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AN INVESTIGATION INTO THE STABILITY OF
SLIMES DAMS

WITH PARTICULAR REFERENCE TO THE NATURE OF THE MATERIAL OF
THEIR CONSTRUCTION AND THE NATURE OF THEIR FOUNDATIONS.

BY

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A thesis submitted for the degree of Master of Science
in Civil Engineering to the University of Cape Town.

DECLARATION:

In submitting this thesis, I hereby declare that:

- (a) this work, with the exception of Chapter II, was done by me personally or under my direct supervision
- and (b) this thesis has not been submitted to any other university as a thesis for a Master's degree.

SIGNED: signature removed

DATE: 22nd August, 1960.

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LIST OF SYMBOLS

A	=	actuating force in stability analyses.
A'	=	effective cross-sectional area of water in the voids of a partially saturated soil.
B	=	limiting ratio of the permeabilities of slimes and foundation soil which will affect the position of the phreatic surface.
C'	=	resultant of effective cohesion for one slice in stability analysis.
c	=	cohesion per unit of area.
c_{cu}	=	cohesion obtained from consolidated undrained shear test.
c_d	=	cohesion obtained from drained shear test.
c_o	=	apparent cohesion of sample rebounded to zero pressure.
c_u	=	cohesion obtained from undrained shear test.
c'	=	effective cohesion.
c_p	=	apparent cohesion due to preconsolidation pressure p .
D_f	=	depth of foundation soil.
D_{90}	=	grain size of sieve passing 90% of material.
D_{85}	=	grain size of sieve passing 85% of material.
D_{15}	=	grain size of sieve passing 15% of material.
D_{10}	=	grain size of sieve passing 10% of material.
e	=	void ratio.
F_s	=	factor of safety.
F_u	=	pore water pressure factor.

- H, H_d = height of dam.
 h = depth to point being considered.
 h_c = maximum height to which all capillaries in the soil are full.
 k = permeability.
 k_h = horizontal permeability.
 k_v = vertical permeability.
 k_1 = permeability of slimes.
 k_2 = permeability of foundation soil.
 P_c = Preconsolidation pressure: maximum intergranular pressure that has existed in the soil during a wetting and drying cycle.
 p = total pressure at any point in the soil.
 pF = logarithm of the pressure deficiency in the water expressed in cms. of water.
 p' = intergranular pressure at any point in the soil.
 p'' = pressure deficiency in the pore water.
 R = resisting force in stability analysis.
 S = shear strength.
 S_f = shear strength of foundation soil.
 S_m = shear strength of remoulded material.
 S_s = shear strength of slimes.
 S_u = shear strength of undisturbed soil.
 u = pore water pressure.
 $\bar{u}$ = negative pore water pressure.
 $\bar{u}_{max}$ = maximum value of negative pore water pressure.

(v)

- α = angle of inclination of the failure plane in a slice.
- γ = density of soil.
- γ_d = dry density of soil.
- γ_s = submerged or buoyant density of soil.
- γ_t = total density of soil.
- γ_w = density of water.
- θ = angle of slope.
- σ = total stress normal to a shearing plane.
- σ' = intergranular stress normal to a shearing plane.
- σ'_{cr} = intergranular stress in rebounded soil.
- σ_1, σ_3 = principal total stresses in triaxial test.
- σ'_1, σ'_3 = principal intergranular stresses in triaxial test.
- τ_f = shearing stress at failure.
- ϕ = angle of shearing resistance or angle of internal friction.
- ϕ_{cr} = angle of shearing resistance with relation to rebound pressure.
- ϕ_{cu} = angle of shearing resistance from consolidated undrained test.
- ϕ_d = angle of shearing resistance from drained test.
- ϕ_m = modified angle of shearing resistance for seepage conditions.
- ϕ_u = angle of shearing resistance from undrained test.
- ϕ' = effective angle of shearing resistance.

RESEARCH INTO THE CONSTRUCTION OF SLIMES DAMS

CHAPTER I. I N T R O D U C T I O N

HISTORICAL BACKGROUND

In November, 1950, the Inspector of Mines, Bloemfontein, approached the Director of the National Building Research Institute of the Council for Scientific and Industrial Research, with regard to a ruling on the maximum permissible height of slimes dams in the new Orange Free State Goldfields. Similar enquiries were received from the National Resources Development Council and from certain mining groups. Several meetings were held with various interested bodies, to discuss mainly the influence of slimes dam building operations on water pollution and land usage. At a meeting of the Consulting Metallurgists of the Chamber of Mines on 28th December, 1950, at which the National Building Research Institute was represented, certain proposals relating to slimes dams in the Orange Free State, put forward by the National Resources Development Council, were considered. In April, 1951, the Chamber of Mines appointed a sub-committee to maintain liaison with the National Building Research Institute in connection with investigations into the construction of slimes dams. A certain amount of research was done and another meeting was held in January, 1953, between the representatives of the Chamber and the National Building Research Institute. At this meeting it was made clear that, although many slimes dams had been built successfully and a large amount of practical experience had been gained, no-one possessed any exact knowledge of the mechanism giving strength to a slimes dam.

As a result of these and several other meetings, the Chamber of Mines entered into a contract with the Council for Scientific and Industrial Research in August, 1953, for an investigation into the stability of slimes dams.

TERMS OF REFERENCE

The terms of reference of this contract were:

1. Scope. On behalf of the Transvaal and Orange Free State Chamber of Mines the Council for Scientific and Industrial Research will investigate the character of the stability of slimes dams with particular reference to the nature of the material of their construction and the nature of their foundation.
2. Steering Committee. The Council will appoint a Steering Committee consisting of three representatives of the Chamber and two representatives of the Council for Scientific and Industrial Research.
The Steering Committee will ensure that the Chamber is consulted on the nature of the investigation and on the manner in which it reports on it.
3. Publication of Results. Neither the Council nor the Chamber will publish the results of the research without the written consent of the other party. Subject to this limitation the Council reserves the right to publish any results of the investigation which it may consider to have scientific value.
4. Litigation between the Chamber and third parties. The Chamber undertakes not to subpoena or require the attendance of any officer or other representative of the Council in any legal or other proceedings between the Chamber and third parties for the purpose of giving evidence in relation to the investigation.

EXISTING METHODS OF BUILDING SLIMES DAMS

As a background to the whole problem of slimes dam building, visits were paid to numerous mines on the reef and in the Orange Free State to study the techniques in use. Publications on slimes dam construction are rather scarce but work by Thomas and Osterloh (1), Westwood (2) and Stitt (3) proved most valuable. Further information was obtained by studying the reports on slimes dams in the Mining Commissioner's office in Johannesburg.

The practice of slimes dam construction has grown with the mining development of the Witwatersrand. The techniques used are based mainly on personal experience and, although the general principles are the same, considerable variations in details are encountered and the explanations for many phenomena are also widely different.

In the early stages of development the metallurgical practice consisted of a two-pulp cyanide process, using sand and slime, which were the products after hydraulic classification. The sand residue was placed on sand dumps and the slimes residue in slimes dams. This process has changed, with more and more slimes being produced and less sand, until today most mines operate on the all-sliming process and produce from 80,000 to 200,000 tons of slime per month. This residue is disposed of by building slimes dams.

Starting a new slimes dam

In the layout of a new mine the main factors demanding attention are the shafts, headgear, offices and reduction works. The area available after these features have been positioned is then allocated for the construction of the slimes dam. The dam layout is then worked out as regards the positions of the delivery points, penstocks and outer walls. As the slimes flows round the dam by gravity, it is natural that the delivery point, which is therefore the highest point on the dam wall, is very often placed at the highest point of the surface topography of the area, and the penstocks which drain off the excess water are placed nearest to the lowest point of the surface topography.

Fig. 1 is a simplified plan of a slimes dam. At the beginning the following trenches are dug:

A. The stormwater and seepage trench. This trench is usually several feet wide and several feet deep. As its name indicates, this trench collects the stormwater run-off from the dam and the seepage water coming from the dam and carries these waters to a suitable disposal point.

B. The toe trench of the outer wall, which is usually 9 feet wide by 3 feet deep, and forms the outer perimeter of the dam itself. Between trenches A and B a strip of about 30 feet in width is left open as a catchment to collect material washed off the dam.

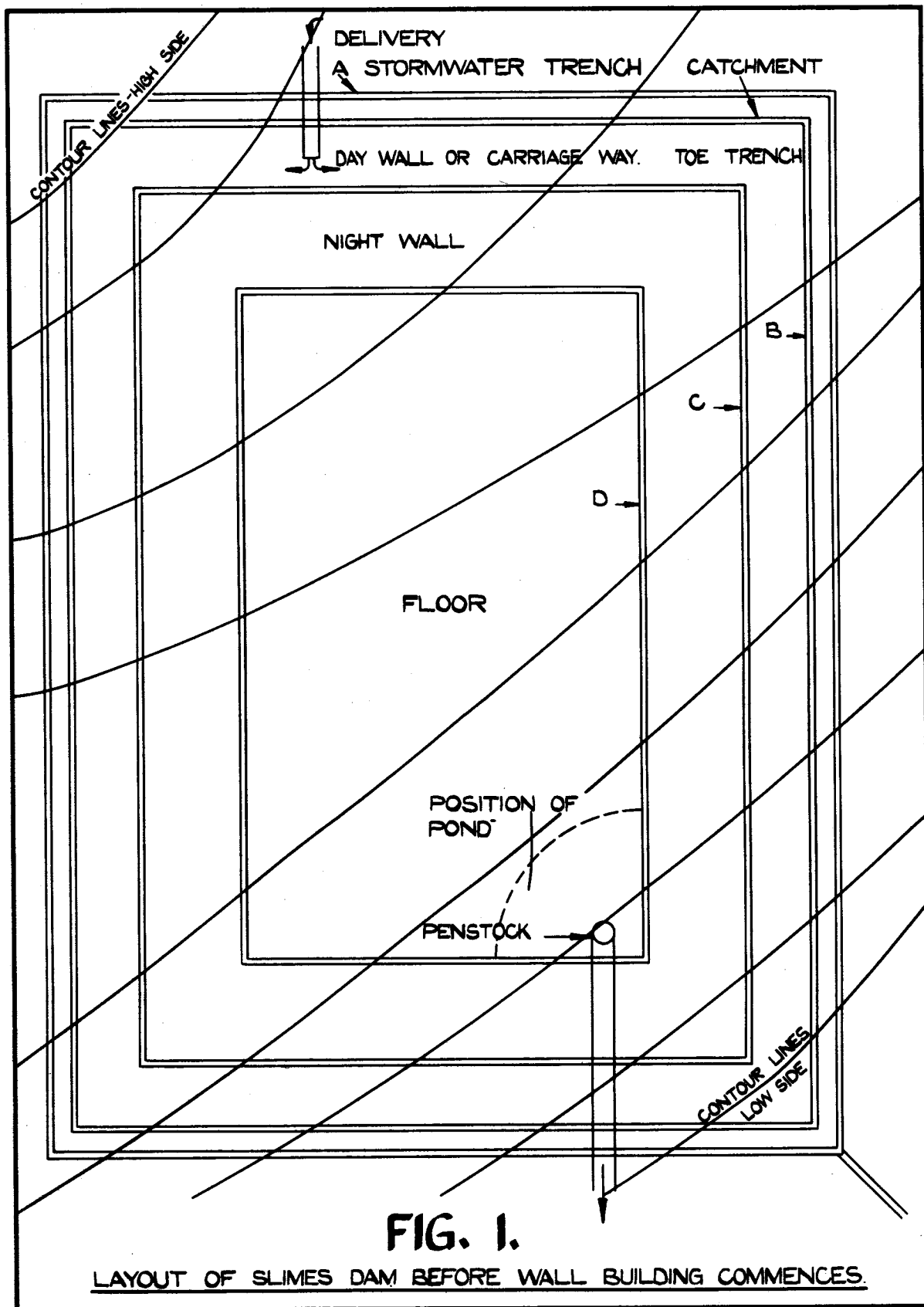


FIG. 1.

LAYOUT OF SLIMES DAM BEFORE WALL BUILDING COMMENCES.

C. The trench forming the foundation for the boundary wall between the day-wall and the night-wall. This trench is also 9 feet wide by 3 feet deep and the day-wall between trenches B and C about 100 feet wide.

D. is the boundary between the night-wall and the floor of the dam. The night-wall between trenches C and D is about 150 feet wide.

Naturally the main penstock is on the floor of the dam, but in the initial stages a subsidiary penstock may be erected in the wall area to cope with excess water until the main penstock comes into operation. As soon as the trenches are completed and the delivery line and penstocks have been installed, the dam is ready for wall-building to commence. A section through the lowest point on the wall is shown in Fig. 2(a).

As soon as slimes is put on to the dam, it is run along trenches B and C until they are full at the lowest point. When these trenches are full, the slimes is allowed to dry a little and then some of the slimes is dug out and packed to form a low wall along each trench as shown in Fig. 2(b). The next step is to run slimes into the day-wall allowing it to bank up against the wall at B. As the slimes dries out, some of it is used to raise the wall at B and then more slimes is run in. As the outer wall rises it is extended along the slope as the trench fills up. The slimes is put on the wall as a slurry and once the solids have settled there is a considerable amount of excess water to be disposed of: as the penstocks cannot yet be used, a system known as "open-end working" is used. A weir, usually a piece of old piping, is placed in the outer wall at its upper limit where its top meets ground level and the excess water is drained off by this means. As the wall rises the "open end" moves up the slope with the end of the wall.

After a period the wall at B will have risen so that the slime level in the day-wall will be at the same level as the wall at C, as shown in Fig. 2(c). From then on the day-wall is raised further by depositing slimes between the two handpacked walls at B and C and by raising these walls. The next stage is reached when the day-wall is about 4 feet higher than the ground level at C, as shown in Fig. 2(d). At this stage slime deposition is started on the night-wall by running the slime against the inside

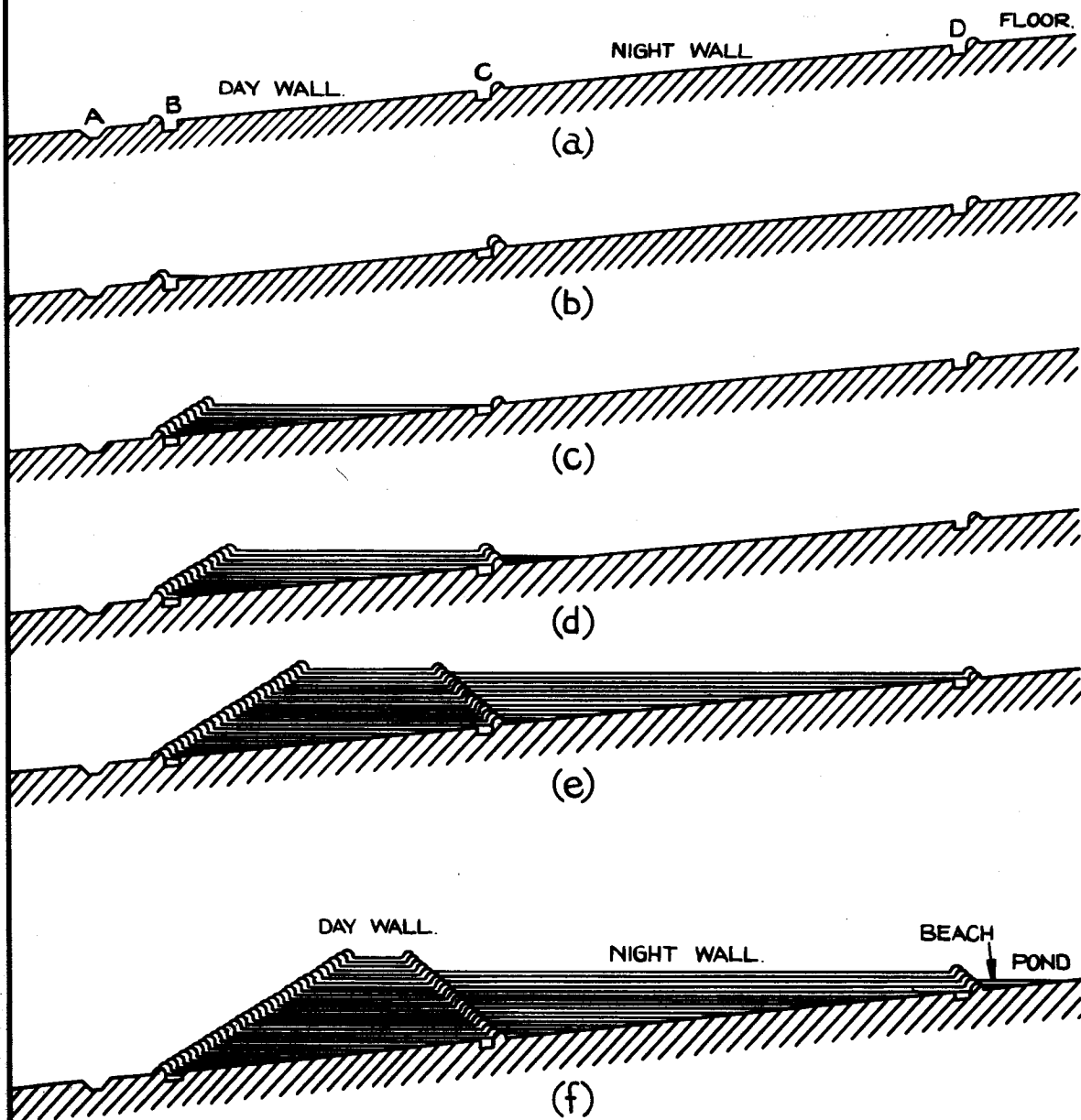


FIG. 2.

INITIAL STAGES IN THE BUILDING OF A SLIMES DAM.

of the day-wall. At the same time the day-wall is raised to keep it 4 or 5 feet above the night-wall.

The disposal of excess water from the night-wall creates a slight problem. Sometimes it is removed by means of an auxiliary penstock installed on the night-wall; otherwise, one end of the day-wall has to be butted to form an "open end" round which the excess water from the night-wall can be led.

Fig. 2(e) shows the stage of construction when the day wall has reached a fair height and the full width of the night-wall has been filled with slimes. Thereafter, the wall at D has to be handpacked as the wall rises. Once the level of the night-wall rises 4 or 5 feet above the floor of the dam at B, slimes can be placed on the floor of the dam by means of a weir from the night-wall. This slimes will form a beach, and the excess water will be forced back towards the main penstock, as shown in Fig. 2(f).

By this method the dam continues to be raised until the main penstock comes into operation. Auxiliary penstocks at lower elevations may again be used to cope with the excess water until the main penstock comes into operation.

As soon as the outer wall has risen high enough to retain an area large enough to cope with emergency conditions, the dam may be considered to have reached the normal operational stage. The outer wall may or may not enclose the total area, depending on the slope of the site.

OPERATION OF AN ESTABLISHED DAM

Wall-building

On an established dam, the wall-building is continued as in the initial stages, keeping the day-wall 4 or 5 feet above the night-wall and the night wall 5 or 6 feet ahead of the floor of the dam. Since the day wall is the outermost portion of the wall, it requires the greatest care in building, and is "loaded" with slimes during the day, when there is full supervision, while the night-wall can be "loaded" during the night. As a means of ensuring adequate freeboard, most of the slimes is used for wall-

building and only the excess material is placed on the floor of the dam. However, since the amount of excess material is usually large, constant care must be taken to ensure that the necessary freeboard is maintained. If there is not enough freeboard, the dam will be overtopped, with resultant erosion and breakaways.

As the dam rises the day-wall becomes narrower. When it reaches a width of about 40 feet, the inside edge of the day-wall is moved back onto the night-wall and the inside of the night-wall is moved back into the floor of the dam so that the respective widths of 100 feet and 150 feet are again established. This stepping back is shown in Fig. 3.

Penstock operation

The penstock, as explained earlier, is built to remove excess water from the slimes dam. The penstock intake, (situated at the lowest point of the dam), is a vertical pipe which can be either rectangular or circular in section and must be so designed that the intake level can be varied. In the rectangular type, 3 sides are fixed and the fourth side consists of horizontal slats which are placed in vertical grooves. By adding or removing slats the level of the intake is raised or lowered. In the circular type, short sections of pipe are added to raise the intake level.

The penstock is raised or lowered to maintain a pond area which will ensure that the water entering the penstock is clean. When a large volume of water is entering the dam, the penstock is raised to slow down the velocity of the water and thus keep it clean. When heavy downpours of rain occur, it is very often impossible to reset the penstock intake level; in these cases, measures are sometimes taken for automatic diversion of penstock water to the stormwater outfalls while the high flow lasts.

Size of dams

The areas of slimes dams vary from a few hundred square yards for small emergency dams to several square miles for the main dams on large mines. The area of a dam is governed by the following factors: expected total slime production of the mine, expected final height of the

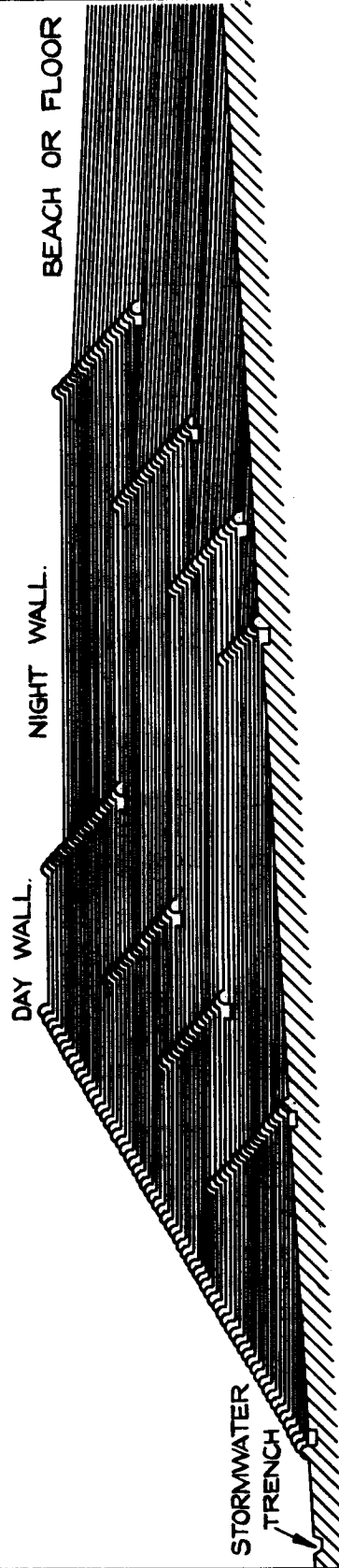


FIG. 3.

SECTION THROUGH WALL SHOWING SUCCESSIVE STEEPINGS BACK OF INNER
BOUNDARIES OF DAY AND NIGHT WALLS.

dam, and allowable rate of building. The last two factors are, of course, dependent on the stability of the outer walls of the dam.

PROBLEMS OF SLIME DAM CONSTRUCTION AND THEIR CAUSES

In many slimes dams, difficulty has been experienced in maintaining the stability of the dams. It was, therefore, necessary to determine the nature of the problem, or problems. The first step was to examine the failures that have already occurred. As a result of this survey, a report (4), summarising the various types of failures on slimes dams, was published in 1954.

This report showed that there were three causes of failure of slimes dams viz. (a) Surface erosion, (b) Seepage erosion and (c) Lack of overall stability resulting in breakaways.

- (a) Surface erosion. As the name implies, this problem is caused by the removal of material from the surface of slimes dams and particularly the outer walls, by wind and water. The material so removed finds its way into water-courses, causing pollution, and leaves gulleys on the dam faces. In the more serious cases these gulleys extend well into the outer walls and weaken the whole structure of the dam.

On many dams a hard outer crust is formed and no problems arise as a result of surface erosion. Where there is no surface crust or binding material in the outer layers of the dam surface, erosion occurs.

- (b) Seepage erosion. Seepage erosion is a common problem, evidence of which can be seen at the toe of the dam. Water deposited on the dam during slimes deposition, or by rainfall, seeps through the slimes material to the base of the dam. If the foundation soil under the dam is permeable and free-draining, the seepage water soaks straight into the foundation soil and is carried away. On the other hand, if the foundation soil is relatively impermeable or is not well-drained, the seepage water cannot escape through the base of the dam and flow is towards the toe of the dam, where the water emerges. This seepage flow keeps the toe of the dam wet and will erode channels where it emerges

from the dam, by removing the slimes particles in its path. In severe cases, the seepage erosion progresses so far as to seriously undercut the outer walls of the dam and reduce their overall stability.

In some cases slimes buttresses have been built over areas subject to seepage erosion, in an endeavour to reclaim such areas. In a short time, however, these buttresses have suffered the same fate as the original wall, since the seepage water has now been forced to find a way out through the buttresses.

- (c) Overall stability. The overall stability of the dam depends on the strength of the material of which the wall is made. This strength determines the safe height and slope at which a slimes dam can be built, since, if a dam is built at too steep an angle or is too high for the strength of the material, a portion of the dam will break away and slide out. Numerous slides of this type have occurred on Reef slimes dams.

As the slimes material is very loose there is always the possibility that when a breakaway occurs, the slimes will liquefy and flow out through the break. The most notable case of this nature occurred at Simmer and Jack in 1937; there was a similar occurrence at Grootvlei Mines in 1956.

Such failures are very costly both in maintenance on the dam and also because production may be held up on the whole mine.

The aim of the research project was, therefore, to find effective means of combatting these failures by making the best use of existing knowledge and techniques.

CHAPTER II.

S U R F A C E E R O S I O N

As mentioned earlier a hard outer crust is formed on many slimes dams while on others there is no crust formation. This hard crust provides very good protection against surface erosion and also facilitates the run-off of rainwater and prevents the water penetrating into the walls and weakening the slimes material.

A full investigation into the formation of crusts on slimes dams was carried out and the detailed results were published in an earlier report (5) which is summarised in this chapter.

EXPERIMENTAL WORK

Several gold mines were visited with the objects of obtaining information concerning the processing of the ore and of examining the various slimes dams. The operations carried out at the mines visited represented a cross-section of the types of ore-processing that are being used in the Union, and also of the different types of slimes dams that have been and are being built. The following mines and their slimes dams have been visited:

Randfontein Estates	-	Randfontein
Daggafontein Mine	-	Springs
Blyvooruitzicht Mine	-	Carletonville
Welkom Mine	-	Welkom
Freddies North Mine	-	Odendaalsrus
Marievale Mine	-	Springs
Government Gold	-	
Mining Areas		Brakpan.

At each of these mines, snatch samples of various materials of the slimes dams were taken for analysis. The materials were analyzed for iron, calcium oxide, sulphate and manganese when it was present in significant quantity.

In addition to the above, some experiments have been carried out into the possibility of achieving rapid and suitable crust formation by the addition of ferrous sulphate or ferric sulphate to finely-pulverised quartz to which

pyrites had been added. Surface treatment, by spraying solutions of these materials, and by using cement or lime slurries have also been experimented with.

DISCUSSION OF ANALYTICAL RESULTS

The stability of a slimes dam is a function of the seepage forces of water draining out of the dam, the rainfall penetrating into the dam, the amount of undercutting and the erosion of the dam walls, and the method of deposition and time of drying. The object in having a hard, somewhat impermeable, surface crust on a slimes dam is to increase stability by preventing rainwater from penetrating into the slimes dam through the wall, and also to present a fairly erosion-resistant surface, so that slimes may not easily be washed away from these walls. The deposition of fine slimes in water channels leads to a reduction in their drainage efficiency, as the channels become clogged with slimes. Moreover oxidation products of pyrites lead to contamination of water off slimes dams.

Examination of the results shows the following to be the features of hard slimes dam crusts:

1. Hard slimes dam crusts are characterised by high iron (Fe^{3+}) contents of the order of 5 or more per cent.
2. The percentage of iron in the crusts appears to increase with the age of the slimes concerned. Thus 16 year and 20 year old crusts taken from Marievale and Daggafontein, slimes dams respectively, contained 11.8% and 13.6% of Fe^{3+} .
3. The computed compound composition of the crusts (based on the assumption that any CaO present is in the form of CaSO_4) shows them to consist essentially of a $\text{Fe}_2(\text{SO}_4)_3$ - Ferric oxide and/or hydrated oxide type of material. The iron present in the form of $\text{Fe}_2(\text{SO}_4)_3$ is generally about 20% - 30% of the total Fe^{3+} content. This percentage of Fe^{3+} present in the form of sulphate, occasionally increased to considerably higher values, e.g. Welkom uranium slimes dam crust 15 months old contains 95% of its Fe^{3+} as $\text{Fe}_2(\text{SO}_4)_3$ and Randfontein Estates 20-year old gold slimes dam crust has all its iron present as $\text{Fe}_2(\text{SO}_4)_3$.

4. The percentage of sulphur present as pyrites (after removal of sulphate sulphur by acid solution) on hard crust samples where such sulphur was determined, was found to be practically zero. The only exception to this was the 5-year old gold slimes dam base for the pyrite extract slimes dam at Daggafontein. The sulphur percentage of 1.0 may be unrepresentative and/or represent undecomposed pyrite in this particular sample. The latter is possible since the high percentage of pyrite in this ore may require a rather long period of time for its complete degradation.

Fairly hard crusts, as well as surface materials which were showing definite signs of crust formation, had pyrite sulphur contents ranging from 0.2% to 0.5%. This may represent an intermediate stage of pyrite degradation.

5. The occasional presence of ferrous iron in hard crust or in hardening crust could be expected as the formation of ferrous sulphate represents a step in the reactions postulated for the degradation of pyrites.
6. An interesting observation was that finely-pulverised crust from 20-year old gold slimes dam at Randfontein Estates which was computed to consist essentially of $\text{Fe}_2(\text{SO}_4)_3$, developed cementitious properties on exposure to the air. Conglomerations appeared to have some elastic properties, because if a conglomeration was pulled away from the rest of the material there was a slight rubbery type of resistance. A markedly reddish-brown colouration developed in the surface layers after exposure in the laboratory for several days. This could not be due to oxidation since all the iron was initially present in the ferric state, and a hydrolysis of the $\text{Fe}_2(\text{SO}_4)_3$ to form, under atmospheric conditions, cementitious compounds containing $\text{Fe}_2\text{O}_3 \cdot x\text{H}_2\text{O}$ types of material must be postulated. Since the crust was originally hard, it must be noted that the secondary hardening of the finely ground material indicated a cementitious compound different in nature from Portland cement which does not develop secondary hardening. Following on from this it was found that the finely ground crust when exposed at ambient temperature over saturated sodium chloride in an evacuated desiccator (relative humidity 75%) for two months did not develop the reddish brown colouration and the cementitious properties of the air-exposed

material. A minimum relative humidity, and not the presence of oxygen, may be necessary for the cementitious compound causing the secondary hardening of slimes dam crust, to form.

7. It is well-known that although $\text{Fe}_2(\text{SO}_4)_3$ is very soluble in water it dissolves extremely slowly, so that if there is any present in a slimes dam crust it will not readily be leached away.

When considering crust formation on slimes dams built up with the residues after uranium extraction and generally gold extraction as well, it must be borne in mind that those slimes dams are very young, ranging in age from a few months to about 2 - 3 years old, and it is, therefore, rather difficult to assess the possibilities of crust formation after this short period. However, the data obtained at the Welkom and Freddie's mines slimes dams, yield some interesting information in this connection. Freddie's 2-year old slimes dam formed a fairly hard crust, and even the 1-year old portion of the slimes dam showed definite signs of crust formation. The uranium slimes dam at Welkom (made up of Freddie's gold slimes and other gold slimes, but not of Welkom gold slimes from which uranium is not extracted) had formed a fairly hard crust after 15 months, and definite signs of crust formation were evident even on 6-months old material. This implies that the actual uranium processing itself is of little significance in the subsequent development of surface hardening. Freddie's ore contained approximately 3% of FeS_2 , and there were fairly high iron contents in these uranium slimes dam crusts. On the other hand, Welkom gold slimes of known low pyrite content showed absolutely no signs of an iron type surface crust even on the 2 - 3 year old portion of the slimes dam. The surface material of the Welkom gold slimes dam is also characterised by a remarkably low iron content of less than 1%. Concerning the other uranium slimes dams of recent origin, examined, it was found that the sulphur contents, present as pyrites, are of the order of 0.3% except for the Daggafontein material which contained only 0.07% sulphur. It, however, contained 2.9% of Fe^{3+} .

Welkom 2-year old gold slimes crust contained 0.3% and its 15 month old uranium slimes crust 0.5% sulphur present as pyrites, and some inside material (approximately 6 inches behind the surface) from this 15 month old portion

of the uranium slimes dam contained 0.7% sulphur as pyrites.

On the basis of the evidence available thus far it would seem that for any given slimes grading there is a minimum percentage of pyrites, which is necessary in order to form an adequate crust in any reasonable period or at all. With increasing pyrite percentage the process of crust formation will most probably be accelerated as more pyrite is available. The surface pyrite content also decreases with increasing age of the slimes dam, until it eventually becomes zero. At any intermediate stage the surface pyrite content is probably a function of the original pyrite content and the rate of decomposition. It appears that sulphur contents, derived from pyrites of the order of 0.3% - 0.4% may represent pyrite contents around the critical range. Percentages of sulphur greater than this will probably lead to increasing rates of crust formation with increasing pyrite contents and increasing periods of time.

It must also be concluded that pyrite extraction as such will not necessarily lead to the formation of slimes dams which will not form crusts. Provided sufficient pyrite is left in the material, crust formation will in due course take place. Thus the Daggafontein and Government Gold Mining Areas slimes, from which the major portion of the pyrite has been extracted, show definite signs of crust formation. This slimes dam at Government Gold Mining Areas, which is only about 2 years old still has a pyrite content of about 1%. (Original pyrite content before removal of the pyrite by flotation was about 9 - 10%).

The problem, therefore, occasioned by the development of uranium and pyrite extraction from gold ores, is probably not so much the inability of these residues to form an erosive resistant crust, as one of decreased stability. This is the result of having to cope with greater tonnages of material due to monitoring of old slimes dams, as well as one plant having to deal with slimes from several sources and also the fact that these new slimes have a greater water/solids ratio than before (i.e. of the order of 1.5 tons of water per ton of slimes solids).

LABORATORY EXPERIMENTS ON ACHIEVING RAPID CRUST FORMATION ON SLIME MATERIALS

Very finely pulverised clean quartz sand (surface area 3673 sq.cm./gram) which had been prepared by ball-milling, was intimately mixed with very fine pyrite as prepared in the flotation plant at Government Gold Mining Areas. This pyrite contained 80 - 85% FeS_2 and in terms of fineness is typical of pyrite normally present in slimes dams. This mixture was used as the slime material in these experiments. Initially an attempt was made to develop an accelerated laboratory technique for crust formation by utilising hydrogen peroxide as an oxidising agent for pyrites. It was considered that such an accelerated technique might permit the rapid assessment of the crust forming potentialities of slimes of varying composition. This was, however, not successful and this approach was abandoned.

Pats of slimes material with various additions were prepared in bakelite moulds by mixing with water to a stiffish consistency. These pats were 7 cm. diameter at the bottom surface, 6 cm. diameter at the top surface and 4 cm. in height. The pats were subjected to about six wetting and drying cycles under laboratory atmosphere conditions. After an initial drying period of 3 - 5 days they were well wetted by gentle spraying with water and allowed to get fairly dry again before the next wetting. Surface treatments of these test pats were also carried out immediately after preparation of the relevant pats and continued for 4 - 6 cycles of wetting and drying.

DISCUSSION OF RESULTS

Admixture of 3% $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$ to a slime containing $1\frac{1}{2}\%$ of pyrite resulted in a rather promising crust being formed. A poor yellowish flaky crust, which exhibited a tendency to peel off and also showed some whitish-yellow efflorescence, was formed when 3% of $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$ was added to the finely pulverised quartz without pyrite addition. On the other hand with a $12\frac{1}{2}\%$ pyrite content and only a $1\frac{1}{2}\%$ $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$ addition, only poor crust formation resulted, with this crust also exhibiting some brownish-white efflorescence and a tendency to flakiness. The above results

seem to imply that the presence of some pyrite is necessary for the formation of a suitable crust when $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$ is added to the slimes material. An appreciable minimum quantity of $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$ also appears to be necessary to ensure crust formation.

Ferric sulphate, $\text{Fe}_2(\text{SO}_4)_3$, as an additive to slimes materials, did not confer any good crust-forming properties in these tests. A 3% addition of this material to a slime containing $1\frac{1}{2}\%$ of pyrite resulted in a weak yellowish-brown crust being formed. This is in contrast to the 3% addition of $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$ to a similar slime material, when a good crust developed. When no pyrite was present in the slimes even a 6% addition of $\text{Fe}_2(\text{SO}_4)_3$ caused the development of a brownish efflorescence rather than any real crust. With a $12\frac{1}{2}\%$ pyrite content and 6% $\text{Fe}_2(\text{SO}_4)_3$ content in a slimes, a yellowish-brown efflorescence was formed.

Surface treatment of slimes material containing $1\frac{1}{2}\%$ pyrites, by spraying with a 5% solution of $\text{Fe}_2(\text{SO}_4)_3$, resulted in a firm and hard darkish-brown crust developing. Spraying of the same slime with a 5% $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$ solution produced a rather weak yellowish-brown crust. This crust, however, appeared to harden with age. The effect of spraying the solutions, therefore, appears to be contrary to that found with the admixture of the solids, when $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$ produced the best crust.

Surface treatment with an ordinary Portland cement slurry and with a high-calcium lime slurry to a nominal thickness of $1/16$ inch, led to the formation of hard but somewhat brittle crusts which adhered fairly well to the underlying material.

X-ray diffraction data have been obtained on some gold slimes dam crust samples from the Government Gold Mining Areas slimes dam. Before examination these were enriched by settling in alcohol where the coarser quartz particles settled first. The major detectable constituent (excluding quartz) was coquimbite, the hexagonal form of $\text{Fe}_2(\text{SO}_4)_3 \cdot 9\text{H}_2\text{O}$. According to the literature this compound crystallises in hexagonal plates by evaporating to dryness a solution of hydrated ferric oxide in an excess of sulphuric acid. Such conditions will be achieved in practice in slimes dams during the process of degradation of pyrites. It is possible that this compound may play a

role in the cementing agent of natural slimes dam crusts. Some further X-ray diffraction data have also been obtained on samples of experimental crusts which had been scraped away from the test pats. These were generally enriched by settling in alcohol prior to examination by X-ray diffraction. Coquimbite was found to be present in both of the good crusts, i.e. 3% $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$ addition and spraying with 5% $\text{Fe}_2(\text{SO}_4)_3$ solution. $\text{FeSO}_4 \cdot 4\text{H}_2\text{O}$ was a common constituent of virtually all the crusts examined, including those where only $\text{Fe}_2(\text{SO}_4)_3$ was added to slime containing no pyrite.

Although the reaction taking place in actual slimes dams and even in these laboratory experiments on crust formation appear to be rather complex, it nevertheless seems that there are possibilities of producing in a reasonable period, crusts on slimes dams that will not normally develop them, or on those that will only develop them very slowly. A possible way of achieving this is by dusting or spraying ferrous or ferric sulphates or a mixture of these on to the slimes dam walls. The crusts formed might be similar in appearance and properties to the natural type that normally develop.

CONCLUSIONS

On the basis of the results of chemical analyses it is concluded that hard slimes dam crusts are characterised by high Fe^{3+} contents of 5 or more percent, the actual Fe^{3+} percentage increasing with age. Generally about 20 - 30 percent of this iron is present in the form of $\text{Fe}_2(\text{SO}_4)_3$ with the balance of the iron present as oxides and/or hydrated oxides. The hardest crust contained scarcely any pyrites although fairly hard crusts contained 0.4 - 1.0 percent of pyrites.

The actual processing of the ore for uranium, gold and pyrites has not direct relationship to crust formation, in that such slimes still contain the minimum pyrite content of about 0.7 percent apparently essential for the formation of hard crusts. However, because of the recent development of uranium and pyrite extraction greater tonnages of slimes material at a higher water/solids ratio than the normal gold extraction process are being deposited at certain mines and this aggravates the problem.

Attempts to form crusts on a laboratory scale on small test pats have shown that it is possible that dusting or spraying the surface with ferrous or ferric sulphates or a mixture of these may lead to the formation of adequate crusts in a reasonable period. X-ray diffraction data have shown that the hard natural slimes dam crusts as well as the harder crusts which were developed during the laboratory experiments, contained coquimbite, the hexagonal form of $\text{Fe}_2(\text{SO}_4)_3 \cdot 9\text{H}_2\text{O}$. Surface stabilisation of slimes dams by coating with a cement or lime slurry was also found to be promising in these experiments. Further research along these lines on pilot slimes dams may perhaps be the most useful future approach.

RECOMMENDATIONS

- (a) If the pyrites content of the slimes is below 0.7%, crust formation may be encouraged by the use of additives.
- (b) Gullies should be filled before they become serious.
- (c) The planting of suitable vegetation on the slopes of dams.

CHAPTER III.

SEEPAGE PROBLEMS

ANALYSIS OF SEEPAGE FLOW

Wherever water is impounded behind an earth embankment, some of it will penetrate through the embankment as seepage water. The amount of water flowing out and the rate at which it flows will be determined by the permeability of the material in the embankment, the hydrostatic head causing flow, and the length of the flow paths.

If water is placed in a dry dam, some of that water will immediately start to penetrate through the voids in the embankment and work its way to the existing water table. Initially the water will move vertically downward until its path is blocked by some impermeable barrier, and then it will move horizontally as well. This horizontal spread will continue until the water finds its way into the water table. Fig. 4 illustrates the above process diagrammatically.

Behind the advancing waterfront there will be a saturated zone of seepage bounded by the outer, or upper, flow-line, which begins at the intersection of the water level in the dam and the inner face of the dam wall. This boundary is known as the phreatic surface (something similar to ground water level). As the zone of seepage advances the phreatic surface will alter to suit the flow conditions. Eventually, after the percolating water has reached its goal, the seepage flow will come to equilibrium and the phreatic surface will remain constant.

The equilibrium conditions that are reached for any particular head of water and set of drainage conditions, are known as "steady state seepage". The position of the phreatic surface, flow-lines and equi-potential lines for the "steady seepage" case, under any set of conditions, can be determined theoretically by mathematical processes. The solutions can be obtained graphically (6), by relaxation methods (7) (8), or by model studies and various analog techniques. However, when the conditions become very complicated an accurate determination of the flow net becomes exceedingly difficult to obtain, and when conditions are almost infinitely variable as well as being complicated,

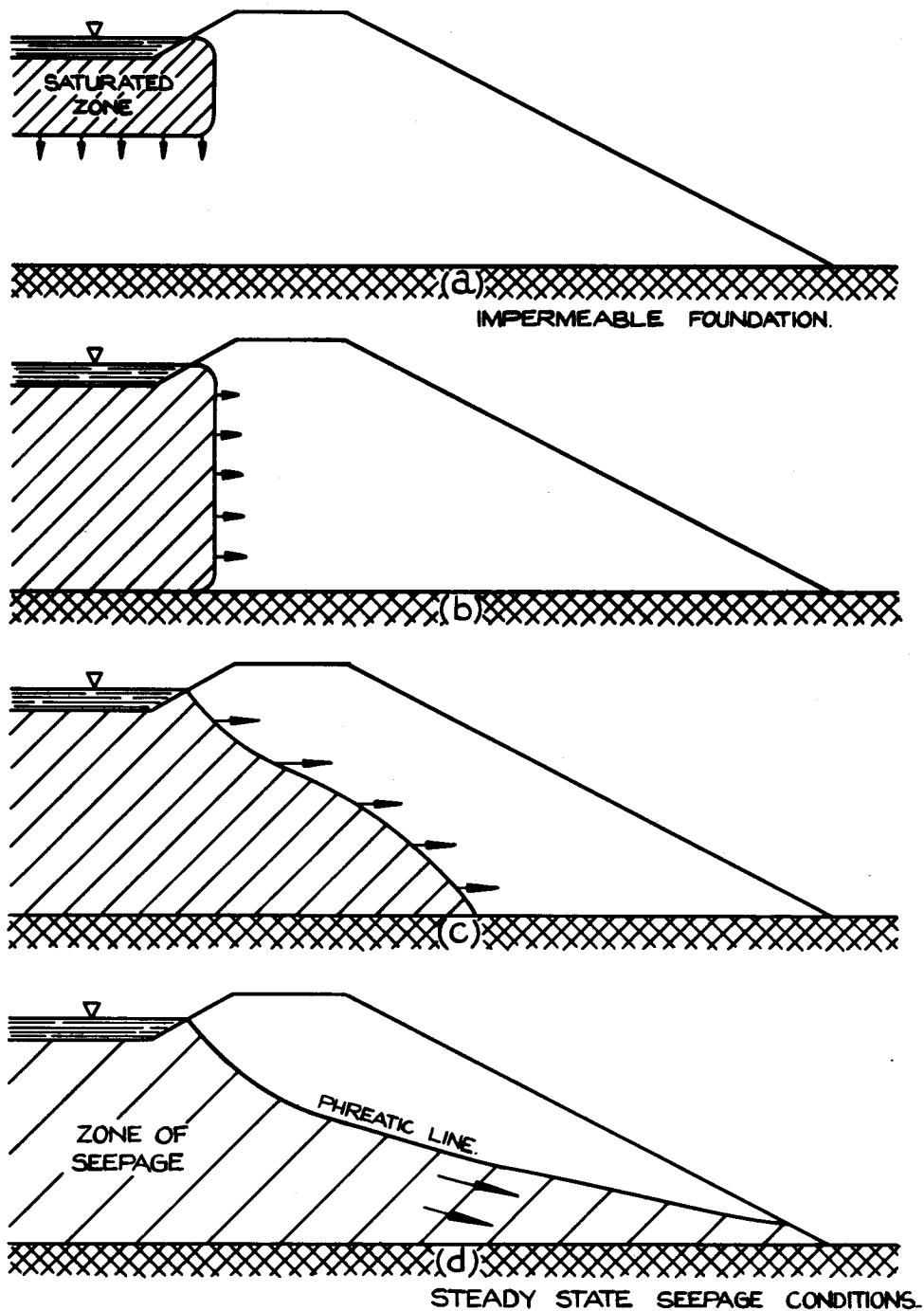


FIG. 4.

SCHEMATIC ILLUSTRATIONS TO SHOW STAGES IN THE DEVELOPMENT
OF A ZONE OF SEEPAGE FLOW FROM A DRY CONDITION.

derivation of an accurate solution becomes virtually impossible.

By way of illustration, the flow nets for various simplified slimes dam sections will be considered. The following assumptions will be made:

- (a) The slimes material, as placed on the slimes dam, is uniform, homogeneous and isotropic, and in particular has the same permeability in all directions.
- (b) The foundation soil is a uniform homogeneous, isotropic material.
- (c) There is a permanent pool of water on the floor of the dam which is maintained at a constant level.
- (d) There is no water on the dam outside the pond.

The effect of these assumptions will be dealt with later.

Case 1.

The slimes dam is built on a deep layer of material having a much greater permeability than the slimes, and the water table is deep. Water will seep vertically down through the slimes into the more permeable material where it will percolate down to the water table. As the voids in the coarse material will never be filled with water no flow net can be drawn in this zone. This case is illustrated in Fig. 5.

Case 2.

The slimes dam is built on an impermeable layer of soil. No water can escape into the foundation soil and so all the seepage water must flow through the wall of the dam. This case is shown in Fig. 6.

Comparing Cases 1 and 2 we see that in Case 1, where there is full drainage, the outer portion of the wall is left dry,

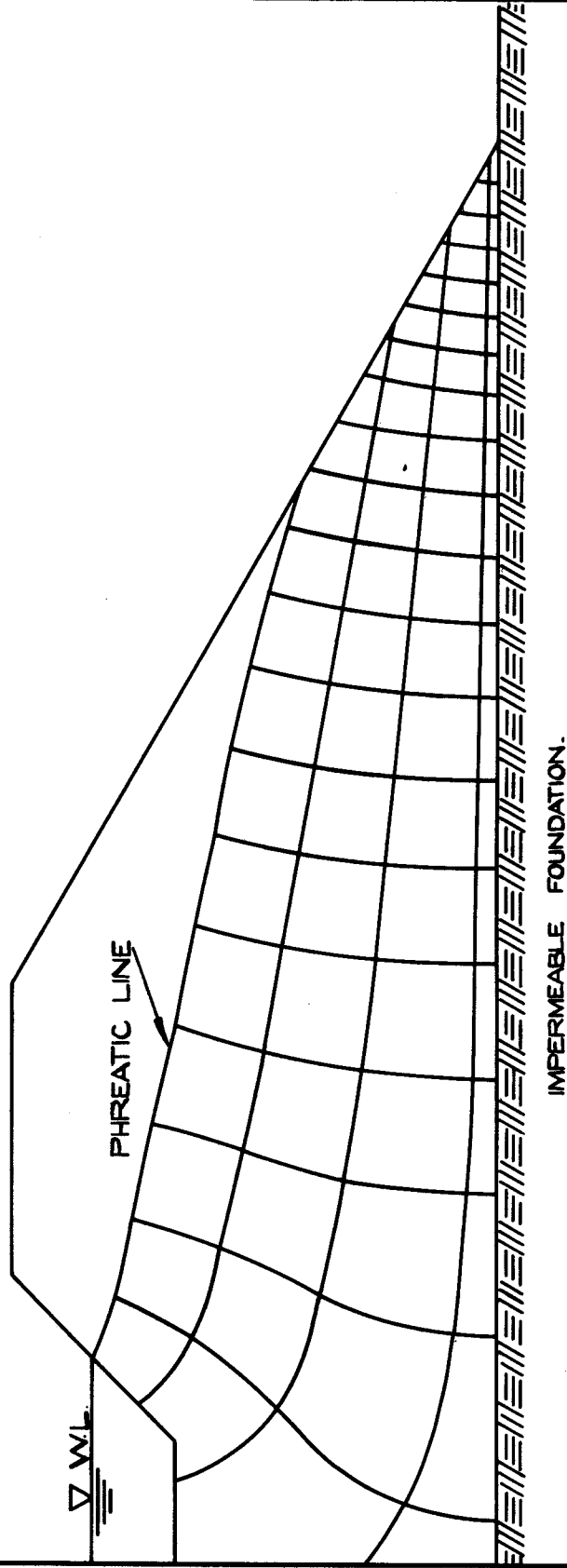


FIG. 6.

