make electric readings than it is to instal hydraulic lines and to measure hydraulic pressures. Also there is much more flexibility in the installation of an electric cable since it can be brought out at any point in the dam, whereas, with hydraulic Piezometers there are definite limitations, especially if the pressures are to be measured in the lower part of the dam or foundation cut-off trench. Different types of electric Piezometers are described below:

5.6.1. Goldbeck Cells. These are the oldest of the widely adopted electric Piezometers which employed air pressure to balance the water pressure and to break an electrical contact. These were used in many dams in the United States starting approximately in the mid 1930s. Usually they failed either by rupture of the diaphragm or by internal corrosion as a result of water condensation from the compressed air inside.

5.6.2. Strain gauge electric Piezometers. This type of electric Piezometer uses SR-4 strain gauges to measure the deflection of the diaphragm. These Piezometers can be made quite precise; however for a number of reasons (principally drift of the gauges and changes in the electric resistance of the cables), they have not proved reliable for long term measurements and have not been installed frequently in earth dams. Also when cables are cut accidentally in the field-as they frequently are - and must be spliced, it is difficult to know the influence of the new cable on the calibration curve for the Piezometer.

<u>5.6.3. Vibrating wire strain gauge principle electric Piezometers:</u> Since about 1955 Piezometers using the vibrating wire strain gauge principle have been installed in about a dozen major dams by European engineers and the experiences have been generally quite favourable. In these Piezometers, changes in pore water pressures acting on a diaphragm or piston cause changes in the tension and frequency of vibration of a fine steel wire, (figs. 5.7 and 5.8). In the instrument the wire is set into vibration with a magnet and the frequency is measured allowing a very precise determination of the pore water pressure. Most of these instruments are manufactured by (i) Telemac, Paris, France, (ii) Maihak, Hamburg, Germany and (iii) Carlson, California, U.S.A.

The primary advantage of the vibrating wire type Piezometer is that the readings are not dependant on the electrical resistance of the cable or of a strain gauge and, consequently, corrosion or temperature of the cable and the necessity for splicing it when cut during construction, does not influence the results. Because of this factor and general advantage of electric cells as discussed above, it is probable that this type of Piezometer will be more widely used in the