FLOW NET FOR A DAM WITH AN IMPERMEABLE FOUNDATION.

while in Case 2, where there is no drainage, seepage water is passing through the greater part of the wall leaving only a small dry area at the top.

These two cases represent the extremes for the slimes dam being considered. There are an infinite number of intermediate cases where the position of the phreatic surface will be governed by (i) the ratio of the permeability of the foundation soil to the permeability of the slimes material in the dam; (ii) the ratio of the depth of the permeable layer below the dam to the height of the dam; (iii) the position of the water table and the natural drainage of the site. These factors are very closely interrelated and it is exceedingly difficult to separate them. However, in the following pages an attempt will be made to treat each factor on its own and as a start, the effect of the ratio of the permeabilities between foundation soil (permeability k_A) and slimes material (permeability k_s) will be considered.

Case 3. Fig. 7.

The permeability of the foundation soil is the same as that of the slimes material and the depth of the foundation soil, D_f is $1/3$ of the height of the dam H_d . The ground-water level outside the dam is level with the foundation soil surface.

Case 4. Fig. 8.

The permeability of the foundation soil is twice that of the slimes material and the depth of this layer D_f is $1/3$ of H_d . The ground-water level outside the dam is level with the foundation surface. -

Case 5. Fig. 9.

The permeability of the foundation soil is 5 times that of the slimes material and the depth of the layer D_f is $1/3$ of H_d . The ground-water level outside the dam is level with the foundation soil surface.

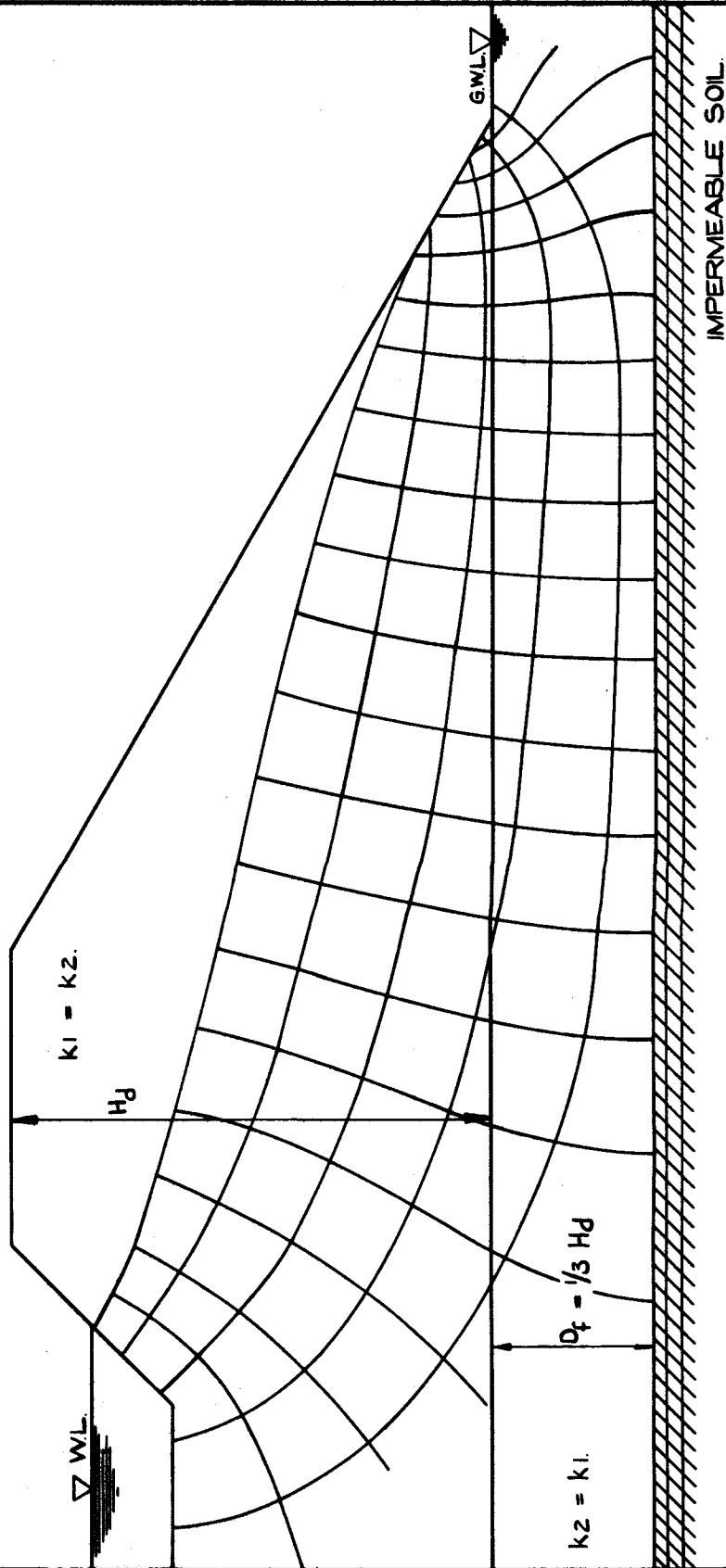


FIG. 7.

FLOW NET FOR CASE 3.

$$k_2 = k_1$$

$$D_f = \frac{1}{3} H_d$$

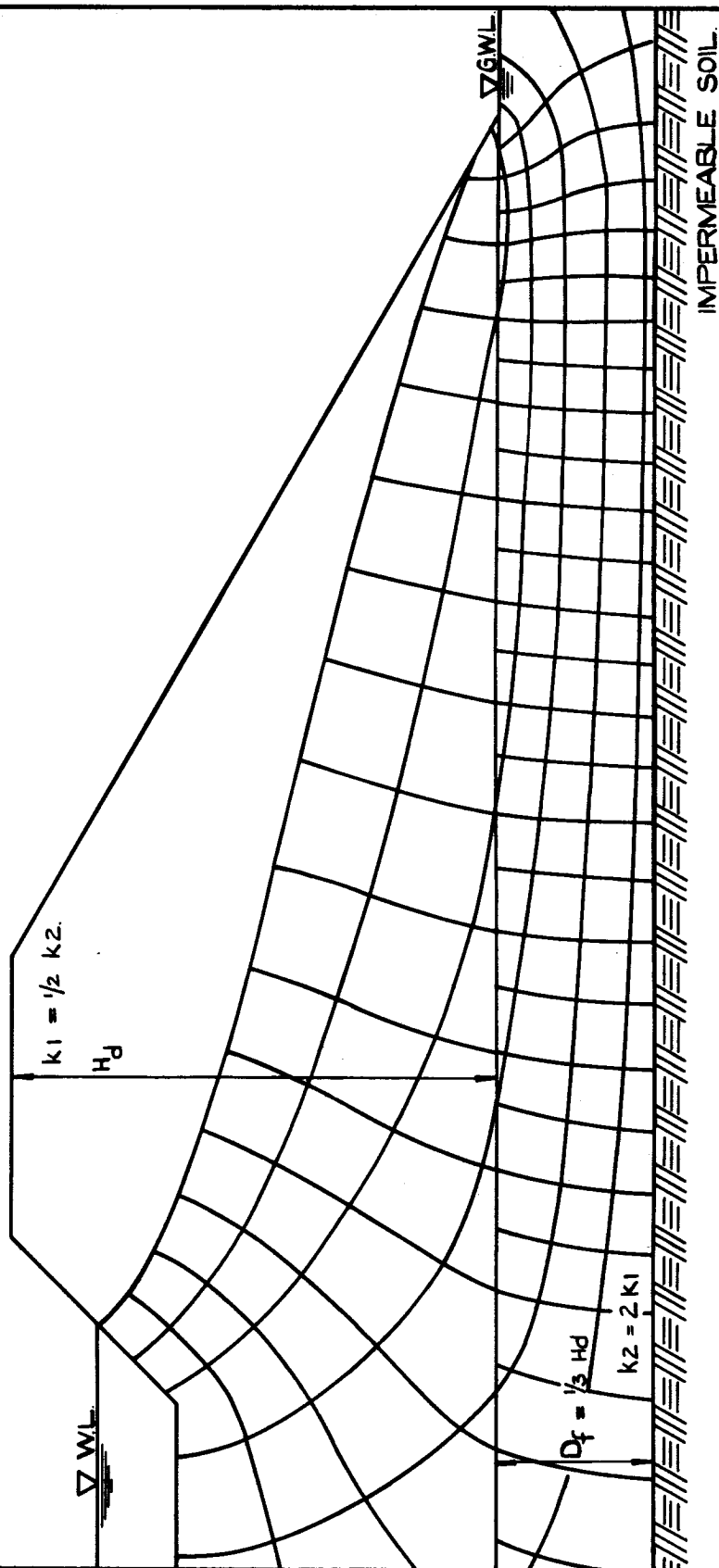


FIG. 8.

FLOW NET FOR CASE 4.

$$k_2 = 2 k_1.$$

$$D_f = \frac{1}{3} H_d.$$

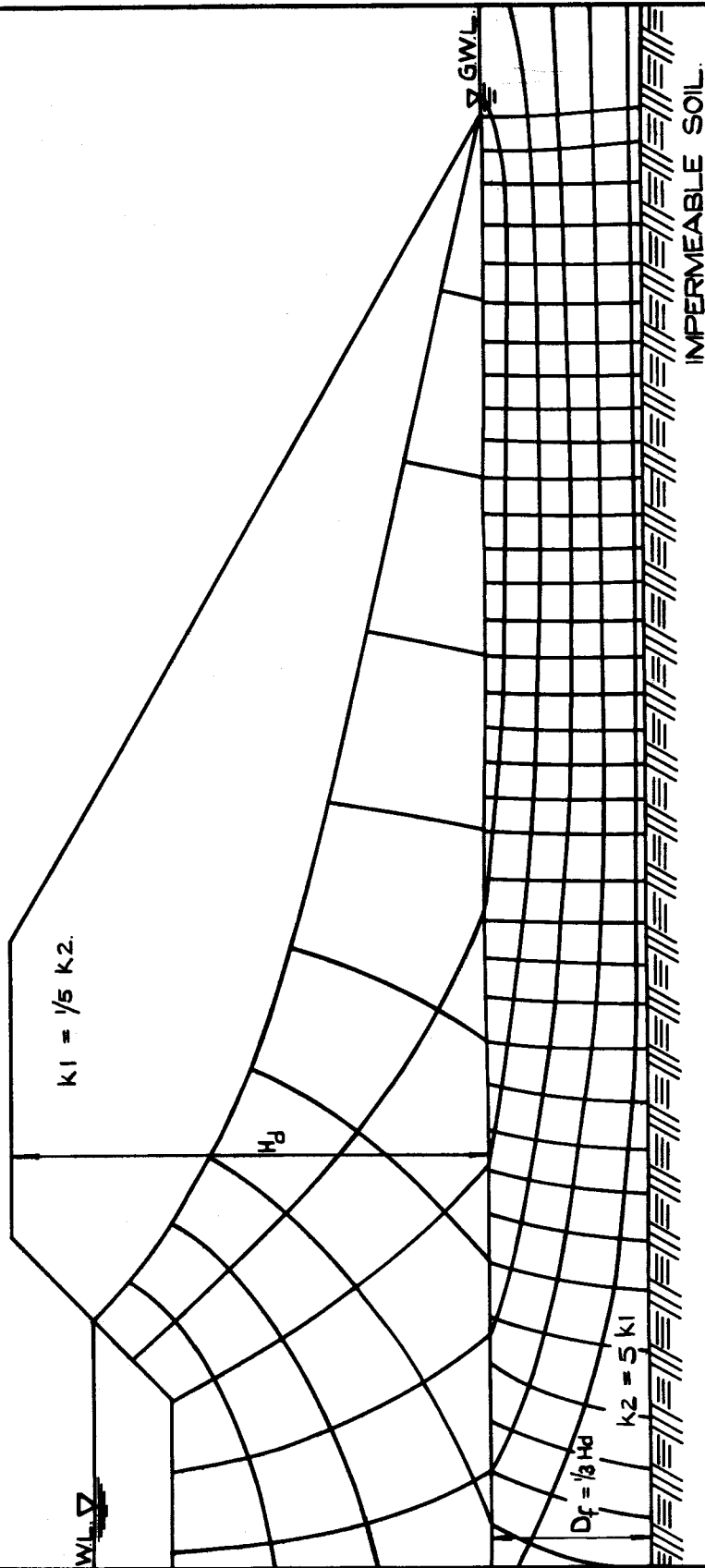


FIG. 9.

FLOW NET FOR CASE 5.

$$k_2 = 5 k_1$$

$$D_f = \frac{1}{3} H_d$$

Case 6. Fig. 10.

Permeability of the foundation soil is 10 times that of the slimes material and the depth of this layer D_f is $1/3$ of H_d . The ground-water level outside the dam is level with the surface of the foundation soil.

Case 7. Fig. 11.

The permeability of the foundation soil is the same as that of the slimes material and the depth of the foundation soil D_f is $2/3$ of H_d . The ground-water level outside the dam is level with the surface of the foundation soil.

Case 8. Fig. 12.

The permeability of the foundation soil is twice the permeability of the slimes material and $D_f = 2/3 H_d$. The ground-water level outside the dam is level with the surface of the foundation soil.

Case 9. Fig. 13.

The permeability of the foundation soil is 5 times the permeability of the slimes material and $D_f = 2/3 H_d$. The ground-water level outside the dam is level with the surface of the foundation soil.

Case 10. Fig. 14.

The permeability of the foundation soil is 10 times the permeability of the slimes material. The ground-water level outside the dam is level with the surface of the foundation soil.

Cases 2, 3, 4, 5 and 6 refer to a dam where D_f is $1/3$ of H_d and the ground-water level outside the dam coincides with the ground surface. The effect of the variation of the ratio of the permeabilities of the slimes material and the foundation material on the position of the phreatic surface for this dam, is shown in Fig. 15.

Similarly, Cases 2, 7, 8, 9 and 10 refer to a dam where D_f is $2/3$ of H_d and the ground-water level outside the dam coincides with the ground surface. Fig. 16

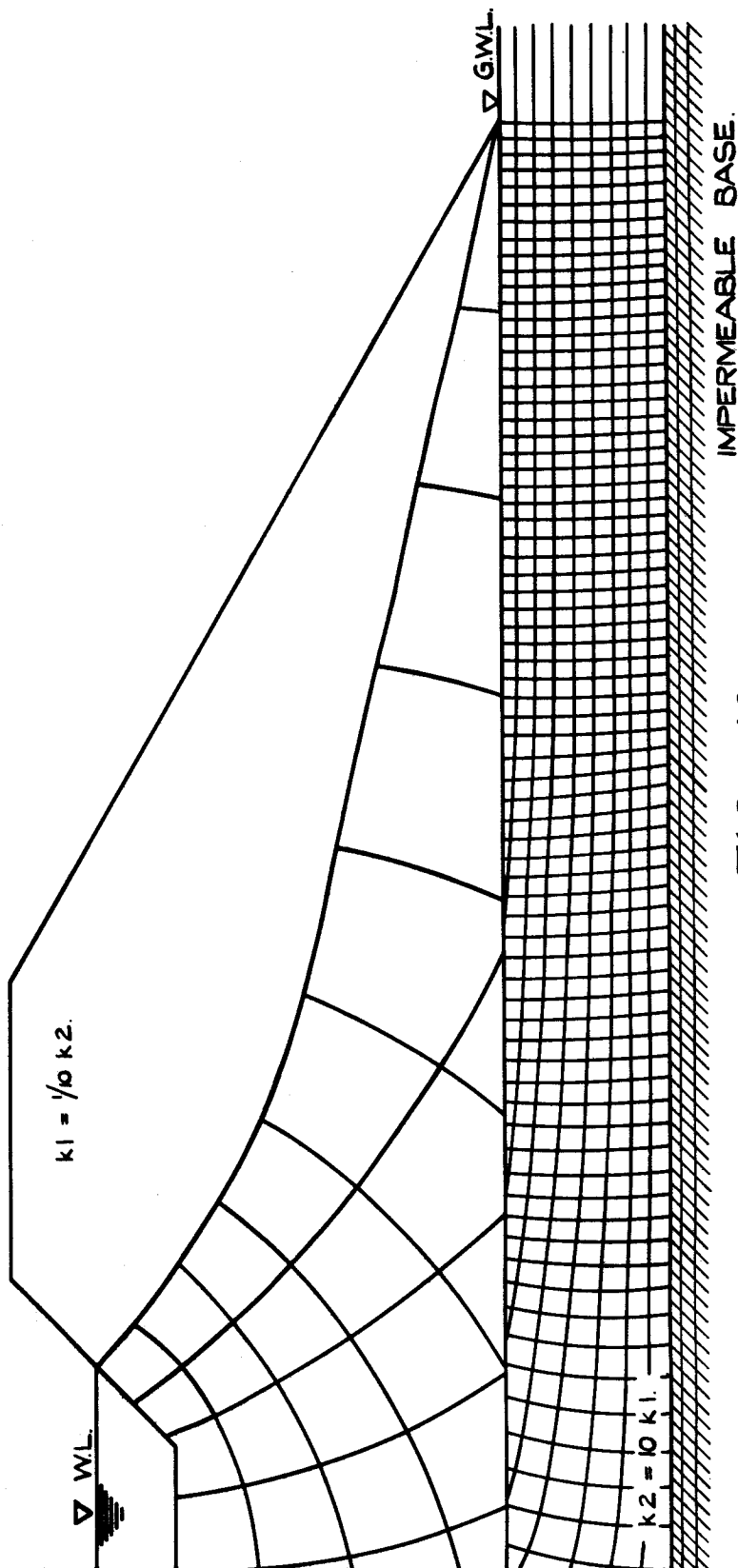


FIG. 10.

FLOW NET FOR CASE 6.

$$k_2 = 10 k_1$$

$$D_f = \frac{1}{3} H_d$$

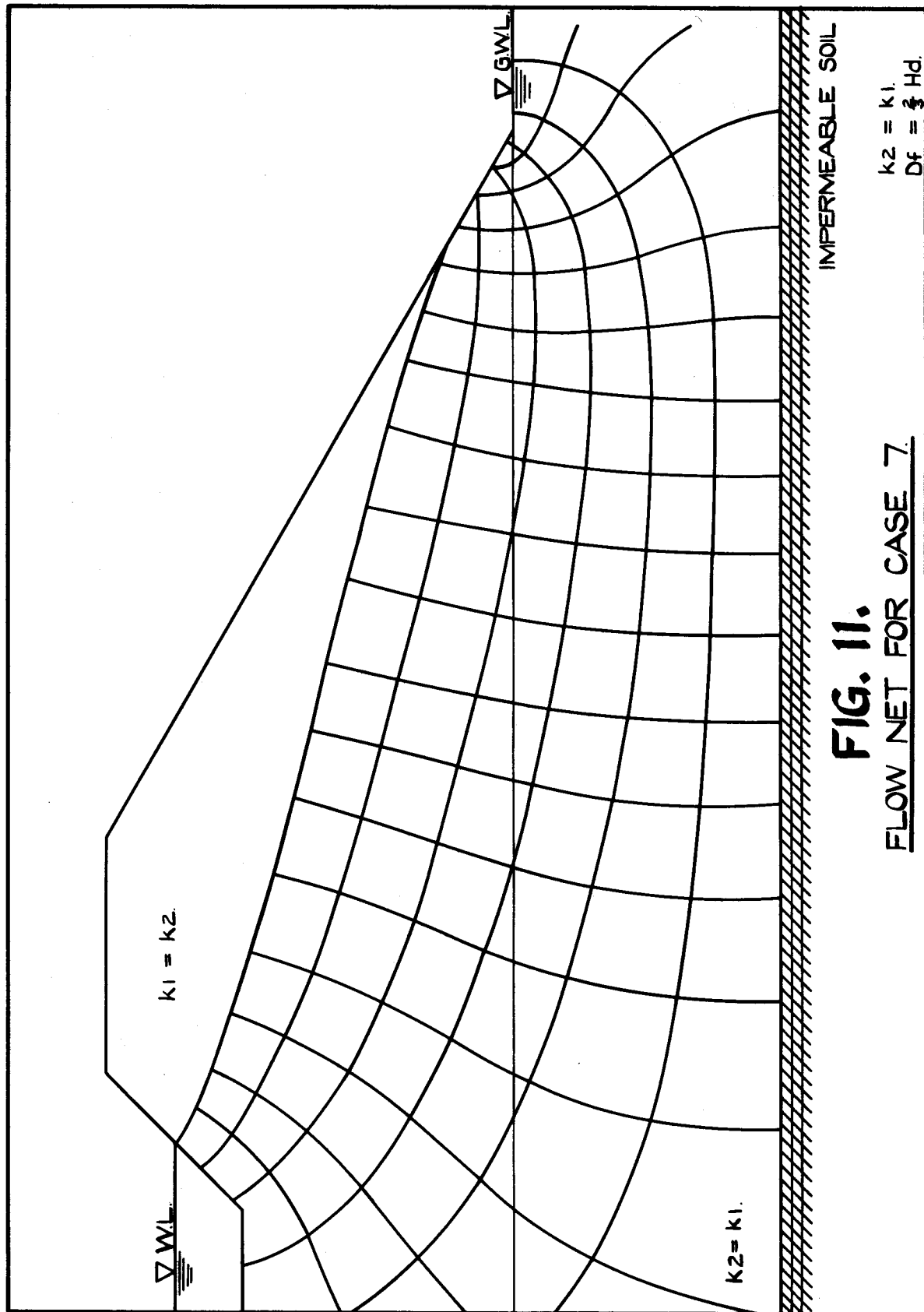


FIG. 11.
FLOW NET FOR CASE 7.

$$k_2 = k_1$$

$$D_f = \frac{1}{2} H_d$$

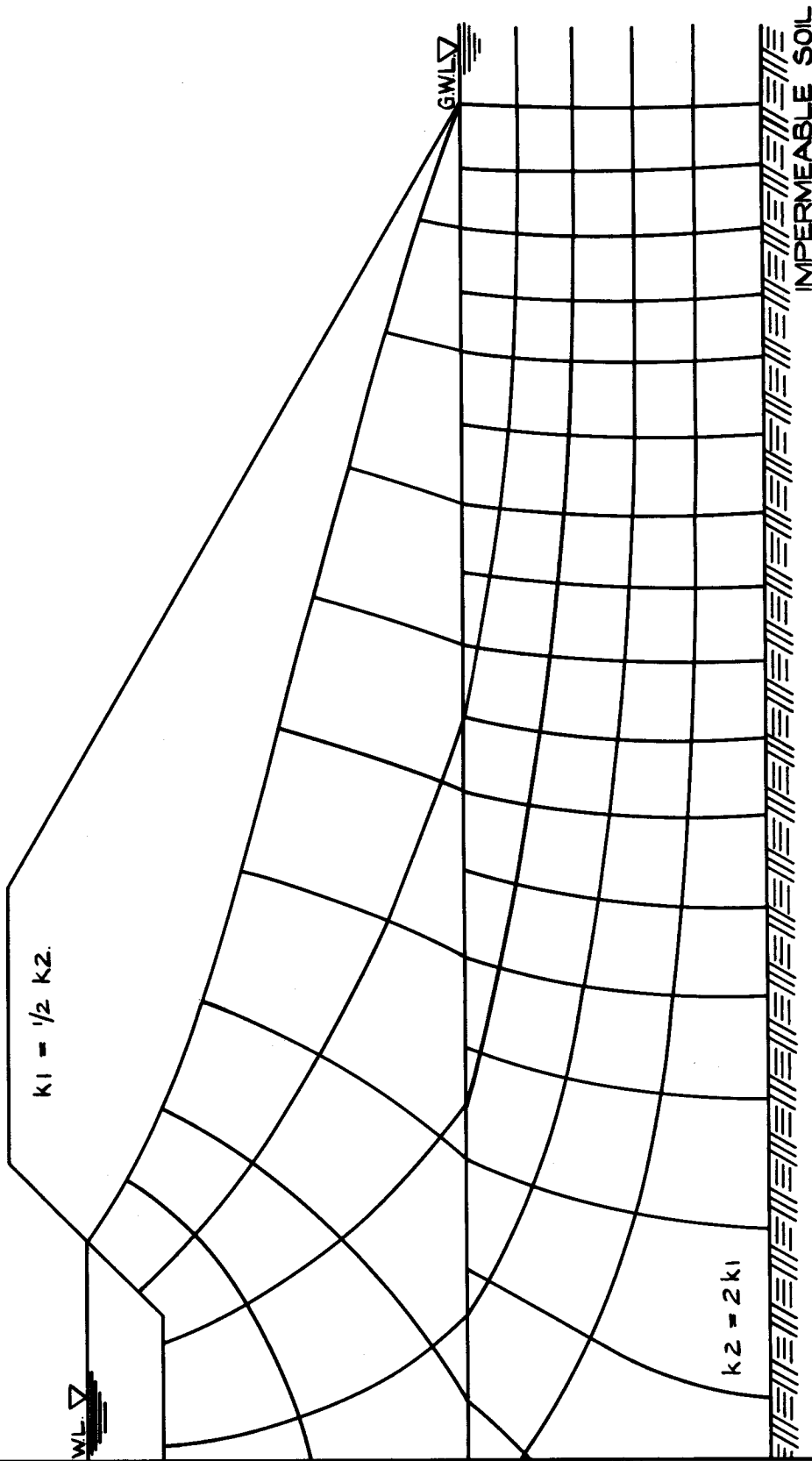


FIG. 12.

FLOW NET FOR CASE 8.

$$k_2 = 2 k_1$$

$$D_f = \frac{2}{3} H_d$$

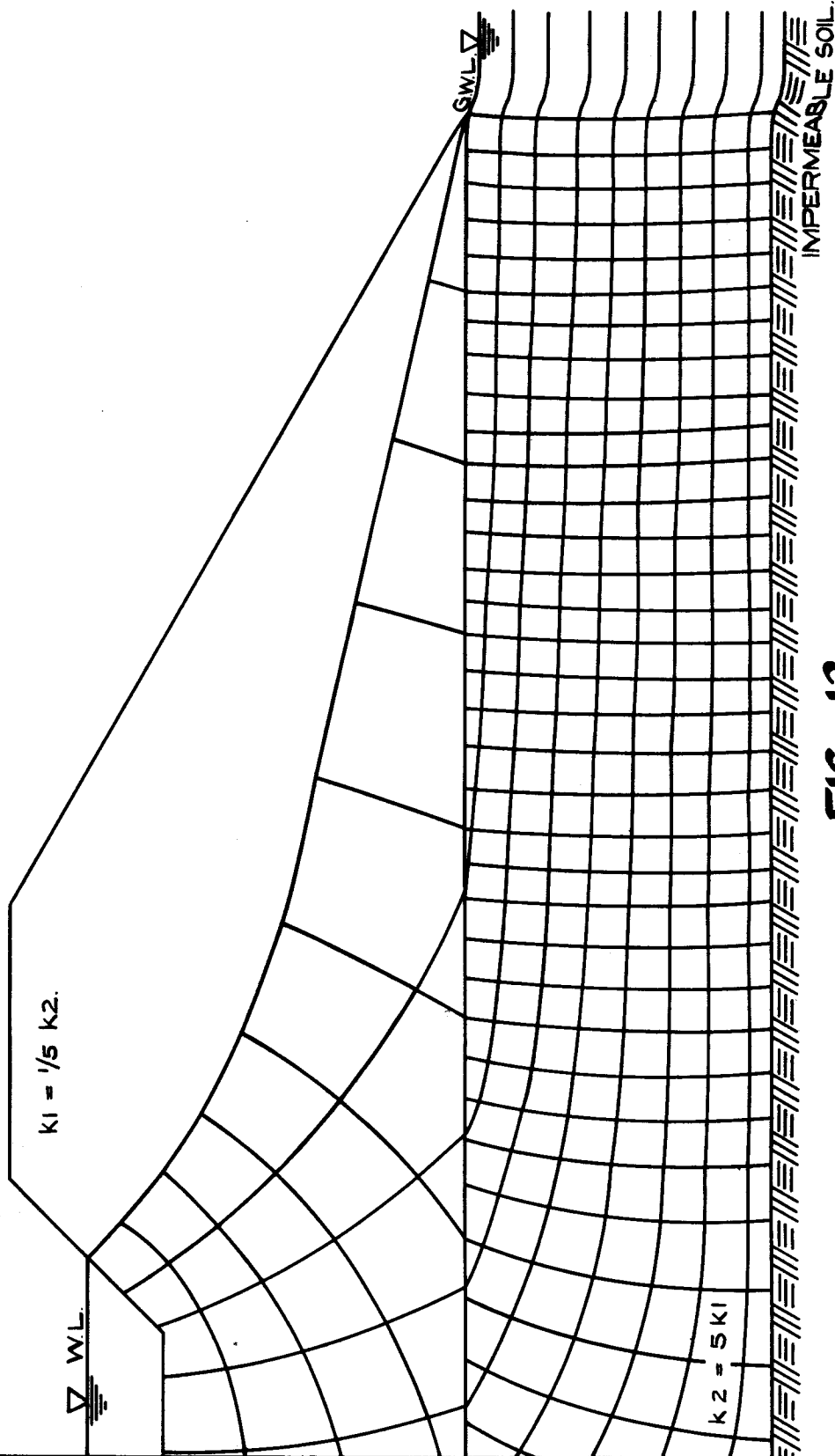


FIG. 13.

FLOW NET FOR CASE 9

$$k_2 = 5 k_1$$

$$D_f = \frac{5}{3} H_d$$

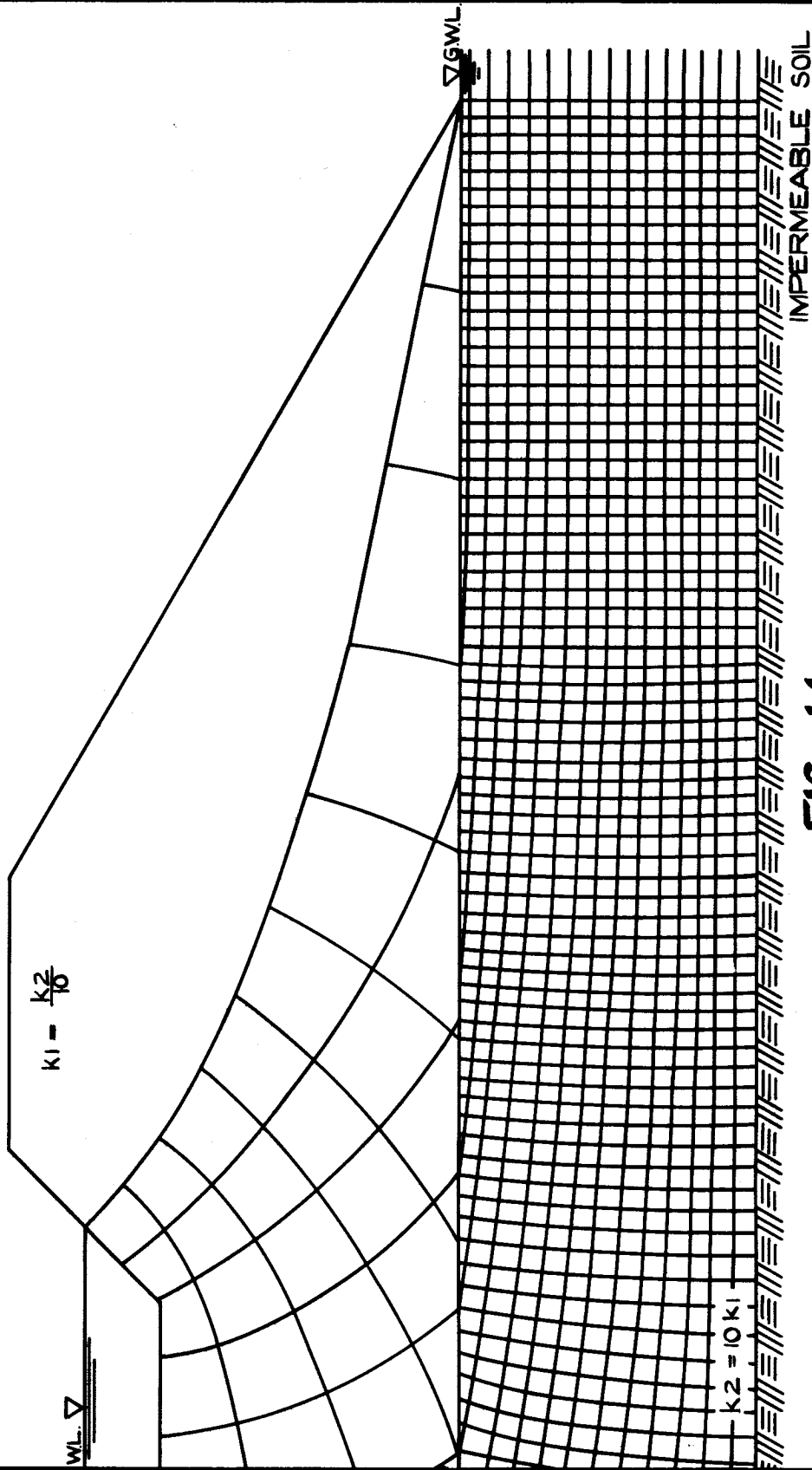


FIG. 14.

FLOW NET FOR CASE 10.

$$k_2 = 10 k_1$$

$$D_f = \frac{2}{3} H_d$$

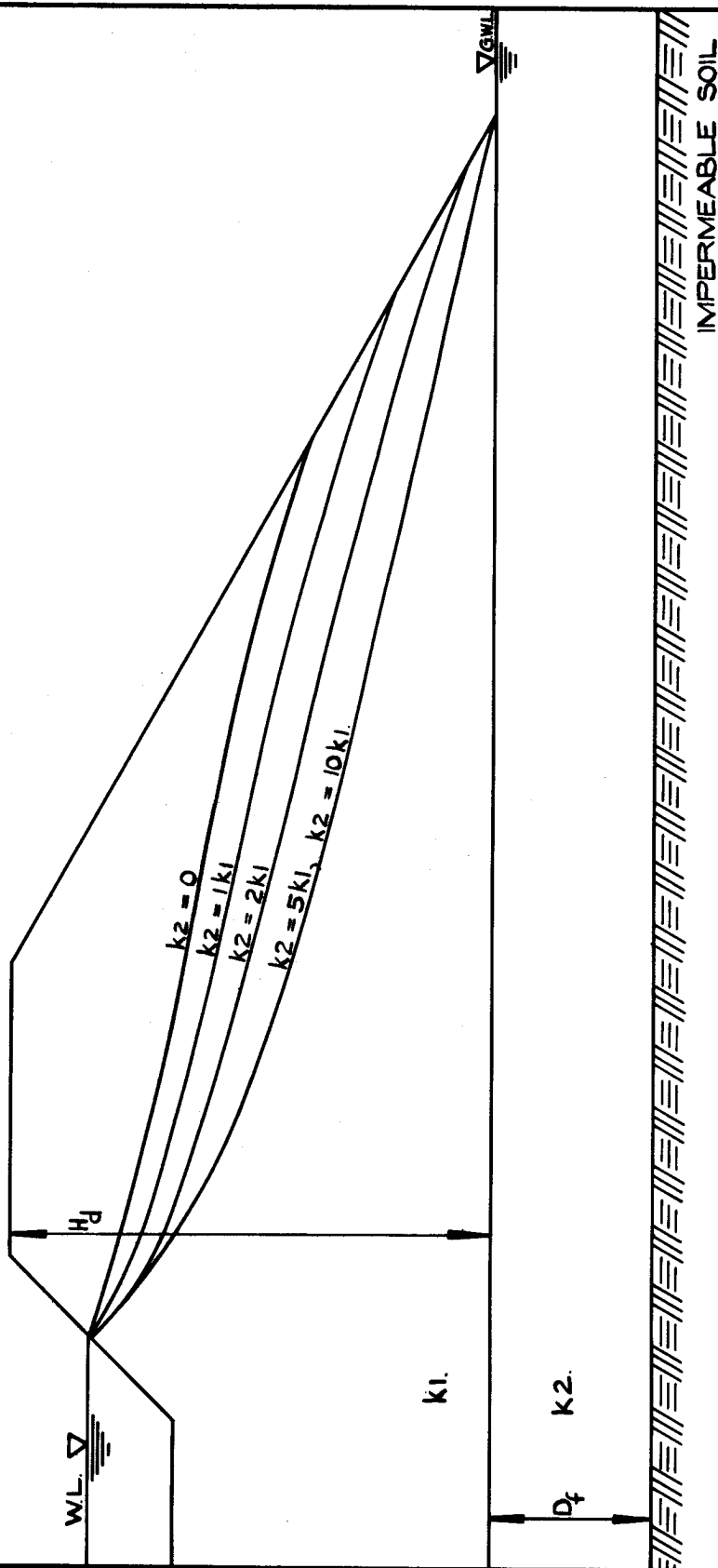


FIG. 15.

COMPARISON OF PHREATIC SURFACES FOR CASES 2, 3, 4, 5 AND 6.

$$D_f = \frac{1}{3} H d.$$

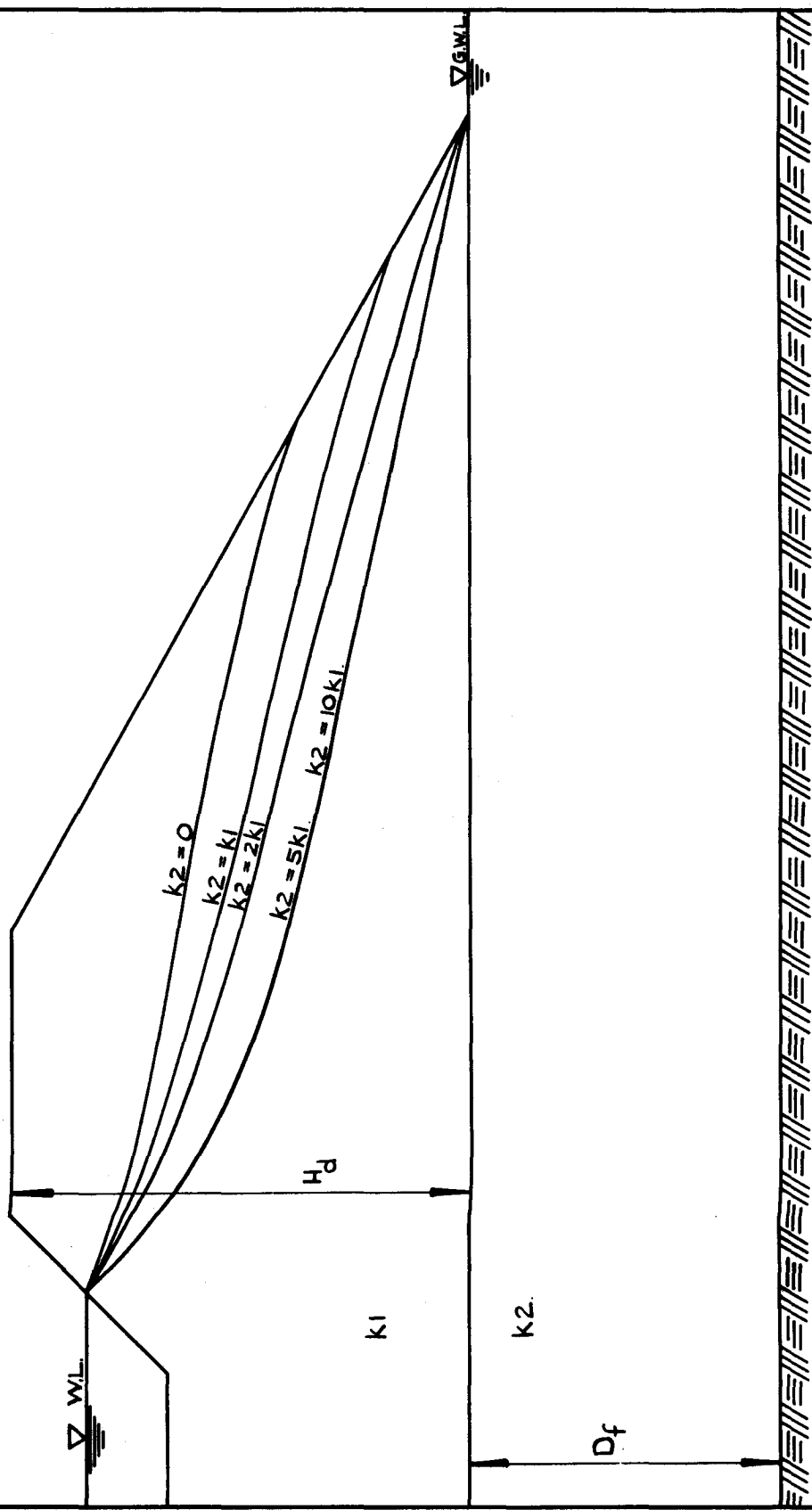


FIG. 16.

COMPARISON OF PHREATIC SURFACES FOR CASES 2, 7, 8, 9 AND 10.

IMPERMEABLE SOIL

$$D_f = \frac{2}{3} H_d$$

combines the phreatic surfaces for these cases, showing the effect of the ratio of permeabilities between the slimes material and the foundation soil on the position of the phreatic surface.

It will be seen from Figures 15 and 16 that the phreatic surface becomes lower as the ratio between the permeabilities of the foundation soil and the slimes material becomes greater, until the case where the foundation permeability k_2 is equal to about 5 times the slimes permeability k_1 , after which any further increase of the ratio makes very little difference to the position of the phreatic surface. It therefore becomes apparent that for any specific dam section with a certain depth of foundation soil and a fixed position of the ground-water level the position of the phreatic line will be lowered as the ratio between the permeability of the foundation soil and the slimes material increases until this ratio reaches a limiting value, of B, above which no sensible alteration in the position of the phreatic line will take place.

The determination of this limiting factor B is not simple but, for the purpose of this investigation, it is enough to know that such a limitation exists. It can therefore be seen that the presence of highly permeable foundation soil will not necessarily ensure adequate drainage of the dam if other factors are unfavourable.

The second factor to be considered is the effect of the ratio of the depth of the permeable layer to the height of the dam. Comparing Figures 15 and 16 it will be seen that there is very little difference in the positions of the phreatic lines for the same permeability ratios. This shows that the effect of the depth of the permeable layer alone, on the position of the phreatic surface, is very small.

Combining the above two conclusions it will be seen that if the dam is built on a deep layer of highly permeable soil, this will not necessarily ensure that the dam is adequately drained; however, it has definitely been shown that a relatively impermeable layer, or very shallow layer of not very permeable material, will cause seepage trouble.

The third factor to be considered, is the effect of the position of the ground-water level, which cannot really be considered on its own as its position is dependent on the conditions of natural drainage of the site. As the ground-water level is itself a phreatic surface in

the foundation soil, it cannot be considered as an end condition for the phreatic surface in the dam, since any additional water seeping from the dam may also influence the position of the ground-water level. The position of the ground-water table and the natural drainage of the site are affected by the topography and geology of the site.

In view of the infinite number of combinations of the various influences that can be found on any site, it is impossible to deal with them separately and to analyse the effect of each on the seepage water in the dam.

It has already been shown in Figures 15 and 16 that where there is no sub-soil drainage ($k_2 = 0$) the phreatic surface intersects the outer wall of the dam above the toe. As the drainage improves (i.e. k_2/k_1 increases) the point of intersection of the phreatic surface and the wall moves down until it coincides with the toe of the wall. The point of intersection of the phreatic surface and the outer wall of the dam will remain at the toe, irrespective of permeability ratios, as long as the ground-water level outside the dam remains level with the foundation soil.

If the natural sub-surface drainage of the site is sufficient to cope with all the natural water from the site plus the additional seepage water from the dam, the ground-water level will be drawn below the ground surface at the toe of the dam and the phreatic line will pass through the base of the dam into the foundation soil. As the sub-surface drainage improves, the point of intersection of the phreatic surface and the base of the dam will move in towards the pond until, when there is unlimited sub-surface drainage, the condition shown in Fig. 5 is reached. Some of the possible phreatic surfaces for conditions varying from an impermeable base to unlimited sub-surface drainage, are shown in Fig. 17.

As was stated in the opening paragraphs, the phreatic surface is the upper boundary of the zone of seepage flow. It will therefore be seen that where the phreatic surface passes through the toe of the dam, or intersects its outer face, seepage water will flow out through the toe of the dam and lower part of the wall. This phenomenon is commonly called "weeping". This seepage water erodes the slimes material from the toe of the wall, undercutting the wall and endangering its stability. Seepage erosion can therefore, be expected where the phreatic line emerges at or above the toe of the dam.

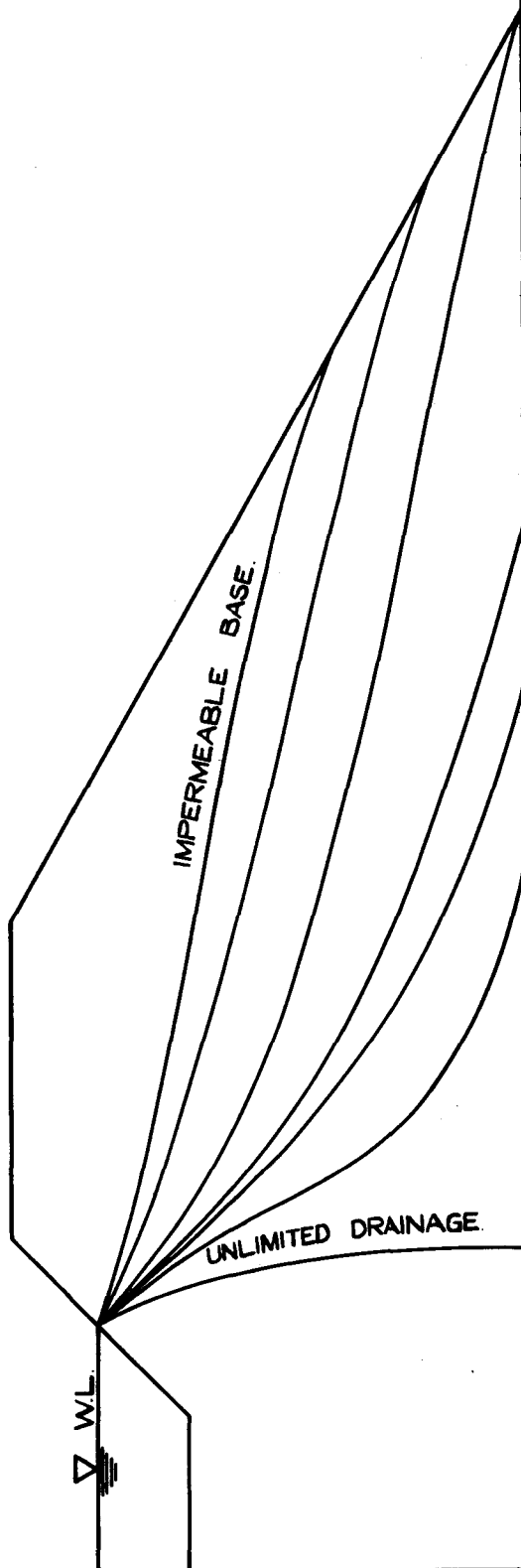


FIG. 17.

POSITIONS OF PHREATIC SURFACE FOR DIFFERENT CONDITIONS OF SUBSURFACE DRAINAGE.

From the foregoing study of flow patterns in slimes dams it can be stated that there will definitely be seepage problems under any of the following conditions:

- (a) where the foundation soil is relatively impermeable, i.e. has a lower permeability than the slimes material, e.g. solid rock, clay, sandy clay, shale;
- (b) where the water table is very shallow, in other words, water-logged ground;
- (c) where natural drainage conditions are bad, e.g. vleis, pans, marshy ground.

The examples given are fairly obvious but other combinations of unfavourable circumstances may not be easily recognised and it is recommended that the site for any proposed slimes dam should be surveyed by an expert on drainage problems and should be zoned into areas of good, medium and bad sub-surface drainage.

If drainage conditions are bad or likely to become so, the solution to the problem is to provide artificial drains which will trap the seepage water inside the dam and bring it out of the dam without damaging the walls. The practice of removing seepage water from soil embankments by the use of artificial drains is standard practice in the design of earth dams for water storage purposes.

THE USE OF ARTIFICIAL DRAINAGE

One form of artificial drainage is suggested by the rather uncommon case where there is a fairly deep layer of reasonably permeable soil under the dam but, for some reason, the ground-water level is high and thus prevents adequate drainage of the dam. In this case, by digging a deep drainage trench round the outside of the dam, and by keeping the water level in this trench low, the phreatic surface in the foundation soil and consequently in the dam, will be lowered. The effect of a trench in permeable soil on the position of the phreatic surface is shown in Fig. 18.

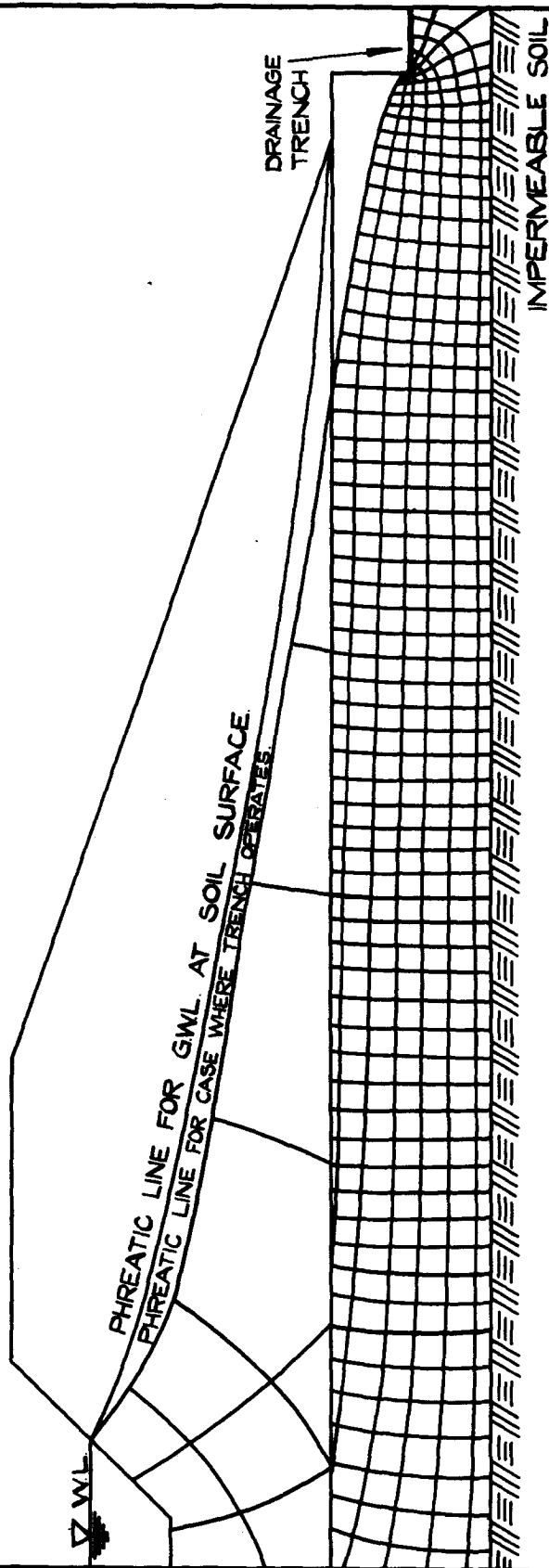


FIG. 18.

EFFECT OF DRAINAGE TRENCH ON THE POSITION OF THE
PHREATIC LINE.

This method has been used successfully with one of the slimes dams at Robinson Deep which is founded on jointed and fissured rock. The trench drains the channels in the rock which in turn drain the slimes material.

The other form of drainage, which is the one used in earth dam construction, involves the provision of artificial under-drains under the dam itself. These drains are usually incorporated in the original design of the dam at those places where seepage flow is expected.

The simplest under-drain is the toe drain. This is a bank of free draining material, usually crushed rock, which is placed to form the toe of the dam as illustrated in Fig. 19. The toe drain catches the seepage water before it emerges from the dam and thus prevents seepage erosion but it does not lower the phreatic surface much. The toe drain will only be effective if it is big enough in proportion to the cross-sectional shape of the dam where it is to be used.

It is not only necessary to prevent the seepage water eroding the toe but also to keep the zone of seepage as small as possible inside the dam. The reasons for this requirement will be made clear in the chapters dealing with the shear strength and overall stability of slimes dams.

Blanket drains consist of a layer of free-draining material placed under the dam from the toe inwards, as shown in Figures 20, 21, 22 and 23. In these figures it will be seen that the phreatic surface is lowered and that all the seepage flow enters the drain in a very short length of drain. Figures 20 and 21 show how the phreatic surface becomes lower as the drain is extended further into the dam. Comparison of Figures 21, 22 and 23 shows that with a drain, as is the case without a drain, an increase in the permeability of the foundation soil will result in a lowering of the phreatic surface.

The effect of using under-drains has been clearly shown; it now remains to consider the requirements necessary in the design of an efficient drainage system, namely:

1. the drain should be large enough and have sufficient gradient to pass all the seepage water without flowing full;
2. the drain should be protected against becoming blocked or choked.

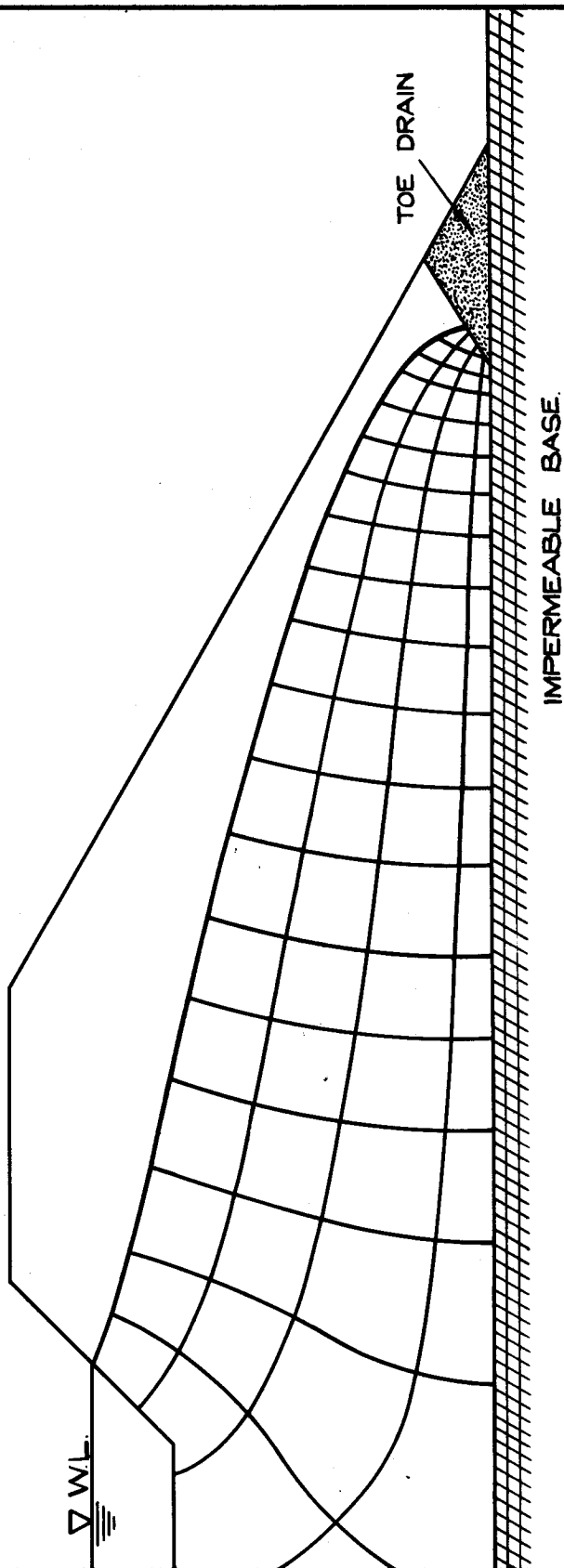


FIG. 19.
EFFECT OF TOE DRAIN ON THE POSITION OF THE PHREATIC LINE.

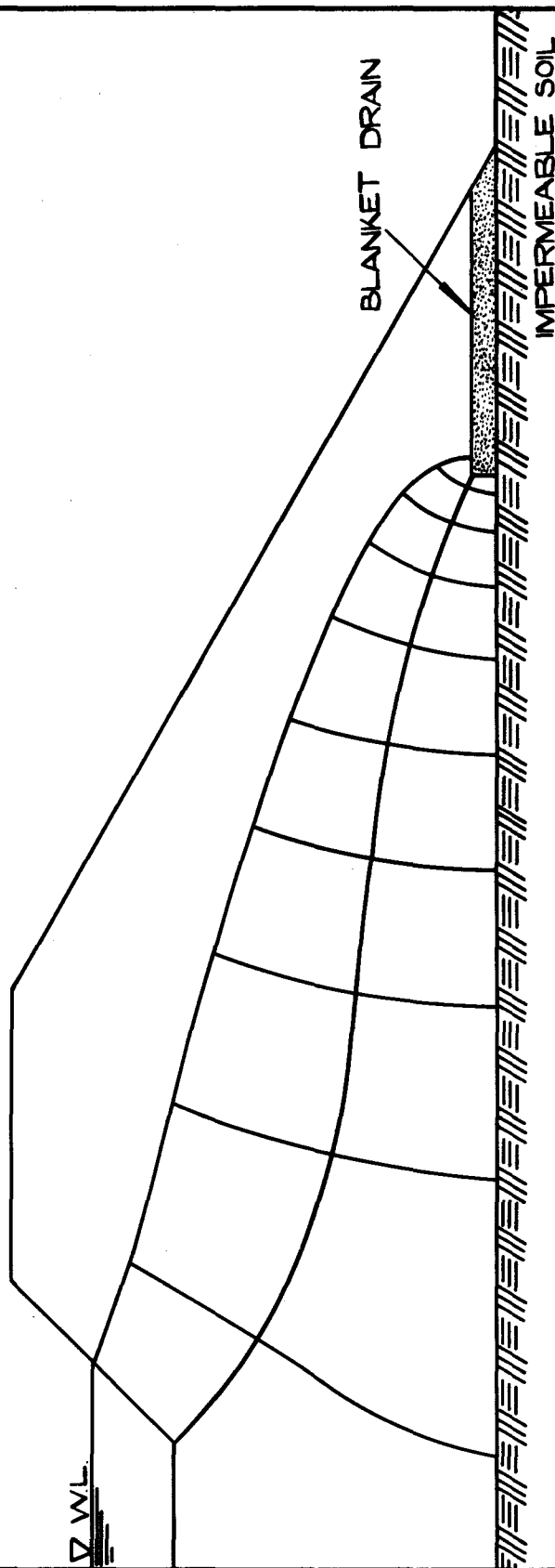


FIG. 20.

EFFECT OF SHORT BLANKET DRAIN ON THE POSITION OF THE
PHREATIC LINE.

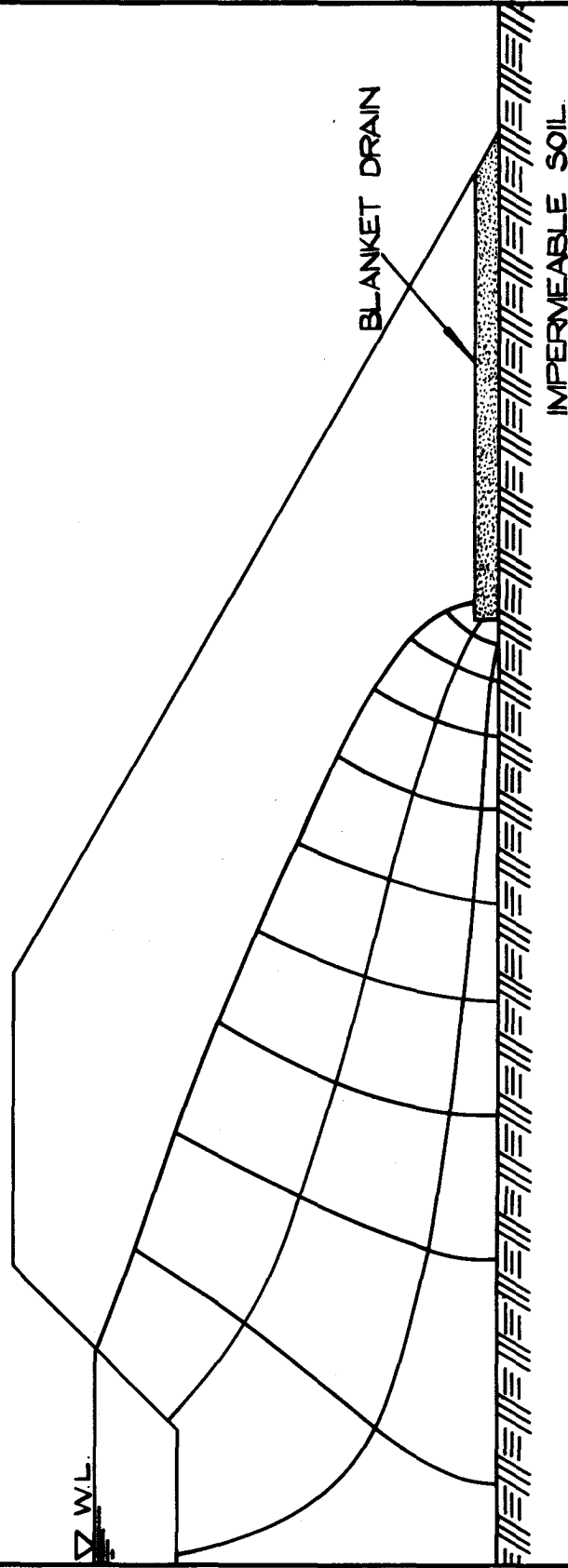


FIG. 21.

EFFECT OF A LONGER BLANKET DRAIN ON THE POSITION OF THE PHREATIC
SURFACE.

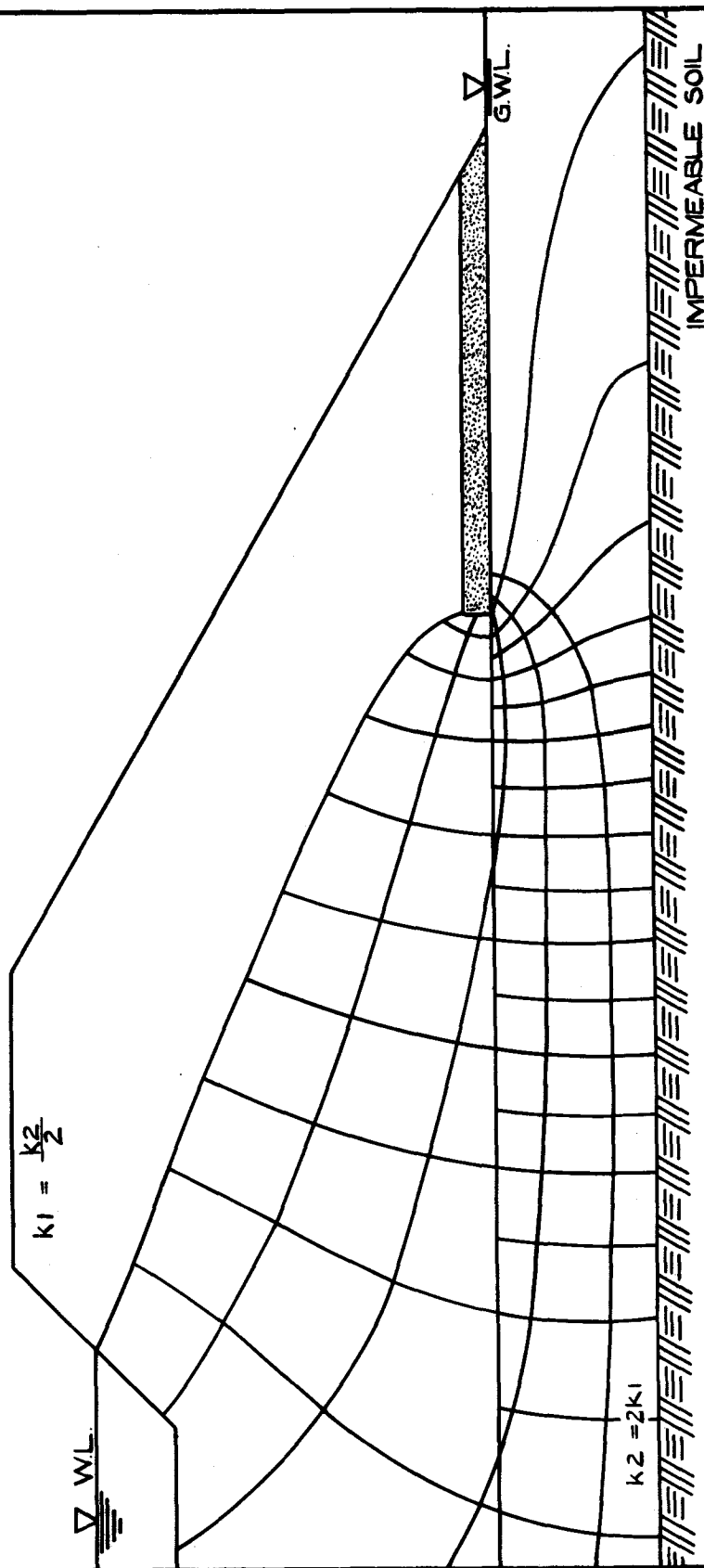


FIG. 22.

FLOW NET FOR A BLANKET DRAIN USED OVER A LAYER OF
PERMEABLE SOIL.

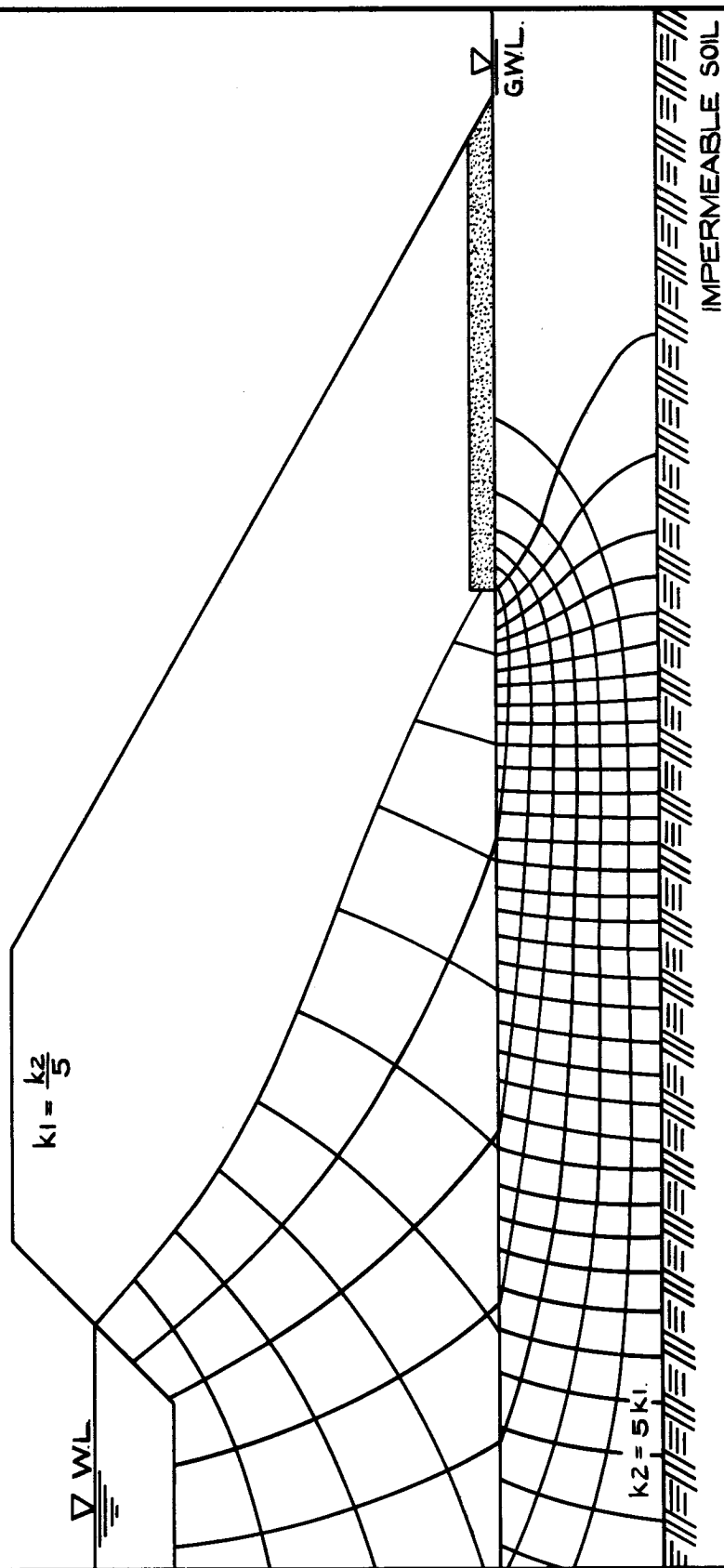


FIG. 23.

FLOW NET FOR BLANKET DRAIN OVER LAYER OF MORE PERMEABLE SOIL.

Considering (1) the amount of seepage water that will enter the drain can be calculated from the flow net. It is then necessary to ensure that the size of the drain, the size of rock used in the drain, and the hydraulic gradient will be such that the drain will not be waterlogged. It can be seen from Figures 20, 21, 22 and 23 that most of the seepage water enters the drain in a very short distance near its inner end, the rest of its length being used simply to carry the water out through the dam wall. This leads to another type of drain, of sufficient capacity to catch all the seepage water, and built inside the dam; the water is then channelled out of the dam in drainage trenches or pipes as shown in Figure 24. In this way, a relatively small area of drain can be used most efficiently.

The protection of the drains from clogging or choking is most important. If the drain is allowed to become clogged by slimes material, the effect of the drain will be nullified. It is possible to exclude the entry of fine-grained material into coarse-grained material by the design of properly graded filters, as outlined in the next section. It is also necessary to take precautions to avoid slimes material being washed into the drain at the toe by stormwater and run-off. A third probable cause of trouble, about which very little is known as yet, is clogging due to precipitation of insoluble substances, e.g. the oxidation of iron salts in the seepage water, leaving a deposit of iron oxides in the drain. It is suggested that some allowance should be made for this factor by making the drain larger than is absolutely necessary, so that if part becomes clogged, the remainder will still be adequate to handle all the seepage flow.

DESIGN OF FILTERS

In the design of drains it is advantageous to use large aggregates so that there will be large void spaces to facilitate the easy flow of the seepage water. At the same time, precautions must be taken to see that these voids do not become blocked by the ingress of fine material and this is done by placing one or more layers of filter material between the finest material and the coarse rock. The efficiency of the filter will not be determined by the number of layers used, but by the ratio of the particle size gradings of the various layers to one another.

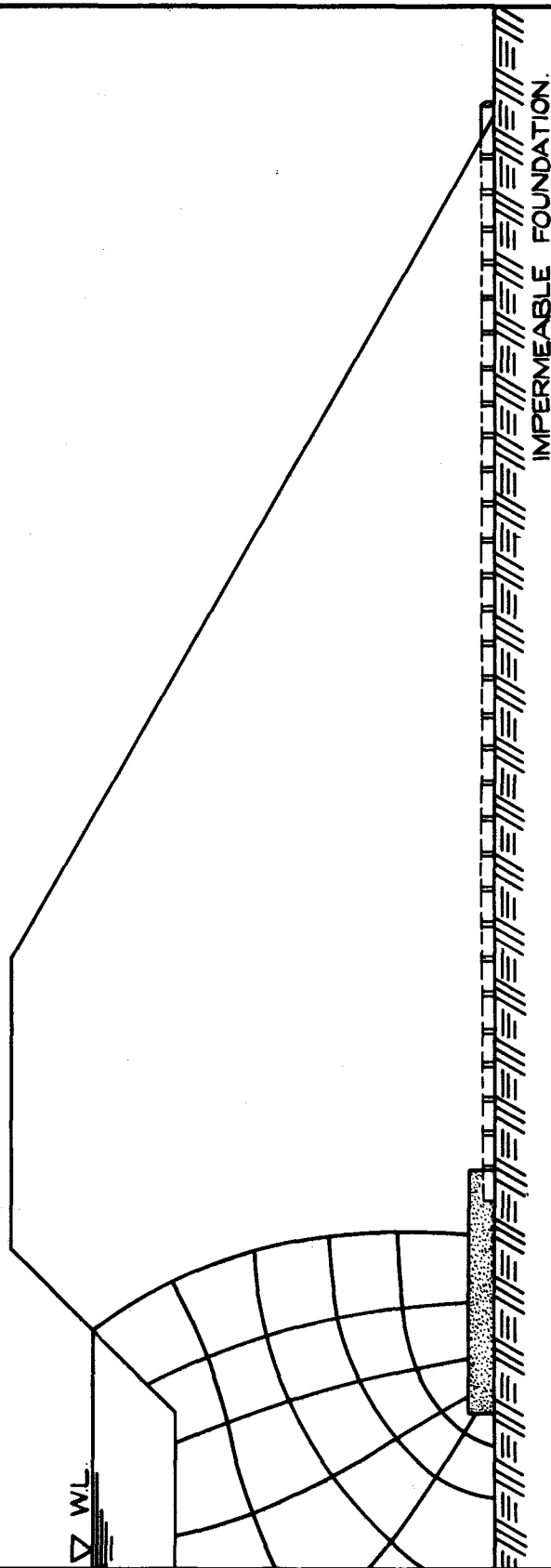


FIG. 24.

DRAIN LOCATED TO GIVE THE EFFECT OF UNLIMITED SUBSURFACE DRAINAGE.

Research has shown (9) that the following relationship defines a good filter material

$$\frac{D_{15} \text{ (of filter)}}{D_{85} \text{ (of finer material)}} < 5 < \frac{D_{15} \text{ (of filter)}}{D_{15} \text{ (of finer material)}} \quad (1)$$

where D_{15} = 15 per cent size and D_{85} = 85 per cent size. The first term implies that to prevent the finer material being washed into the pores of the filter layer, the 15 per cent size of the filter material must not exceed 5 times the 85 per cent size of the finer material. The last term covers the second requirement; to keep seepage forces in the filter to permissibly small magnitudes, the ratio of the 15 per cent sizes should be greater than 5. The 15 per cent and 85 per cent sizes of the materials are found by carrying out particle size analyses according to A.S.T.M., B.S.S. or other recognised soil mechanics laboratory standards and plotting the results as shown in Fig. 25. The D_{85} and D_{15} can then be read off.

Material A in Fig. 25 has $D_{85} = 0.014$ mm. and $D_{15} = .0048$ mm. Material B has $D_{85} = .240$ mm. and $D_{15} = .06$ mm. The ratio of the D_{15} of B to the D_{85} of A is therefore $\frac{.06}{.014} = 4.3$ and the D_{15} ratio of B to A is $\frac{.06}{.0048} = 12.5$. Material B would thus be suitable for use as a filter layer with material A.

It will be noted from Fig. 25 that both material A and material B are fairly uniformly graded, i.e. the range of sizes in the material is small. However, it is possible to obtain materials which have a wide range of particle sizes or which may be gap-graded i.e., consist of two or three separate particle sizes without the intervening particle sizes being present. These materials might not be filters within themselves and if they are used as filter layers the finer fraction of the material may be washed out into the next coarser layer of the filter, allowing fines to be washed into the layer, from the preceding layer, thus causing its breakdown and the breakdown of the whole filter. Research work at the University of the Witwatersrand (10) has established that for a material to be a filter within itself it should have a uniformity coefficient of D_{85}/D_{15} not greater than 5 or of D_{90}/D_{10} not greater than 7.

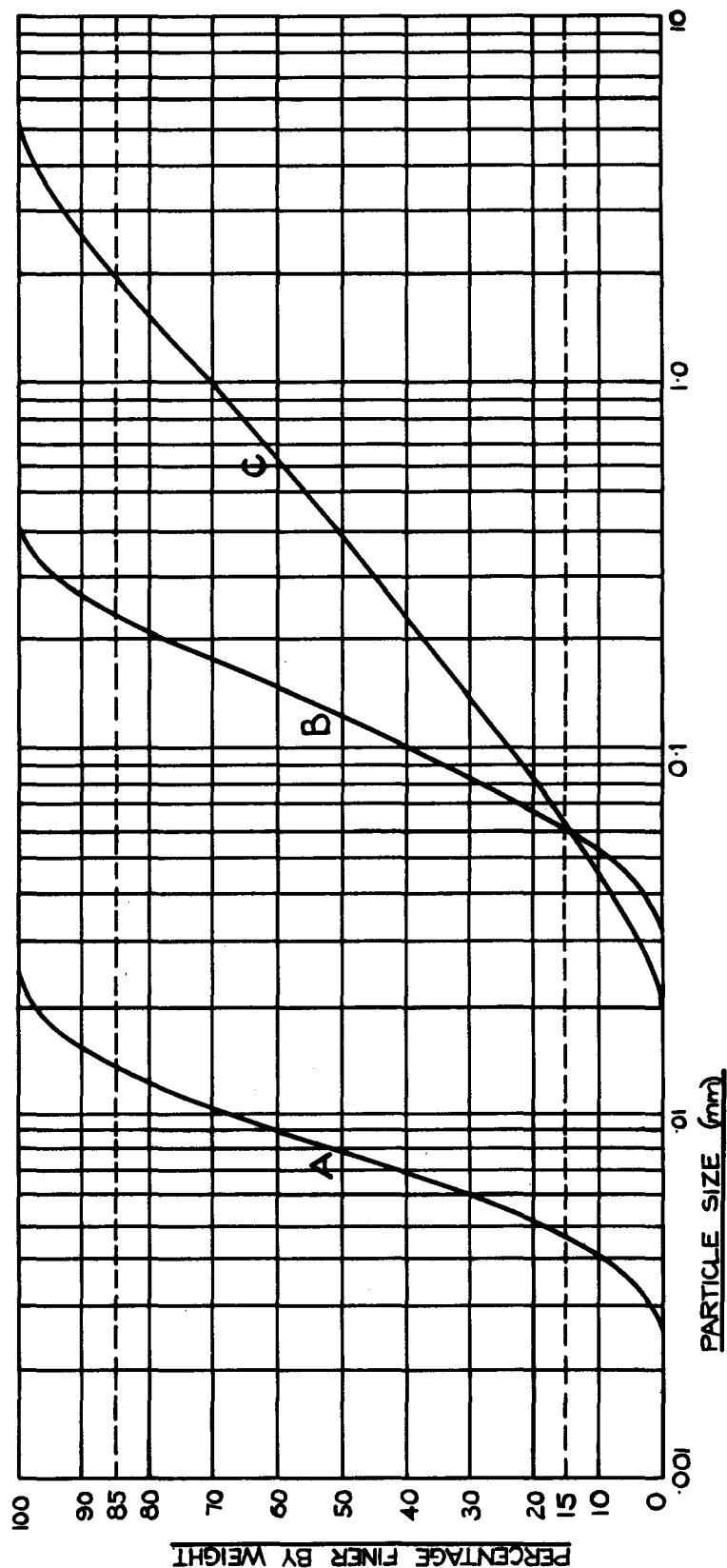


FIG. 25.

PARTICLE SIZE DISTRIBUTION CURVE FOR POSSIBLE FILTER MATERIALS.

In Fig. 25 the D_{85}/D_{15} ratio for material A is $\frac{0.014}{.0048} = 2.9$ and for material B $D_{85}/D_{15} = \frac{.240}{.06} = 4.0$ which shows that these materials are both filters within themselves. Material C in Fig. 25 has $D_{15} = 0.06$ and $D_{85} = 2.0$ and therefore a uniformity coefficient of $\frac{2.0}{0.06} = 32$, which shows that the material is not a filter within itself. Both material B and C comply with the condition set out in equation (1) but while material B complies with the condition set out in (2) material C falls well outside this limit; therefore, only material B could be used as a filter layer with material A.

Ideally, the various filter layers should be of materials which fall just inside these limits so that the smallest number of layers will be required. In practice, however, it is usually most economical to use readily-available local material even if this entails using one or two additional layers. A practical example of the design of filter layers is given in Appendix 1: Experimental Wall at St. Helena Slimes Dam.

The drain must be protected along the bottom and sides as well as along the top, when the foundation soil is permeable, since there is just as much danger of foundation soil being washed into the drain as there is of slimes material being carried in.

REVIEW OF INITIAL ASSUMPTIONS

In the beginning of this section the following assumptions were made:

- (a) the slimes material, as placed on the slimes dam, is homogeneous and isotropic, in particular has the same permeability in all directions;
- (b) the foundation soil is a uniform, homogeneous isotropic material;
- (c) there is a permanent pool of water on the floor of the dam maintained at a constant level;
- (d) there is no water on the dam outside the pond.

It is now necessary to investigate the validity of these assumptions and their effect on the conclusions drawn in the earlier part of this section.

- (a) The mineral composition of slimes is mostly quartz and the other minerals present depend on the ore that is being milled. The grading of the slimes is kept fairly uniform on any particular mine but does change from time to time. As the walls are built up of layers of slimes deposited during a long period it is easy to see that the material will not be identical in each layer. A typical size distribution curve for slimes in Fig. 26 shows the considerable range of particle sizes in ordinary slimes. According to the sedimentation principle expressed in Stoke's Law the coarser particles will settle out first and the finest particles last, resulting in horizontal stratification, due to size, within each layer of deposition. This stratification can be clearly seen in any specimen of slimes taken from the top of a dam.

The permeability of a material is related to the grain size of the material and, therefore, there will be a different permeability for every size stratum within each layer. As the stratification is horizontal, the permeability in a horizontal plane in each stratum will be uniform while the permeability in the vertical direction cutting across the strata will vary considerably. When flow in the vertical direction is impeded by the strata of fine material it will tend to follow the horizontal direction of the strata of coarser particles. This stratification cannot be numerically assessed as such but its effect can be gauged by treating the slimes as a uniform, homogeneous material with differing horizontal and vertical permeabilities.

The flow nets for a typical section, with horizontal to vertical permeability ratios of 2 and 10, are shown in Figures 27 and 28 respectively. The permeabilities of the foundation soil are assumed to be same as for the slimes. These figures can be compared with Fig. 7 which shows the flow net for the same cross-section with a permeability ratio of

$k_h/k_v = 1$. The phreatic surfaces for the three different k_h/k_v ratios are shown in Fig. 29 where it will be seen that the phreatic surface is pushed outwards as k_h/k_v increases.

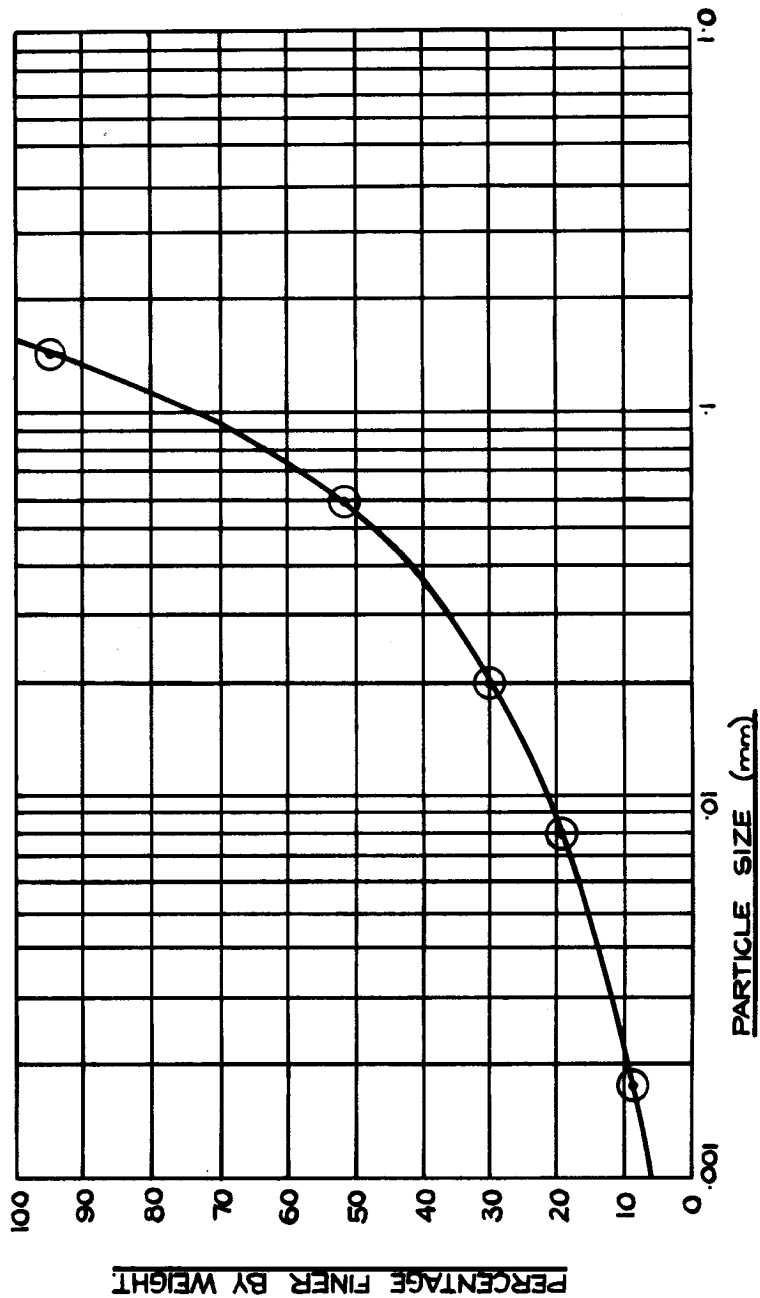


FIG. 26.

PARTICLE SIZE DISTRIBUTION CURVE FOR SLIMES.

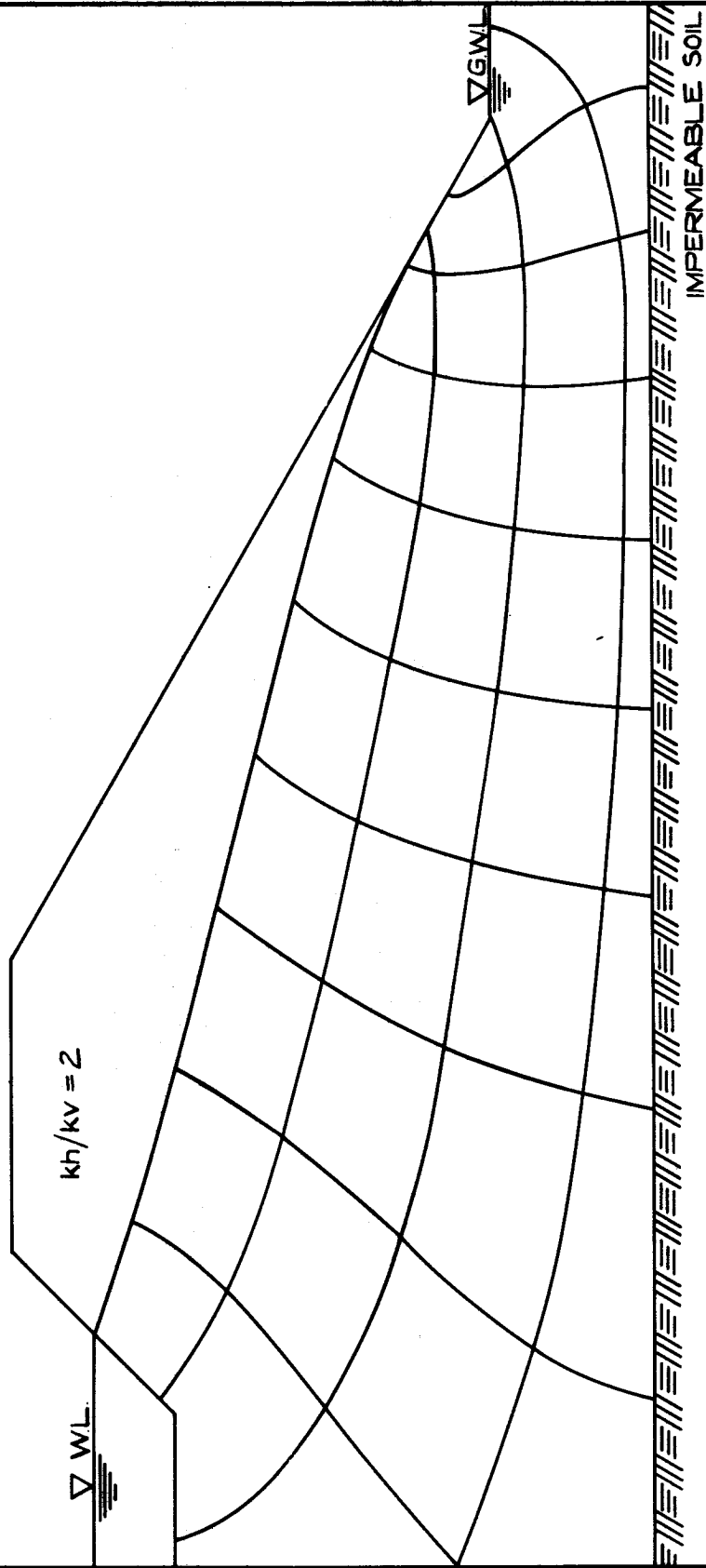


FIG. 27.

FLOW NET FOR CASE WHERE $kh/kv = 2$.

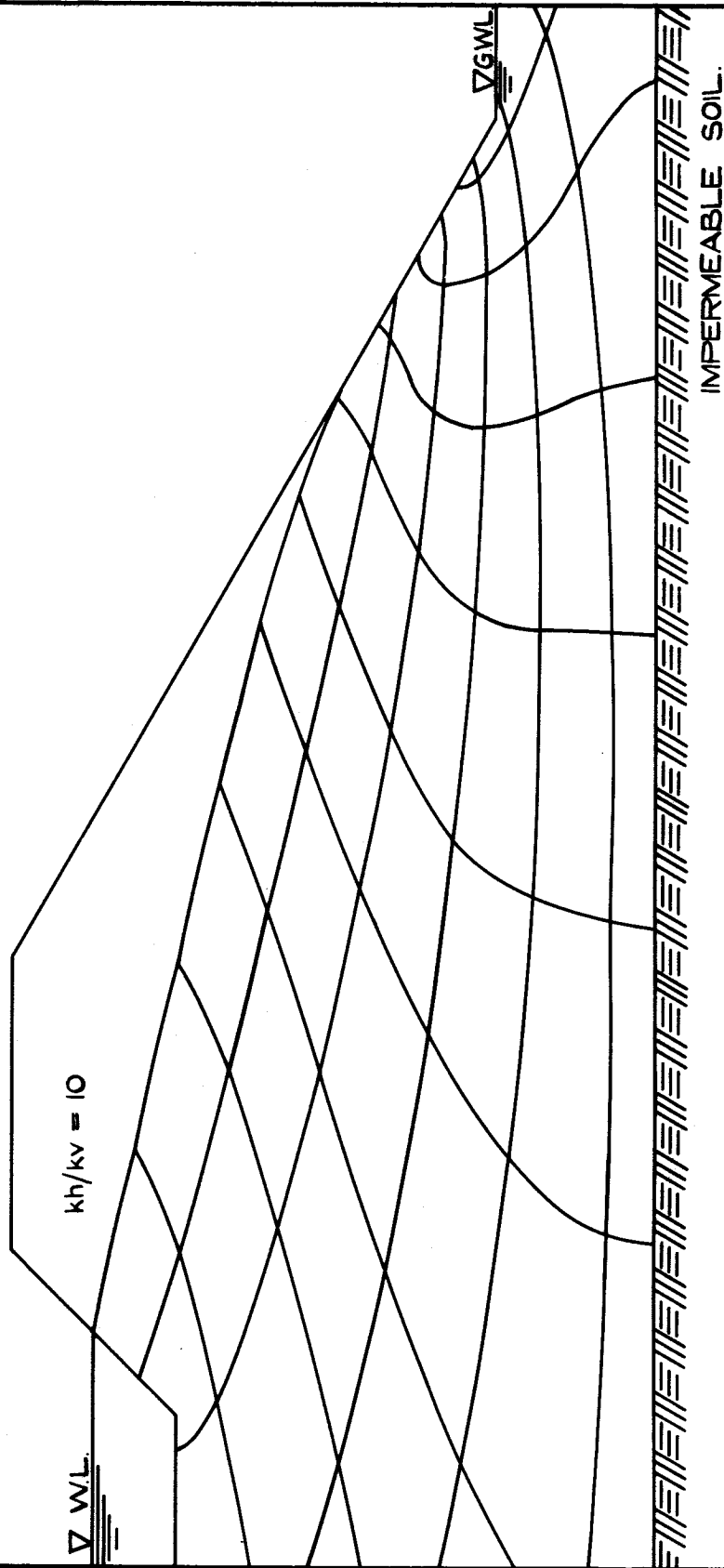


FIG. 28.
FLOW NET FOR CASE WHERE $kh/kv = 10$

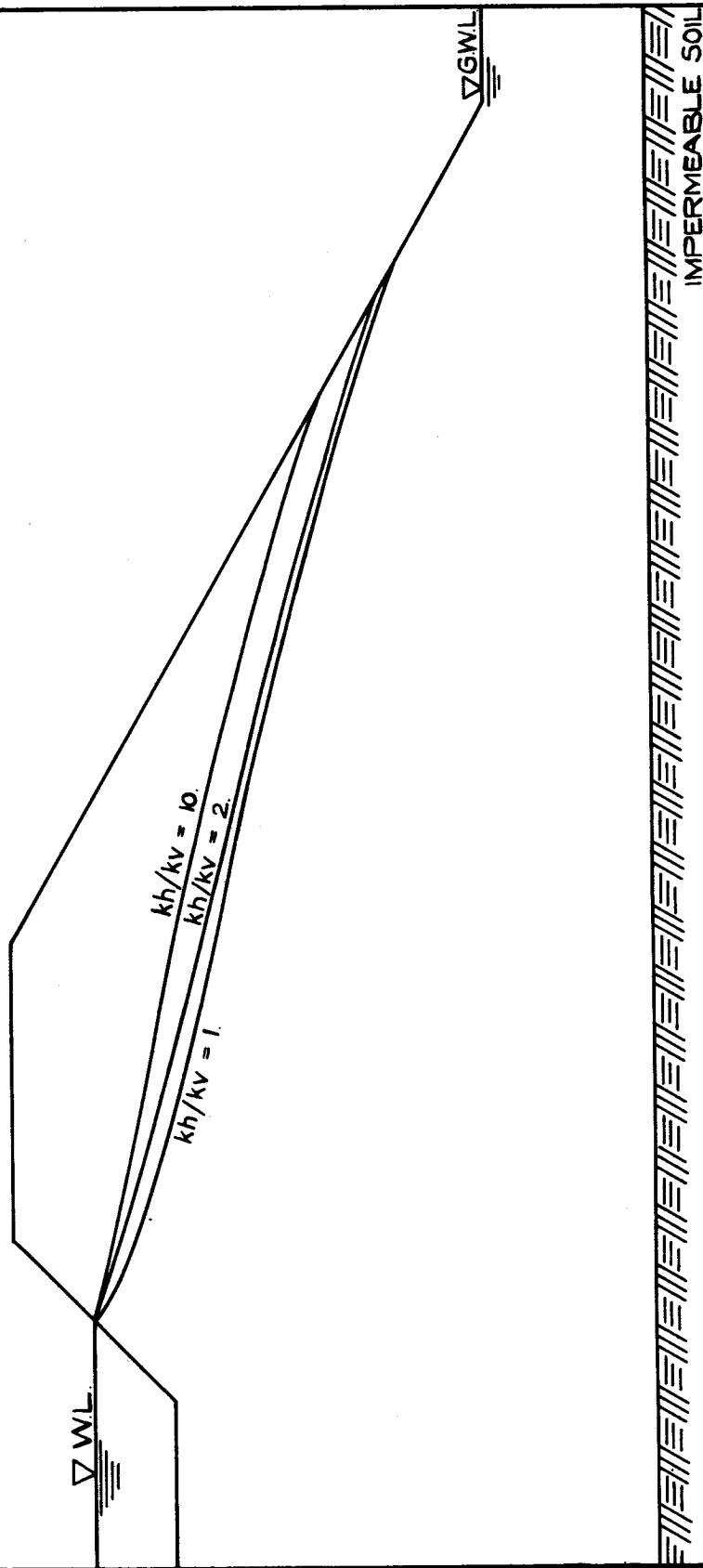


FIG. 29.

COMPARISON OF THE EFFECTS OF VARIOUS k_h/k_v VALUES ON THE POSITION
OF THE PHREATIC SURFACE

Although this horizontal distortion will occur in all cases where k_h/k_v is not unity, the basic effects of drainage layers, ratios of permeabilities, etc., on the position of the phreatic surface will be the same for the stratified material as for the homogeneous isotropic material originally considered.

- (b) Not only do foundation soils differ from one dam site to another, but they may also differ from one section of a particular dam to another section of the same dam. As in each case one particular section of a dam has been considered, it is reasonable to assume that the foundation soil in that particular section is uniform, although it may be anisotropic. In each case, the foundation soil conditions will have to be evaluated carefully before the seepage conditions in any section can be assessed. Nevertheless, this will not affect the basic principles already stated.
- (c) Around each penstock there is usually a pool of water on the floor of the dam and thus where a large dam is divided into several sections with separate penstocks there will be several pools of water. The assumption, however, is that there will be a pool of water for each specific section considered and this will hold for each case. The level of the pond will, however, be subject to fluctuations owing to the level of the penstock intake setting, the amount of slimes being put on to the particular section of the dam, and stormwater collection in the pond. A study of the flow nets shows that the inter-section of the water level in the pond and the inner face of the wall is the upper limit of the phreatic surface and, therefore, as the level of the pond rises and falls, the phreatic surface will be adjusted to follow. The higher the level of the pond, the higher will be the position of the phreatic surface.

In the case where, in dry weather, a section is not worked and the pond dries up, the main source of seepage water will be removed and the zone of seepage will shrink until all the seepage water in the wall has flowed out, whereupon seepage will stop. As soon as water is put in the pond again, seepage will start once more.
- (d) Wherever slimes is being deposited on the walls and during rainy weather there will be an accumulation of water outside the pond. As long as the water remains

lying there, it will seep into the wall, as shown in Fig. 30(a), and when the water on the surface of the outer wall has disappeared, the zone of moisture will move down towards the base of the dam and some of the moisture will be drawn up by capillary forces, as shown in Fig. 30(b). As the wetting of the outer wall is a periodic occurrence it is conceivable that there will be a number of these zones of moisture representing different wettings moving down through the wall. This moisture will eventually be picked up by the zone of seepage or be collected by the permeable sub-strata or a drainage layer. These zones of moisture will only affect the phreatic surface slightly when they enter the zone of seepage.