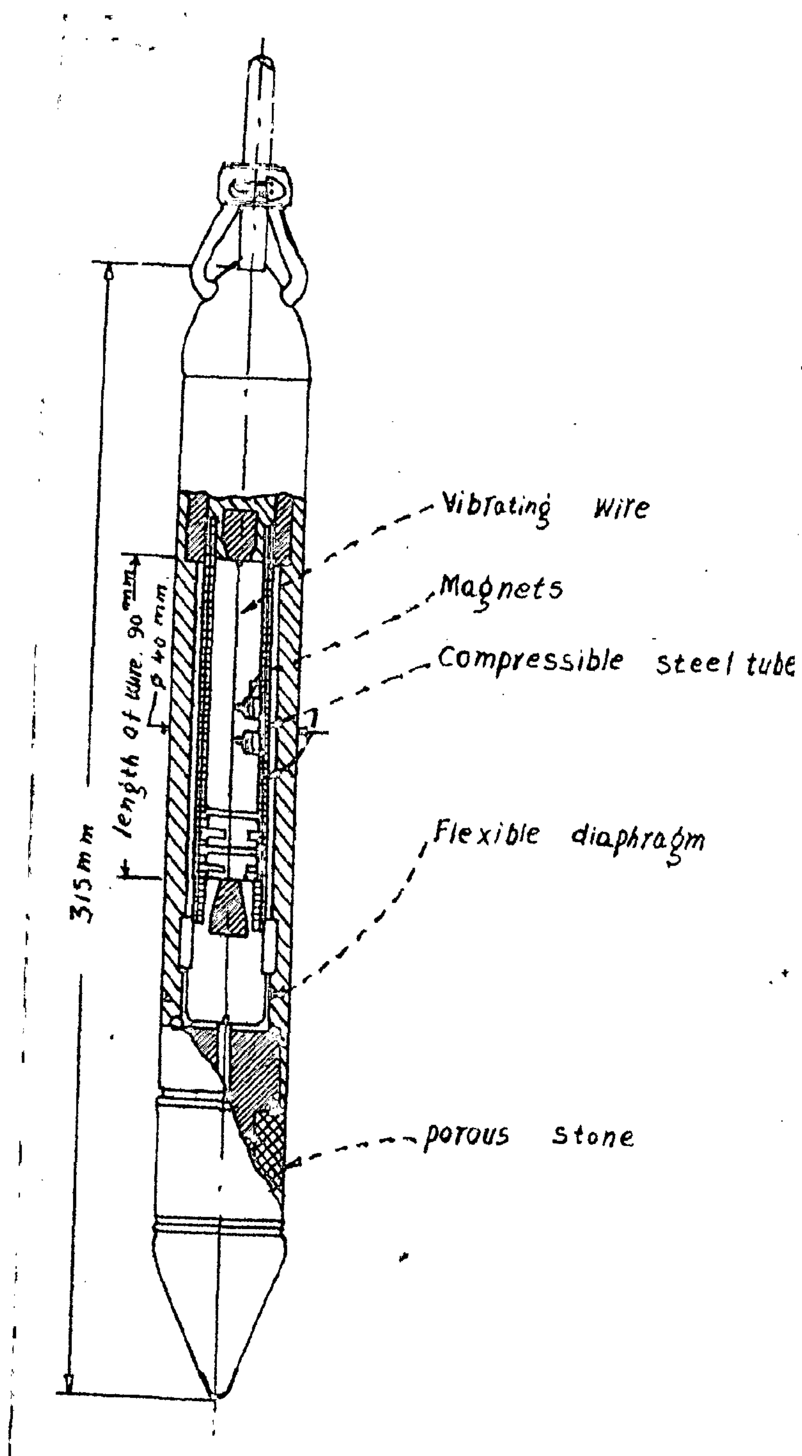


Figure 5.7:- Telemac Vibrating Wire piezometer (

future. In one interesting application, the cells of the telemac type were installed in the high Serre-Pencon Dam which was completed in the French Alps in 1960. At several locations in the core of this Dam, the vibrating wire cells were located directly adjacent to standard USBR hydraulic Piezometers. Both sets of Piezometers gave almost identical results during construction and during the first years of reservoir operation.