However, if the water remains almost constantly on the outer wall or if the wetting is very frequent with only short dry periods, the moisture zones may become linked to form a large zone of saturation which may even extend throughout the wall and steady seepage conditions may be set up as shown in Fig. 31. In this case, the water level on the outer wall determines the phreatic surface, with the result that the whole wall becomes a zone of seepage.

It is, therefore, obvious that as long as the outer wall is kept fairly dry the position of the phreatic surface will not be materially affected, but if water is allowed to lie on the outer wall for any length of time the whole picture will be changed.

This examination has shown that, while the initial assumptions were not absolutely valid, the differences between them and the actual conditions in slimes dams do not materially affect the conclusions drawn

PRACTICAL APPLICATIONS OF THE RESULTS OF RESEARCH ON SEEPAGE

1. Site Investigations

Foundation conditions play a major role in seepage problems in slimes dams. It is, therefore, essential that a survey be carried out on the site for any proposed slimes dam so that the zones of good and poor drainage conditions can be defined. In some cases it may be desirable to shift the dam to a completely different location, but it is recognised that this is seldom possible, because of the

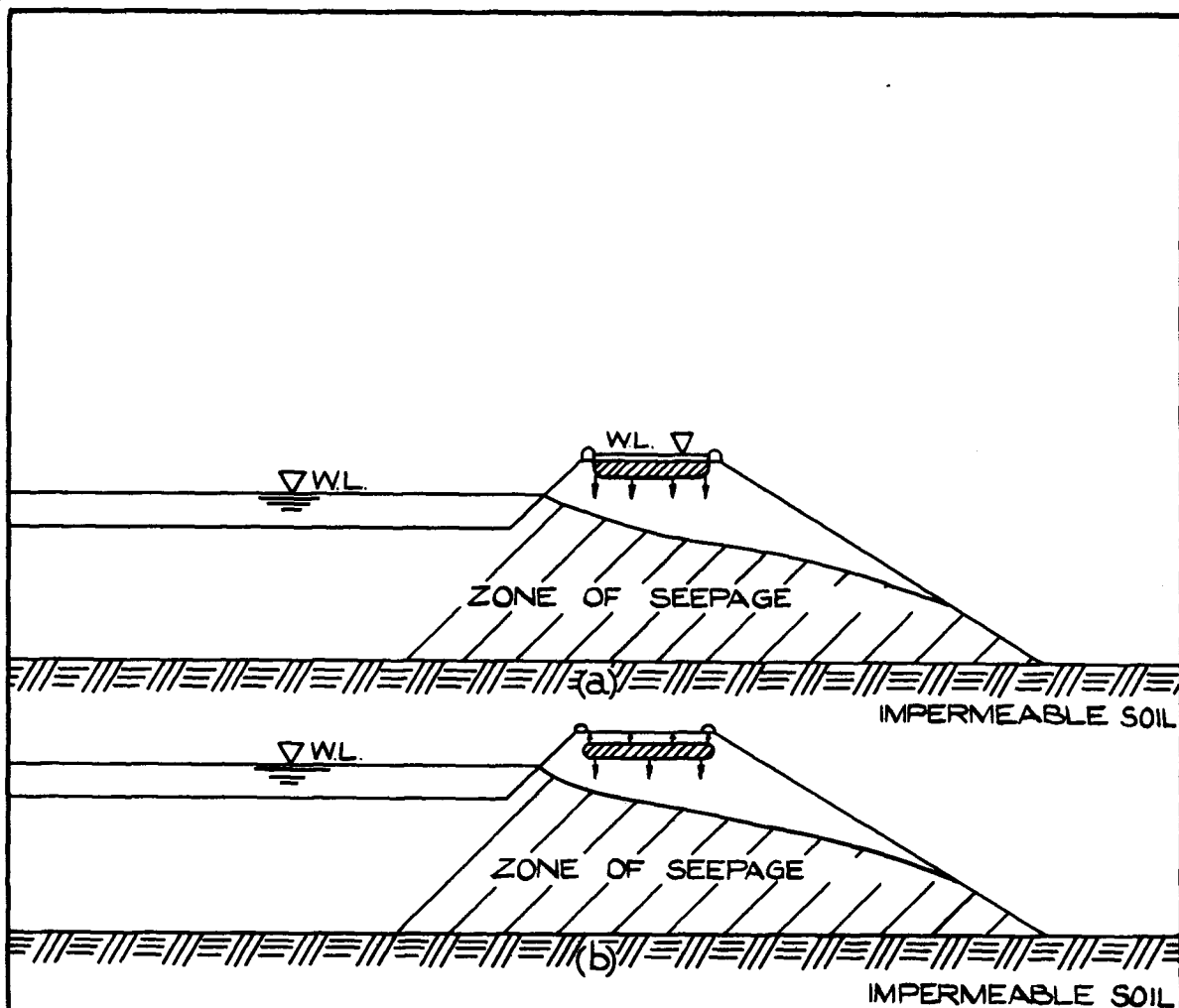


FIG. 30.

ZONE OF MOISTURE MOVING DOWN THROUGH
SOIL ABOVE ZONE OF SEEPAGE.

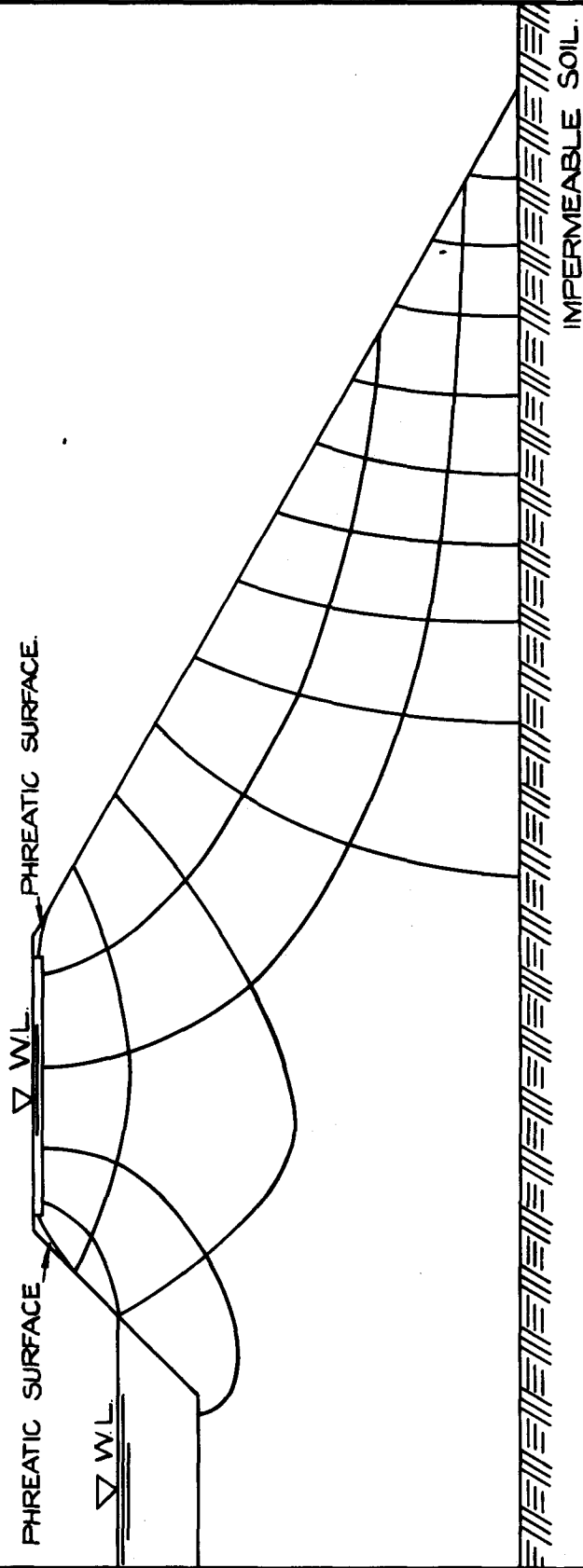


FIG. 31.

FLOW NET FOR STEADY STATE SEEPAGE CONDITIONS FOR WATER
ON OUTER WALL.

other factors governing the choice of the site for a slimes dam. Nevertheless, by using the results of the drainage survey intelligently, the likelihood of seepage problems can be limited, and a minimum amount will have to be expended on protective measures.

2. Layout of the Dam in relation to Site Survey Data

It has been shown that for seepage problems to occur the foundation drainage conditions must be inadequate and there must be a pool of water on the floor of the dam to supply the seepage water. Therefore, if the pool can be kept away from the areas of bad drainage and can be located over areas of good drainage, seepage problems can be reduced or avoided; the seepage water from the pond will then find its way into the well-drained foundation soil and be carried away before it has a chance to reach the badly-drained areas and create a problem there.

The obvious conclusion is that the penstocks with their ponds should be located on the best drainage areas. It is realised that in the initial stages, due to topography of the site, it may be necessary to use auxiliary penstocks located on unfavourable soil until the main penstocks can be brought into operation. However, with suitable location of the delivery pipes and controlled building of the walls, there is no reason why the pool of water on the floor of the dam cannot be maintained at any desired point.

In areas where the drainage conditions are bad, artificial drainage must be provided.

3. The Use of Drains in Slimes Dam Construction

Depending on the results of the site survey, a variety of combinations of the various types of drain can be used to control the seepage flow. The methods outlined here are a few which appear to be the most economical for dealing with particular situations.

- (a) Where the whole dam is founded on unsuitable material a blanket drain can be located under the wall adjacent to the pond to lower the phreatic surface. At the points far removed from the pond some seepage may occur but the phreatic surface will be low and it

will only be necessary to protect the toe with rock toe drains. The position of the two types of drain in relation to the pond is shown on the plan in Fig. 32. Section A.- A shows the blanket drain adjacent to the pond with the collection trench and outlet for taking the water away. Section B - B shows the toe drain in use where the phreatic surface is low; the further section B - B is removed from the pond the lower the phreatic surface will become.

- (b) This is a similar type to (a) but the blanket drain is replaced by longitudinal drainage trenches inter-linked with cross-drains. As will be seen in section A - A in Fig. 33, the phreatic surface will be drawn towards the first drain which will collect all the seepage water while the second drain will be mainly a standby in case the first becomes blocked, and will also help to collect water moving down through the dam wall.

In both these cases care must be exercised to see that the cross-drains and drain outlets are placed so that no water is trapped in the drain by an adverse gradient.

- (c) As an extension of method (b) the drainage trenches can be taken into the dam to encircle the area of the pond, as shown on the plan of Fig. 34. This will result in the whole zone seepage being confined to the area surrounded by the drains as shown in section A - A₁ with the result that no drainage will be necessary on other parts of the dam unless, of course, there is another pond on the dam.

- (d) Herringbone drains are used to drain large areas of ground and have been recommended (3) for slimes dams because of this reason, and such an installation is shown in Fig. 35. However, it is felt that there are several drawbacks in using herringbone drains as compared with blanket or longitudinal drains and cross-drains. The main drawback is that there is only one outlet drain through which all the water collected in the system must pass. This means that the location of the drain is governed to a large degree by the topography of the site as every point on the system must be made to drain through the single outlet drain and also a blockage in the main drain will put the whole system out of operation. Secondly, if the side-drains are spaced far apart seepage flow

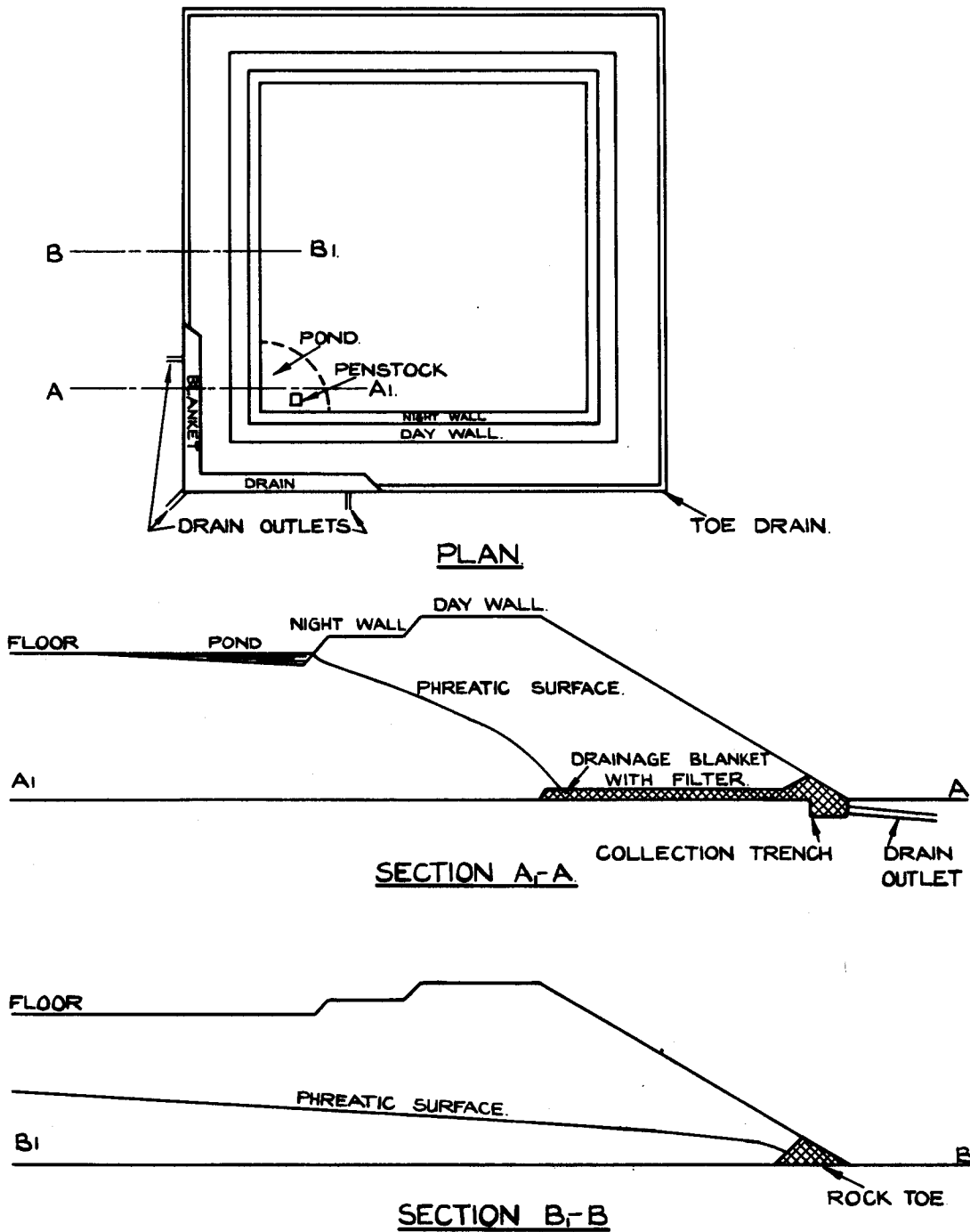
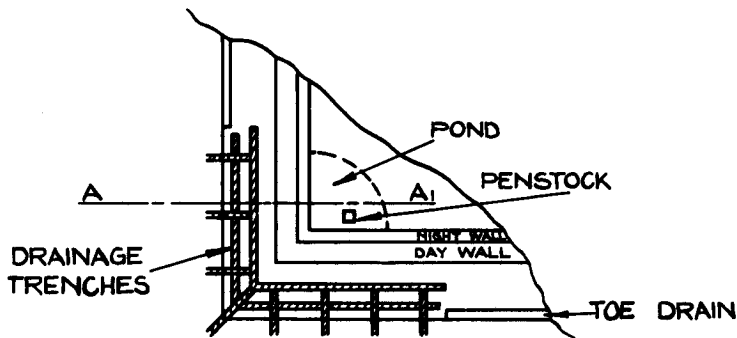
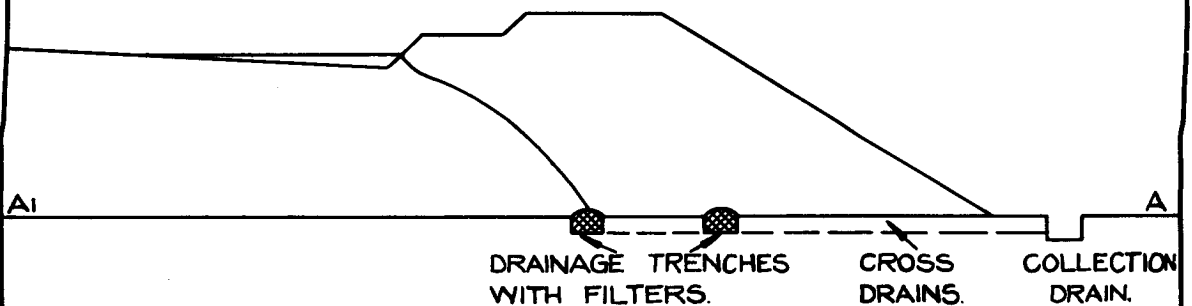


FIG. 32.

CONTROL OF SEEPAGE FLOW BY THE USE OF BLANKET DRAINS.



PART PLAN



SECTION A1-A

FIG. 33.

CONTROL OF SEEPAGE FLOW BY THE USE OF
LONGITUDINAL DRAINAGE TRENCHES.

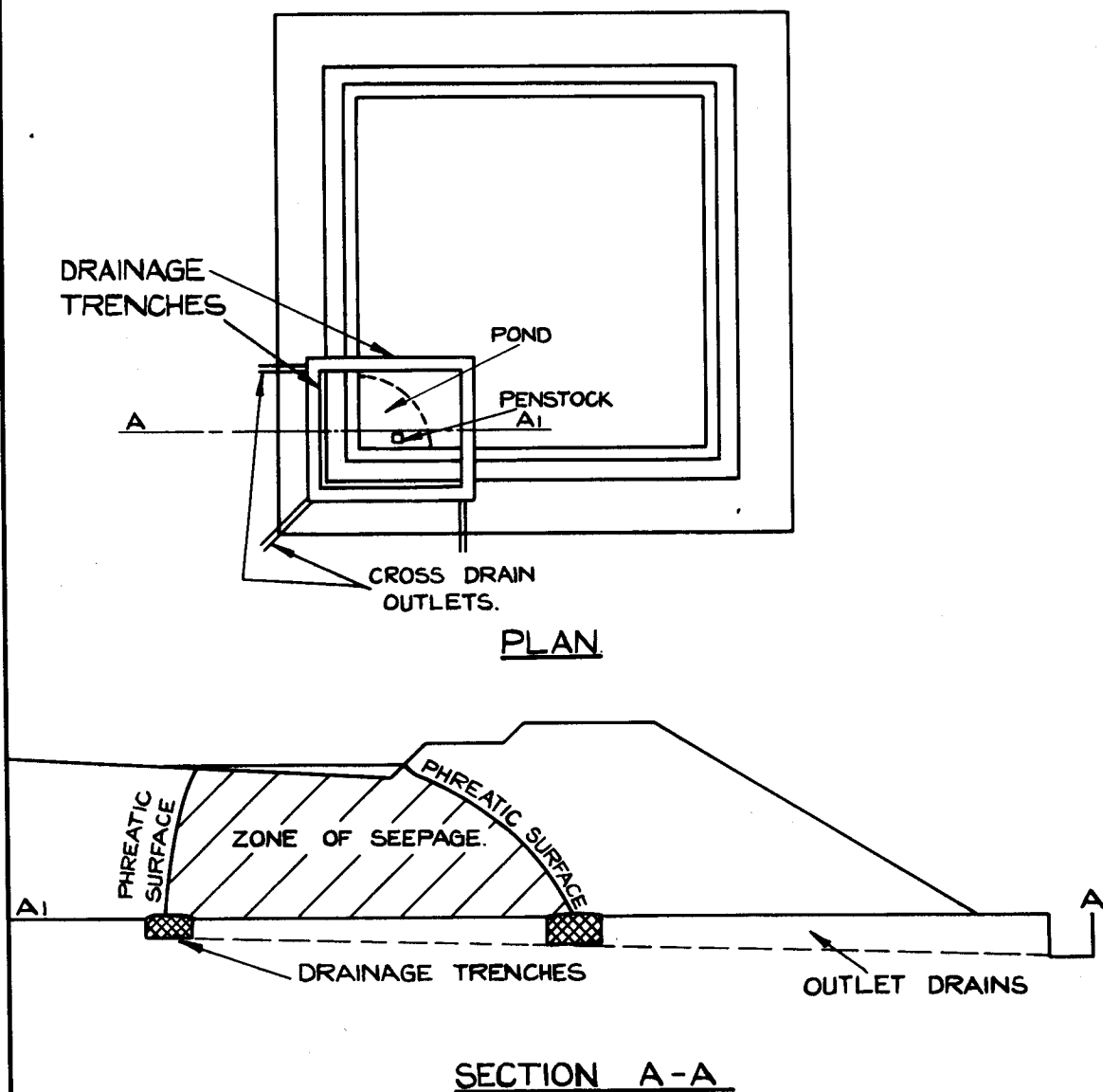


FIG. 34.

RESTRICTING THE AREA OF SEEPAGE FLOW BY THE USE
OF DRAINAGE TRENCHES.

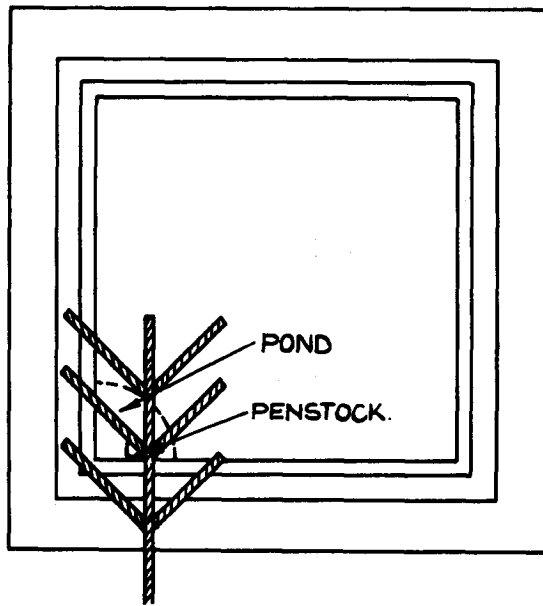


FIG. 35.

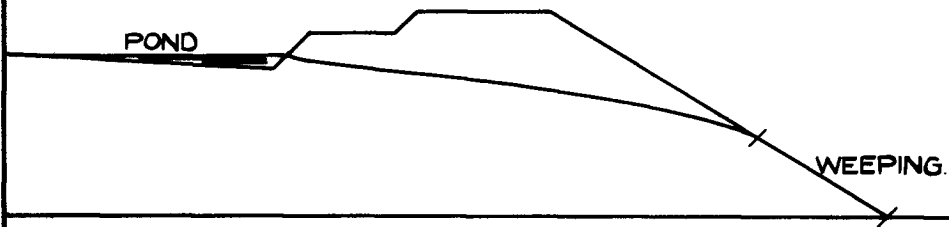
HERRINGBONE DRAINAGE SYSTEM FOR CONTROLLING
SEEPAGE FLOW.

may be set up in the zones between the areas of influence of the side drains. Finally, those portions of the drain immediately below the pond will draw a large amount of water due to the short flow path, and as a result, larger drains will be required than for those skirting the outside of the pond. Bearing these drawbacks in mind there is no reason why an efficient herringbone system cannot be designed.

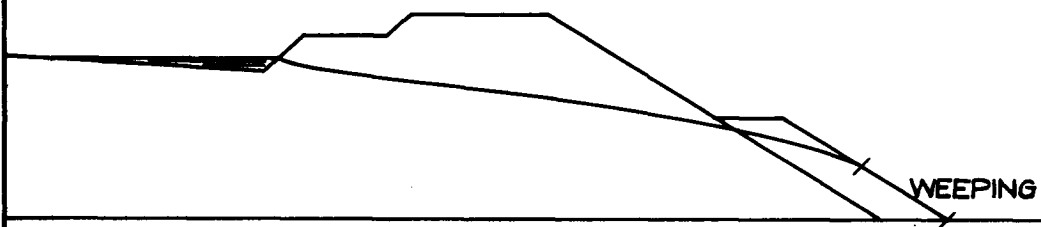
- (e) Many existing slimes dams are beset by seepage problems and there is a good deal of visual evidence of unsuccessful attempts to cure this trouble. The usual method of repair is to build a buttress wall against the affected area on the outer wall of the dam. Fig. 36(a) shows a section through the original wall with the phreatic surface, and the area where the seepage water is emerging. A slimes buttress built against this wall will in no way alter the drainage conditions of the foundation under the wall and will act either, as shown in Fig. 36(b), as an extension of the dam, with the phreatic surface passing into the buttress and the weeping occurring at the toe of the buttress, or, as in Fig. 36(c), where the buttress is not very high, weeping will occur at the point where the buttress joins the main wall and also at the toe of the buttress. In either case the problem has not been solved but the area of the trouble has simply been moved slightly to a new location.

In some cases rock buttresses have been used but very often these are either filled with slimes during construction or become filled with slimes washed in by the seepage water. Once the rock buttress is filled with slimes it behaves in exactly the same way as the slimes buttresses mentioned above.

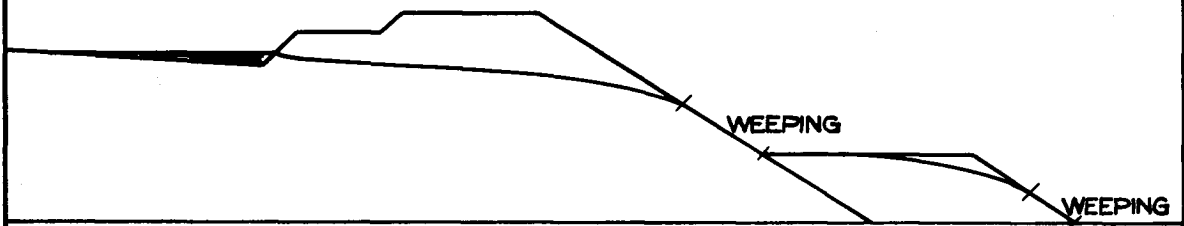
The ideal solution would be to under-drain the wall itself but it will be readily appreciated that this would be extremely expensive and, from the practical point of view, almost impossible. A suitable solution is, therefore, to use a buttress wall of slimes with blanket or longitudinal drains. The buttress should be raised to a height where the danger of the conditions outlined in Fig. 36(b), developing, is avoided. This buttress can even be carried to the full height of the existing wall and from there on be used as the outer wall in the normal way, as shown in Fig. 37(a).



(a)



(b)



(c)

FIG. 36.

SEEPAGE PROBLEMS IN EXISTING DAMS.

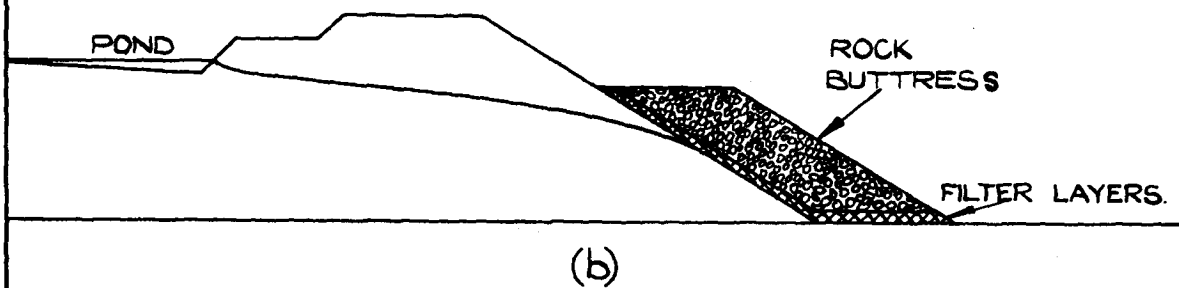
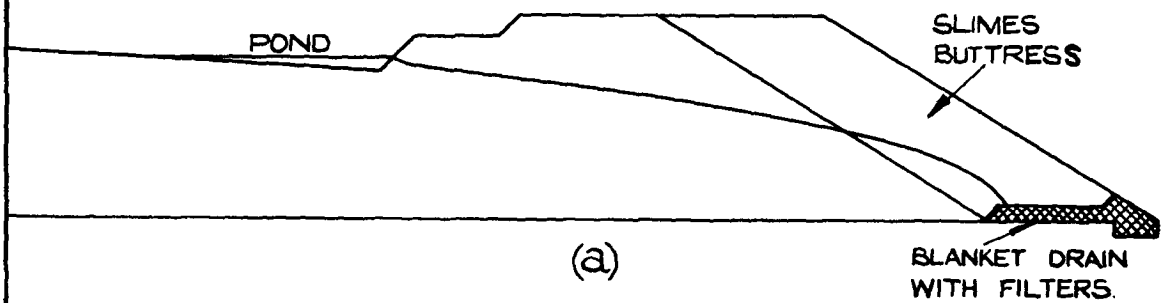


FIG. 37.

RECOMMENDED METHODS OF PROTECTING THE TOE OF
EXISTING DAMS SUBJECT TO SEEPAGE PROBLEMS.

If it is decided to use a rock buttress then a filter must be placed between the rock and the slimes to prevent the slimes being washed into the rock; on no account should slimes be used to fill the rock voids. A rock buttress with filter is shown in Fig. 37(b).

So far, practical experience with a blanket drain has been gained at St. Helena (see Appendix 1) and ~~longitudinal~~ drains (rock-filled trenches) have been used effectively at Luipaardsvlei and Durban Roodepoort Deep Mines although it was impossible to obtain any information about the design of these systems. Herringbone drains have been installed on some slimes dams in recent years and longitudinal drains have been put into the new slimes dam at Winkelhaak, but in these cases the drains have not yet been tested under full operating conditions although they appear to be quite successful to date.

The photographs in Fig. 38 show views of the drainage system being installed at Winkelhaak.

CONTROL OF SEEPAGE WATER OUTSIDE THE DAM

Once the seepage water has been brought out of the dam it should be collected and treated in the same way as water from the penstocks.. As this water may contain objectionable chemicals it is not considered advisable to release it directly into watercourses without prior testing and treatment where necessary.

CONTROL OF WATER LEVEL IN THE POND

The level of the water held in the pond is controlled by the height of the penstock intake which can be raised or lowered as desired. Usually the penstock intake is set at a level which will maintain a pond of sufficient size to allow all the suspended material in the water to settle out before the water enters the penstock. The water leaving the penstock is collected in a sump for testing and treatment before being recirculated or put into natural watercourses. During periods of rainfall the penstock may pass too great a flow of water to be dealt with by the treatment plant; to avoid this, the penstock intake is raised to hold the excess stormwater in the pond



Longitudinal drains



Graded material in drainage trench.

Fig. 38.
Drainage system at Winkelhaak Mines
slimes dam.

until the treatment plant is able to deal with it.

It has been shown that the extent and level of the pond will govern the extent of the zone of seepage and thus the magnitude and cost of precautionary measures that will be required. Therefore, the level of the water in the pond should be kept as low as possible, in fact just high enough to ensure that the water reaching the penstock is clear of suspended solids. Some mines have obtained permission from the Department of Water Affairs to install a counterweighted bypass valve in the outlet from their penstocks. During peak periods of rainfall the weight of water passing through the penstock opens the bypass valve and passes the water directly into the watercourses without going through the treatment plant. This has been allowed as it is felt that the large amount of stormwater will reduce the concentration of chemicals in the penstock discharge to a harmless level.

This system is recommended as it means that the water level in the pond will be kept at a permanently low level and it also reduces the danger of overtopping due to inadequate provision of freeboard on the outer walls.

CHAPTER IV. O V E R A L L S T A B I L I T Y:
C O N S O L I D A T I O N O F M A T E R I A L
I N A S L I M E S D A M

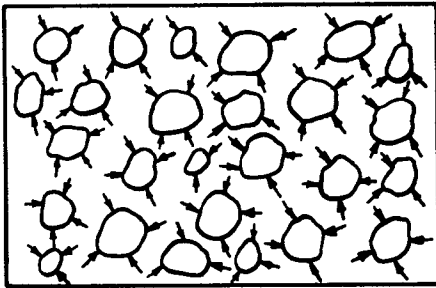
CONSOLIDATION THEORY FOR A SLURRY

The consolidation of a soil is the process whereby it is inelastically compressed under its own weight or under an applied load, and is accompanied by the driving out of water and an increase in intergranular pressure. In slimes dams the process can conveniently be considered in two stages, the consolidation of the material to the fully consolidated state under its own weight, and further consolidation due to applied loads.

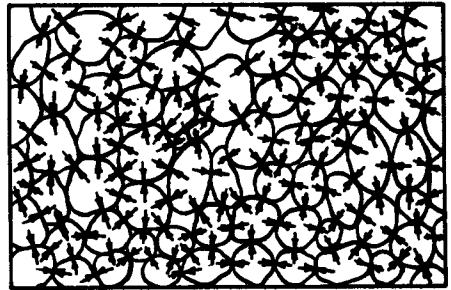
If a slurry consisting of a suspension of slimes particles in water is placed on a slimes dam, each particle will be surrounded by the water and there will be no intergranular pressure between the particles. The density of the suspension will be the density of water (γ_w) plus the submerged density of the solids (γ_s). At any depth h below the surface of the suspension the pressure in the suspension will be $h (\gamma_w + \gamma_s)$ and the intergranular pressure will be zero. This state is illustrated in Fig. 39(a). Consolidation will occur as the water between the particles is squeezed out; the particles will make contact with each other and the intergranular pressure will increase until the full submerged weight of the solids is carried as intergranular pressure and the fluid pressure is the hydrostatic head of water. At any depth h in the fully consolidated material the intergranular pressure will be $h \gamma_s$ and the fluid pressure will be $h \gamma_w$. This state is shown in Fig. 39(b).

This process is illustrated diagrammatically in Fig. 40. The pressure distribution in a deposition of slurry of depth h is represented by the triangle ABC where $BC = h (\gamma_w + \gamma_s)$. In the fully consolidated soil the intergranular pressure distribution is represented by the shaded triangle ABD and the hydrostatic pressure distribution by the triangle ADC.

A knowledge of the actual process of consolidation, i.e. the transfer of pressure from the pore-fluid to the intergranular structure, is of vital importance as it enables the pressure distribution for both intergranular and



(a) FLUID STATE.



(b) SOLID STATE.

FIG. 39.

ARRANGEMENT OF SLIMES PARTICLES DURING DEPOSITION AND CONSOLIDATION.

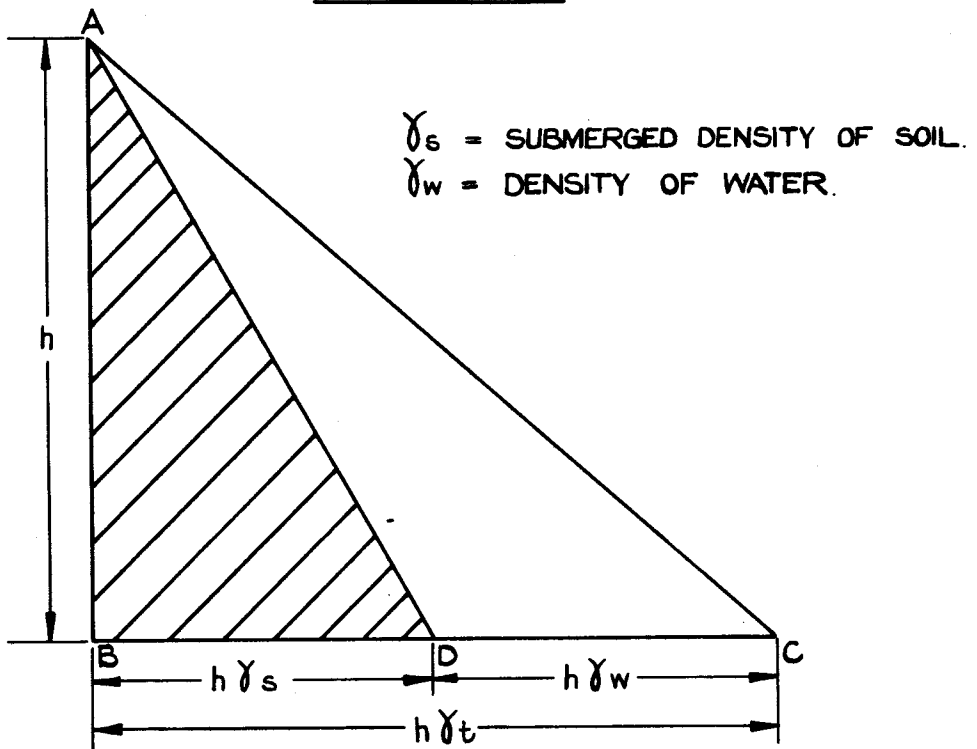


FIG. 40.

PRESSURE DISTRIBUTION IN A SUSPENSION OF SLIMES.

fluid pressure to be determined at any stage of consolidation in a consolidating mass. In 1936 Terzaghi and Frölich (11) published mathematical theories for the consolidation process for 3 cases, namely:

- (a) An instantaneously deposited sediment on an impermeable base;
- (b) An instantaneously deposited sediment on a permeable base;
- (c) A sediment increasing in depth at a uniform rate of deposition on an impermeable base.

These theories were checked and extended to cover a further case:

- (d) A sediment increasing in depth at a uniform rate of deposition on a permeable base;

and were theoretically applied to slimes dams by Kantey (12).

Attempts were made to verify these theories by tests in a settlement tank in the laboratory as cases (c) and (d) are the consolidation processes occurring in slimes dams. These experiments were unsatisfactory because of the type of apparatus available and the techniques in use at that time. Coarser materials, such as fine sand, settled and consolidated in a matter of seconds and it was therefore impossible to record the consolidation process using this material. The use of finer materials such as slimes or clays was also unsuccessful owing firstly to the difficulty of obtaining and keeping a large volume of this material as a uniform suspension and secondly to the fact that the suspension entered the piezometer tubes and gave distorted pressure head readings.

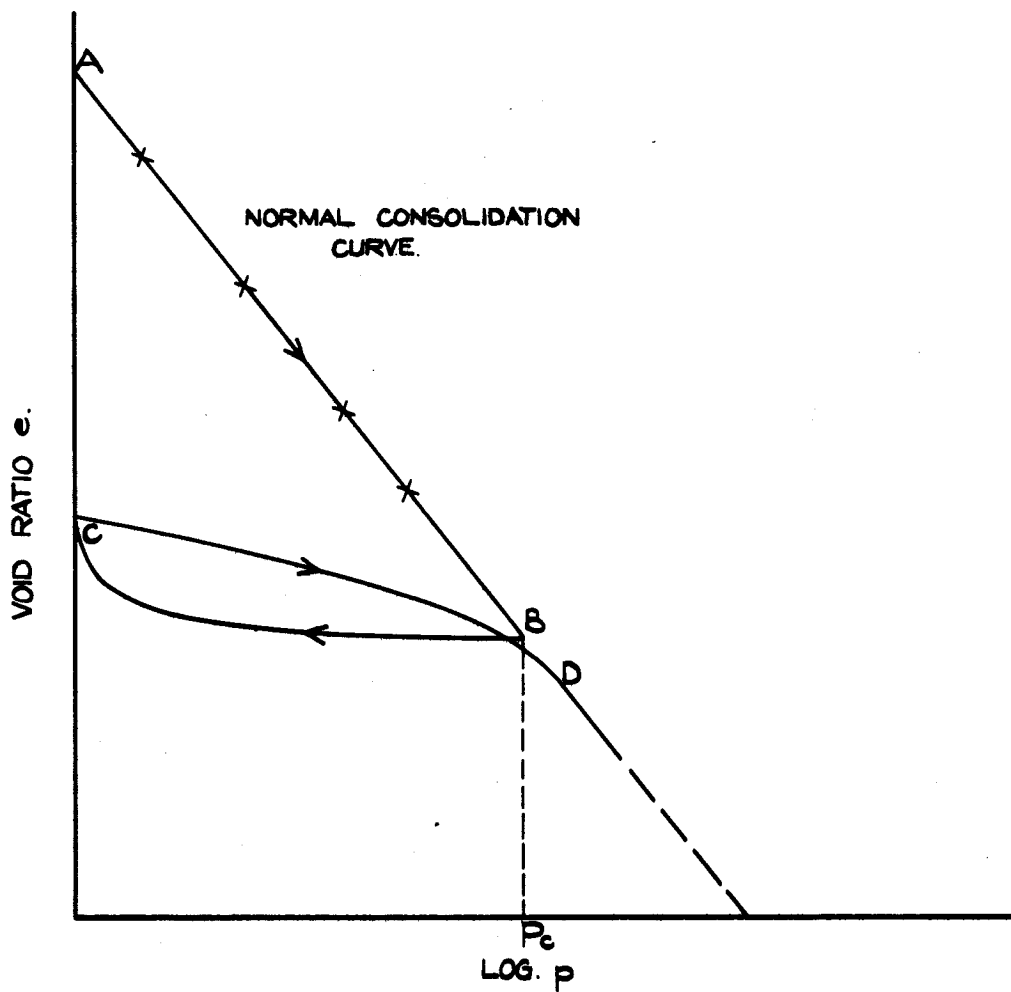
CONSOLIDATION OF A SOIL UNDER AN APPLIED LOAD

If an additional load is applied to a soil, fully or partially consolidated under a previous load, the additional load will be carried entirely by the pore fluid in the voids between the particles and there will be no increase in intergranular pressure until the pore fluid

flows out to a zone of lower pressure and the excess pressure, i.e. the difference between the pressure in the pore fluid and the normal hydrostatic pressure at that point is transferred to the grain structure as intergranular pressure. When the pressure in the pore fluid is normal hydrostatic pressure, and all the weight of the soil itself and any applied loads are carried on the grain structure, the soil is considered to be fully consolidated under those particular loads.

As has been already mentioned, the transfer of load from the pore fluid to the grain structure can only occur when some of the pore water flows out to an area of lower pressure and the grains are brought into closer contact with each other. Thus, as the intergranular pressure in any soil sample is increased, the volume of voids in the sample will decrease. This relationship is expressed by plotting a curve of the void ratio, e (the ratio of the volume of voids in the sample to the volume of solids in the sample) against the logarithm of the intergranular pressure. When this curve is plotted for a soil consolidated from a slurry under increasing loads, the points generally fall on a straight line, as indicated by the line AB and its extension, which is known as the natural or virgin consolidation curve in Fig. 41. If the soil is fully consolidated to point B and the load is then released, the sample will regain the elastic compression that has occurred and will rebound along curve BC to C. If the sample is then reloaded the recompression curve will fall along CD where the initial portion of the curve will be fairly flat and will then curve sharply at a point near the maximum load to which it had previously been consolidated (in this case P_c), to rejoin the virgin consolidation curve at D. The maximum pressure to which the soil had previously been consolidated is known as the preconsolidation load and is denoted by P_c .

The preconsolidation pressure for any soil sample can be determined in the laboratory by conducting standard consolidation tests on the sample and applying a graphical construction to the consolidation curve. If the preconsolidation pressure is equal to the overburden pressure the soil is termed normally consolidated and if the preconsolidated pressure is greater than the overburden pressure the soil is overconsolidated.



VOID RATIO VS. LOG. P CURVE FOR NORMALLY CONSOLIDATED AND OVERCONSOLIDATED SOILS.

FIG. 41.

CONSOLIDATION RESULTS FOR MATERIAL FROM A SLIMES DAM

Several boreholes were put down at East Geduld Slimes dam (see Appendix 2: Field Sampling and Testing of Slimes at East Geduld Slimes Dam) and undisturbed samples from various depths were subjected to consolidation tests. A typical plot for one of these tests is shown in Fig. 42. The full line is the consolidation curve and the dotted lines are the construction, developed by Casagrande, and described by Taylor (13), for the determination of the preconsolidation pressure, P_c , which in this case was 68 lb./sq.in. Tables I, II and III give the results of the consolidation tests of the samples from Boreholes 1, 2, 3 and 3a at East Geduld Slimes Dam.

These preconsolidation pressures plotted against depth are shown in Fig. 43, where the normal consolidation curves calculated using submerged density, dry density and total density of the slimes, have also been drawn. It will immediately be seen that all the preconsolidation pressures fall well above the normal consolidation curve for submerged density. The condition of full consolidation under submerged density would be valid if the material had been put down under water and the water level had been maintained at the surface of material. However, as it was known that the water level was below the surface of the slimes it was expected that the preconsolidation pressures would be greater than those due to submerged density.

The normal consolidation curve for dry density represents the curve along which the preconsolidation pressures would fall if the material were consolidated only under its own weight. As the water table drops, some water is held in the pores of the material above it and the weight of this water contributes to the consolidating force. This additional weight is taken into account by using the total density of the material for points above the water table. Although the majority of test results fall above this curve some results do fall between the submerged density curve and the total density curve. These results are due to one of two factors:

- either (a) the sample was taken from below the water table or phreatic surface, and the consolidation pressure at that point was due to the total density of the material above the phreatic surface plus the submerged density of the material below the water table;
- or (b) the sample had been disturbed during sampling or handling.

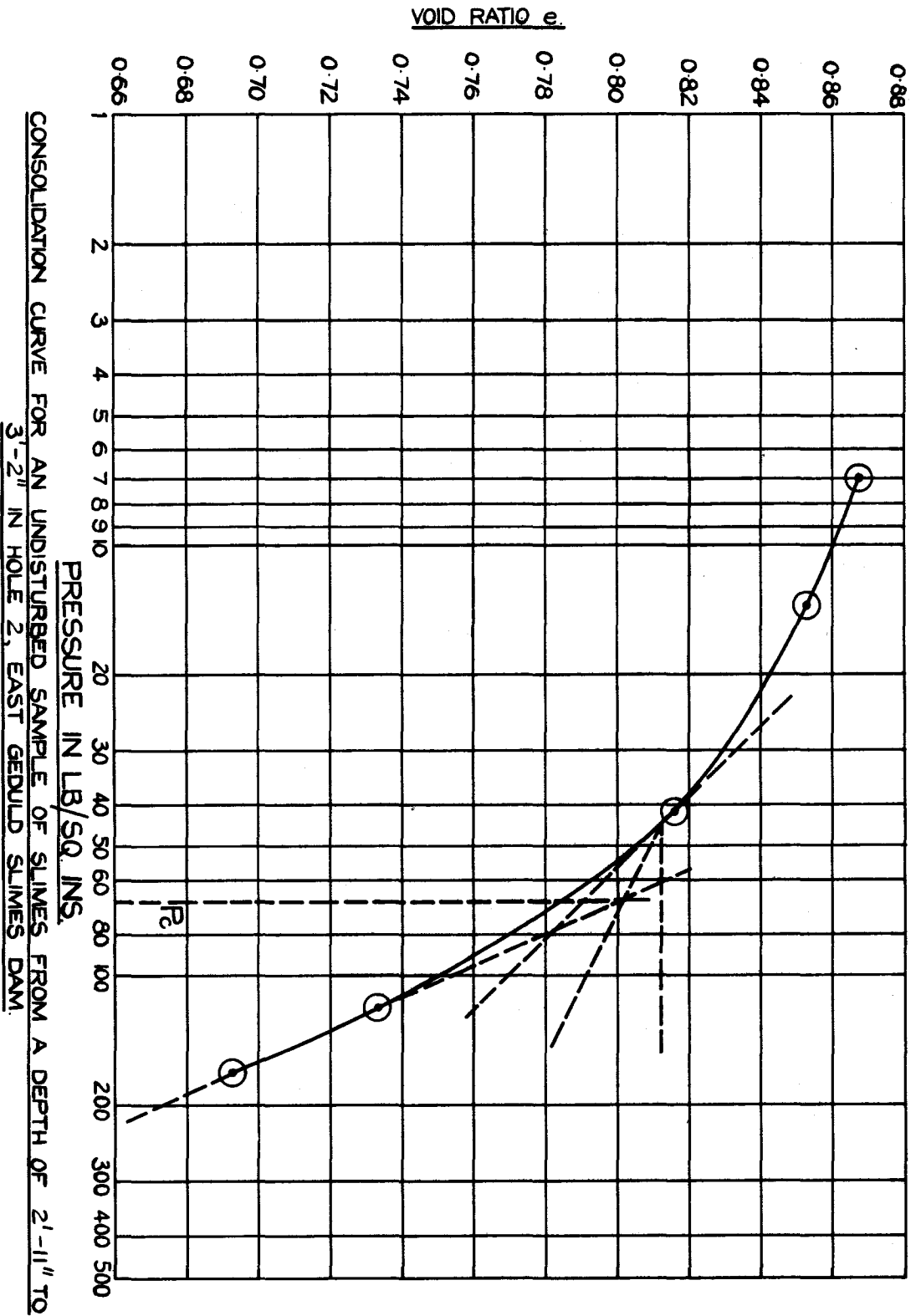


FIG. 42.

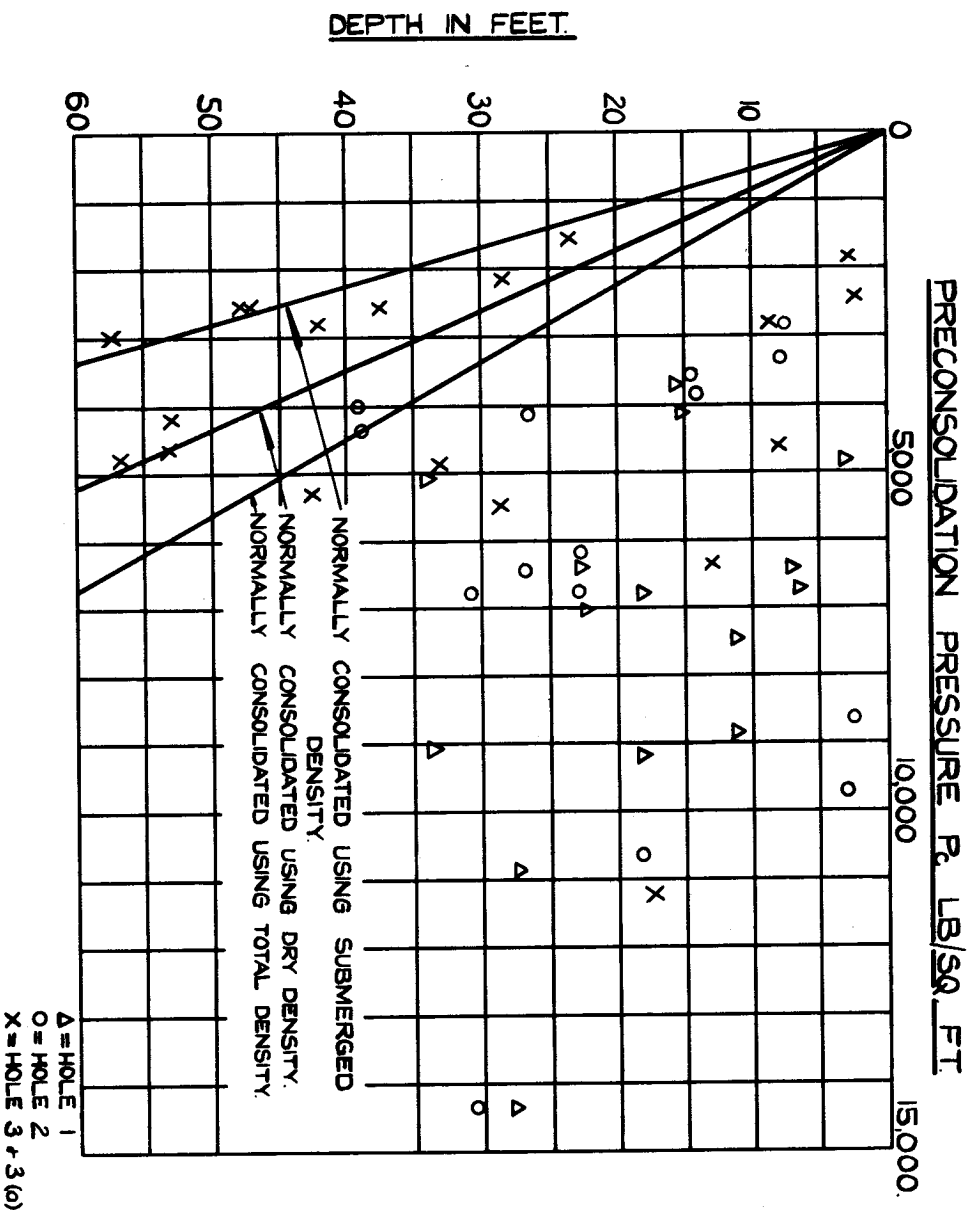


FIG. 43.

PRECONSOLIDATION PRESSURES VS DEPTH PLOT FOR UNDISTURBED
SAMPLES FROM EAST GEDULD SLIMES DAM.

TABLE I.

Results of consolidation tests on undisturbed samples
from borehole no. 1, East Geduld Slimes Dam

Sample Depth		Natural moisture content %	Natural void ratio e	Preconsolidation pressure P _c	
from	to			lb./sq.in.	lb./sq.ft.
3'-3"	3'-6"	11.3	0.94	34	4,895
6'-8"	6'-10"	35.0	0.95	47	6,755
6'-10"	7'-0"	33.7	0.91	45	6,480
11'-0"	11'-2"	38.7	0.92	62	8,930
11'-2"	11'-4"	38.9	0.97	52	7,490
15'-1"	15'-3"	36.1	0.83	29	4,165
15'-3"	15'-5"	30.0	0.70	26	3,740
18'-1"	18'-3"	11.8	0.84	48	6,805
18'-3"	18'-5"	30.2	0.95	64	9,210
22'-6"	22'-9"	13.4	0.98	49	7,050
22'-9"	23'-0"	36.8	1.05	45	6,480
27'-3"	27'-6"	20.2	0.99	76	10,945
27'-8"	28'-0"	17.5	0.86	100	14,400
34'-0"	34'-3"	23.9	0.68	63	9,060
34'-3"	34'-6"	28.1	0.70	35	5,045

TABLE II.

Results of consolidation tests on undisturbed samples
from borehole no. 2, East Geduld Slimes Dam

Sample Depth		Natural moisture content %	Natural void ratio e	Preconsolidation Pressure P _c	
from	to			lb./sq.in.	lb./sq.ft.
2'-6"	2'-11"	21.0	0.94	60	8,645
2'-11"	3'-2"	14.9	0.90	68	9,795
7'-4"	7'-7"	27.1	0.95	20	2,880
7'-7"	7'-10"	32.3	0.84	23	3,310
14'-1"	14'-6"	28.9	0.77	27	3,885
14'-6"	15'-0"	26.8	1.07	25	3,600
18'-0"	18'-3"	-	1.26	74	10,660
22'-4"	22'-7"	22.8	0.77	43	6,195
22'-7"	22'-10"	29.6	0.66	47	6,755
26'-6"	26'-9"	36.4	0.73	29	4,165
26'-9"	27'-0"	26.8	0.78	45	6,480
30'-6"	30'-9"	35.1	1.03	100	14,400
30'-9"	31'-0"	37.2	1.07	48	6,805
38'-9"	39'-3"	29.3	0.74	30	4,320
39'-3"	39'-7"	38.3	0.94	28	4,032

TABLE III.

Results of consolidation tests on undisturbed samples from
boreholes nos. 3 and 3a, East Geduld Slimes Dam

Sample Depth		Natural moisture content %	Natural void ratio e	Preconsolidation Pressure P _c	
from	to			lb./sq.in.	lb./sq.ft.
Hole 3					
2'-3"	2'-8"	11.5	0.84	17	2,446
2'-8"	3'-1"	8.7	0.79	13	1,872
8'-2"	8'-5"	29.5	0.91	32	4,608
8'-5"	8'-7"	29.5	0.80	19	2,836
13'-3"	13'-7"	23.6	0.81	44	6,336
13'-7"	13'-11"	29.0	1.00	44	6,336
17'-6"	17'-10"	38.6	1.19	78	11,232
23'-0"	23'-3"	16.7	0.88	19	2,836
23'-3"	23'-6"	16.0	0.91	11	1,584
28'-6"	28'-9"	38.4	0.98	15	2,160
28'-9"	28'-11"	40.8	1.13	38	5,462
33'-0"	33'-5"	51.0	1.27	19	2,836
33'-5"	33'-10"	26.7	0.64	34	4,895
37'-9"	38'-0"	15.6	0.95	18	2,592
38'-0"	38'-3"	20.1	0.95	24	3,456
42'-3"	42'-7"	21.1	1.09	19	2,836
42'-7"	43'-0"	24.4	1.29	37	5,328
47'-6"	47'-9"	30.3	0.79	18	2,592
47'-9"	48'-0"	24.0	0.81	18	2,592
Hole 3a					
52'-10"	53'-0"	15.8	0.61	29	4,166
53'-0"	53'-3"	20.1	0.39	32	4,608
56'-11"	57'-2"	16.5	0.35	34	4,896
57'-2"	57'-6"	25.4	0.61	21	3,024

Unfortunately it was impossible to determine the level of the phreatic surface during sampling operations but it was later found to be at a depth of about 40 feet and it is considered that these low values are probably due to a combination of both factors.

However, the most important fact is that the majority of samples appear to have been over-consolidated to a high degree. Over-consolidation in a soil is due to an additional load, once applied and since removed. In the case of slimes dams it is known that no additional external load has been applied and the additional consolidating pressure must therefore be generated within the slimes material itself. The obvious conclusion is that this pressure is due to forces in the pore water of the slimes set up during drying out of the material

CONSOLIDATION BY DESICCATION

Terzaghi (14) has stated that a soil can be consolidated by the forces holding columns of capillary water above the water table. The increment of intergranular pressure due to a column of capillary water of height h will be

$$\Delta p' = h \gamma_w \dots\dots\dots (3)$$

where γ_w = density of water.

If h_c is the maximum capillary height at which all the capillaries in the soil remain full, then, for this case, the increment of intergranular pressure due to the capillary forces will be

$$\Delta p' = h_c \gamma_w \dots\dots\dots (4)$$

When the capillaries are no longer full, i.e. the soil is non-saturated, drops of water will collect at the contact points between particles and Terzaghi further mentions (15) that these solid particles will be drawn together by a force equivalent to the tension in the water. Aitchison (16) has dealt thoroughly with the conditions of non-saturation and their effects on pore water and intergranular pressure.

Consider the consolidation of a point in a mass of slimes in terms of effective pressures. (17) The intergranular pressure p' on any horizontal plane in a

saturated soil mass is that portion of the total pressure p on that plane which is carried by the grain structure, while the remainder of the total pressure will be carried by the pore water as pore water pressure u , i.e.

$$p' = p - u \quad \dots\dots\dots (5)$$

The total pressure p at the bottom of a suspension of slimes of total density γ_t and depth h , and the pore water pressure at that point will both be equal to $h \gamma_t$. From (5) the intergranular pressure will be zero.

As the particles settle out, some of the total load is transferred to the grain structure until the pore water is left under normal hydrostatic head. At this stage $p = h \gamma_t$ as there has been no change in total load and $u = h \gamma_w$ where γ_w is the density of water. Substituting in (5)

$$\begin{aligned} p' &= h \gamma_t - h \gamma_w = h(\gamma_t - \gamma_w) \\ &= h \gamma_s. \end{aligned}$$

Where γ_s = submerged density of slimes.

As the intergranular pressure p' is the consolidating pressure acting on the slimes the process can be illustrated by the Void Ratio vs Log Intergranular Pressure in Fig. 44. As the consolidation takes place the point considered will move down the curve to point A, where it will remain as long as the water table is maintained at the surface of the deposit and the total pressure is not increased.

During the second stage of consolidation the water table starts to fall and eventually reaches depth h , at which stage $u = 0$ and

$$p' = h \gamma_t.$$

This stage is represented by the progress down the consolidation curve from A to B. The third stage is reached when the water table drops below the point considered, putting the pore water at the point considered into tension. This tension is usually designated the negative pore water pressure, $\bar{u}$, and in the same way as in the positive pore pressure case, the intergranular pressure is given by

$$p' = p - \bar{u} \quad \dots\dots\dots (6)$$

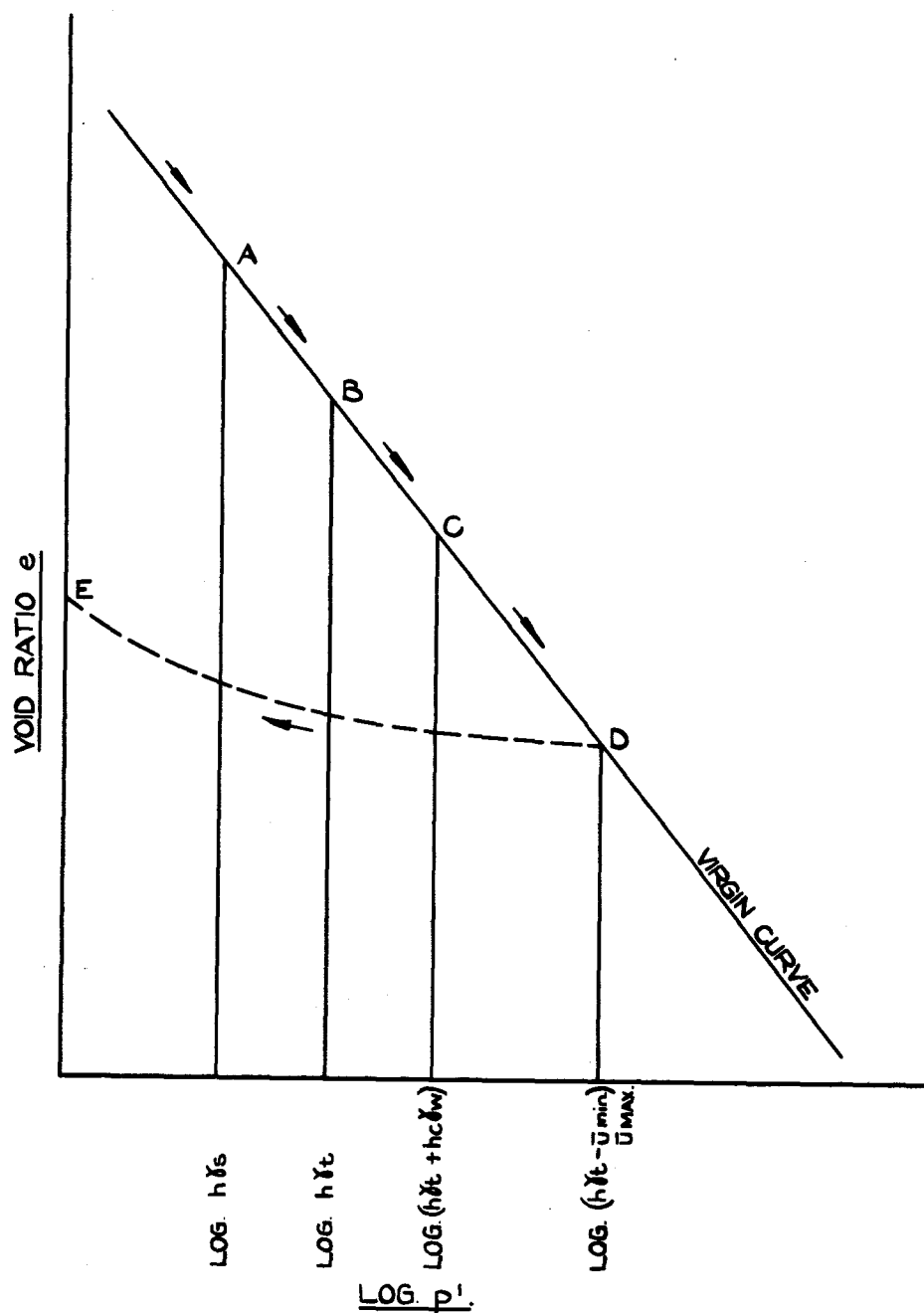


FIG. 44.

CONSOLIDATION BY DESICCATION.

The third stage continues until the water table has dropped to a depth h_c below the point considered where h_c is the maximum height to which all capillaries in the sample will be full. At this stage $\bar{u} = h_c \gamma_w$ and therefore substituting in (6)

$$p' = h \gamma_t + h_c \gamma_w.$$

This third stage is depicted by section BC of the virgin consolidation curve.

In the fourth stage the pores of the soil are no longer completely full of water. Nevertheless, water will collect at each point of contact between grains and will be held there by surface tension. This tension in the water will increase the intergranular force between the solid particles. On any plane, the force due to the tension in the pore water, will be the sum of all the small cross-sectional areas of water lenticals multiplied by the tension in the water. For the sake of continuity, this force is averaged over the whole cross-sectional area and still termed negative pore water pressure and designated by $\bar{u}$. If at any moisture content the effective cross-sectional area of water is A' and the tension in the water, otherwise called the pressure deficiency is p'' .

$$\bar{u} = -A'p'' \dots\dots\dots (7)$$

where $A' = \frac{\text{Cross-sectional area of water in voids}}{\text{Total cross-sectional area of voids}}$

As the moisture content of the soil decreases as it dries out, A' will remain constant or will decrease and p'' will increase. The result is that $\bar{u}$ will increase numerically until it reaches a maximum negative value $\bar{u}_{\max}$ which will depend on the nature of the material. During this fourth stage therefore, the consolidating pressure or intergranular pressure increases from

$$p' = h \gamma_t + h_c \gamma_w \text{ to } p' = h \gamma_t - \bar{u}_{\max} = h \gamma_t + (A'p'')_{\max}.$$

This fourth stage is represented in Fig. 44 by the section CD of the virgin curve.

As it was known that very high values of p'' obtain in fine grained soils at low moisture contents, it was thought that very high preconsolidation pressures would

be induced in these materials by the fourth stage of consolidation described above and it is suggested that this type of consolidation is responsible for the high preconsolidation pressures plotted in Fig. 43.

CONSOLIDATION BY DESICCATION OF A SLIMES DAM

The four stage consolidation process described is an idealised version of what happens on the wall of a slimes dam. In this case a point at a depth h was considered while the depth of material remained constant and the positive pore pressure was gradually reduced to zero and then the negative pore pressure was gradually increased to a maximum value u_{max} giving a continuously increasing intergranular pressure which results in the Void Ratio vs Log Pressure curve following the virgin consolidation line through A, B and C to D.

In practice the slimes is deposited in a shallow layer and the first stage of consolidation occurs quickly. The excess water is then drained off. Evaporation occurs and stage 2 is entered and as the desiccation continues the slimes will pass through stage 3 and into stage 4 and if conditions are very favourable may even reach a point represented by D in Fig. 44. The next layer of slimes is then deposited and water from this layer enters the layer below; it will release some, and perhaps all, of the negative pore water pressure, causing the sample to rebound along curve DE until it is in equilibrium with the new intergranular pressure under the altered conditions. As the material dries out again the intergranular pressure increases, and the e vs $\log p$ relationship will follow the loop shown in Fig. 41 and may or may not reach the value of D again before the next wetting, when it will rebound once more. Thus for each wetting and drying cycle there will be a loop of the consolidation curve.

On the other hand the point D may not be reached on the first wetting and drying cycle and the curve will rebound earlier. Once again a hysteresis loop in the consolidation curve will represent each wetting and drying cycle for the point considered. During one of the subsequent cycles the effective intergranular pressure may reach its greatest value, and may even reach point D which is the greatest possible value attainable. The

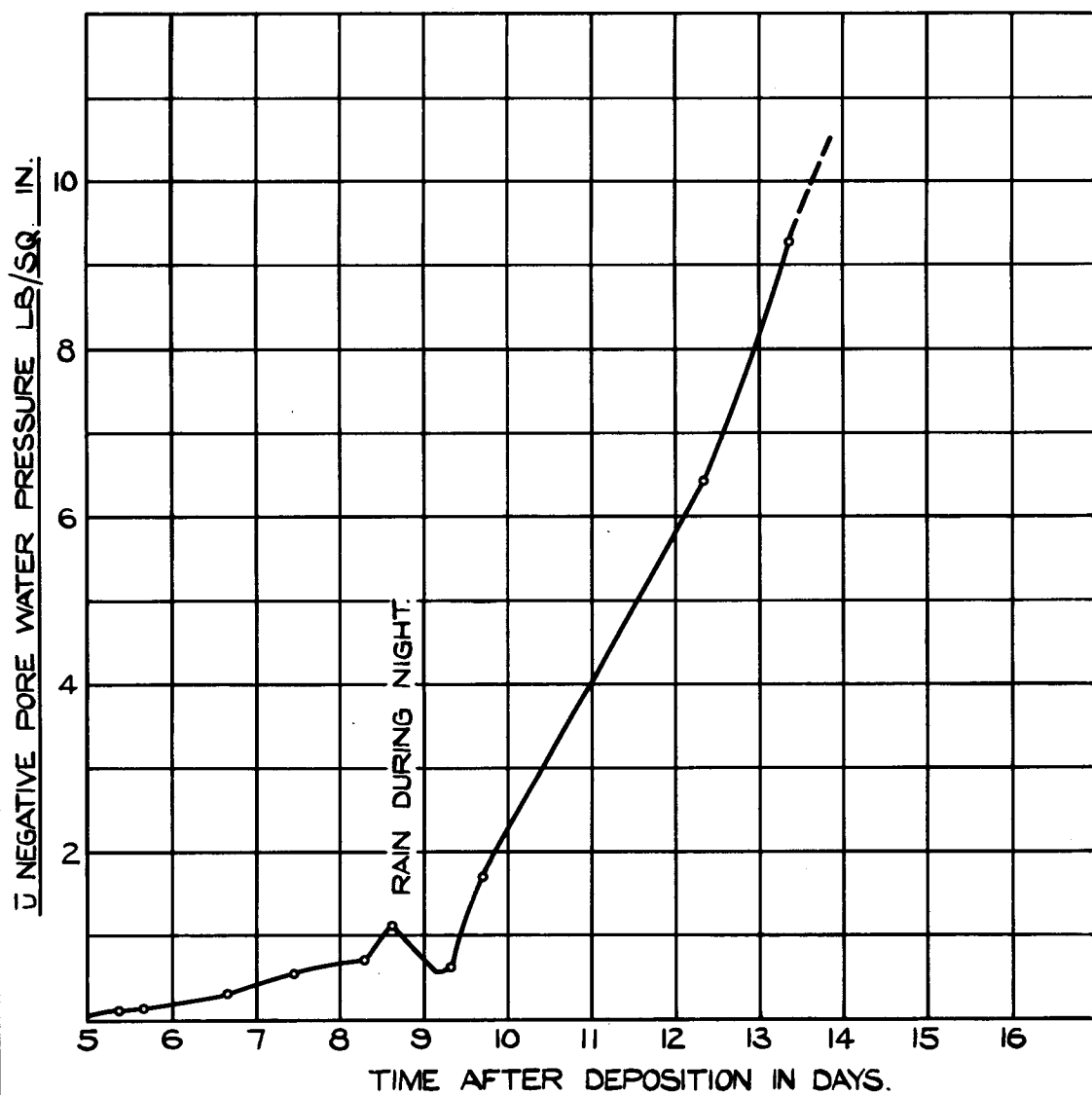
information in Tables I, II and III and in Fig. 43 shows that at some stage in the history of each sample a maximum intergranular pressure P_c was reached, but as the exact position on the consolidation curves, at the time of testing, is not known for any of these samples, the values of natural moisture content and natural void ratio give no correlations with preconsolidation load or with each other.

LABORATORY DESICCATION TESTS

In order to obtain a better understanding of the nature of consolidation by desiccation, it was decided to carry out laboratory desiccation tests during which the negative pore water pressure could be measured. In this way it was hoped to find a correlation between $\bar{u}$ and preconsolidation load.

It is exceedingly difficult to measure negative pore water pressures and pressure deficiencies in soils. Nevertheless, a simple pore pressure device (described in Appendix III: Laboratory Desiccation Tests) was used, and small negative pore pressures were measured. In the first test, a container of slimes slurry was exposed to the weather and readings of $\bar{u}$ were taken at various times. Tests 2, 3 and 4 were similar to test 1 except that the container of slimes was dried out in the laboratory by fans. In tests 5 and 6, four pore water pressure gauges were installed at various depths in a mass of slimes slurry and a series of readings of all the gauges was taken as the slimes dried out. At the completion of tests 2 to 6 samples were cut from the dried-out slimes and tested in the consolidometers to determine the preconsolidation pressures.

The negative pore water pressure for test 1 is plotted against time after deposition and draining in Fig. 45. For the first five days the water pressure remained at normal hydrostatic head which, in this case, was very nearly zero and the first readings of negative pore water pressure were obtained on the sixth day and the negative pore water pressure built up slowly during the seventh and eighth days and more rapidly during the ninth day. During the night of the ninth day it rained and the suction in the soil was partially relieved. As soon as the rain ceased, evaporation started again and it will be seen that the negative pressure build up was very rapid



DESICCATION TEST No. 1.

FIG. 45.

until air entered the measuring device, when the pressure was too great for the range of this apparatus. Tests 2 and 3 gave similar curves to test 1.

In test 4 water was put on to the slimes when the negative pore water pressure had built up to almost the limit of the measuring apparatus and the results of this test are plotted on Fig. 46. It will be seen that the addition of water released the negative pore water pressure almost instantaneously, the actual time taken being fifteen minutes.

In test 5 considerable difficulty was experienced with the measuring apparatus and readings were obtained for only very low values of negative pore water pressure. The negative pore water pressure versus time curves for test no. 6 are plotted in Fig. 47. As was to be expected there was a time lag increasing with depth between the upper and lower gauges. It will be noted that there is a good deal of unevenness in these curves, which can be accounted for by the fact that it was often necessary to flush out the measuring system to clean out air bubbles and some of this water escaped into the soil surrounding the gauge, causing a temporary break in the build-up of negative pore water pressure at that point. Fig. 48(a) shows the relationship between negative pore water pressure and depth at various stages of test 6, where it can again be seen that the greatest build-up of negative pore water pressure is at the top and that it diminishes with depth. These values of negative pore pressure have been added to the intergranular pressure due to the total density of the material, to give the total intergranular pressure which is plotted against depth in Fig. 48(b). In this figure the intergranular pressure for any particular time is almost uniform, irrespective of depth as the depth effect on the negative pore water pressure has been compensated for by the increase in intergranular pressure with depth due to the weight of the material itself. It will further be seen that these intergranular pressures are well in excess of the pressures due to weight of the material only.

Consolidation tests were carried out on samples taken from the slimes in tests 2 to 6, and the results of one of these tests are shown plotted in Fig. 49. The load in the consolidation tests was gradually increased to 32 lb./sq.in., then gradually released to 1 lb./sq.in., then gradually increased to 128 lb./sq.in. and finally decreased to zero. The rebound loop thus obtained appears

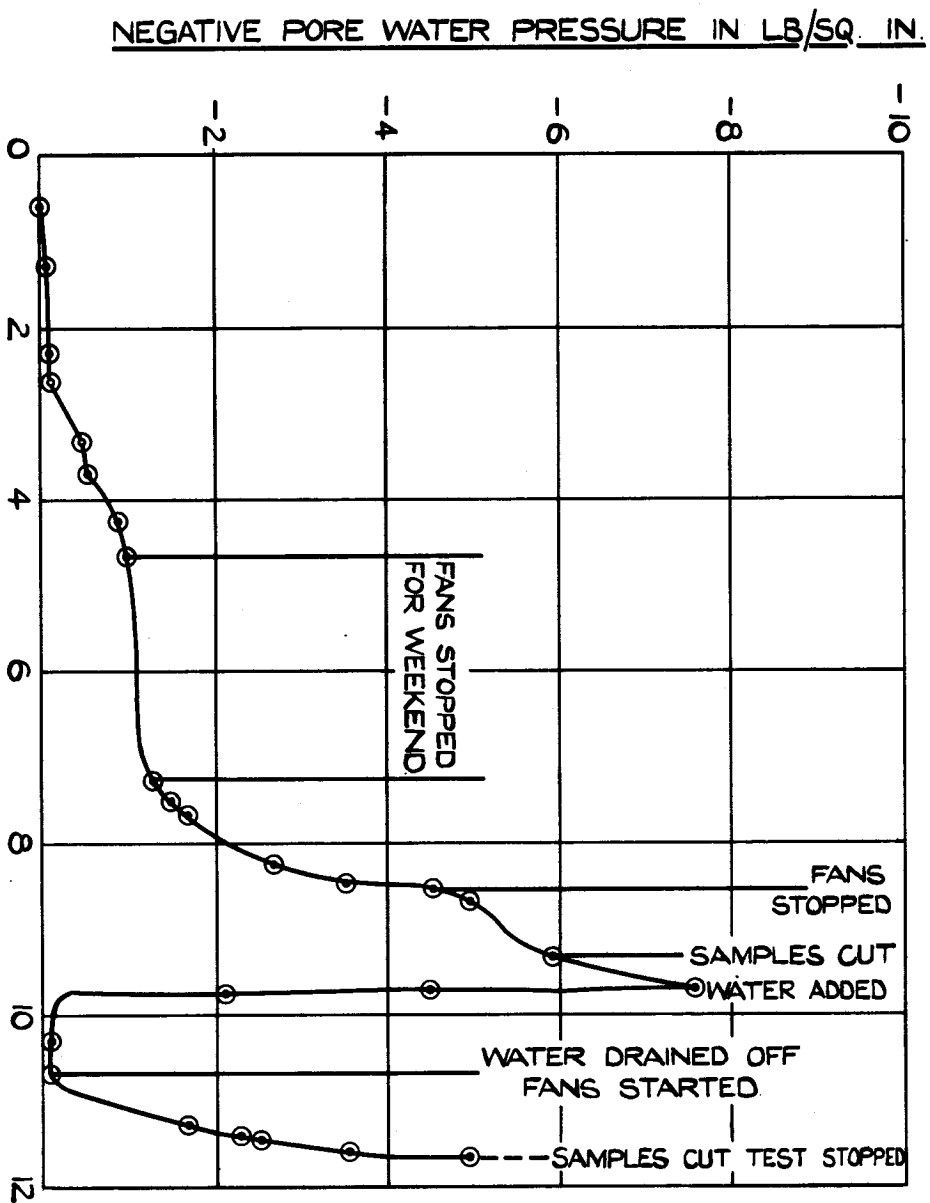
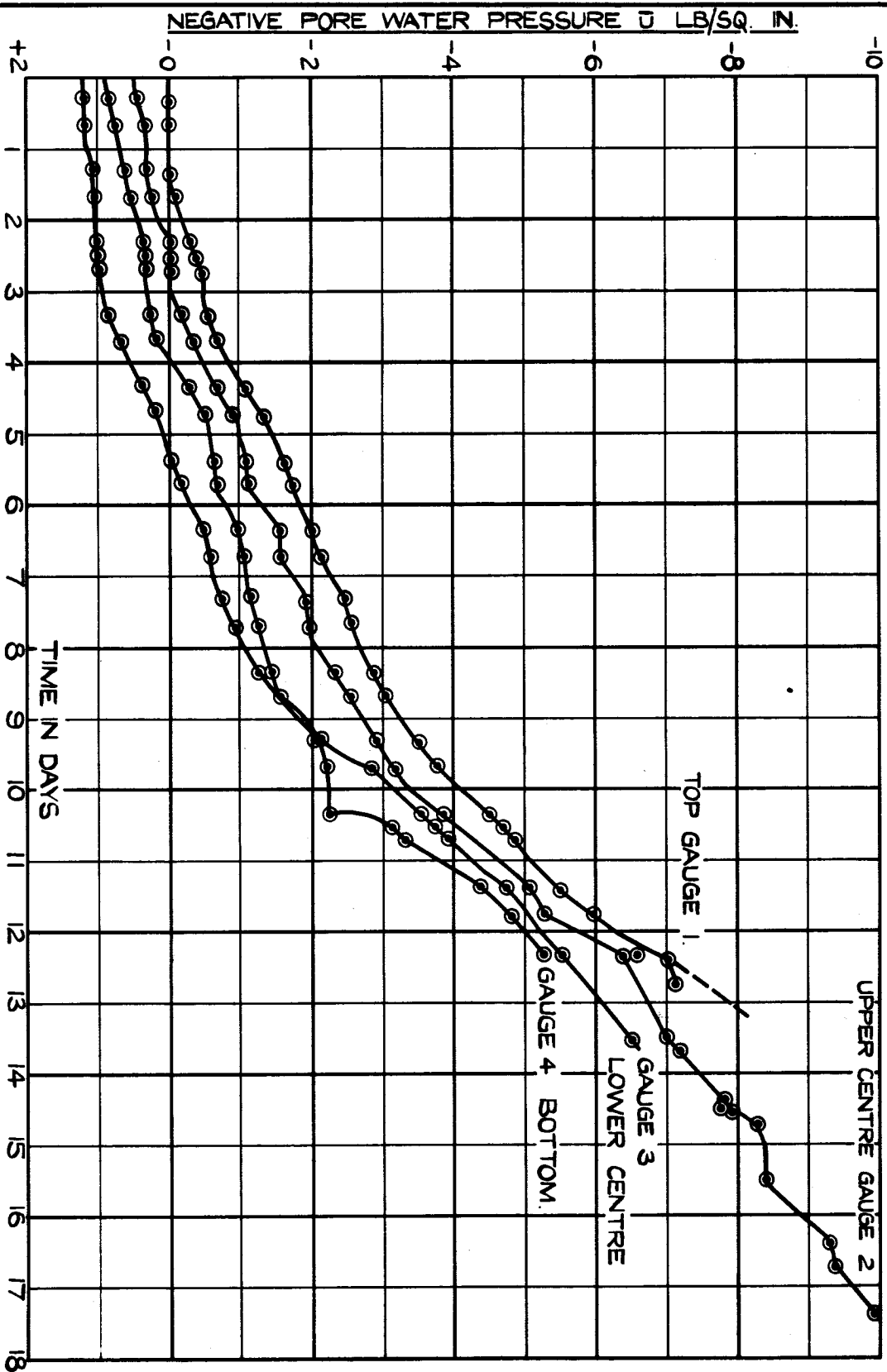
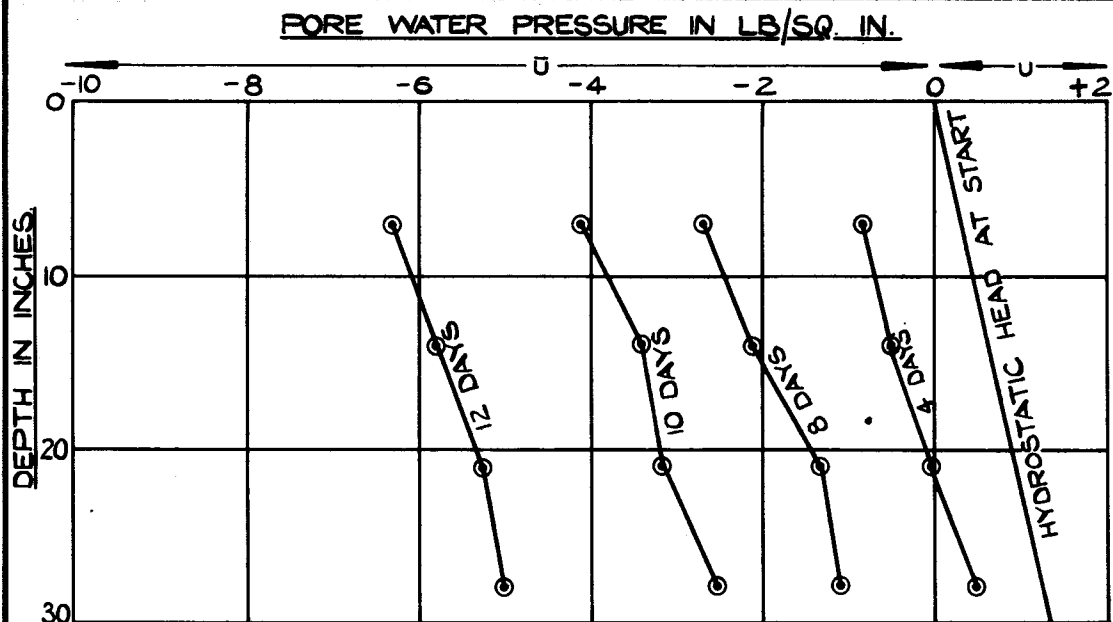


FIG. 46.

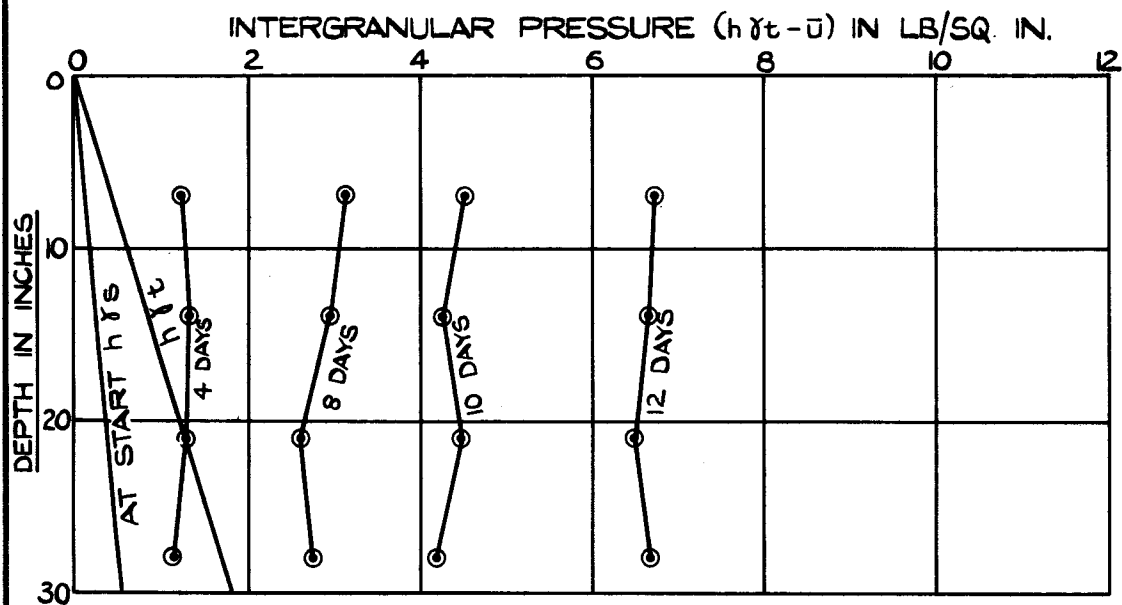
FIG. 47.

DESICCATION TEST No. 5.





(a) PORE WATER PRESSURE VS DEPTH.



(b) INTERGRANULAR PRESSURE VS DEPTH.

DESICCATION TEST No. 6.

FIG. 48.

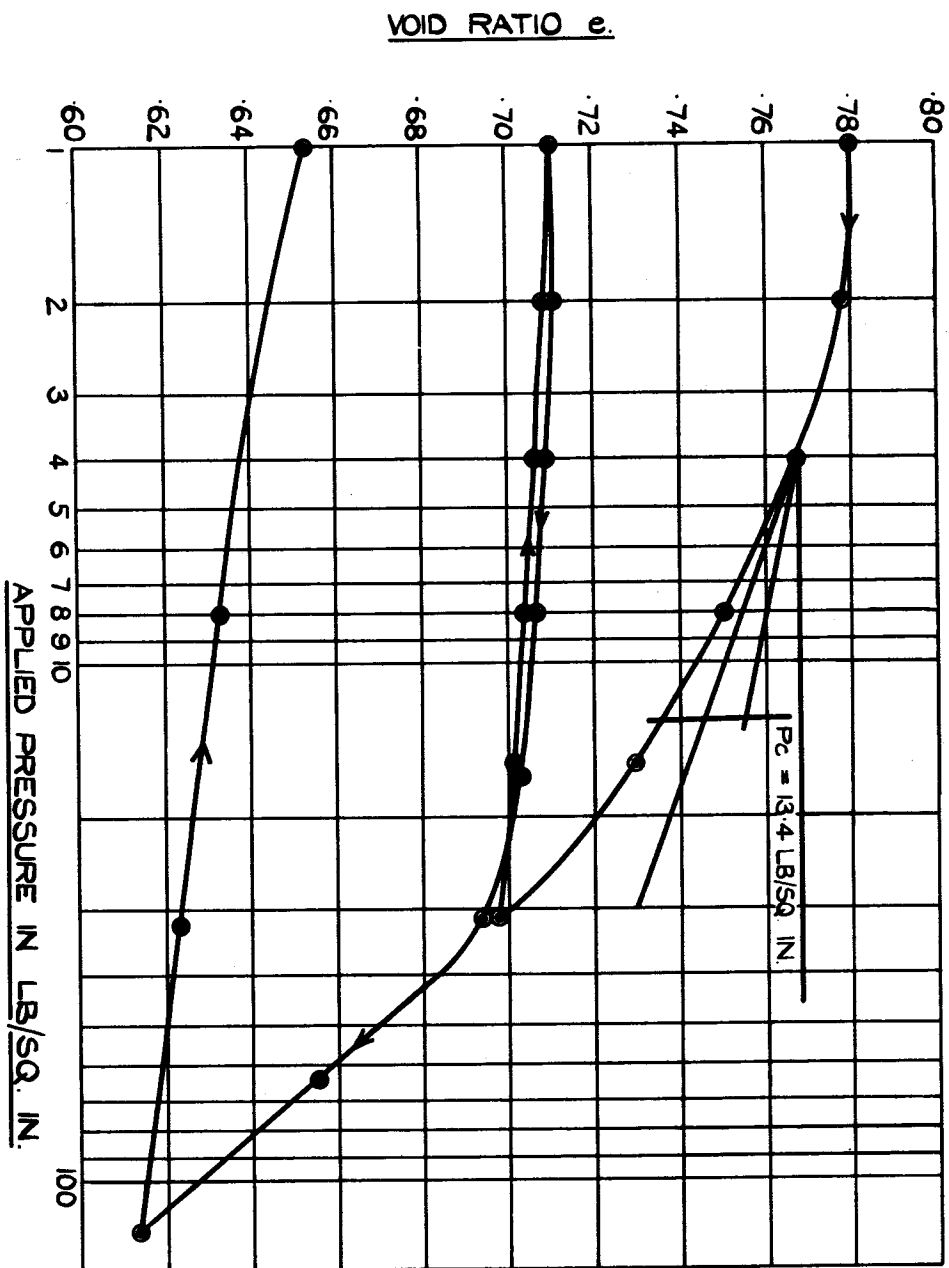


FIG. 49.

CONSOLIDATION CURVE — DESICCATION TEST NR. 6.

in the middle of Fig. 49. Similar rebound loops were determined in three other tests and the curves were almost identical.

An obvious point about Fig. 49 is the sharp curve of re-entry of the mechanical rebound loop into the virgin curve, whereas the upper portion of the consolidation curve (which is the recompression leg of a rebound loop after consolidation by desiccation) has a much flatter curve. This could possibly be explained if the total negative pore water pressure in the sample had not been fully released and the remnant of negative pore water pressure left would thus have to be added to the applied load to determine the effective intergranular pressure or consolidating pressure. If this is the case, the points on the top curve would be shifted to the right, closer to the virgin curve, and a curve similar to the mechanical rebound recompression curve would result. As the determination of the preconsolidation pressure depends on the shape of the curve, an alteration in the shape of the curve would probably also mean a higher or lower preconsolidation pressure. However, it is felt from studying these curves that even if the shape of the curve is not correct, any alterations to the curve in the manner suggested, would only result in minor changes in the preconsolidation pressure as determined from the curves.

The preconsolidation pressures, maximum negative pore water pressures measured and resultant maximum intergranular pressures are given in Table IV. It will be seen that, with three exceptions, all the preconsolidation pressures measured in the consolidation test are higher than the maximum effective intergranular pressure measured during the desiccation test. This is illustrated in Fig. 50 where preconsolidation pressures are plotted against maximum measured intergranular pressures on a log-log scale. The number of the preconsolidation pressures which greatly exceed the maximum measured intergranular pressures are due to the fact that some time elapsed after the range of the gauges was exceeded before consolidation tests could be performed, and thus additional desiccation had occurred during this period. It can therefore, be said that in this range the negative pore water pressure could be treated in the same way as positive pore water pressure in relation to effective intergranular pressure.

TABLE IV.

Preconsolidation pressures, negative pore water pressures, etc. for desiccation tests

Test No.	Gauge No.	Max. $\bar{u}$ lb./sq. in.	$h \gamma_t$ lb./sq. in.	$h \gamma_t - \bar{u}$ lb./sq. in.	P_c lb./sq. in.	Moisture content before consoli- dation %	Void ratio be- fore con- solidation test
1	1	9.3	-	-	-	-	-
2	1	-7.2	0.45	7.6	24.0	26.1	0.82
					16.0	28.6	0.72
					17.5	27.4	0.90
					30.0	25.1	0.87
3	1	-9.7	0.45	10.1	31.0	31.1	0.82
					19.0	32.5	0.81
					14.0	33.3	0.68
4	1	During test -5.8	0.45	7.9	7.3	25.8	0.74
					7.5	24.5	0.79
		-7.5	0.45	7.9	24.0	18.1	0.64
					7.0	15.9	0.75
5	1	-3.5	0.53	4.0	11.6	25.9	0.70
					12.0	26.1	0.76
	2	-4.2	1.06	5.2	48.0	25.7	0.66
	3	-2.7	1.60	4.3	32.5	27.9	0.78
					8.2	23.6	0.70
6	4	-7.9	2.13	10.0	7.0	33.9	0.83
	1	-6.9	0.53	7.4	14.6	0.8	0.91
					18.2	1.6	0.87
	2	-9.8	1.06	10.9	13.4	10.0	0.76
	3	-6.8	1.60	8.4	11.8	10.1	0.81
					10.9	10.3	0.78
	4	-5.8	2.13	7.9	11.8	8.8	0.83

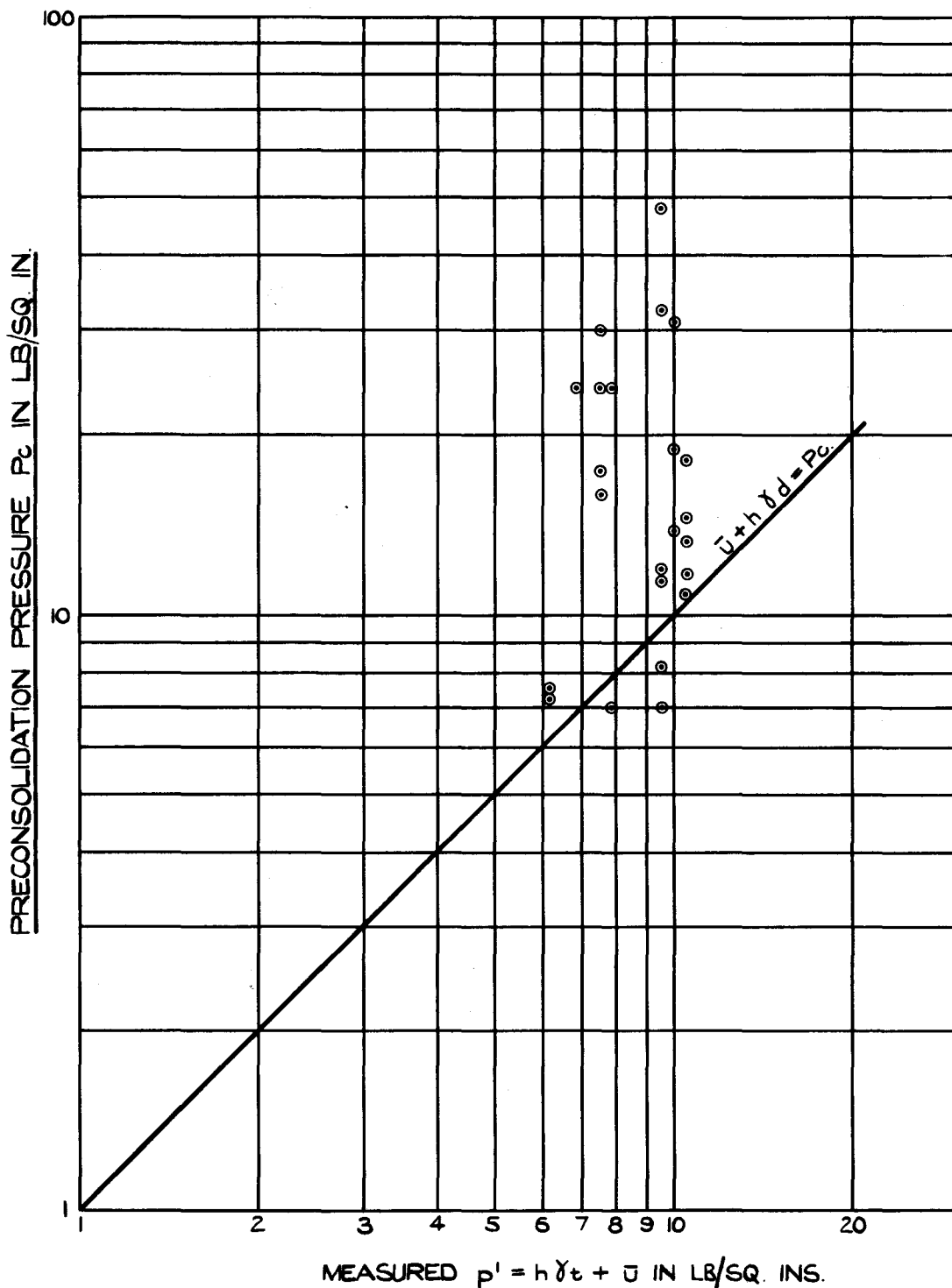


FIG. 50.

PRECONSOLIDATION PRESSURE VS MEASURED INTERGRANULAR PRESSURE
FOR DESICCATION TESTS.

pF-MOISTURE CONTENT RELATIONSHIP

Unfortunately no method exists of measuring the negative pore water pressure, as such, outside the range already dealt with. However, it is possible, using the latest soil physics techniques to measure the moisture content of a soil under conditions of controlled pressure deficiency and in this way to determine a moisture content-pressure deficiency relationship. Since the range of possible pressure deficiencies is enormous, Schofield (18) has proposed a convenient scale for pressure deficiency with his pF concept where

$$pF = \log_{10} h \dots\dots\dots (8)$$

where h is the pressure deficiency (p") expressed in centimetres of water.

Using suction-plate and micro-desiccator methods similar to those used by Croney, Bridge and Coleman (19) it was possible to determine the pF-moisture content relationship for slimes. This is shown in Fig. 51. It will be seen that points have been obtained at the ends of the curve and the centre portion has had to be interpolated as no apparatus is yet available which can be used in this pF range.

APPLICATION OF pF-MOISTURE CONTENT RELATIONSHIP TO CONSOLIDATION PROCESS

Using this pressure deficiency - moisture content relationship and assuming $A' = 1$, i.e. $\bar{u} = -p''$ the theoretical maximum effective intergranular pressure p' was calculated for all those consolidation tests where the moisture content at the start of the test was less than 20%. These results are given in Table V and plotted in Fig. 52.