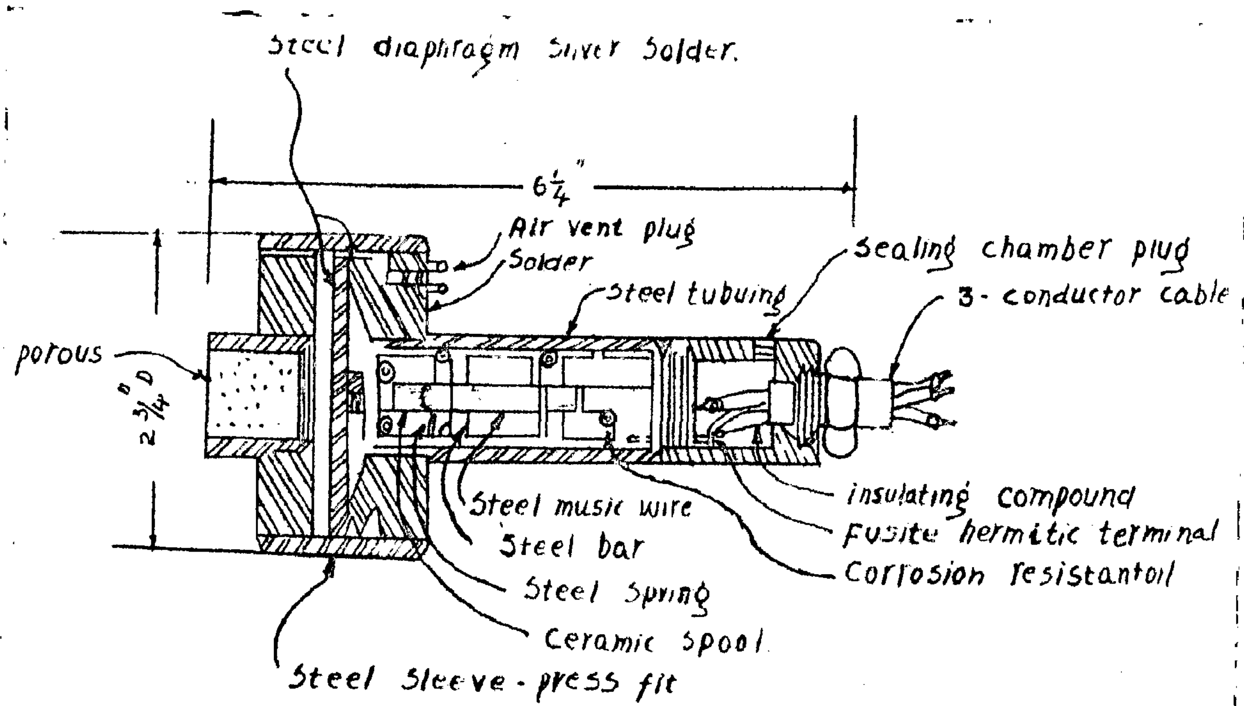


Figure 5.8 :- Carlson pore pressure cells

5.7. Advantages and disadvantages of vertical and horizontal Piezometer tubes.

The main advantage with vertical tube Piezometers is that less troubles develop with air bubbles. This has been explained in detail in para 5.5. In addition, Piezometers with vertical riser tubes have another advantage in that the tubes are usually shorter. The greater the distance between the Piezometer and the measuring point the more likely are poor results from all causes including tube leakages, bubbles and time lags. In some large dams, where the tubes are carried horizontally to a terminal well at the downstream toe, Piezometer lines have been as long as 1500 feet to 2000 feet.

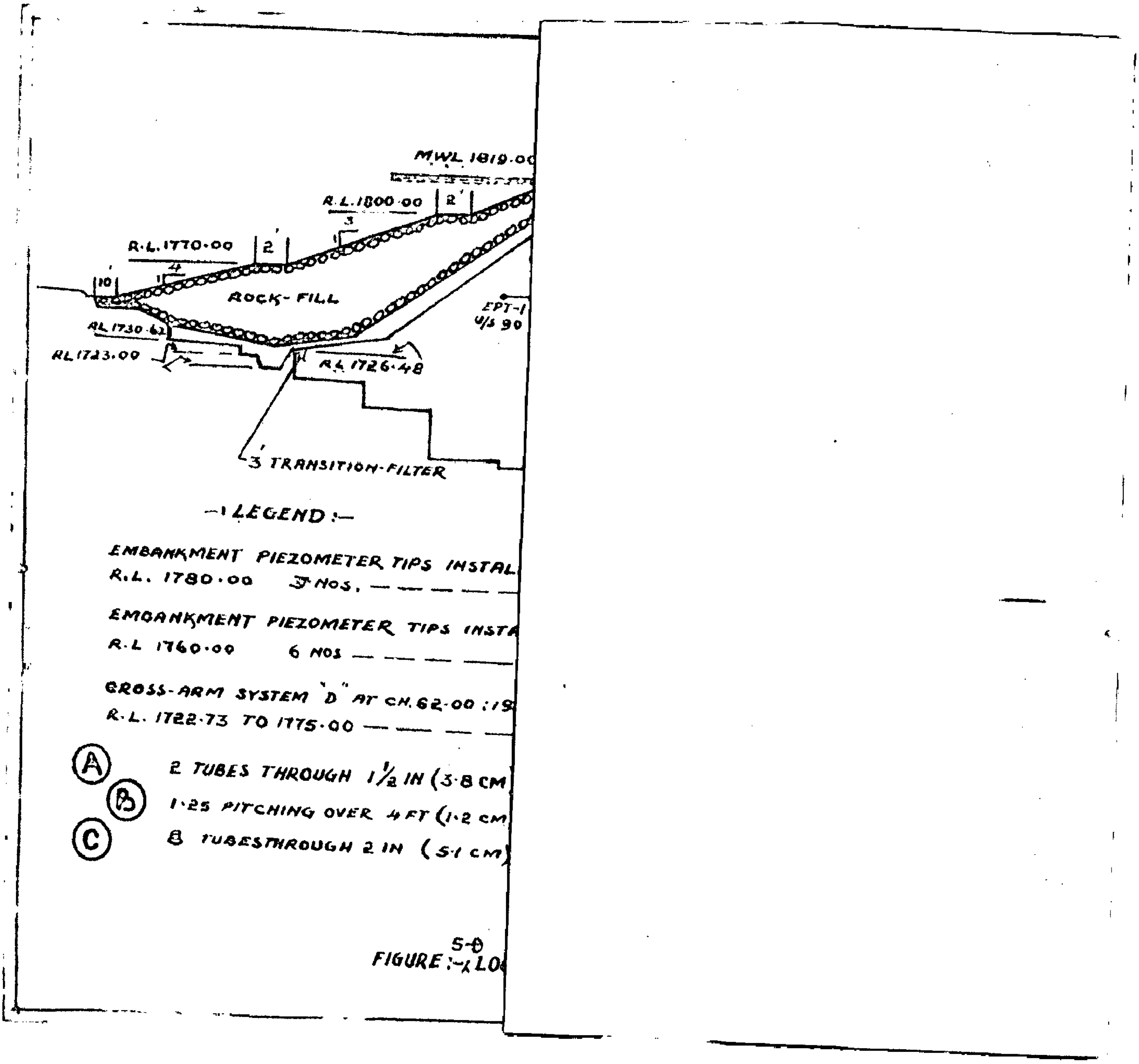
Horizontal Piezometer tubes have two main advantages over vertical tubes: (i) Regardless of the pore pressure a closed system is obtained in which no flow from the soil surrounding the Piezometer is necessary and hence theoretically there is no time lag. (ii) The lines are burried and got out of the way whereas vertical tubes installed during construction have to be continuously protected and extended above the construction surface.

5.8. Success of Piezometer Installations.

Good instruments, good installation, competent maintenance and reading are very much required to obtain reliable information from the Piezometers regarding pore pressures. Poor or suspect reading are so common as to be almost the rule. In cases where the engineer is seriously counting on the validity of the measurements to assure him of the safety of the dam he should instal at least two and preferably three different types of Piezometers. Leak-proof valves and connections are an essential requirement in the installations. The success of Piezometers depends on how carefully they are installed. For this reason many engineers exclude Piezometer installation from the main construction contract and instal them with their own personnel.

5.9. Piezometer installations at Sharavathi Project[57]

The Linganamakki Earth dam of the Sharavathi Project is the first earth dam wherein Piezometers have been installed in Mysore State. The maximum height of the earth dam from the deepest foundation is 170 feet. The construction of this dam took over 4 seasons to complete and involved placing of 1000 lakh Cft. of earth in addition to rockfill of 400 lakh Cft. The filling of the reservoir started in July 1964 and the reservoir filled partially during 1964-65 and 1965-66.



Originally it was programmed to instal 30 embankment Piezometer tips at Ch.62+00 and 30 more at Ch.68+00 at various elevations and a few foundation tips. But the programme suffe-rred a great set back due to non-arrival of the equipment in time and the scheme had to be pruned down to the installation of only 9 embankment Piezometer tips in all at Ch. 62+20 in two tiers, 6 at R.L. 1760.00 and 3 at R.L. 1780.00. Of these, 4 of the lower

tier and 1 of the upper tier are in the impervious core portion and 2 of the lower tier and 2 of the upper tier are installed down stream of the inclined filter. Leads from these Piezometers have been taken to an R.C.C. terminal well constructed between chainages 63+10 and 63+20 at the downstream toe; vide fig.5.9. The Piezometers and accessories used are of Indian manufacture. Since Indian mahufacturers for this kind of equipment do not