There is a wide scatter of the points in Fig. 52 both above and below the 45° line representing P_c measured = p' calculated. The points which fall above the line can be explained by the sample having taken up moisture after reaching its maximum pressure deficiency; thus, when sampled, the existing moisture content gave a lower calculated value of p' than had previously existed - the sample had rebounded.

TABLE V.

Calculated intergranular pressures for samples
used in consolidation tests

Precon- solidation pressure P_c lb./sq.in.	Moisture content %	$h \gamma_t$ lb./sq.in.	$P_c - h \gamma_t$ lb./sq.in.	Pressure deficiency p'' lb./sq.in.	Inter- granular pressure $p' = h \gamma_t + p''$
Undisturbed field samples					
34	11.3	2.61	31.4	41.0	43.6
48	11.8	13.9	36.1	35.7	49.6
49	13.4	17.3	31.7	23.1	40.4
100	17.5	21.3	79.7	7.3	28.6
68	14.9	2.3	65.7	14.9	17.2
17.1	11.5	1.9	15.2	39.2	41.1
13.0	8.7	2.3	10.7	71.3	73.4
19.0	16.7	17.6	1.4	9.2	26.8
* 11.0	16.0	17.9	-	11.3	29.2
* 18	15.6	30.2	-	12.4	42.4
Desiccation tests					
24.0	18.1	0.5	23.5	5.9	6.4
7.0	15.9	0.5	6.5	11.6	12.1
13.4	10.0	1.0	12.4	57.9	58.9
11.8	10.1	1.6	10.2	56.6	58.2
10.9	10.3	1.9	9.0	55.3	57.2
11.8	8.8	2.2	9.6	80.0	82.2

* In these samples P_c is less than $h \gamma_t$ and therefore, it can be safely assumed that they have been disturbed.

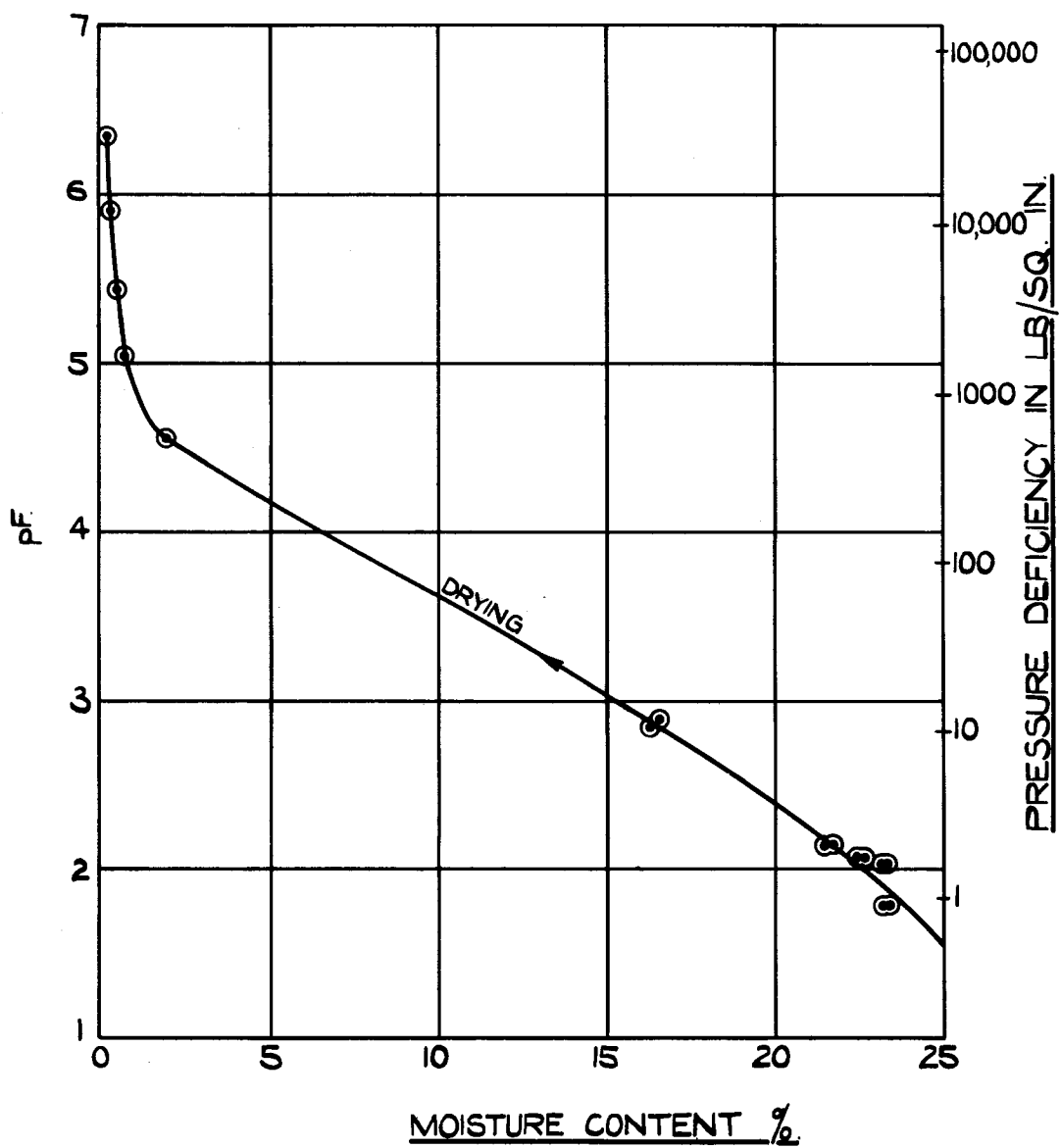
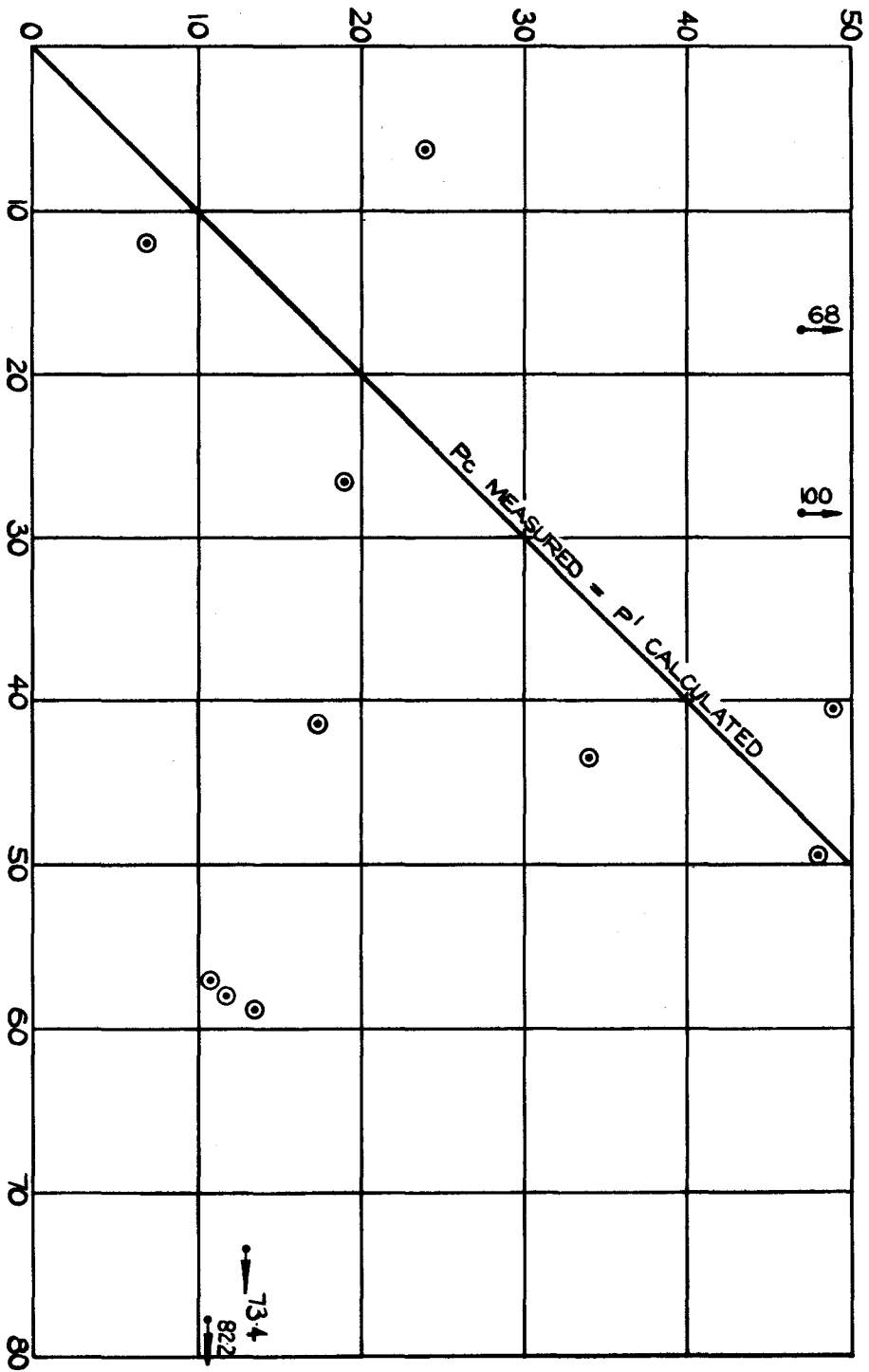


FIG. 51.

pF VS MOISTURE CONTENT RELATIONSHIP FOR SLIMES.

PRECONSOLIDATION PRESSURE P_c LB/SQ. IN.



CALCULATED INTERGRANULAR PRESSURE $P_i = h \gamma_c + P''$, ($-P'' = U$) LB/SQ. IN.

FIG. 52.

PRECONSOLIDATION PRESSURE VS CALCULATED INTERGRANULAR PRESSURE FOR
SAMPLES BELOW 20% MOISTURE CONTENT.

The points falling below the line can be explained in one of two ways, viz:

- (a) The samples were disturbed during sampling and handling as has already been shown in Table V for two samples. The material is very sensitive and thus very susceptible to disturbance during handling.

or

- (b) It was stated in equation (7) that

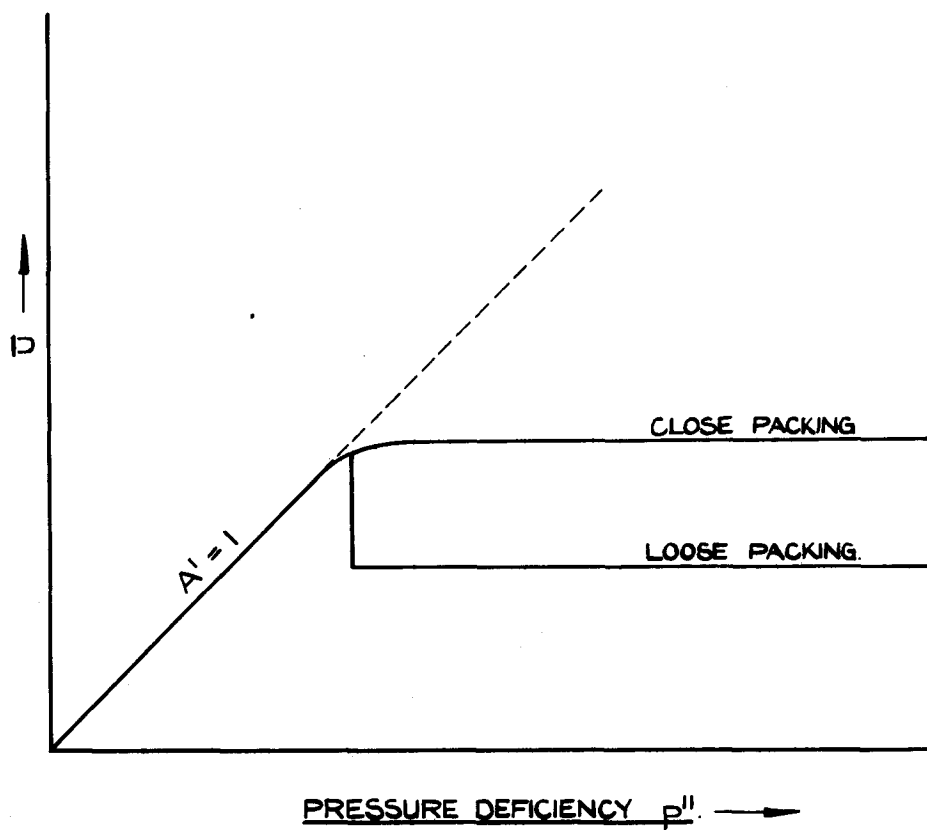
$$\bar{u} = -A'p''$$

and, for the purpose of these calculations, it was assumed that $A' = 1$, i.e. $\bar{u} = -p''$. If A' is less than unity the actual intergranular pressure p' would be less than that calculated assuming $A' = 1$.

Reason (b) requires investigation. Aitchison (16) has stated that for compressible soils (clays) $A' = 1$ for pressure deficiencies of up to 30 kg./sq.cm. (430 lb./sq.in.) but that in granular soils like sands $A' = 1$ initially until a certain value of p'' is reached whereafter $A'p''$ remains constant. This is shown in Fig. 53 giving the relationship between $\bar{u}$ and p'' for idealised material consisting of uniform spheres. The values of p'' where A' ceases to be unity and the value of $A'p''$ is constant for any soil will not give exactly the same curves as a mass of uniform spheres owing to the shape of the grains and the variations in grain size but the curves will be similar in shape.

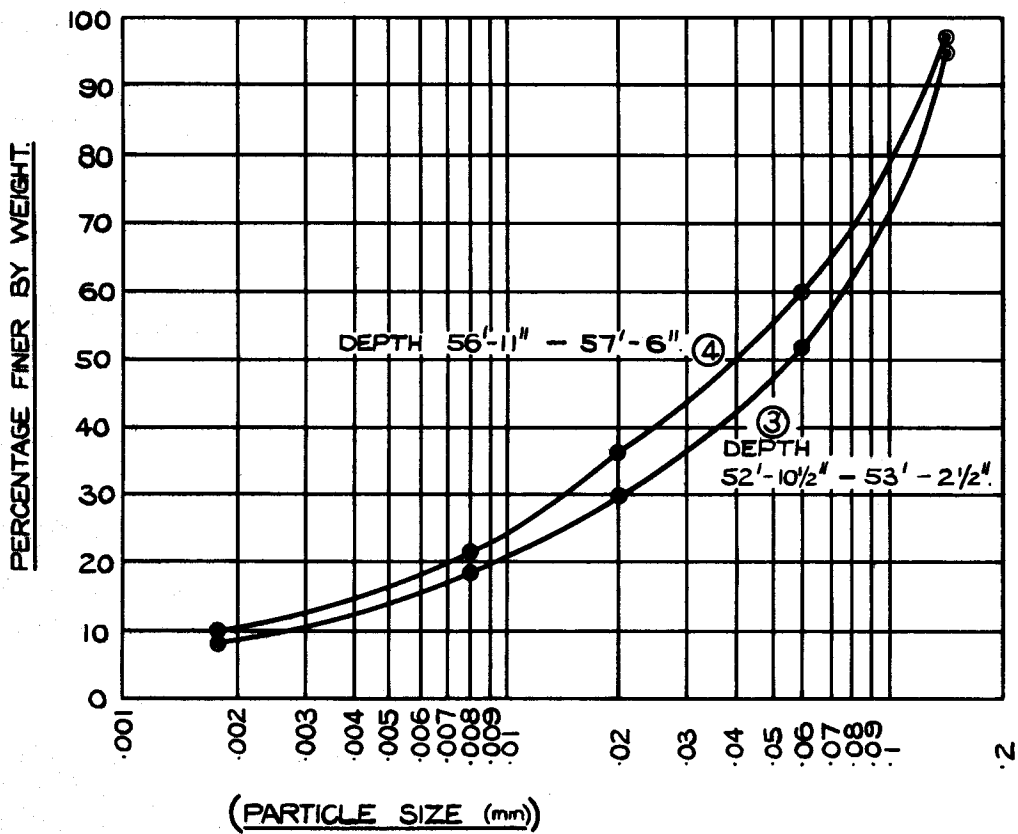
Slimes material covers a wide range of particle sizes as can be seen from the particle size distribution curves in Fig. 54. It has already been stated that there is a certain amount of segregation of particles during settlement on slimes dams and thus one would expect higher maximum values of $\bar{u}$ in places where the finer material predominates and lower values where the material is coarser. This is another factor contributing to the scatter of the preconsolidation pressures plotted in Fig. 43.

It would seem, therefore, that in the best conditions where the slimes is very fine and $A' = 1$, and where moisture content is reduced to a very low value, thus developing a high pressure deficiency p'' , intergranular pressures of the magnitude of the highest preconsolidation loads, e.g. 100 lb./sq.in., would be developed and in general the maximum effective intergranular pressures developed will be governed by:



RELATIONSHIP BETWEEN u AND p'' FOR A MASS OF UNIFORM SPHERES.

FIG. 53.



PARTICLE SIZE DISTRIBUTIONS OF SLIMES FROM
BOREHOLE 3 (a), EAST GEDULD.

FIG. 54.

- (a) the maximum pressure deficiency developed in the slimes during its history; and
- (b) the maximum value of $\bar{u}$ which is governed by the nature of the material.

It was felt that with the techniques available, no further progress could be made in studying these relationships. *

CONCLUSIONS

The conclusions drawn from the research into the consolidation of slimes material can be summarised as follows:

1. Field investigation of slimes dams showed that the material had been over-consolidated with maximum preconsolidation pressures of 14,400 lb./sq.ft. and an average preconsolidation pressure of about 5000 lb./sq.ft.
2. Over-consolidation is caused by the desiccation of the slimes which results in high pressure deficiencies in the pore water, in turn causing high intergranular pressures.
3. The maximum intergranular pressure developed will depend on the maximum pressure deficiency developed and on the grain size of the material.
4. As a corollary to 3. the greatest intergranular pressures will be developed in the finest material. This is a well-known practical fact which is used by slimes dam builders who say that the finer slimes "binds" better.
5. As a further corollary to 3., it has been shown that the slimes is layered and each layer may have a different limiting maximum intergranular pressure depending on its grain size.

* Note: Further work along these lines is envisaged in a project being undertaken on a national basis at the National Building Research Institute.

6. As the slimes is subjected to repeated cycles of wetting and drying which result in the intergranular pressure being raised and lowered, the material is being consolidated or rebounded all the time and the preconsolidation load as determined in the laboratory merely indicates the maximum intergranular force which was once developed in the particular sample tested and does not necessarily bear any relationship to the conditions obtaining in the soil at any time thereafter.

PRACTICAL APPLICATIONS OF CONSOLIDATION RESULTS

The main application of the work on the consolidation of slimes is to the study of the shear strength of slimes.

It has been shown that the maximum intergranular pressures are determined by the nature of the material and by the maximum pressure deficiency that is developed. The nature of the material is governed primarily by gold production and not convenience of slimes dam building. However, the slimes dam builder can assist in the development of the maximum pressure deficiency in the soil by seeing that the material is kept as dry as possible by the following means.

1. Depositing the material in shallow layers, because a shallow layer dries out more quickly and more thoroughly. The 2 inch layer used on most dams is satisfactory;
2. Draining all excess water from the deposition area as soon as possible;
3. Allowing as long a period as possible between successive depositions of material. Although it is not feasible to stipulate a definite period of drying, since this will depend on the thickness of the layer, the depth to which moisture has penetrated below the surface and the prevailing weather conditions, it is recommended that a minimum of fourteen days drying should elapse between successive depositions, if reasonable consolidation is to take place;
4. Removal of stormwater from the surface of the dam, especially the outer walls, as quickly as possible;

5. Ensuring adequate under-drainage for control of seepage flow. If seepage water is flowing through the wall causing a high phreatic surface, the pressure deficiency built up in the upper layers, by evaporation will be released by water drawn up from the phreatic surface. A large pressure deficiency will never develop and the material will not be well consolidated.

CHAPTER V. OVERALL STABILITY:
SHEAR STRENGTH OF SLIMES
MATERIAL

SHEAR STRENGTH OF SOILS

In calculating the stability of slopes, bearing capacity and other soil properties the shearing strength of the soil is used. This consists of two components, firstly cohesion, which is due to the attraction of the particles to each other and secondly, a frictional component due to friction developed by rubbing the soil grains against one another. This concept is contained in Coulomb's Law (20)

$$S = c + \sigma \tan \phi \dots\dots\dots (9)$$

where S = shearing strength of soil
 c = cohesion of the soil
 σ = stress in the soil normal to the plane of failure
and ϕ = friction angle of the soil or angle of shearing resistance.

This is shown diagrammatically in Fig. 55. In clayey soils c is usually large with ϕ small and in sands and other granular materials c is small, often zero, with a large ϕ .

The effect of stresses in the pore water during shear is considerable, thus, depending on the drainage conditions obtaining at the time of determining the shearing strength, both in the field and in laboratory testing, the values of c and ϕ vary; the different conditions are denoted by suffixes. Some common examples are given below:

$$S = c_u + \sigma \tan \phi_u \dots\dots\dots (10)$$

for the undrained case where no drainage of the pore water takes place during shearing.

$$S = c_{cu} + \sigma \tan \phi_{cu} \dots\dots\dots (11)$$

for the consolidated undrained case, where the material is consolidated under a certain pressure and is then sheared at that pressure while no drainage of the pore water takes place.

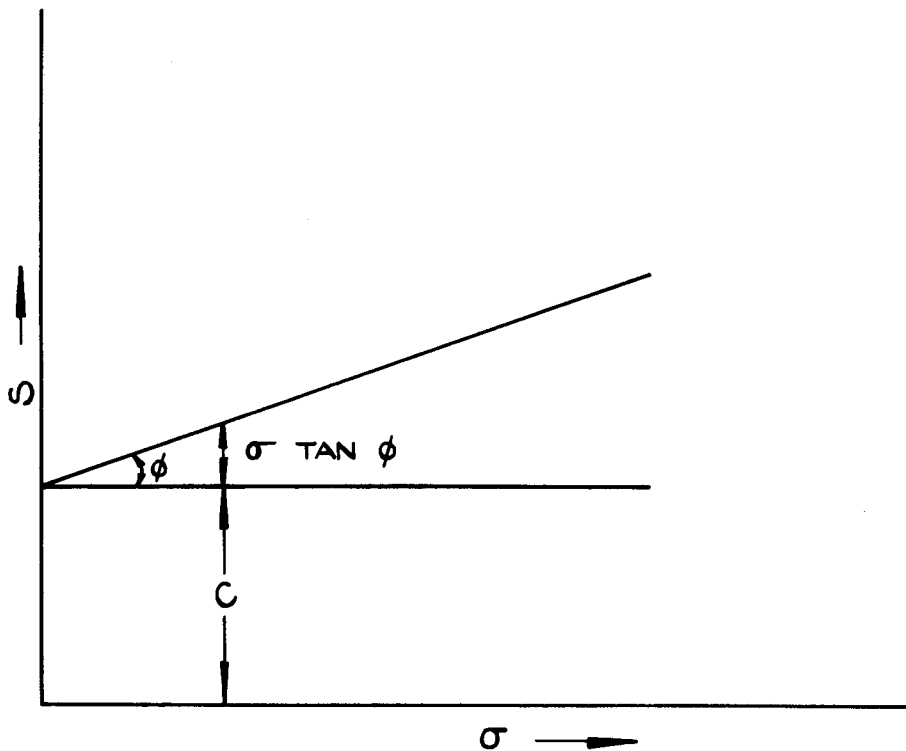


FIG. 55.

SHEAR STRENGTH DIAGRAM.

$$S = c_d + \sigma \tan \phi_d \dots\dots\dots (12)$$

for the drained case where complete drainage occurs during shearing.

Equations (10, (11) and (12) relate to σ , the total stress normal to the plane of failure for a given set of external conditions. However, if intergranular stresses are used, an equation is derived which can be applied to any set of conditions where σ' can be determined

$$S = c' + \sigma' \tan \phi' \dots\dots\dots (13)$$

where c' = effective cohesion
 ϕ' = effective angle of friction
 σ' = intergranular stress.

As intergranular stress and intergranular pressure are the same,

$$\sigma' = p' \dots\dots\dots (14)$$

and $\sigma = p$ for total stress and pressure..... (15)

and equation (5) used for consolidation studies becomes

$$\sigma' = \sigma - u \dots\dots\dots (16)$$

and (6) becomes $\sigma' = \sigma - \bar{u} \dots\dots\dots (17)$

Of the four main types of shear equation listed here, equation (13) is closest to the true values of c and ϕ .

The value of σ' in equations (16) and (17) will depend on the values of u and $\bar{u}$ in exactly the same way as was explained in the previous chapter for p' .

LABORATORY DETERMINATION OF SHEAR PARAMETERS c AND ϕ

In order to study the shear strength of slimes it was necessary to know the shear parameters c and ϕ . At the beginning of the study the only method suitable for measuring the shear strength of slimes was the shearbox test (21), otherwise known as the direct shear method. As no control could be exercised over drainage and no measurement of pore pressure was possible, only a generalised c and ϕ could be obtained from the test. A further difficulty was that, as the samples were consolidated in the shearbox from

a slurry, they could not be tested at a controlled density. The results of this series of tests are plotted in Fig. 56. As would be expected in a granular material $c = 0$ and the friction angle is $\phi = 39^\circ$.

It was felt that a more accurate determination of ϕ was needed and particularly that ϕ' should be determined. After some experiments, a successful technique was evolved for consolidating and testing slimes samples in a standard triaxial compression machine (see Appendix 4: Method of Triaxial Testing for Consolidated Slimes Material).

The normal method of plotting triaxial compression test results is by means of Mohr diagrams as explained by Terzaghi and Peck (22). During the test there is the cell pressure surrounding the sample, σ_3 , and the compressive stress on the sample $\sigma_1 - \sigma_3$ which reaches a maximum value when the sample fails. The value of σ is plotted on the Mohr diagram with σ_3 as shown in Fig. 57. From the geometry of the figure it will be seen that the shearing stress at failure is

$$\tau_f = \frac{(\sigma_1 - \sigma_3)}{2} \cos \phi \dots\dots (18)$$

If the pore water pressure u has been measured, intergranular stresses can be used for plotting as

$$\begin{aligned}\sigma_1' &= \sigma_1 - u \\ \sigma_3' &= \sigma_3 - u.\end{aligned}$$

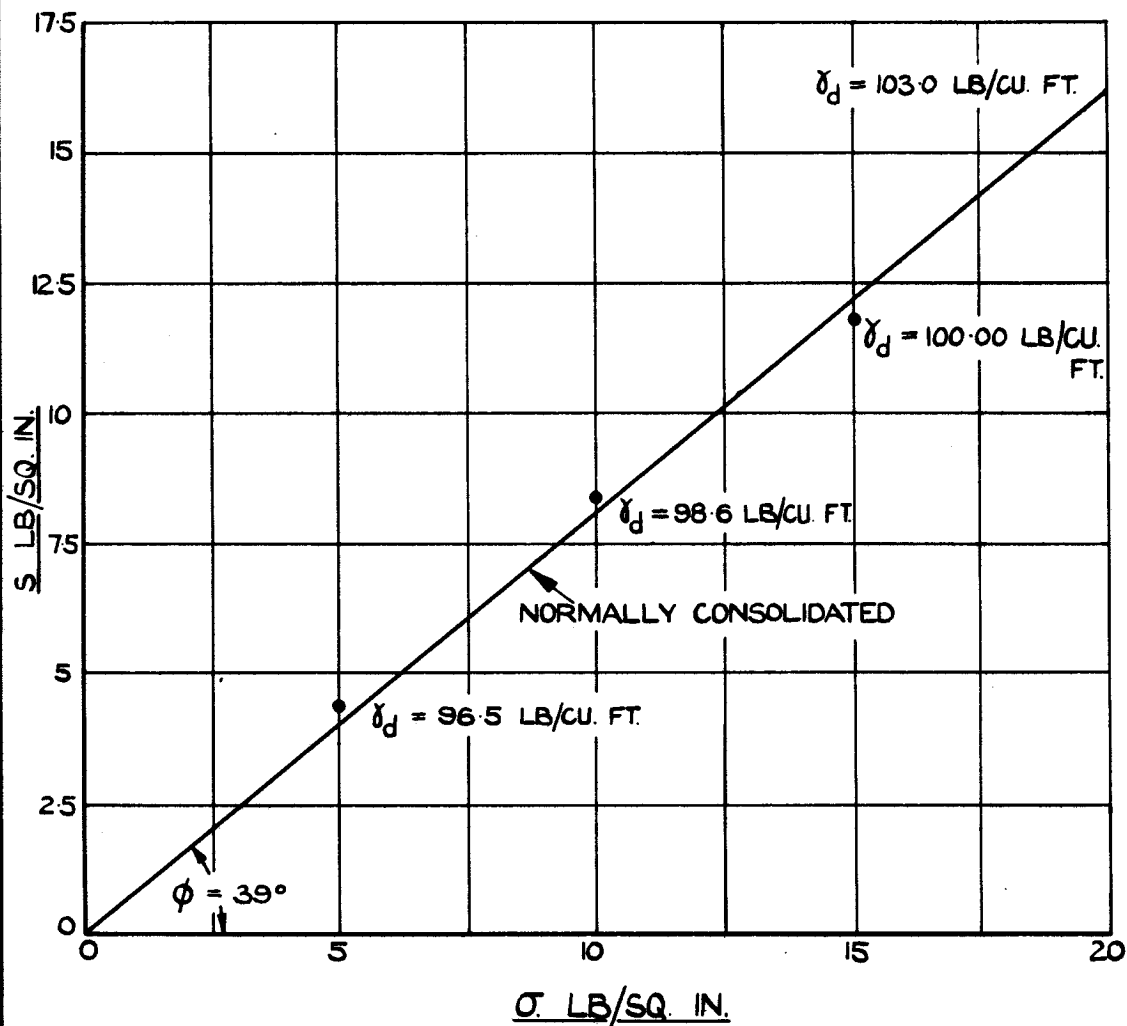
The intergranular stress case is shown by the dotted lines in Fig. 57 and for this case

$$\tau_f = \frac{(\sigma_1' - \sigma_3')}{2} \cos \phi' \dots\dots (19)$$

It is also obvious that $(\sigma_1' - \sigma_3') = (\sigma_1 - \sigma_3)$. The shearing stress at failure τ_f is considered as the shear strength S of the material.

The first series of tests consisted of a number of samples which were consolidated to certain cell pressures and were then tested at those cell pressures with no drainage and with the pore water pressure measured. The results of these tests are given in Table VI. There are two schools of thought as to when failure occurs during a triaxial test: namely

- (a) when $(\sigma_1' - \sigma_3')$ reaches a maximum; or
- (b) when σ_1'/σ_3' reaches a maximum.



RESULTS OF SHEARBOX TESTS ON NORMALLY CONSOLIDATED SLIMES.

FIG. 56.

SHEAR STRESS AT FAILURE τ_f .

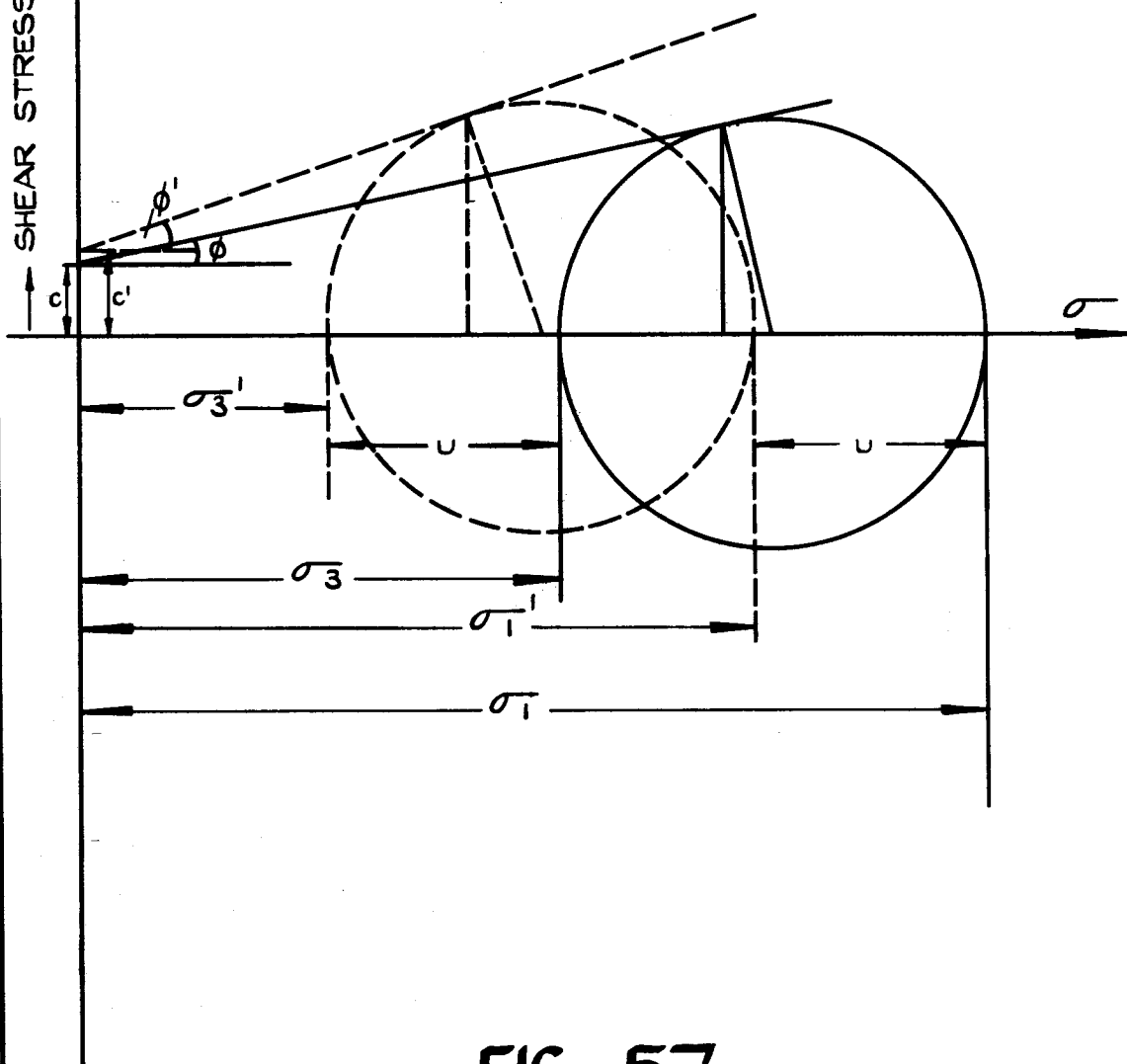


FIG. 57.

MOHR CIRCLES FOR TRIAXIAL COMPRESSION TEST RESULTS.

The graphs for $(u, \sigma_3, \sigma_1 - \sigma_3, \sigma_1$ and σ_1/σ_3 for test no. 25 are shown in Fig. 58. It will be seen that $(\sigma_1 - \sigma_3)$ never reaches a maximum value during the test but is increasing slowly at the end of the test, this small increase is attributed to the dilatancy of the sample and in these cases the 20% strain value of $(\sigma_1 - \sigma_3)$ is used as the maximum value of $(\sigma_1 - \sigma_3)$. In Table VI both failure criteria are used.

The Mohr circles for these tests are plotted in Fig. 59. From Table VI it will be seen that the total stress relationship of σ_3 and $(\sigma_1 - \sigma_3)$, which is the same as $(\sigma_1 - \sigma_3)$, varies considerably, and this will be further shown in Fig. 63. In spite of this variation, the effective stress diagrams in Fig. 59 plot very well to give a straight line passing through the origin in both cases. It is, therefore, seen that, as both diagrams give about the same results, the slimes has no effective cohesion; i.e. $c' = 0$, and the corresponding effective friction angle is $\phi' = 35^\circ$.

STRENGTH OF SLIMES MEASURED IN SITU

Vane shear tests were carried out in-situ at East Geduld Slimes Dam at the same time as the drilling operations (see Appendix 2: Field Sampling and Testing of Slimes at East Geduld Slimes Dam). During these tests, the undisturbed and remoulded shear strengths were measured at various depths and the "sensitivity" of the slimes was calculated. "Sensitivity is usually associated with clays and denotes loss of cohesion in the soil due to remoulding. In this case the term denotes loss of shear strength due to remoulding. The results of these tests are given in Tables VII, VIII and IX, and are plotted against depth in Fig. 60.

It will be seen from these results that the values obtained show a wide scatter similar to those for preconsolidation pressures plotted in Fig. 43. It will also be seen that the material has a "sensitivity" of 4 or more; this may account for some of the low values obtained during the testing.

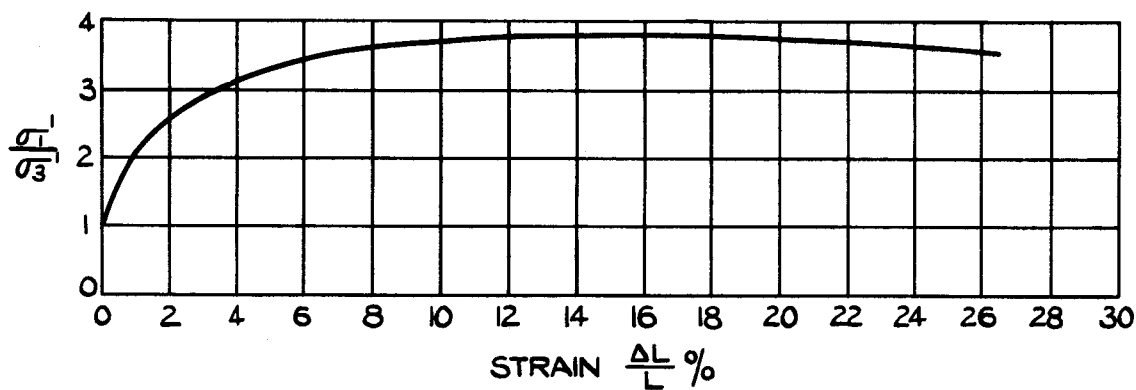
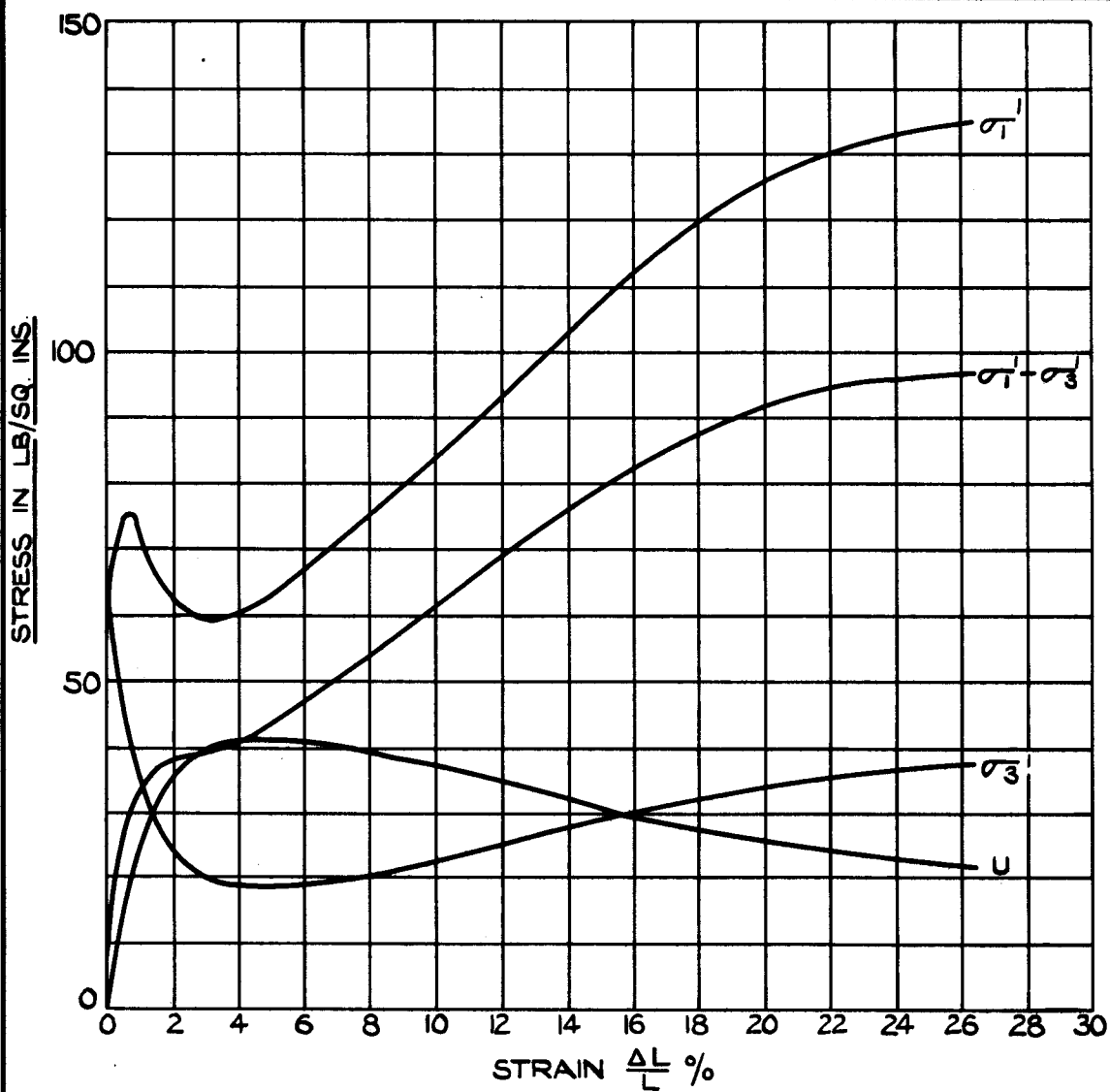


FIG. 58.

TRIAXIAL TEST RESULTS FOR SAMPLE NO. 25.

TABLE VI.

Results of undrained triaxial compression tests on
normally consolidated slimes samples
Pressures in lb./sq.in.

Nominal cell pressure σ_3	Test No.	Dry density lb./cu.ft.	$(\sigma_1 - \sigma_3) = \text{maximum}$				$\sigma'_1/\sigma'_3 = \text{maximum}$			
			$(\sigma'_1 - \sigma'_3)$	u	σ'_3	σ'_1	$(\sigma'_1 - \sigma'_3)$	u	σ'_3	σ'_1
10	11	95.2	42.2	2.0	12.0	54.2	34.2	1.3	8.7	42.9
20	4	93.7	28.8	9.8	10.3	39.1	22.2	12.7	7.4	29.5
	6	94.8	34.7	2.0	18.0	52.7	33.5	3.4	16.6	50.1
40	7	99.2	67.7	2.3	37.7	105.4	67.7	2.3	37.7	105.4
	16	91.5	83.7	9.9	30.1	113.8	79.2	12.0	28.0	107.2
50	3	-	91.8	14.6	35.4	127.2	83.0	20.6	29.4	112.4
60	24	100.0	130.8	14.4	45.6	176.4	107.0	23.7	36.3	143.3
	25	97.5	92.8	25.1	34.9	127.7	86.9	29.0	31.0	117.9
	28	102.0	83.1	32.3	27.7	110.8	66.9	39.1	20.9	87.8
	29	103.3	109.0	19.7	40.3	149.3	109.0	19.7	40.3	149.3
80	2	98.0	92.1	47.0	32.7	124.8	85.5	49.5	29.7	115.2
	5	103.4	200.1	11.6	68.2	268.3	196.3	12.9	66.9	263.2
	22	96.2	121.7	38.9	41.1	162.8	97.1	48.0	32.0	129.1
	23	96.0	159.5	22.7	57.3	216.8	155.9	25.0	55.0	210.9

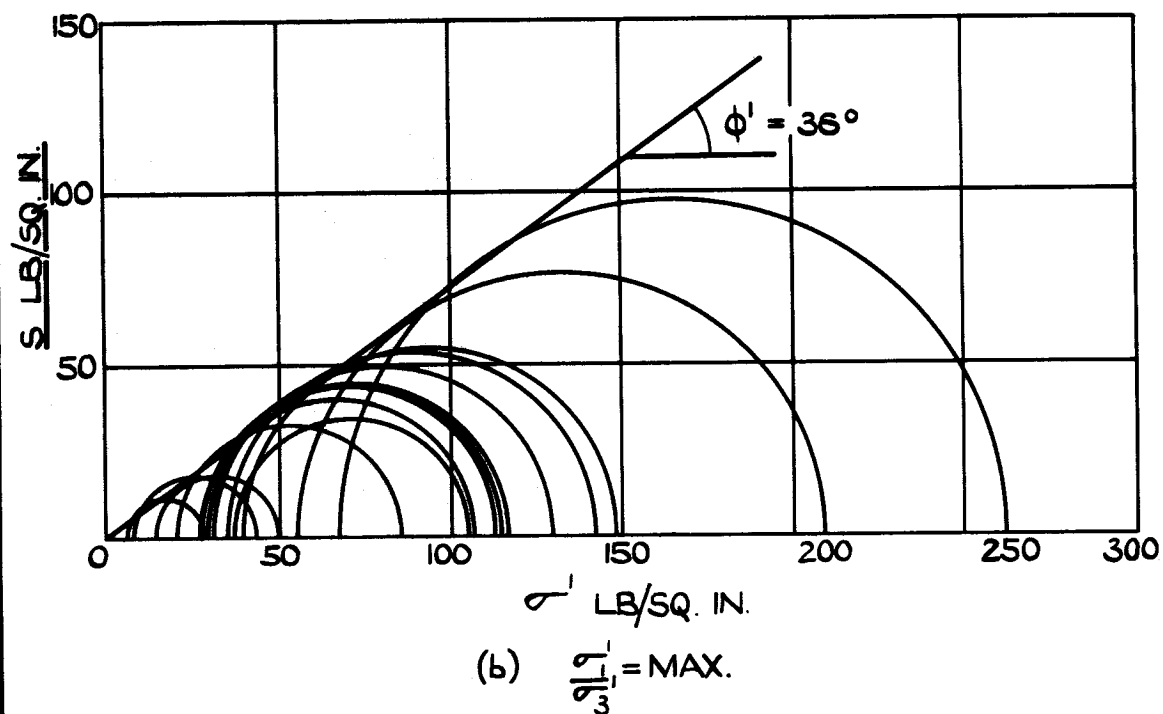
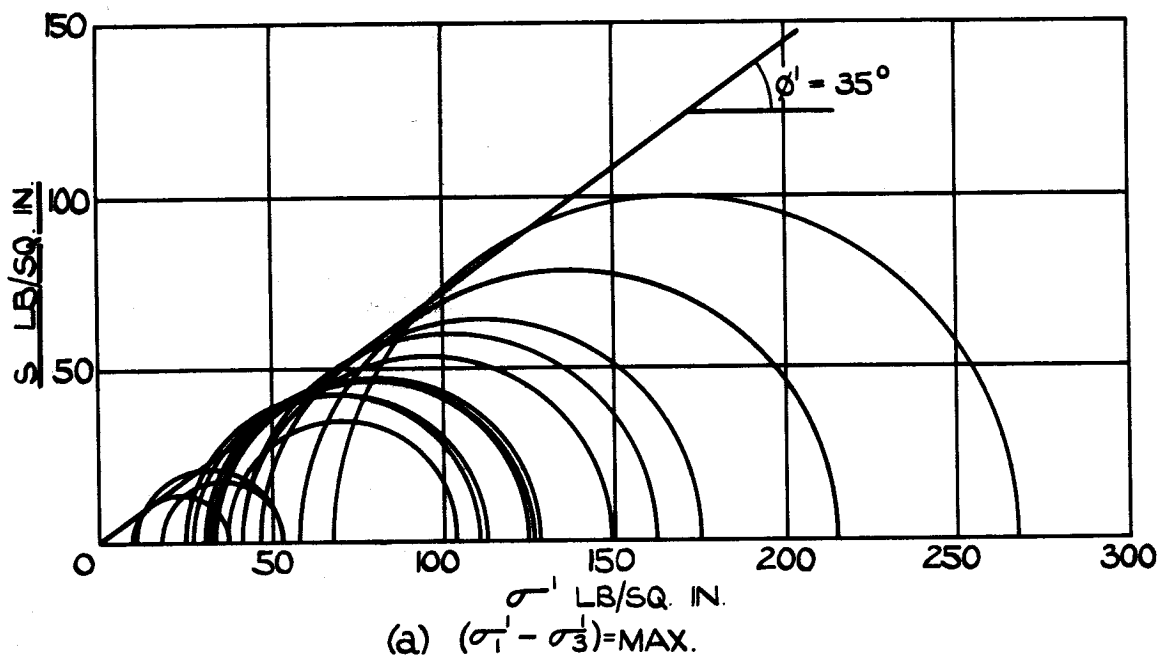


FIG. 59.

MOHR CIRCLES, EFFECTIVE STRESS ANALYSIS, TRIAXIAL RESULTS.

TABLE VII.

In-situ vane shear test results for hole 1.

Depth	Undisturbed shear strength lb./sq.ft. S_u	Remoulded shear strength lb./sq.ft. S_m	Sensitivity S_u/S_m
5'-0"	1787	-	-
9'-0"	1377		
13'-0"	931	No readings were taken.	
21'-0"	2140		
25'-0"	2512		
29'-0"	2382		
35'-0"	2717		
38'-0"	2757	726	3.8

TABLE VIII.

In-situ vane shear test results for hole 2.