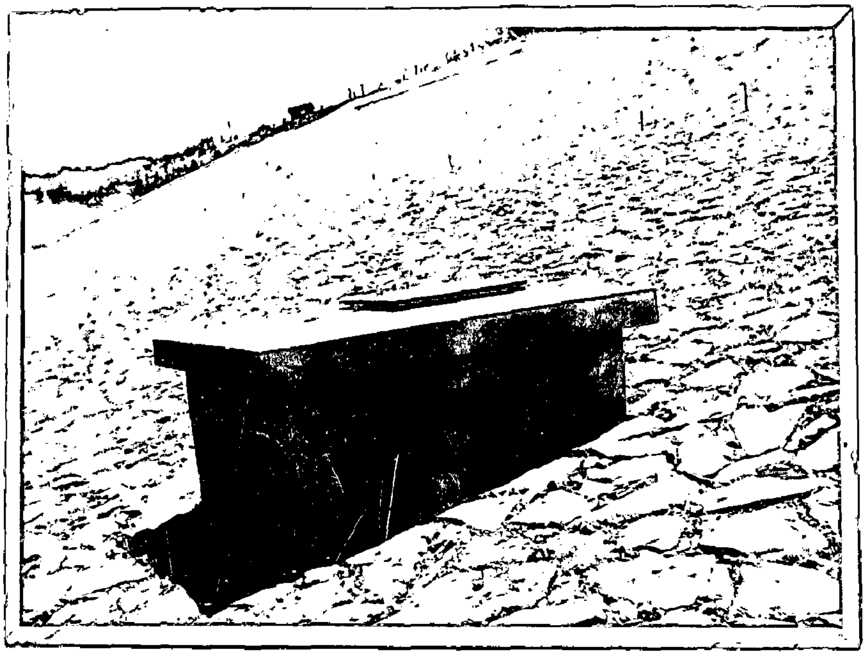


Fig. 5.10: R.C.C. Terminal Well, (Sharavathi Project)

yet have enough experience, it is possible that the equipment are not upto the standard of similar equipments manufactured by standard foreign firms. As such, the observations have to be treated with caution till their reliability is established. Some of the foreign manufactures who deal with this type of equipments are: (i) Messrs. Soil Tests Inc. U.S.A. (ii) Messrs. Carlson and Company, Berkeley-8, California, U.S.A., (iii) Messrs. Soil Mechanics Equipments Ltd., London, (iv) Messrs. Soil Instruments Limited, London and (v) Messrs. Geonor and Sons, Oslo. In India Piezometer tips are being manufactured by (i) Messrs. Kailash Bros. (Private) Limited, New Delhi, and (ii) Concrete and Soil Research Laboratory, Madras.

5.9.1. Difficulties experienced with indigenous equipments:

The pore pressure equipments and accessories were manufactured and supplied by Messrs. Kailash Brothers (Private) Ltd., New Delhi. Though the Piezometers withstood the testing pressures yet they were not of the standard type. The inlet and outlet ends cemented to the tip were not of sufficient length and not bent in standard shape nor were they of correct dimensions. These required modifications with the plastic ends. The installation of the tips in position in the off-set trench and leading the leads into the main trench also presented certain difficulties. The plastic tubes supplied by the firm were hard and strong to withstand test pressures but were not ~~fx~~ flexible. This resulted in curling of the tubes and breaking very frequently, necessitating the use of more number of field joints. Due to the curling nature, carrying the plastic tubes through G.I.pipes while crossing the inclined filter and rock-fill was also difficult. The Piezometer gauges supplied by the firm were of bottom connections and were not filled with anticorrosive liquid while it was specified for top connections filled with anti-corrossive liquid and sealed with metallic threaded caps. This resulted in mounting the gauges upside down on the panel boards. The reliability of these gauges for continued observations over a long period has to be established. The needle valves supplied were not of standard type and the field instrumentation engineers had to spend innumerable hours in checking and correcting the various connections notwithstanding the reliability of the gauges.

The Sharavathi Project engineers[34] have opined that "in general the indigenous equipments were not of a standard

Fig. 5.11: Looking down into the Terminal Well, (Sharavathi Project)

comparable with those of foreign make of ✱ reputation and their reliance for precision measurements cannot therefore be ensured.** Where refinement and precision are the criteria, procuring instruments from abroad from reliable organisations should be entertained and processed. In addition, to build up service records as well as corelations, the measurements may be dupli-

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**Note: The installation, maintenance and reading of Piezometers is an intricate work and difficulties therein may be experienced with both indigenous and foreign instruments. Comparison of performance can only be made by embedding different kinds of Piezometers in the same embankment and comparing results.

cated wherever possible by installing indigenous equipments side by side with standard foreign equipments. Certain firms who have handled research equipments should be sponsored to promote and develop refined equipments pertaining to instrumentation which should be periodically checked by organisations like the Council of Scientific and Industrial Research, The National Physical Laboratory, and such other National Laboratories. Such an arrangement will promote manufacture of better equipments as well as prevent marketing of spurious equipments thereby facilitating the study of structural behaviour of earth dams and eliminate getting erratic data from bad equipments".

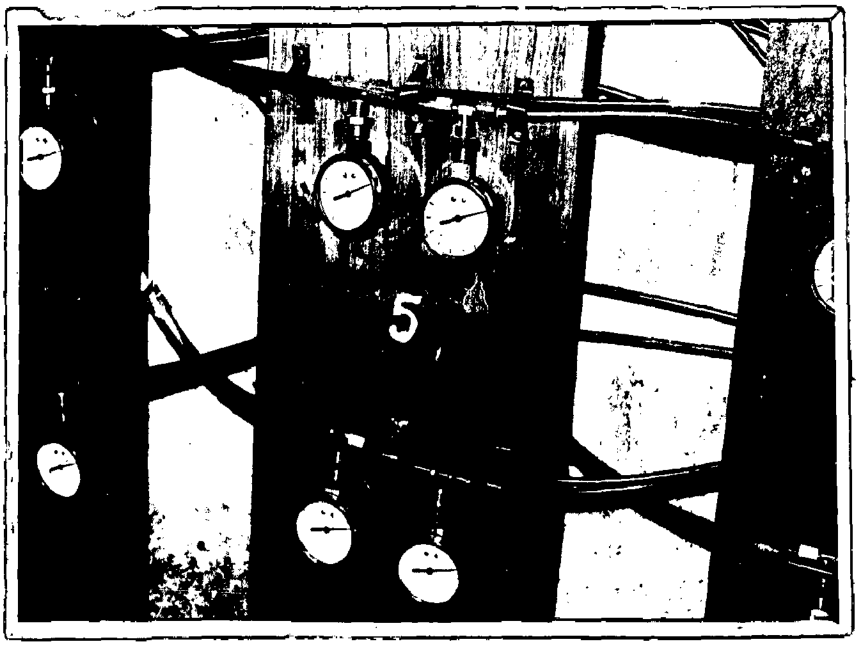

Fig. 5.12: View of the Gauges in the terminal well, (Sharavathi Project)

CHAPTER VI.

CONCLUSIONS

1. The science of soil mechanics among Civil Engineering Sciences is comparitively of a recent origin. There too the concept of pore pressure and effective stresses is of still more recent origin (It was Karl Terzaghi who first introduced this concept in 1923). Due to its recent origin there are still many unsolved problems and unanswered questions regarding even the basic concepts. The laws that interconnect cause and consequence in this field are not so simple and lucid as those in many other fields of Civil Engineering. Many of the problems regarding pore pressures, effective stresses, and shear parameters cannot be solved adequately by theory alone, because of the complexity of the physical properties of soil as a structural material. Inspite of these inherent difficulties an intimate knowledge of the relations between cause and effect increases the capacity of the engineer for a rational design of the earth dam. Whatever theory that is available for this purpose should be supplemented by the knowledge gained from experiences in the field. For evolving

economical and efficient designs the earth dam engineer must not only evaluate lessons from his own experience but should familiarise himself with the practice and experience of other engineers working on similar projects and problems elsewhere, and this is not an easy task. It is unfortunate, but quite true, that much of the valuable knowledge on many interesting projects is completely suppressed by the engineers in charge for fear of criticism by others, and much more, for fear of the information being made use of against themselves in the event of any damage to or failure of the structure at a later date. And where the damage or failure has already taken place the findings regarding the real causes thereof are concealed from public knowledge much more carefully.