Depth	Undisturbed shear strength lb./sq.ft. S_u	Remoulded shear strength lb./sq.ft. S_m	Sensitivity $S = S_u/S_m$
6'-0"	1873	130	15.2
10'-0"	763	335	2.3
17'-5"	2103	484	4.4
21'-6"	1582	614	2.6
25'-6"	2066	465	4.5
29'-3"	1861	428	4.4
36'-0"	2401	595	4.0
42'-0"	744	130	5.8

TABLE IX.

In-situ vane shear test results hole 3.

Depth	Undisturbed shear strength lb./sq.ft. S_u	Remoulded shear strength lb./sq.ft. S_m	Sensitivity $S = S_u/S_m$
5'-0"	1787	186	9.6
10'-0"	1545	298	5.2
15'-0"	1973	112	17.6
⌘ 20'-0"	2698 +	-	-
21'-7"	2643	354	7.5
25'-0"	2216	298	7.4
30'-0"	2959	521	5.7
35'-0"	1973	93	21.0
⌘ 41'-0"	3855 +	-	-
45'-0"	1768	354	5.0
50'-0"	1117	167	6.7

⌘ Vane sheared off.

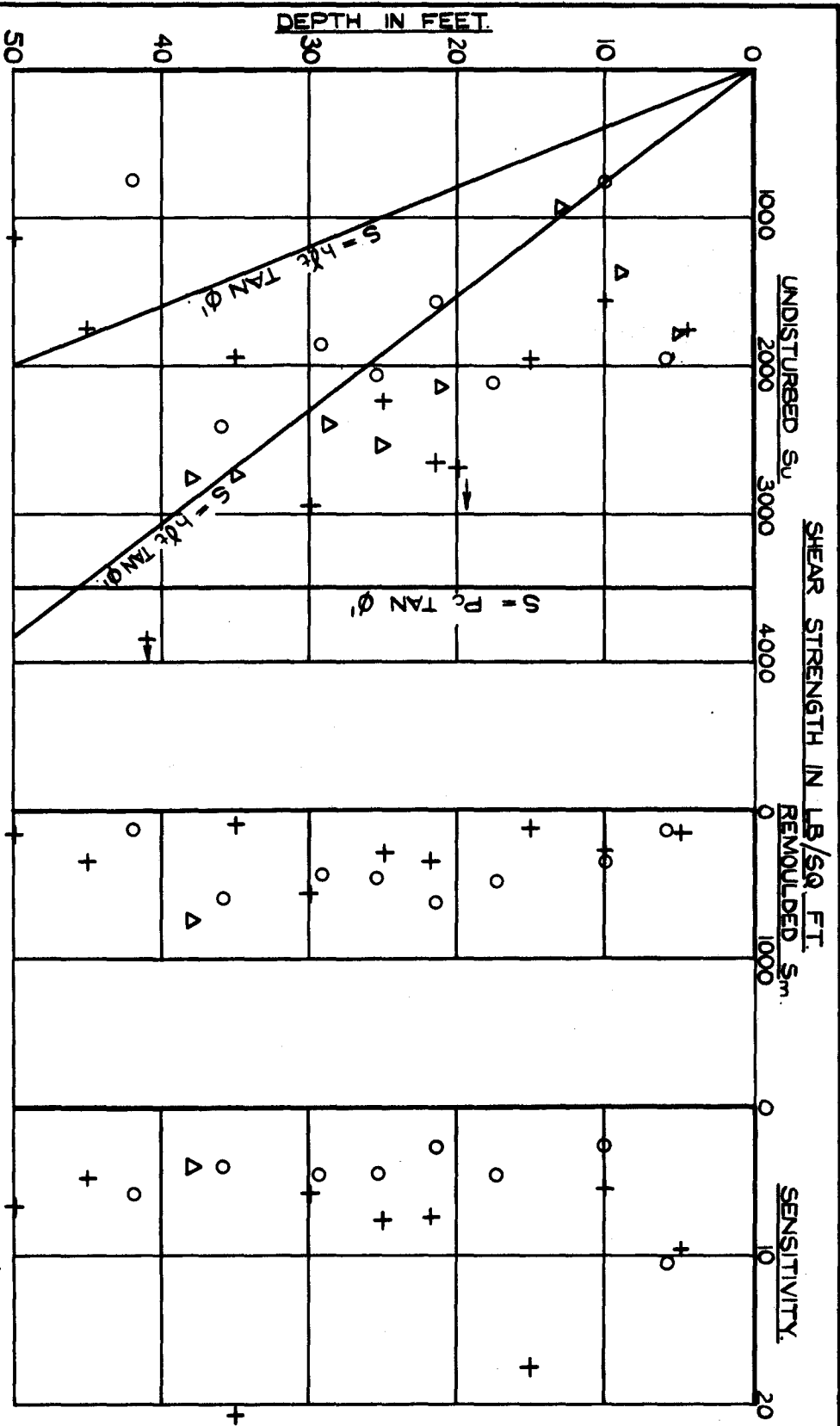


FIG. 60.

RESULTS OF IN-SITU VANE SHEAR TESTS, EAST GEDULD.

Also shown on Fig. 60 are the predicted shear strengths using the effective angle of friction $\phi' = 35^\circ$ and intergranular stresses based on submerged density, total density and the average preconsolidation load $P_c = 5000 \text{ lb./sq.ft.}$ estimated from Fig. 43.

Assuming the ground-water level in the dam coincided with the surface of the slimes the intergranular stress would be based on submerged densities and this would be the minimum intergranular stress which could occur. All but three of the values measured fall above this minimum condition, and the three low values of shear strength are probably the result of partial disturbance of the soil before or during the tests.

It will be seen that a large number of the strengths obtained for the undisturbed testing fall between the calculated values using total density for calculating intergranular stress, and the values assuming intergranular stress equal to the preconsolidation pressure P_c ,

$$h \gamma_t < \sigma' < P_c \dots\dots\dots (20)$$

This bears out the conclusion already stated in the previous chapter that the slimes at the time of sampling is on a rebound cycle somewhere between the preconsolidation pressure and $h \gamma_t$ for points above the water table.

These high shear strength values are due to apparent cohesion owing to increased effective intergranular stress at the time of testing caused by either

- (a) the existence of a negative pore water pressure at the time of testing;
- or (b) the effects of over-consolidation of the slimes.

INCREASE IN SHEAR STRENGTH OWING TO NEGATIVE PORE WATER PRESSURE AT THE TIME OF TESTING

The effective intergranular stress at the time of testing was

$$\sigma' = h \gamma_t - \bar{u} \dots\dots\dots (21)$$

where $\bar{u}$ is the negative pore water pressure and from (20)

$$0 < \bar{u} < P_c - h \gamma_t \dots\dots\dots (22)$$

In other words the shear strength at the time of testing was

$$S = (h \gamma_t - \bar{u}) \tan \phi' \dots\dots\dots (23)$$

The shear strength at any point will thus vary between $S = h \gamma_t \tan \phi'$ and $S = P_c \tan \phi'$ as $\bar{u}$ fluctuates when the slime dries out and is rewet.

Fig 61 shows the shear strength vs depth plot and strength plots for values of $\bar{u}$ of 0, -5, -10 and -15 lb./sq.in. It can thus be seen that the relatively small values of $\bar{u}$ will produce an apparent cohesion which will explain these high shear strength values.

However, the mean value of the majority of natural moisture contents in the field, tabulated in Tables I, II, III, XVIII^x, XIX^x, XX^x (^xAppendix 2) is greater than 25%. Relating these moisture contents to the pF-moisture content curve in Fig. 51, $\bar{u}$ for 25% will give an increase in intergranular stress of less than 1 lb./sq.in. It can, therefore, be seen that while some of the apparent cohesion may be due to negative pore water pressure, a number of values appear to be due to some other factor.

THE EFFECT OF OVER-CONSOLIDATION ON THE SHEAR STRENGTH OF SLIMES

Hvorslev (23), and others since, have shown that clay soils subjected to over-consolidation acquire additional shear strength due to the precompression, and on a plot of shear strength versus rebound pressure a hysteresis loop is formed as shown in Fig. 62. The shear strength of the rebounded material is much greater than that of normally-consolidated material at the same overall pressure.

Although this has been proved for cohesive materials it was felt that it would most probably also be true for slimes, which is not strictly cohesive but has a very fine grading. A further series of triaxial tests (Appendix 4) was performed on samples consolidated at 80, 40 and 20 lb./sq.in and then rebounded to a series of lower pressures before testing. This lower, rebound, pressure will be designated σ'_{cr} . As σ'_3 for these tests was not known, the σ'_1/σ'_3 ratio could not be determined and the $(\sigma'_1 - \sigma'_3)_{max}$ value has been used for calculating the shear strength which, from equation (19), becomes

$$S = (\sigma'_1 - \sigma'_3)_{max} \cos \phi' \dots\dots\dots (24)$$

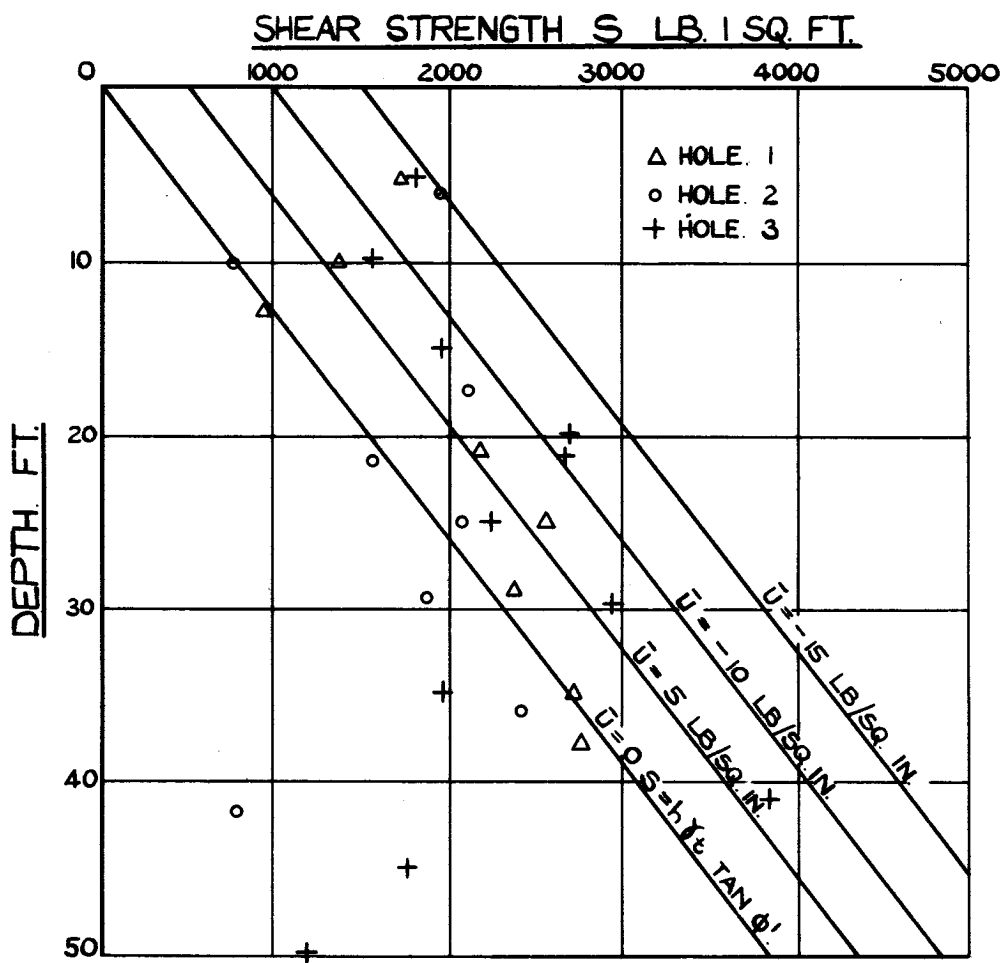
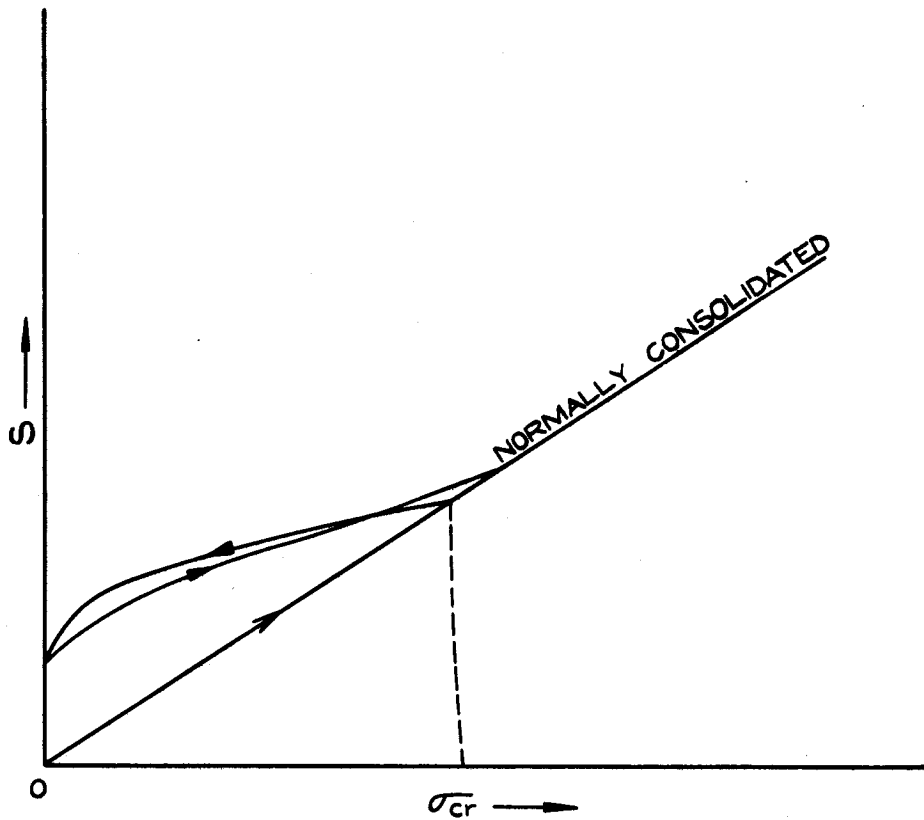


FIG. 61.

INCREASE IN SHEAR STRENGTH FOR VARIOUS VALUES OF
NEGATIVE PORE WATER PRESSURE.



EFFECT OF OVERCONSOLIDATION OF A CLAY SOIL
ON ITS SHEAR STRENGTH.

FIG. 62.

$$S = \frac{(\sigma_1' - \sigma_3)}{2} \cos \phi' \text{ LB/SQ. IN.}$$

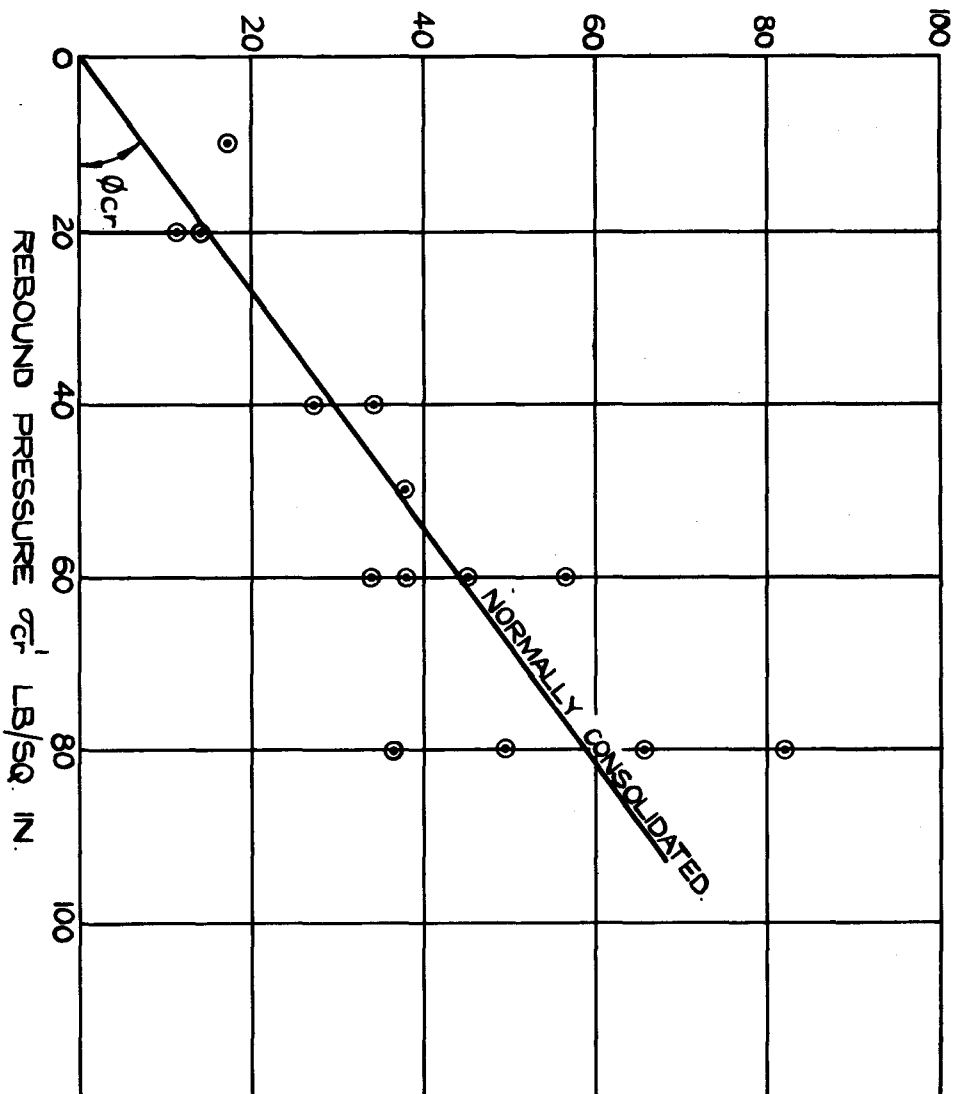


FIG. 63.

RELATIONSHIP BETWEEN SHEAR STRENGTH AND REBOUND PRESSURE FOR
NORMALLY CONSOLIDATED SLIMES.

The rebound pressure σ'_{cr} is taken as the apparent intergranular pressure at the beginning of shearing; in other words no account is taken of $\bar{u}$ or an increment of σ' due to over-consolidation. In the case of laboratory triaxial samples σ'_{cr} = cell pressure - u at the beginning of the test and in the field σ'_{cr} = $h \gamma_t - u$ or in both cases $\bar{u}$ if the pore water pressure is negative. In this series of tests $u = 0$ at the start of the tests and therefore, $\sigma'_{cr} = \sigma_3$, the cell pressure at start of test.

The results of this series of tests are given in Tables X, XI and XII.

The values for the normally consolidated case are taken from Table VI for $(\sigma'_1 - \sigma'_3)_{max}$ and are shown plotted in Fig. 63. The straight line is chosen to be the best average line and the slope of this line is ϕ_{cr} , the angle of shearing resistance with relation to σ'_{cr} , and is not necessarily equal to ϕ' . In other words

$$S = \sigma'_{cr} \tan \phi_{cr} \dots\dots\dots (25)$$

In this case $\phi_{cr} = 36^\circ$.

The very low total shear strength value of test 2, which plotted on the effective stress line in Fig. 59, is due to the very high pore water pressure measured during the test and it is almost certain that this high pressure was due to leakage through the membrane round the sample.

The shear results from Tables X, XI and XII are plotted in Fig. 64 from which it will be seen that there is a marked increase in shear strength due to over-consolidation of slimes. It will also be seen that the shear strength remains almost constant under rebound conditions until a value is reached from where the shear strength drops rapidly until it reaches almost zero when $\sigma'_{cr} = 0$. The recompression curves will, therefore, lie very close to the normally consolidated line.

Fig. 65 shows a simplified version of the curves in Fig. 64 which allows a set of equations to be set up for the strength of the precompressed material. These equations read as follows:

In the first range between $\sigma'_{cr} = 0$ and the flatter slope

$$S = c_0 + \sigma'_{cr} \tan 57^\circ \dots\dots\dots (26)$$

TABLE X.

Results of laboratory shear tests on samples pre-consolidated to 20 lb./sq.in. and rebounded

Rebound pressure σ'_{cr} lb./sq.in.	Test No.	$(\sigma'_1 - \sigma'_3)_{\max}$ lb./sq.in.	$S = \frac{(\sigma'_1 - \sigma'_3)_{\max} \cos \phi'}{2}$ lb./sq.in.	γ_d lb./cu.ft.
20	4	28.8	11.8	93.7
	6	34.6	14.2	94.8
15	32	34.4	14.1	99.7
10	9	30.7	12.6	92.3
	36	31.9	13.1	93.0
5	35	26.6	10.9	93.0
0	34	16.0	6.6	92.0

TABLE XI.

Results of laboratory shear tests on samples pre-consolidated to 40 lb./sq.in. and rebounded

Rebound pressure σ'_{cr} lb./sq.in.	Test No.	$(\sigma'_1 - \sigma'_3)_{max}$ lb./sq.in.	$S = \frac{(\sigma'_1 - \sigma'_3)_{max}}{2} \cos \phi'$ lb./sq.in.	γ_d lb/cu.ft.
40	7	67.5	27.7	99.2
	16	83.6	34.4	91.5
30	27	62.7	25.8	99.8
20	13	66.4	27.2	95.8
	33	54.0	22.2	101.2
10	14	54.4	22.3	95.1
5	31	30.5	12.5	94.5
0	26	14.0	5.7	96.5

TABLE XII.

Results of laboratory shear tests on samples pre-consolidated to 80 lb./sq.in. and rebounded

Rebound pressure σ'_{cr} lb./sq.in.	Test No.	$(\sigma'_1 - \sigma'_3)_{max}$ lb./sq.in.	$S = \frac{(\sigma'_1 - \sigma'_3)_{max}}{2} \cos \phi'$ lb./sq.in.	γ_d lb./cu.ft.
80	2	92.1	37.8	98.0
	5	200.1	82.0	103.4
	22	121.0	49.6	96.2
	23	159.6	65.4	96.0
60	18	96.5	39.6	-
	21	112.2	46.0	-
50	1	106.7	43.7	-
40	8	101.7	41.7	94.9
	17	96.6	39.6	95.2
30	19	90.2	37.0	-
	20	99.0	40.5	-
20	10	85.0	34.8	103.6
10	12	51.6	21.2	99.2
	30	54.0	22.2	101.5
0	15	10.4	4.3	96.3

The second, flatter range

$$S = c_p + \sigma'_{cr} \tan 15^\circ \dots\dots\dots (27)$$

where c_p = apparent cohesion due to precompression and suffix p denotes precompression pressure.

The change-over occurs where $\sigma'_{cr} = (\sigma'_{cr})_p$ and equations (26) and (27) are equal.

$$\begin{aligned} c_o + (\sigma'_{cr})_p \tan 57^\circ &= c_p + (\sigma'_{cr})_p \tan 15^\circ \\ \therefore (\sigma'_{cr})_p (\tan 57^\circ - \tan 15^\circ) &= c_p - c_o \\ \therefore (\sigma'_{cr})_p &= \frac{c_p - c_o}{\tan 57^\circ - \tan 14^\circ} \\ &= 0.787 (c_p - c_o) \dots\dots\dots (28) \end{aligned}$$

Below this value, equation (26) is used and for σ'_{cr} greater than $(\sigma'_{cr})_p$ equation (27) is used, until the normal consolidation occurs once more as the precompression curve rejoins the normally consolidated line and

$$S = \sigma'_{cr} \tan 36^\circ \dots\dots\dots (29)$$

This occurs when $\sigma'_{cr} = (\sigma'_{cr})_n$ and equations (29) and (27) are equal.

$$\begin{aligned} c_p + (\sigma'_{cr})_n \tan 15^\circ &= (\sigma'_{cr})_n \tan 36^\circ \\ \therefore (\sigma'_{cr})_n (\tan 36^\circ - \tan 15^\circ) &= c_p \\ \therefore (\sigma'_{cr})_n &= \frac{c_p}{.4586} \\ &= 2.14c_p \dots\dots\dots (30) \end{aligned}$$

Summarising, we have, therefore

$$\begin{aligned} \text{for } 0 < \sigma'_{cr} < 0.787 (c_p - c_o) \\ \text{Equation (26)} \quad S &= c_o + \sigma'_{cr} \tan 57^\circ \\ \text{for } 0.787 (c_p - c_o) < \sigma'_{cr} < 2.14c_p \\ \text{Equation (27)} \quad S &= c_p + \sigma'_{cr} \tan 15^\circ \end{aligned}$$

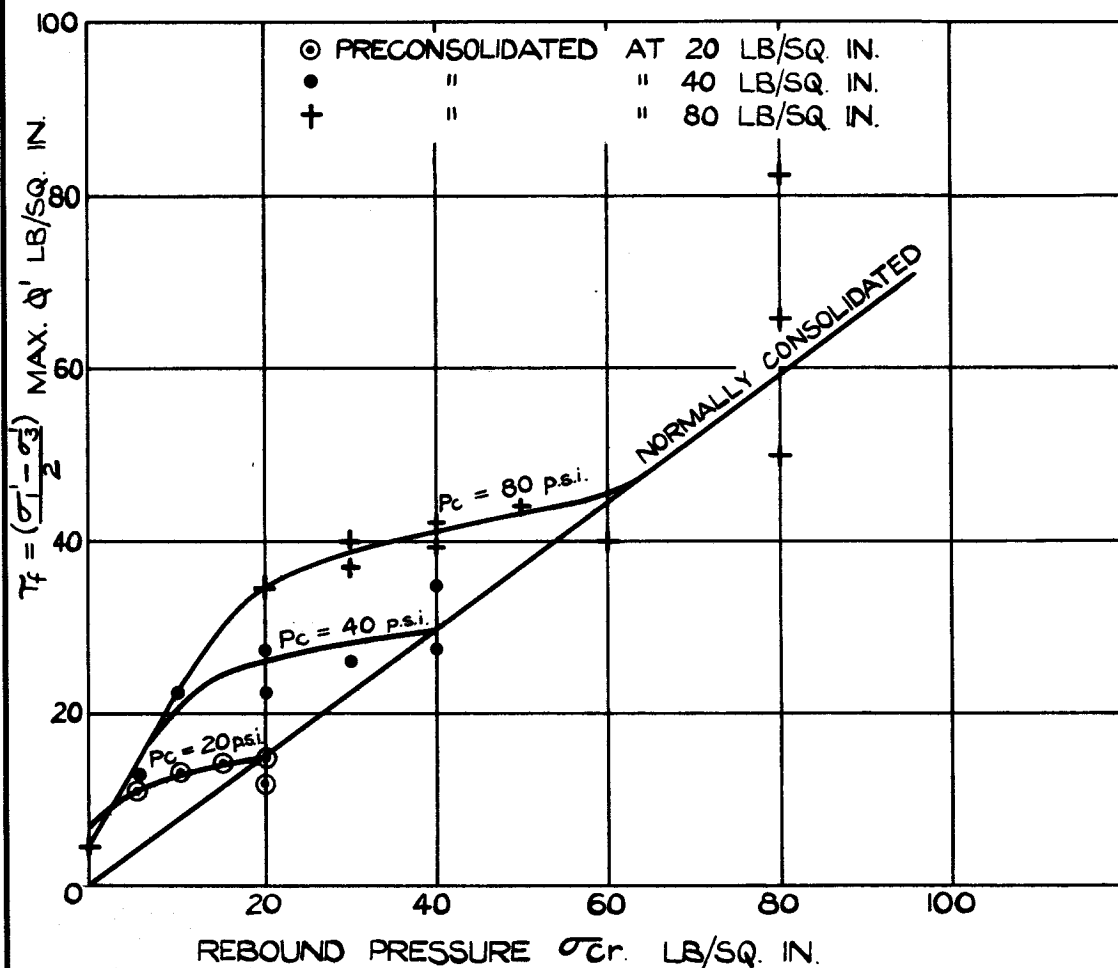


FIG. 64.

SHEAR STRENGTH VS REBOUND PRESSURE PLOT FOR TRIAXIAL TESTS ON OVERCONSOLIDATED SAMPLES.

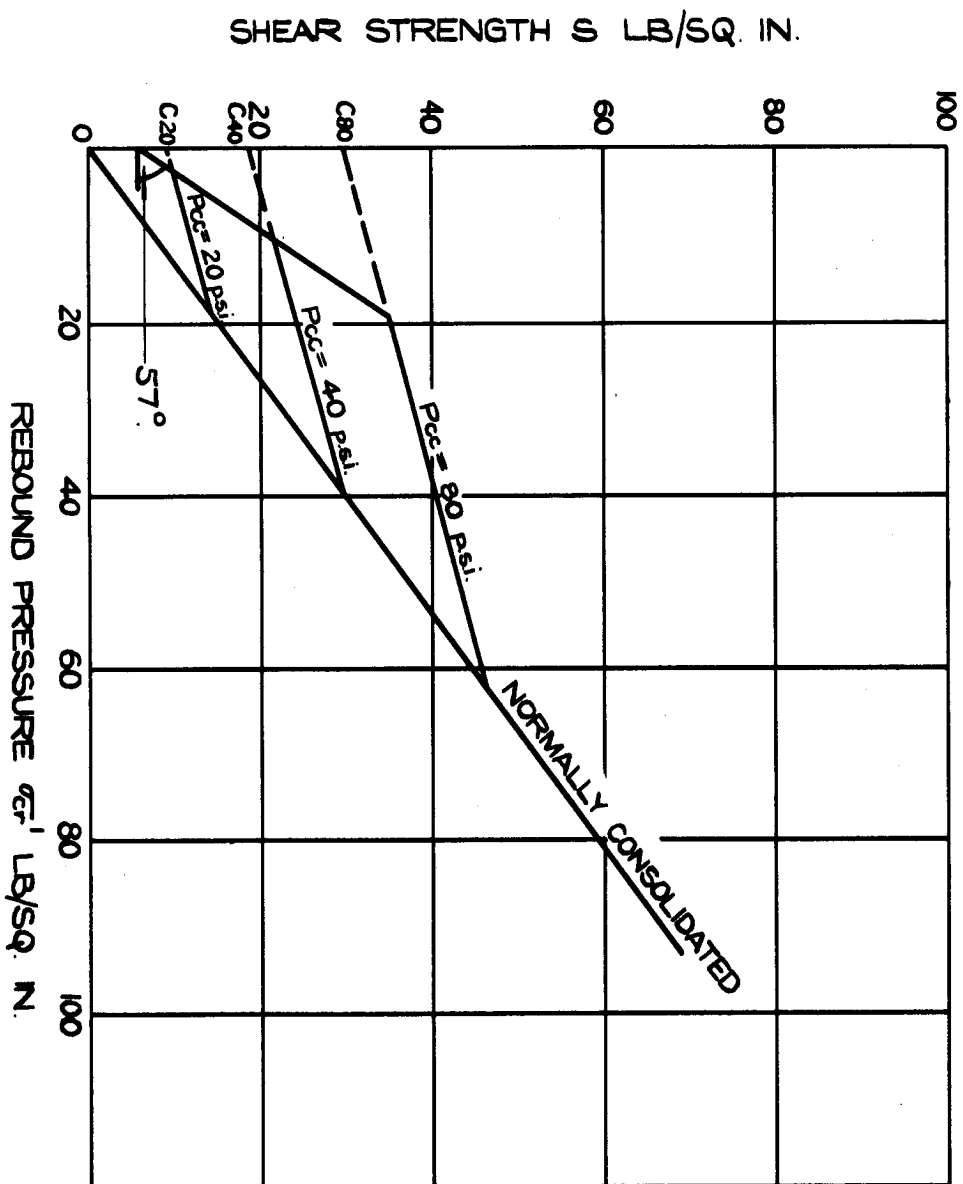


FIG. 65.

SIMPLIFIED RELATIONSHIP BETWEEN SHEAR STRENGTH AND REBOUND PRESSURE.

for $\sigma'_{cr} > 2.14c_p$, normally consolidated case.

Equation (13) $S = \sigma'_{cr} \tan 36^\circ$.

For the three preconsolidation pressures used, the values of c_o and c_p are:

for all cases $c_o = 6 \text{ lb./sq.in.}$

for $P_c = 20 \text{ lb./sq.in.}$, $c_{20} = 9 \text{ lb./sq.in.}$

for $P_c = 40 \text{ lb./sq.in.}$, $c_{40} = 19 \text{ lb./sq.in.}$

for $P_c = 80 \text{ lb./sq.in.}$, $c_{80} = 30 \text{ lb./sq.in.}$

Substituting these values in equations (26), (27), (28), (29) and (13) the following are found for each preconsolidation pressure:

$P_c = 20 \text{ lb./sq.in.}$:

for $0 < \sigma'_{cr} < 2.5 \text{ lb./sq.in.}$

$$S = 6 + \sigma'_{cr} \tan 57^\circ \dots\dots\dots (30a)$$

for $2.5 < \sigma'_{cr} < 20 \text{ lb./sq.in.}$

$$S = 9 + \sigma'_{cr} \tan 15^\circ \dots\dots\dots (31)$$

for $\sigma'_{cr} > 20 \text{ lb./sq.in.}$

$$S = \sigma'_{cr} \tan 36^\circ \dots\dots\dots (32)$$

$P_c = 40 \text{ lb./sq.in.}$:

for $0 < \sigma'_{cr} < 10.4 \text{ lb./sq.in.}$

$$S = 6 + \sigma'_{cr} \tan 57^\circ \dots\dots\dots (33)$$

for $10.4 < \sigma'_{cr} < 40 \text{ lb./sq.in.}$

$$S = 19 + \sigma'_{cr} \tan 15^\circ \dots\dots\dots (34)$$

for $\sigma'_{cr} > 40 \text{ lb./sq.in.}$

$$S = \sigma'_{cr} \tan 36^\circ \dots\dots\dots (35)$$

$P_c = 80 \text{ lb./sq.in.}$:

for $0 < \sigma'_{cr} < 18.9$

$$S = 6 + \sigma'_{cr} \tan 57^\circ \dots\dots\dots (36)$$

for $18.9 < \sigma'_{cr} < 64.0$

$$S = 30 + \sigma'_{cr} \tan 15^\circ \dots\dots\dots (37)$$

for $\sigma'_{cr} > 64.0$

$$S = \sigma'_{cr} \tan 36^\circ \dots\dots\dots (38)$$

In the laboratory it is simple to determine the rebound pressure σ'_{cr} at the beginning of shear. It has already been shown that the intergranular pressure in the slimes varies in cycles as the slimes is dried out and re-wet. In the consolidation theory the preconsolidation pressure P_c was defined as the highest intergranular pressure ever attained by the sample tested.

In dealing with shear strength the problem becomes more complex. It will be seen from Fig. 64 that if the slimes is rebounded to $\sigma'_{cr} = 0$, the shear strength becomes almost zero, irrespective of the preconsolidation pressure. On recompression the shear strength will be the same as for a normally consolidated sample. As the material is continually being compressed and rebounded, the shear strength will be governed by the characteristics of the rebound cycle concerned, and it is essential for an accurate determination of the shear strength to know:

- (a) the rebound pressure σ'_{cr} , which is the apparent intergranular pressure at the start of shearing;
- (b) whether the slimes is on a recompression leg or rebound leg of a rebound cycle;
- (c) if it is a compression leg, what the minimum rebound pressure was at the start of the compression leg and what the immediately preceding highest compression P_{cc} was.
- (d) if it is a rebound cycle, what the highest preceding compression P_{cc} was, as this will determine the position of the point in the shear rebound cycle as will the data required under (c).

From all the foregoing work, it will be obvious that it is impossible to answer these questions with regard to the field conditions. However, assuming $\sigma'_{cr} = h \gamma_t$ in equations (3) to (38) and further assuming a rebound leg, the shear strength of the slimes has been

calculated as shown in Table XIII and plotted with the field test results in Fig. 66, which is really the application of Fig. 65 to Fig. 60. It will be seen that most results fall between $P_{cc} = 2880 \text{ lb./sq.ft.}$ and $P_{cc} = 5720 \text{ lb./sq.ft.}$ with an average value of $P_{cc} = 3200 \text{ lb./sq.ft.}$ as compared with average $P_c = 5000 \text{ lb./sq.ft.}$ This would seem to give a very good fit to the field results obtained. It might be argued that σ'_{cr} is in fact not $h \gamma_t$ but an increased quantity due to $\bar{u}$, but any increase in σ'_{cr} would not affect the value of S greatly in the cohesive range of the curves (steep vertical slope) and will tend to alter all the boundaries in such a way that the whole strength envelope will be lifted. Such a change in the position of the strength envelope will give a much worse fit for the majority of points although this higher value of σ'_{cr} may exist in one or two cases.

For this particular case it can be said that the average shear strength of the slimes can be defined by the following equations

$$\begin{aligned} \text{for } 0 < \sigma'_{cr} < 620 \text{ lb./sq.ft.} \\ S &= 864 + \sigma'_{cr} \tan 57^\circ \text{ lb./sq.ft.} \dots\dots\dots (39) \end{aligned}$$

$$\begin{aligned} \text{and for } 620 < \sigma'_{cr} < 3400 \text{ lb./sq.ft.} \\ S &= 1700 + \sigma'_{cr} \tan 15^\circ \text{ lb./sq.ft.} \dots\dots\dots (40) \end{aligned}$$

$$\begin{aligned} \text{and for } \sigma'_{cr} > 3400 \text{ lb./sq.ft.} \\ S &= \sigma'_{cr} \tan 36^\circ \dots\dots\dots (41) \end{aligned}$$

where σ'_{cr} is calculated as follows for a point at depth h below the surface and where Z is the depth to water table or phreatic surface

$$\begin{aligned} \text{for points above the water table i.e. } h \leq Z \\ \sigma'_{cr} &= h \gamma_t \dots\dots\dots (42) \end{aligned}$$

$$\begin{aligned} \text{and for points below the water table i.e. } h \geq Z \\ \sigma'_{cr} &= h \gamma_t - (h - Z) \gamma_w \dots\dots\dots (43) \end{aligned}$$

As can be seen from equations (42) and (43), σ'_{cr} becomes less as the water table rises above the point concerned, which is still another reason for using underdrainage to

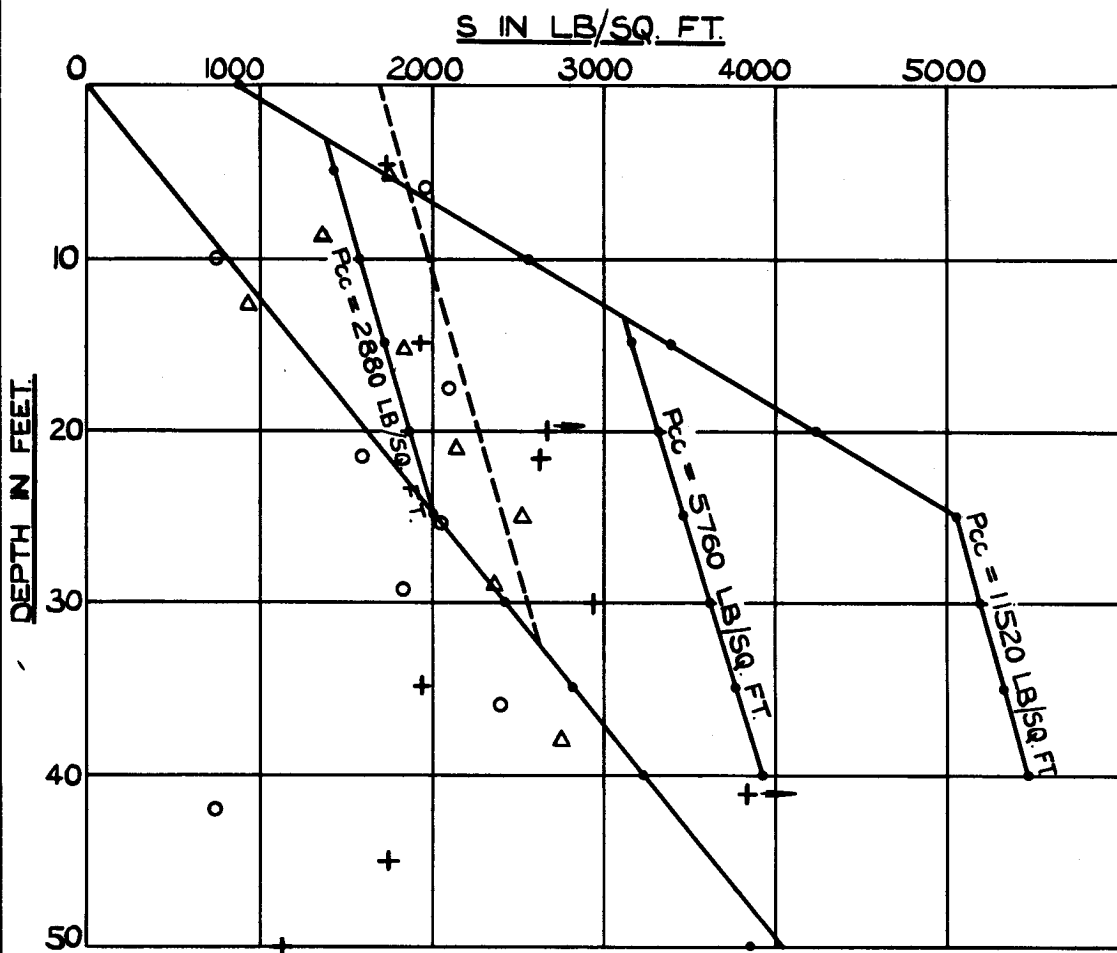


FIG. 66.

CALCULATED REBOUND STRENGTHS VS DEPTH, WITH
IN-SITU STRENGTHS.

TABLE XIII.

Calculated shear strength using laboratory results

h Depth in feet	$\sigma'_{cr} = h \gamma_t$		$\frac{S}{P_{cc} = 20}$ lb./sq.in. = 2880 lb./sq.ft.	$\frac{S}{P_{cc} = 40}$ lb./sq.in. = 5760 lb./sq.ft.	$\frac{S}{P_{cc} = 80}$ lb./sq.in. = 11,520 lb./sq.ft.
	lb./sq.in.	lb./sq.ft.	lb./sq.ft.	lb./sq.ft.	lb./sq.ft.
0	0	000	864	864	864
5	3.8	550	1442	1708	1708
10	7.6	1100	1588	2553	2553
15	11.5	1650	1735	3175	3396
20	15.3	2200	1881	3321	4239
25	19.1	2750	2027	3467	5051
30	22.9	3300	2425	3613	5196
35	26.8	3850	2832	3759	5343
40	30.6	4400	3238	3906	5489

keep the phreatic surface as low as possible. It should also be remembered that when the water table is at or near the surface, the values of σ'_v will be very low and the shear strength will be the same^{cr} as for a normally consolidated material.

Using equations (39) to (43) the average shear strength for this particular dam can be predicted, but for other cases the validity of this equation would have to be proved or a new one derived by doing in-situ shear tests and comparing these results with those of the laboratory tests, given in Fig. 65, in a similar way to Fig. 66.

CONCLUSIONS

- (a) It has been shown that slimes is a non-cohesive frictional material with an effective angle of internal friction $\phi' = 35^\circ$;
- (b) In-situ shear tests have shown that slimes in the field can have an apparent cohesion due to an increase in effective intergranular stress above that for a normally consolidated soil.
- (c) This increase in intergranular pressure is caused by desiccation and is due either to
 - (i) the negative pore water pressure acting as an increment of the intergranular pressure;
 - or (ii) the effects of over-consolidation by desiccation.
- (d) From (c) it follows that the drier the material is, the greater will be its shear strength.

RECOMMENDATIONS

1. To determine the shear strength of slimes

If it is necessary to determine the shear strength for slimes, for the purpose of stability analysis or for other reasons, the following are the suggested methods, in order of accuracy:

- (a) in-situ shear testing and a programme of laboratory shear testing as was used in this case. These tests will give the full picture for any particular slimes dam;
- (b) relating the information from in-situ testing with the information given in Figures 64 and 65. This method is slightly less accurate than (a) and assumes that all slimes have the same shear characteristics;
- (c) where no testing is to be done, an estimate can be made of the shear strength for the over-consolidated case, using equations (39), (40), (41), (42) and (43) but it is recommended that in these cases (d) be used;
- (d) where no in-situ testing is done to determine whether the material is over-consolidated or not, the safest approach is to use the normally consolidated case, using equation (42) and (43) to determine $\sigma'_{cr} = P_{cc}$.

2. To ensure a reasonable shear strength in a dam

- (e) As the strength of the slimes depends on desiccation, the recommendations here are the same as for obtaining a high preconsolidation pressure, namely to keep the walls of the dam as dry as possible by controlling seepage water by the use of underdrains, by removing excess water from the dam in the speediest manner and by building in thin layers which are allowed to dry out thoroughly.

CHAPTER VI. O V E R A L L S T A B I L I T Y:
S T A B I L I T Y A N A L Y S I S

Stability analysis implies the use of all known data concerning a material to predict the stability of a slope in that material for certain sets of conditions. Previous chapters have dealt with seepage through the dam and consolidation and shear strength of slimes; it now remains to apply the findings to stability analyses of slimes dams.

Every slimes dam wall or part thereof will be governed by conditions which may vary considerably. Therefore, to obtain an accurate picture of the stability of a dam, it would be necessary to carry out a separate stability analysis for each section. This chapter will indicate some general methods of approach to the problem.

Three approaches can be made, using three interpretations of the shear strength determined, as suggested in the previous chapter, namely:

- (a) treating the material as normally consolidated and using the effective stress parameters c' and ϕ' where $c' = 0$ and $\phi' = 35^\circ$; in other words, treating the slimes as purely frictional material;
- (b) averaging the shear strength measured in-situ and treating this strength as an apparent cohesion with no apparent angle of shearing resistance;
- (c) using equations of a similar type to equation (40) for overconsolidated material to give an apparent cohesion and an angle of shearing resistance.

Case (a) dealing with the purely frictional material can be considered as an infinite slope. For the cases of apparent cohesion it is better to treat the slimes dams as simple slopes of finite dimensions.

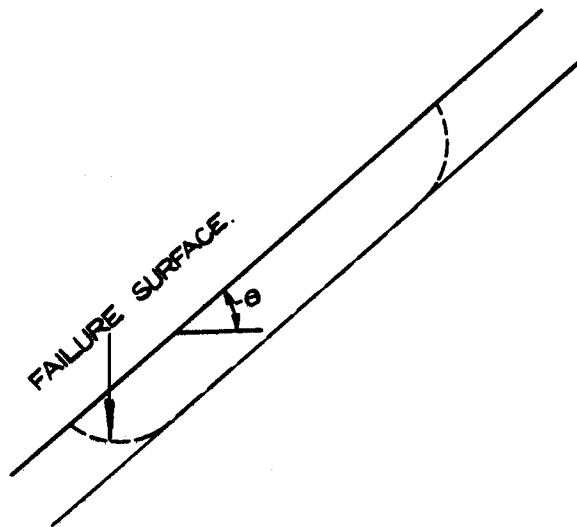


FIG. 67.

FAILURE OF AN INFINITE SLOPE.

INFINITE SLOPES

Taylor (24) defines an infinite slope as "a constant slope of unlimited extent which has constant conditions and constant soil properties at any given distance below the surface of the slope". Although no slope extends infinitely without variation, it is reasonable to assume that the theory governing infinite slopes in granular materials can be applied to slimes dams. The type of failure which occurs in infinite slopes is shown in Fig. 67. It can be proved that the slope will be stable when the material is dry, if the angle of the slope, θ , is less than the angle of friction ϕ' . Therefore, in the case of a dry slimes slope it will be stable if θ is less than 35° for a dam of any height.

If the factor of safety = F_s then the maximum permissible angle of slope θ for any factor of safety will be

$$\theta = \tan^{-1} \left(\frac{\tan 35^\circ}{F_s} \right) \dots\dots\dots (44)$$

Thus for factors of safety of $F_s = 1, 1.5, 2$ and 3 ; θ will be $35^\circ, 25^\circ, 19^\circ$ and 13° respectively for the dry case.

It is known, however, that often a varying quantity of seepage flow takes place through the walls. In applying seepage flow to infinite slopes, the whole slope is considered to be flowing full of seepage water with the upper flow line coincident with the surface. In this case Taylor shows that

$$\theta = \tan^{-1} \left(\frac{\gamma_s}{\gamma_t} \tan \phi \right) \dots\dots\dots (45)$$

where θ = steepest infinite slope

γ_t = total density of saturated soil

γ_s = bouyant density of soil

ϕ = friction angle of soil

Applying this to slimes

$$\gamma_t = 120 \text{ lb./cu.ft.}$$

$$\gamma_s = 58 \text{ lb./cu.ft.}$$

$$\phi = 35^\circ$$

$$\begin{aligned} \therefore \theta &= \tan^{-1} \left(\frac{58}{120} \times 0.7002 \right) \\ &= 18^\circ 40' \end{aligned}$$

This means that for the case where the whole wall is filled with seepage flow, as in Fig. 32, the greatest slope, (that for which the factor of safety is unity), will be $18^{\circ}40'$ for a dam of any height.