2. Because of the unavoidable uncertainties involved in the fundamental assumptions of the theories and in the numerical values of the soil constants, simplicity is of much greater importance than a high degree of accuracy and a highly mathematical approach in the design procedures. If a theory is simple, one can readily judge the practical consequences of various conceivable deviations from the assumptions and can act accordingly. If a theory is complicated, the usually inadequately staffed design offices are unable to make use of it until the results are condensed into graphs or tables that permit rapid evaluation of the final equations on the basis of several different assumptions.

With these considerations the simple procedure of accounting for pore pressures by taking submerged weights for N-forces and saturated weights for T-forces, as discussed in Chapterms II and III is used for the earth dam design by many field engineers instead of evaluating the pore pressures by flow nets etc. Likewise, the procedure of assuming the construction pore pressures as being equal to 1½ times the depth of the impervious fill, etc., as discussed in para 4.6.1 of Chapter IV is also in vogue with many field engineers instead of calculating the same by Hilf's method, Bishop's method, etc.

3. The soil mechanics of unsaturated cohesive soils is still a subject of much conjecture, misconception and controvercy. It has been seriously questioned as to whether pore pressure is a reality in so far as strength characteristics of unsaturated soils is concerned. The reliability of pore pressure measurement obtained during laboratory shear test on unsaturated clays has been doubted. The criterion of failure in laboratory shear tests of compacted cohesive soils in which pore pressures are measured has been the subject of diverse opinions. There is no comprehensive analysis available in the literature of the interaction of water and air in the pore fluid of a soil mass. Such a study is needed both to justify the assumptions used in arriving at a quantitative value for pore water pressure and possibly to explain certain persistent variations from theoretical values which occur in laboratory experiments and in field measurements.

4. In the computations of pore water pressures due to gravity seepage, the influence of capillary forces is neglected since we have no way to evaluate them. Capillary forces tend to prevent the water from flowing out of the pores of fine grained embankments following reservoir draw down by forming menisci at the seepage boundary, and also within the dam, with resulting

tension in pore water. Therefore, ignoring capillary action results in a conservative pore pressure estimate. On the other hand, it is quite conceivable, and even probable, that for many soils capillarity will completely dominate gravity flow pattern, making it impossible for any appreciable positive drawdown pore pressure to exist within the embankment above the reservoir elevation. In future years additional laboratory researches and measurements of draw down pore pressures in dams may demonstrate that our present procedures are excessively conservative except for certain soil types.

5. If the construction water content is on the dry side of optimum the embankment will be too brittle and will settle on saturation after the reservoir fills and settlement cracks develop in the embankment which will provide easy passages for through seepage and lead to failure of the dam. If the construction water content is on the wet side of Proctor's optimum then higher construction pore pressures develop which will lessen the "during construction" stability of the dam. Thus both the alternatives have got their disadvantages. However there are various methods available for controlling the higher construction pore pressures that will develop due to more water content but there are no possibilities available of controlling the post construction settlement on saturation if the water content is on the dry side of optimum during construction. In view of this, for earthen dams, it is desirable to keep the construction water content on the wet side than on the dry side of optimum. The methods of controlling construction pore pressures are described in section 4.8. of Chapter IV.

6. The no-load shearing strength of compacted cohesive soils (i.e., the C component of S in the equation $S = C + N \tan \phi$ when N is zero) cannot be called cohesion in the true sense. This strength is really provided by the capillary water and is of the same order of magnitude as the frictional resistance provided by negative pore water pressure. Progress towards a better understanding of the important property of shearing strength of soils can be achieved by directing research effort to investigating the magnitude of capillary pressures and their variations with void ratio. The phenomenon of settlement on saturation of soils placed very dry of optimum water content has a rational explanation in terms of negative pore pressures. Further study of this phenomenon may lead to an improved and more economical method of setting a lower limit of moisture for compacted cohesive soils in high embankments.

7. As discussed in Chapter II the consolidated undrained tests with pore pressure measurement are carried out for ascertaining the effective shear parameters C' and ϕ' for use in the effective stress method of stability analysis. These tests are still intricate and unless they are performed with great care and the experimental errors evaluated with accuracy the results can be very misleading. One of the principal errors in the measurements, and one difficult to evaluate, results from the fact that the main zone of shear failure is in the centre of the sample. The rigid loading caps and the rubber membrane prevent the specimen from straining as much at the ends as at the centre so that in ~~rapidity~~ rapidly performed tests the pore pressure measured at the ends may be quite different from that in the

zone of failure. Early tests used an internal probe or needle for measuring the pore pressure near the centre of the sample. However, difficulties developed with the testing equipment, and the solid probe had an indeterminable influence on the pore pressure inside, leading most investigators to conclude that it is preferable to take the measurements at the end of the sample. The only practical means so far devised to minimise this error is to run the test slowly so that the pore pressures have an opportunity to equalise throughout the sample. No specific length of time can be prescribed for this although researches indicate that the time required is much greater than has been generally suspected. Other difficulties in the testing procedure which are not yet completely solved arise from the action of air in the sample and from the thixotrophic influence of clayey soils.

8. At the present time some engineers use the effective stress method for stability analysis. Others continue to use the total stress method of analysis because they believe it is easier and more reliable. The methods are equally reliable when applied with the same degree of understanding and judgement. In the manual of practice of Army Engineers of the U.S.A., which is used by all their district offices, and hence, is being applied to studies for a large number of dams, the total stress method is recommended almost exclusively because of the difficulty of estimating pore pressures. On the other hand the engineers of the USBR carry out nearly all analysis in terms of effective stresses. The principal advantage of the total stress method of analysis is its greater simplicity. The advantage of the effective stress method is the fact that the analysis is carried out with a some what more fundamental definition of the shear

strength and it is possible to compare the pore pressures assumed in the design with those which develop in the dam and foundation as measured by Piezometers. For major projects Sherard[11] recommends the effective stress method as the primary analytical procedure and the total stress method as a check for comparison purposes. The two methods often give radically different computed safety factors since the pore pressures assumed for the effective stress analysis are those which are not affected by large embankment shear strains, while, pore pressures which enter the total stress analysis are those which occur at failure.

9. When employing the effective stress analysis we commonly use the pore pressures which would be expected to exist in a stable embankment; i.e., we make an estimate based on experience with other dams or on the basis of laboratory tests in which the specimens are not failed. On the other hand in the total stress method of analysis the pore pressures are those which have developed in the soil at failure, since the shear strengths are used in terms of total stresses at failure. As different pore water pressures are introduced into the analysis by the two methods different safety factors result. The differences in safety factors obtained are dependent to a large degree on the soil type. The difference is greatest for dense, well graded mixtures of gravel, sand and clay which dilate when sheared with a resulting reduction in the pore pressure. The difference is least for very fine grained clayey soils in which shear strains may cause higher pore pressures. The engineer must consider the ~~fxxx~~ fundamental difference between the two methods of approach when evaluating the results of the computations.

10. The results of stability analysis depend on many factors which are difficult or impossible to evaluate reliably. Small changes in the assumed soil strength and pore water pressures give appreciable differences in the results. The safety factor computed depends to a large degree on whether the effective stress method or the total stress method is used, and on whether the calculations are carried out with the standard method of slices or with other methods in which the influence of side forces are considered. We must be more conservative with embankments consisting primarily of fine grained soils because (i) almost all shear slides have occurred in dams constructed of or founded on very fine grained and highly plastic soils and (ii) the difficulty in determining the strength and the pore pressures of clay embankments is greater and the possible error larger.

11. The success of Piezometer installations in indicating the pore pressures accurately depends on how carefully they are installed. For this reason normally this work of installing Piezometers is excluded from the main construction contract and is carried out departmentally. Leak proof valves and connections are an essential requirement and to avoid delay in obtaining a complete set of observations a pressure gauge is to be kept permanently in circuit with each Piezometer point. Reliable readings from Piezometers require a perfect combination of good instruments, good installation and competent maintenance and reading. Poor or suspect reading are so common as to be almost the rule. Hence, in cases where the engineer is seriously counting on the validity of the measurements to assure him the safety of the dam he should instal at least two different types of Piezometers in order to have some assurance of success.

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