Factors of safety are used in the same way as in equation (44) and the maximum permissible slope becomes

$$\theta = \tan^{-1} \left(\frac{\tan 18^{\circ}30'}{F_s} \right) \dots\dots\dots (46)$$

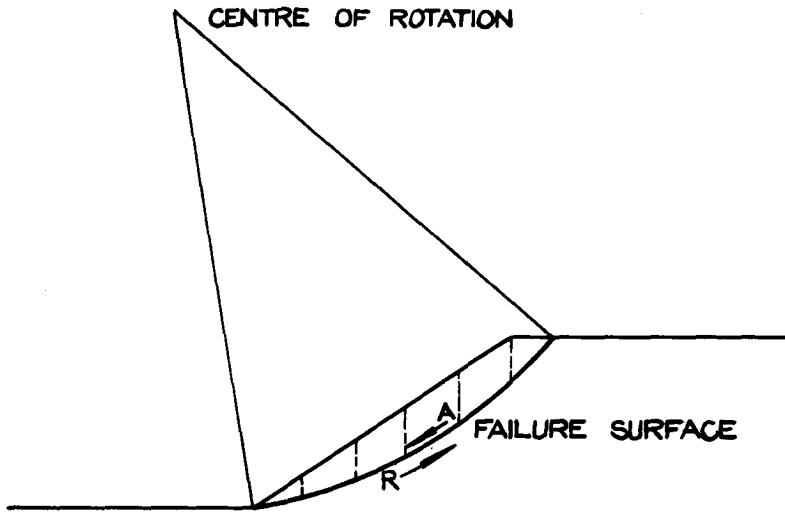
and thus for the case of full seepage flow and factors of safety of $F_s = 1, 1.5, 2$ and 3 ; θ will be $18.5^{\circ}, 12.5^{\circ}, 9^{\circ}30'$ and 6° respectively. It is further evident that for the cases lying between the full seepage condition and the completely dry condition, the greatest slope will be between $18^{\circ}40'$ and 35° . To determine the exact value of the safe angle of slope it would be necessary to know the exact position of the phreatic surface. Alternatively, if no seepage water enters the possible zone of failure ~~and~~ then 35° would be the greatest slope.

This is the case for infinite slopes: an explanation of the precise effect of seepage forces on stability will be given in the following section dealing with simple slopes.

SIMPLE SLOPES

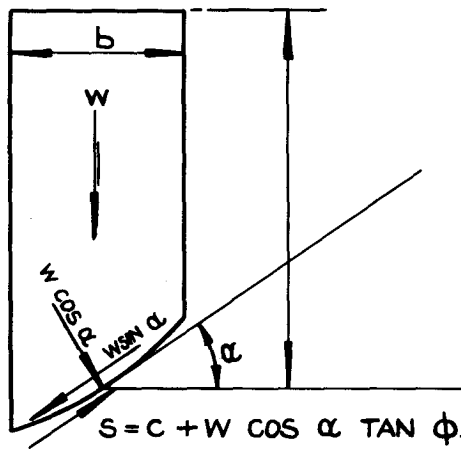
A simple slope is defined as one of constant inclination where the ground is level at the foot and at the top of the slope as shown in Fig. 68(a). When a shear failure occurs, a portion of the slope slides out along a failure surface. It has been shown that in reasonably homogeneous materials this failure surface can be considered as an arc of a circle, as shown in Fig. 68(a), and this circle is known as a slip circle. During failure, the wedge of material above the failure plane rotates about the centre of this circle.

The stability analysis is carried out either by considering moments about this centre of rotation or by resolving the forces acting along the failure plane. The force tending to cause failure is known as the actuating force and consists of the portion of the weight of the wedge which acts tangentially to the failure surface shown in Fig. 68(a) as A. The force resisting sliding and known



(a)

SIMPLE SLOPE WITH FAILURE SURFACE.



(b)

FORCES ACTING ON ONE SLICE.

FIG. 68.

SLIP CIRCLE AND METHOD OF SLICES.

as the resisting force, R , is made up of the cohesive and frictional shear strength acting along the failure surface. The factor of safety of the slope is $\frac{R}{A}$, and if $\frac{R}{A} < 1$ the slope should have failed.

The accepted method of determining the actuating and resisting forces is to divide the failure wedge into vertical slices, as shown in Fig. 68(a) by the dotted lines and to add the relevant forces acting on the slices. An enlarged diagram for one slice is shown in Fig. 68(b). The weight of the slice acting vertically downwards through its centroid is W . The angle of inclination of the failure surface is α and, therefore, the tangential component of the weight is $W \sin \alpha$ and the force component acting normal to the failure plane is $W \cos \alpha$.

The element contributing to the Actuating Force = $W \sin \alpha$ and from equation (13) the shear strength is $S = c' + \sigma' \tan \phi$ where σ' is the intergranular pressure normal to the plane of failure. Considering Forces for this slice

$$S = C' + W \cos \alpha \tan \phi$$

where C' is the total effective cohesion force in the slice and this is the resisting force generated in the slice considered. Summing, for all the slices in a wedge

$$\text{Actuating Force} = \sum W \sin \alpha \dots\dots\dots (47)$$

$$\text{and Resisting Force} = \sum (C' + W \cos \alpha \tan \phi) \dots (48)$$

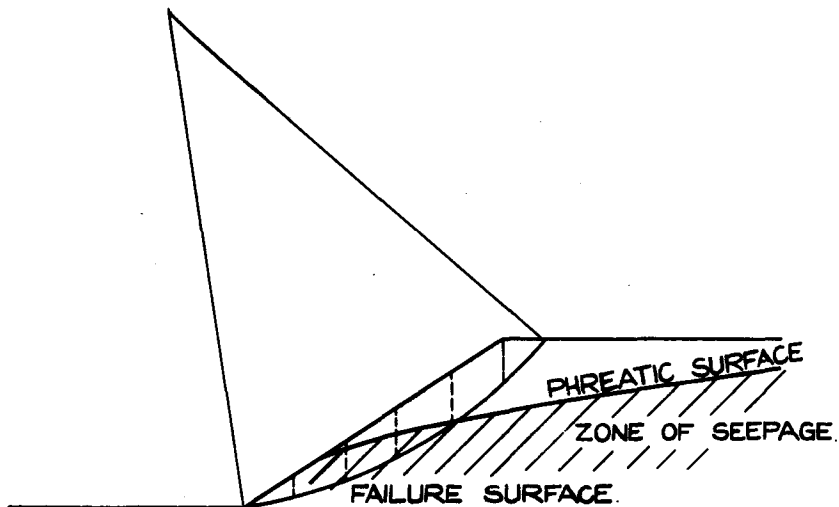
This is for the case where there is no seepage flow through the material and the material is not submerged. The weight of the slice, W , will be

$$W = b (h \gamma_t) \text{ plus any additional external load on the slice } \dots\dots\dots (49)$$

where γ_t is the average total density of the slimes in the slice.

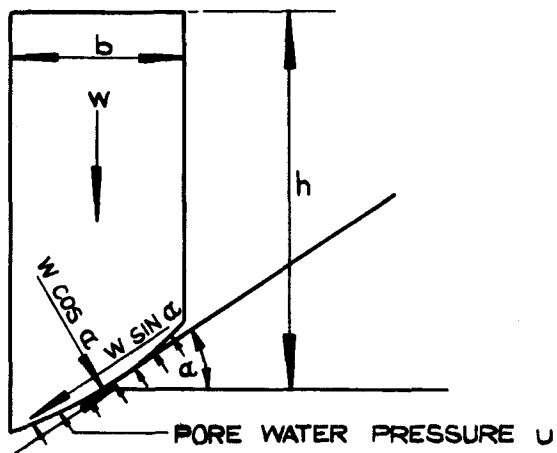
In the case of steady seepage where the whole or part of the slip circle lies below the phreatic surface, as in Fig. 69(a), the pore water pressures due to the seepage water must be taken into consideration.

Considering the slice in Fig. 69(b), the weight of the slice is again W and the tangential component of W is $W \sin \alpha$ and, therefore, the component of the actuating force = $W \sin \alpha$.



(a)

SIMPLE SLOPE WITH FAILURE SURFACE AND PHREATIC SURFACE.



$$S = c' + (W \cos \alpha - \frac{b}{\cos \alpha} u) \tan \phi'$$

(b)

FORCES ACTING ON ONE SLICE.

FIG. 69.

SLIP CIRCLE ANALYSIS WITH SEEPAGE FLOW.

The component of the weight of the slice acting normal to the failure plane = $W \cos \alpha$.

However, there is also pore water pressure u , acting normal to the failure surface and in the opposite direction to $W \cos \alpha$. As the length of the failure plane in the segment is $\frac{b}{\cos \alpha}$, the force due to pore pressure is $\frac{b}{\cos \alpha} u$.

∴ The normal intergranular force on the failure plane = $(W \cos \alpha - \frac{b}{\cos \alpha} u)$ and, therefore, the element of the resisting force due to the slice will be $S = C' + (W \cos \alpha - \frac{b}{\cos \alpha} u) \tan \phi$, and summing for all slices:

$$\text{Actuating Force} = \sum W \sin \alpha \dots\dots\dots (50)$$

$$\text{and Resisting Force} = \sum [C' + (W \cos \alpha - \frac{b}{\cos \alpha} u) \tan \phi] \dots\dots (51)$$

where W is defined as in equation (47).

Comparing equations (47) and (48) with equations (50) and (51) it will be seen that the actuating force in both equations (47) and (50) will be the same, while the resisting force in equation (51) is less than that in equation (48) for a positive pore water pressure and more than equation (48) for a negative pore water pressure. This latter case has to be ignored at present as there is no way of determining or predicting negative pore water pressures as yet.

In analysing any slope, a number of possible slip circles can be drawn through the slope and it is necessary to find out which is the worst case for the slope and conditions considered. Slip circles need not necessarily pass through the toe of the wall as shown in Figures 68(a) and 69(a) but can pass below the toe as shown in Fig. 70. In Fig. 70 a set of circles with a radius of 100 ft. have been drawn through an embankment 40 ft. high with a shear strength of 1000 lb./sq.ft. It will be seen that 6 circles were drawn and their factors of safety were plotted. In this case the worst circle was found to have a minimum factor of safety of 1.33. In order to determine the worst circle for this slope, additional sets of slip circles of different radii would have to be considered to determine the worst circle for those radii and, by comparison of the worst circles for the different radii, the worst circle for the embankment will be found.

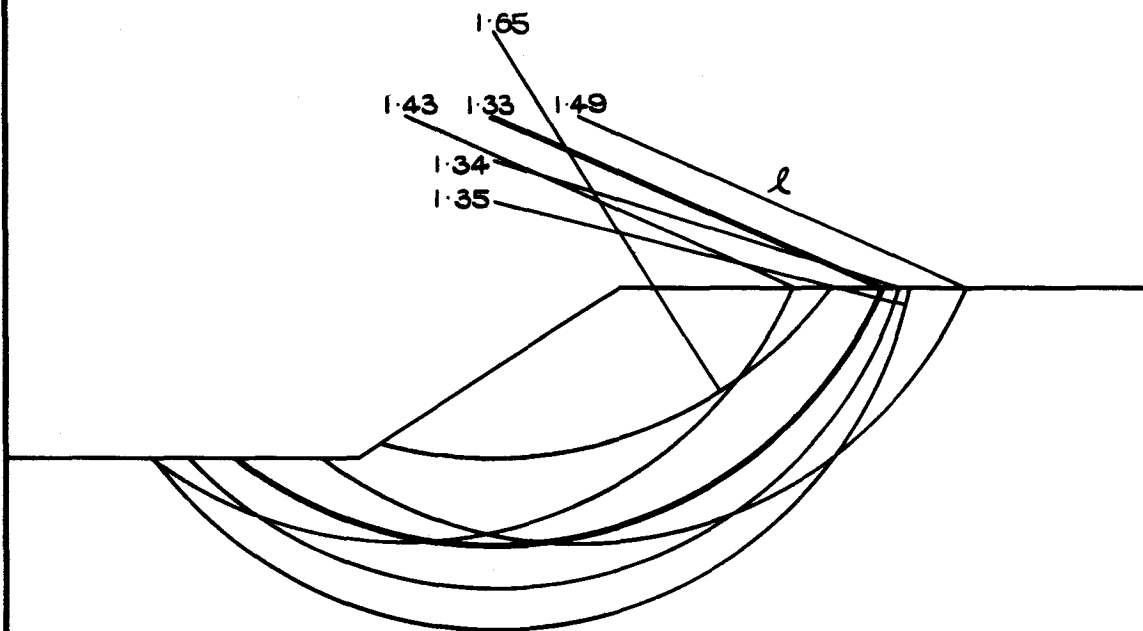


FIG. 70.

SLIP CIRCLES AND FACTORS OF SAFETY FOR A
RADIUS OF 100 FEET.

STABILITY NUMBERS

Taylor (25) and others have analysed a large number of slip circles and have reduced their results to a set of curves from which the stability of an embankment without seepage can be found. These charts give the relationship between the stability number, the angle of the slope and the friction angle of the material where the stability number is $c/\gamma H$ which covers the cohesion, the density of the material and the height of the slope. It will thus be seen that all the relevant variables influencing the stability of the slope are used in these calculations and either the factor of safety against failure of the slope can be determined or, assuming a certain factor of safety, one of the other variables can be determined. Thus the conditions for the worst circle are determined without having to analyse a large number of slip circles.

These stability numbers have been determined not only for failure through the toe but also for the worst circle passing below the toe, in which case a depth factor is also introduced into the chart.

FAILURE THROUGH THE TOE OF A SLIMES DAM WITH NO SEEPAGE

For the purposes of these calculations the material is considered to be saturated although there is no seepage flow and the total density γ_t is taken as 120 lb./cu.ft.

Considering the material as having an apparent cohesion c and no apparent friction, the stability numbers for angles of slopes of 15° , 22.5° , 30° , 45° and 60° were read from the charts. The maximum heights for cohesions of 1000, 2000, 3000 and 4000 lb./sq.ft. and factors of safety of 1, 1.5, 2 and 3 were calculated and these results are tabulated in Table XIV.

These results are plotted on the semi-polar diagrams in Figures 71, 72, 73 and 74. As would be expected, it is seen that for any given strength and factor of safety the permissible height will increase as the angle of the slope is decreased.

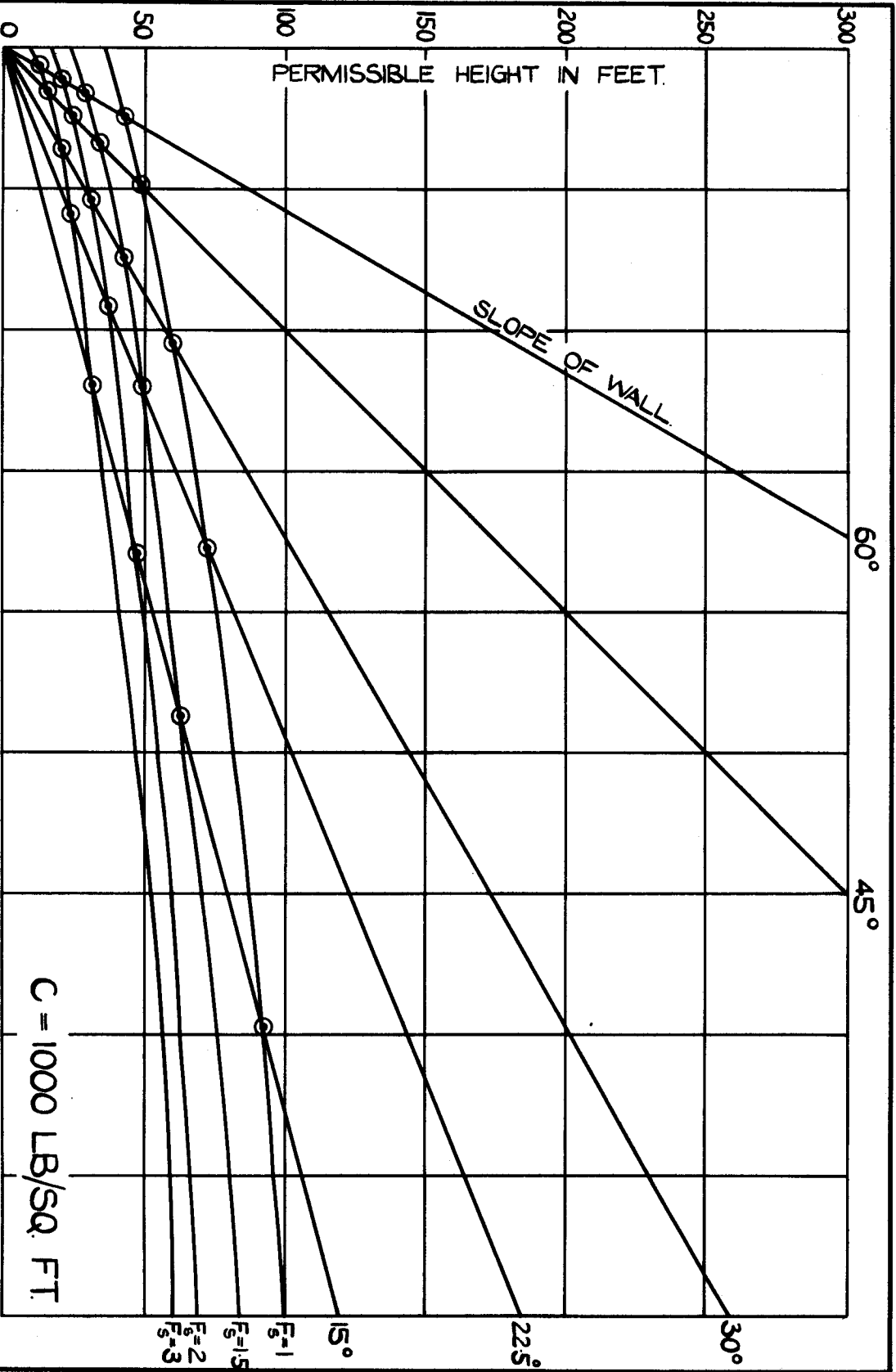


FIG. 71.

PERMISSIBLE HEIGHTS FOR COHESION OF 1000 LB/SQ. FT.

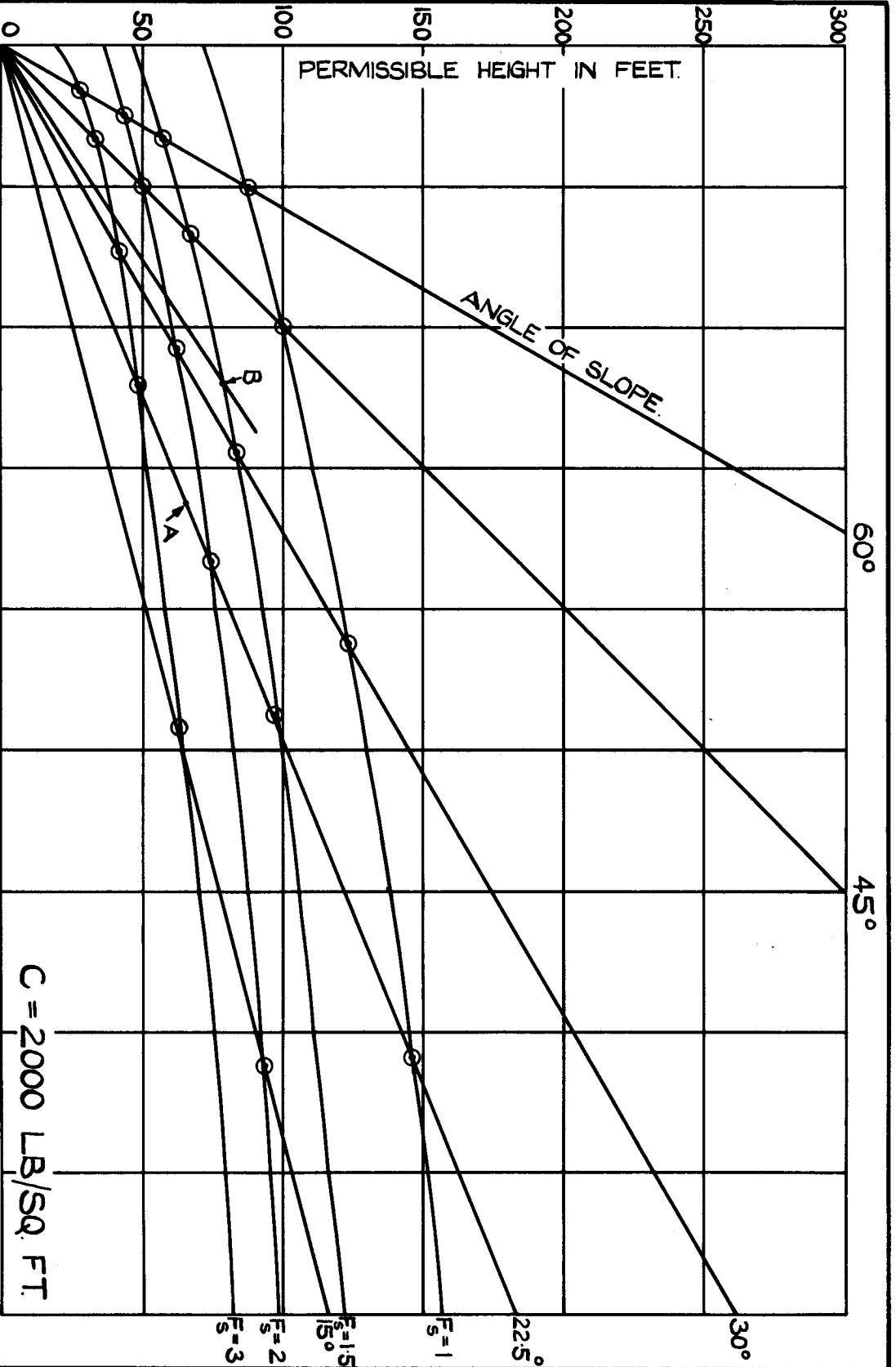


FIG. 72.

PERMISSIBLE HEIGHTS FOR COHESION OF 2000 LB/SQ. FT.

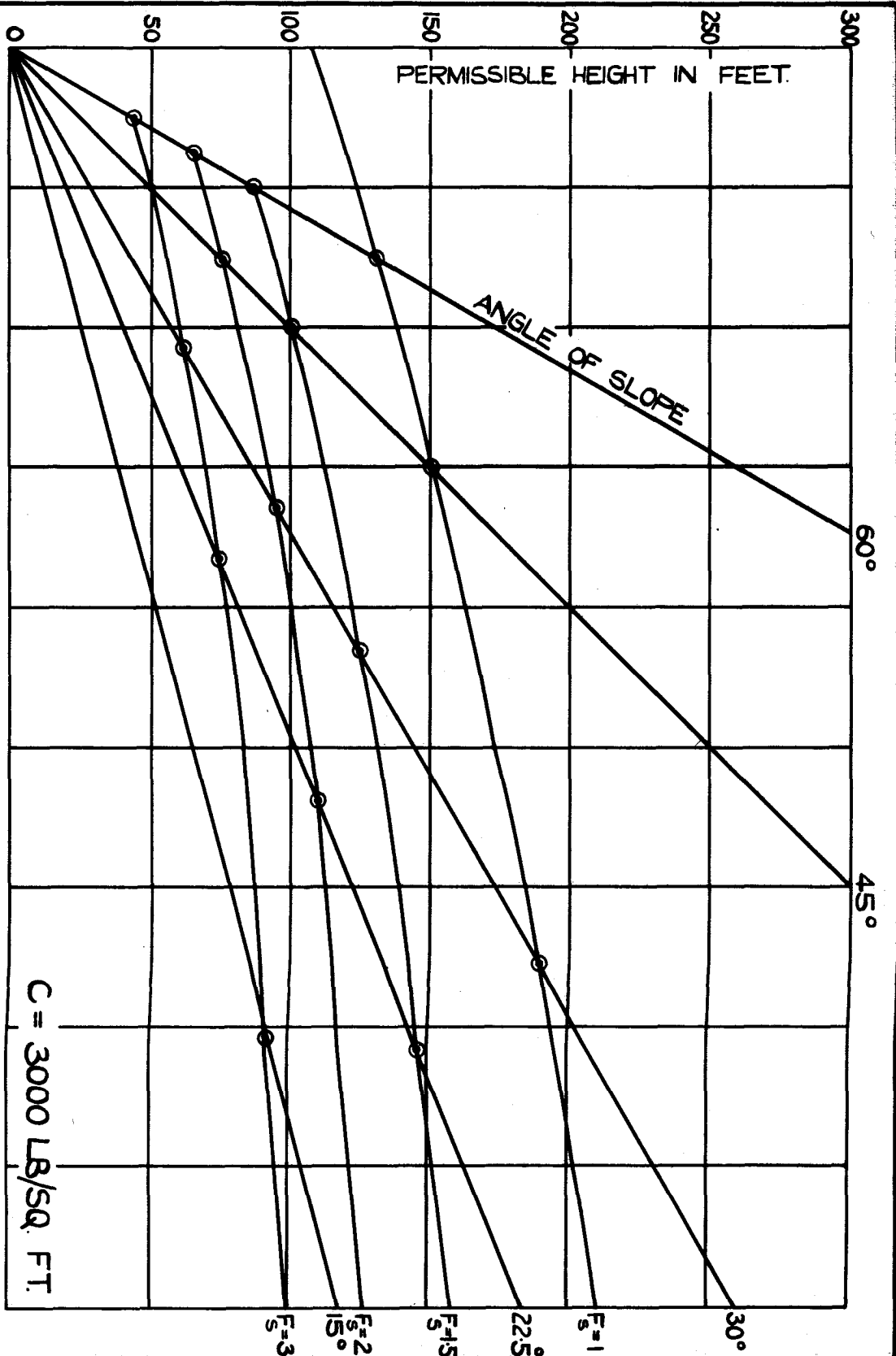


FIG. 73.

PERMISSIBLE HEIGHTS FOR A COHESION OF 3000 LB/SQ. FT.

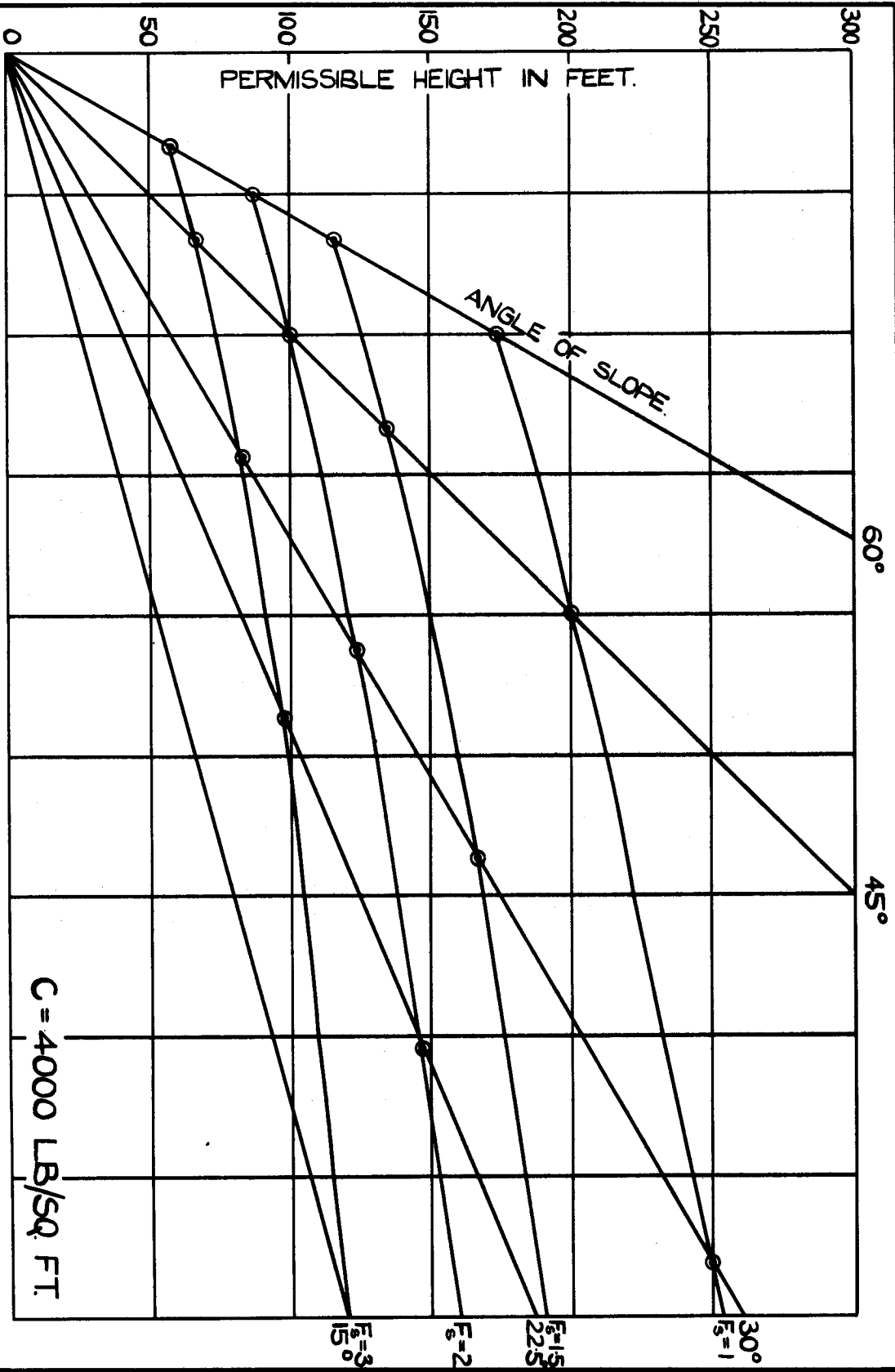


FIG. 74.

PERMISSIBLE HEIGHTS FOR A COHESION OF 4000 LB/SQ. FT.

TABLE XIV.

Calculated safe heights in feet for slip circles
passing through the toes of slimes dams for
various cohesions and factors of safety

Slope θ	Stability No. $c/\gamma H$	c = 1000 lb./sq.ft.				c = 2000 lb./sq.ft.			
		$F_s=1$	$F_s=1.5$	$F_s=2$	$F_s=3$	$F_s=1$	$F_s=1.5$	$F_s=2$	$F_s=3$
60°	0.191	43	29	22	14	87	58	44	29
45°	0.166	50	34	25	16	100	67	50	34
30°	0.134	62	42	31	20	124	83	62	42
22.5°	0.114	73	48	36	24	146	98	73	48
15°	0.089	94	62	47	31	187	125	94	12
		c = 3000 lb./sq.ft.				c = 4000 lb./sq.ft.			
60°	0.191	131	87	66	44	174	116	87	58
45°	0.166	150	100	75	50	200	133	100	67
30°	0.134	186	124	93	62	249	166	124	83
22.5°	0.114	219	146	110	73	292	195	146	98
15°	0.089	281	188	140	94	374	249	188	125

The average shear strength or apparent cohesion in Fig. 60 can be taken as 2000 lb./sq.ft. and, therefore, Fig. 72 applies to this case. The tests were carried out when the dam was 65 feet high and the average slope was $22\frac{1}{2}^{\circ}$. This situation is designated by point A in Fig. 72 and it will be seen that under these circumstances the factor of safety is about 2.3.

Similar curves can be set up for the shear strength derived from equations (31), (34), (37) and (40) where angle of shearing resistance is 15° , apparent cohesion is 1295, 2740, 4320 and 1700 lb./sq.ft. respectively, and $\gamma_t = 120$ lb.cu.ft.

Calculations were carried out as for the previous case and the results are tabulated in Table XV and plotted in Figures 75, 76, 77 and 78.

It will be seen that here the increase in permissible height, for any strength and factor of safety, with decrease in slope, is greater than for the pure cohesive case.

Once again the field strength plotted in Fig. 60 and governed by equation (40) is used and the point A in Fig. 76 again designates a height of 65 ft. and slope of $22\frac{1}{2}^{\circ}$. In this case the factor of safety for the slope is well above 3, approximately 5.

The difference in the values of the factor of safety for the two treatments of this case lies in the fact that the first treatment used average in-situ strength with, it is almost certain, some seepage flow in the lower part of the wall, while the second treatment was based on predicted shear strength using a formula for a dry wall.

TABLE XV.

Permissible heights in feet based on shear strength
equations for preconsolidated slimes

Slope θ	Stability Number $c/\gamma H$	Equ. (31) $P_{cc} = 2880$ $c = 1295 \text{ lb./sq.ft.}$				Equ. (40) $P_{cc} = 3500$ $c = 1700 \text{ lb./sq.ft.}$			
		$F_s=1$	$F_s=1.5$	$F_s=2$	$F_s=3$	$F_s=1$	$F_s=1.5$	$F_s=2$	$F_s=3$
60°	0.117	92	61	46	31	121	81	61	40
45°	0.082	131	88	66	44	173	115	86	58
30°	0.046	234	156	117	78	308	205	154	102
22.5°	0.025	431	288	216	144	566	377	283	188
15°	0	∞	in- determinate			∞	indeterminate		
		Equ. (34) $P_{cc}=5760$ $c = 2740 \text{ lb./sq.ft.}$				Equ. (37) $P_{cc}=11,520$ $c = 4320 \text{ lb./sq.ft.}$			
60°	0.117	195	130	98	65	308	205	154	103
45°	0.082	278	185	139	93	439	293	220	146
30°	0.046	495	330	248	165	782	521	391	261
22.5°	0.025	911	608	455	304	1440	960	720	480
15°	0	∞	in- determinate			∞	indeterminate		

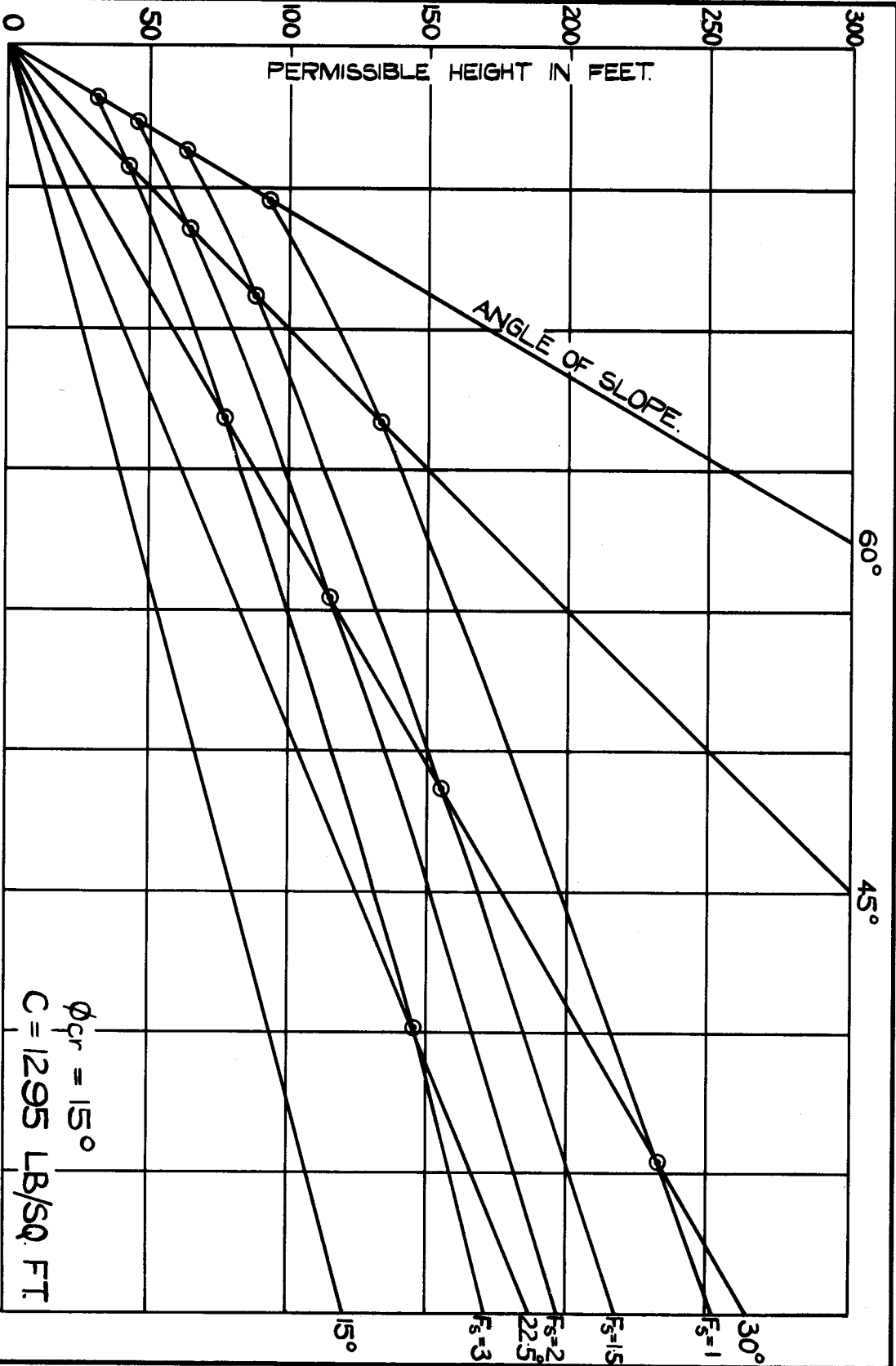


FIG. 75.

PERMISSIBLE HEIGHTS FOR SHEAR STRENGTH CALCULATED FROM EQUATION 31.

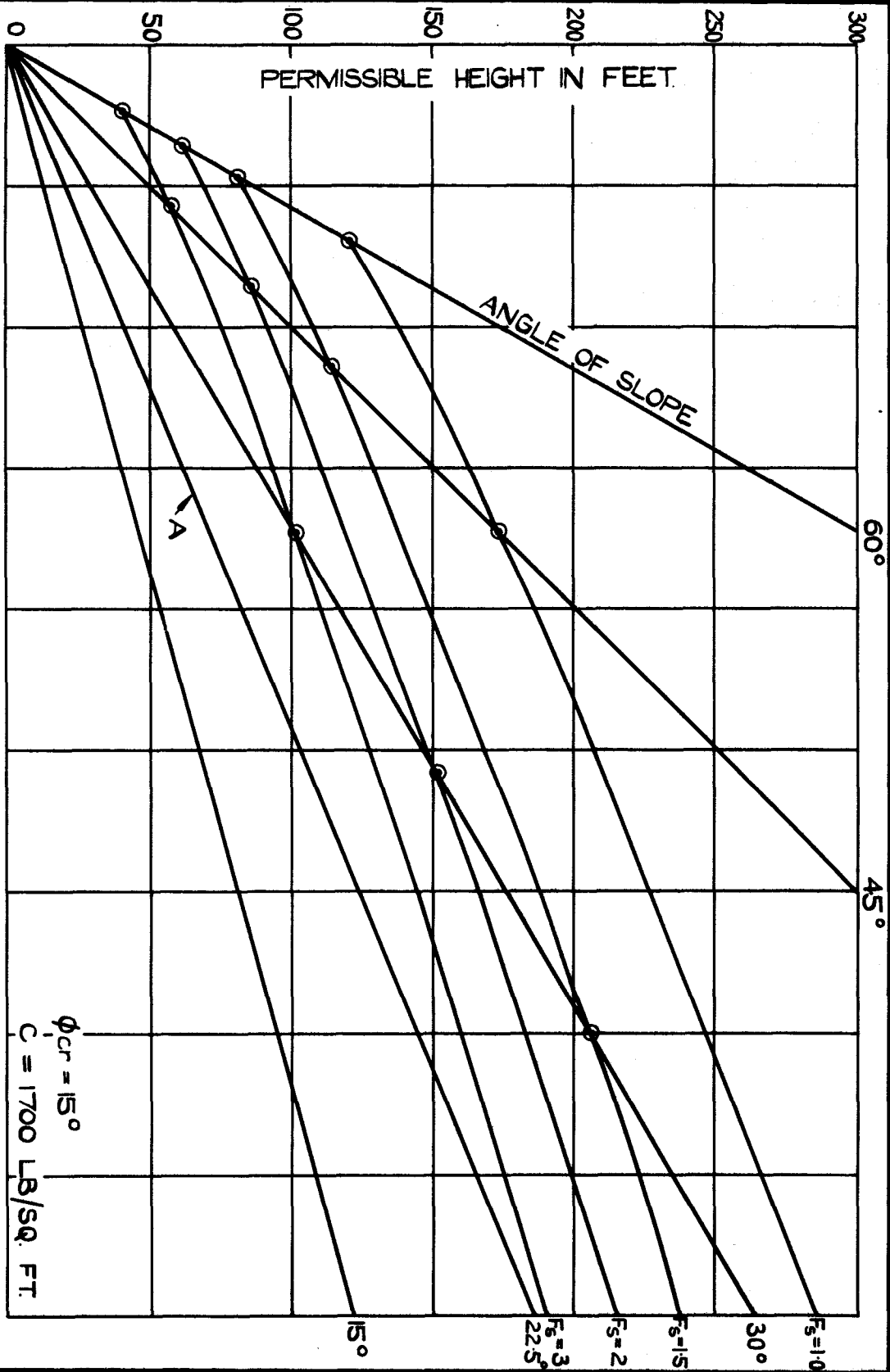


FIG. 76.

PERMISSIBLE HEIGHTS FOR SHEAR STRENGTH CALCULATED FROM EQUATION 40.

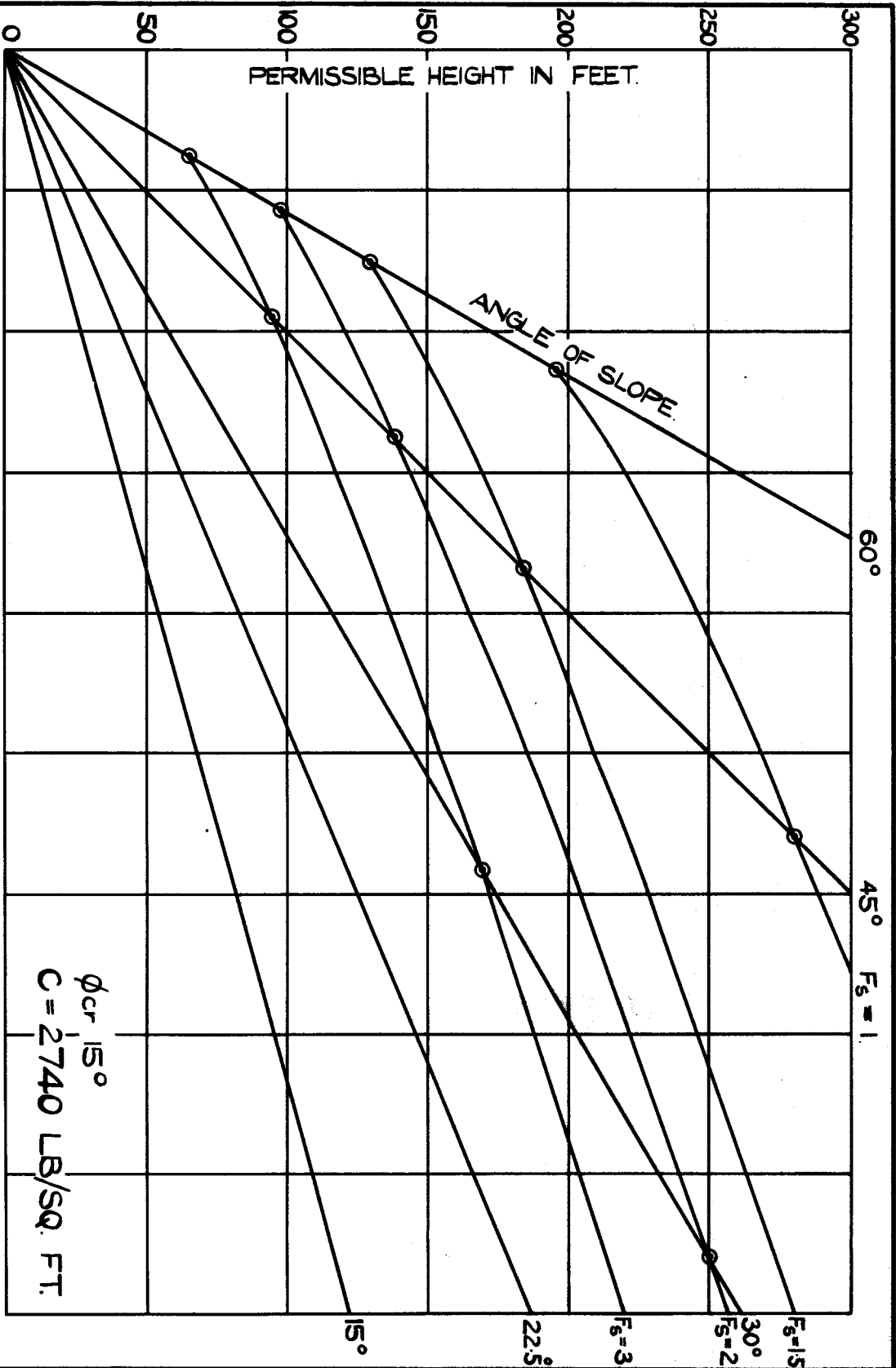


FIG. 77.

PERMISSIBLE HEIGHTS FOR SHEAR STRENGTH CALCULATED FROM EQUATION 34.

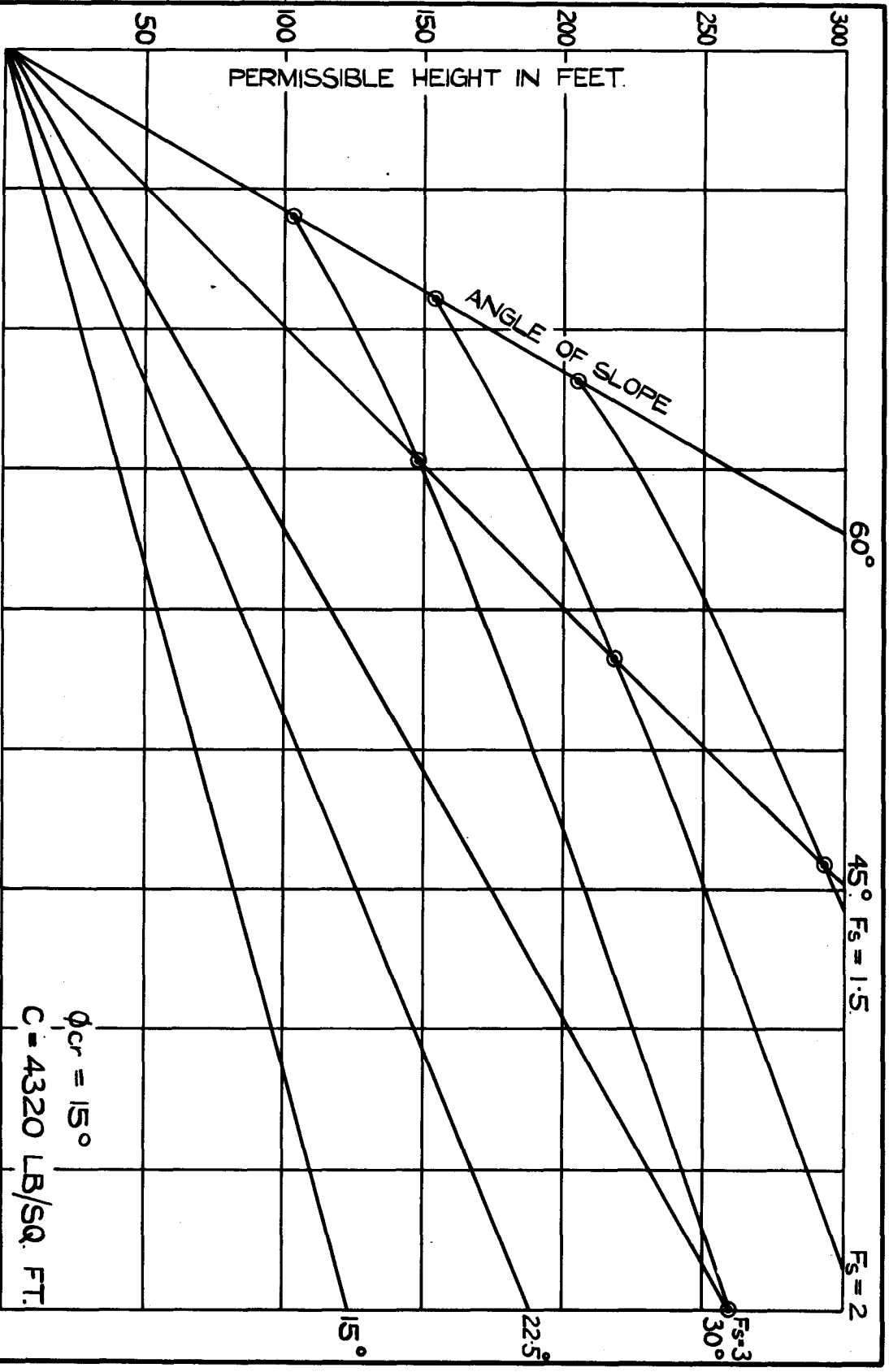


FIG. 78.

PERMISSIBLE HEIGHTS FOR SHEAR STRENGTH CALCULATED FROM EQUATION 37.

FAILURE THROUGH THE TOE OF A SLIMES DAM WITH SEEPAGE FLOW

Without knowing the flow net for any particular case and analysing the stability of that case by the method of slices with pore water pressure allowance, no real assessment of the effect of pore water pressure on the stability of the slope can be obtained.

Studying equations (48) and (51) it would appear that pore water pressure would have no effect on the stability of slopes where the shear strength is made up only of cohesion. This is true for the case where the effective shear parameters c' and ϕ' are used and where $\phi' = 0$. Slimes material has $c' = 0$ and $\phi = 35^\circ$ and the in-situ strengths give only an apparent cohesion. This apparent cohesion can only be used for determining the stability of the embankment under the conditions obtaining at the time of testing. Thus apparent cohesions measured in-situ on a dry slope can only be used for stability analysis of the dry slope, and if the apparent cohesions are measured in-situ on a slope under full seepage flow, they can only be used for analysing the slope under full seepage flow.

It is suggested that allowance can be made for seepage flow in stability number analysis by modifying the angle of internal friction for various conditions of seepage.

Comparing equation (48)

Resisting Force = $\sum (C' + W \cos \alpha \tan \phi')$ for the dry case with the equation (51) for the seepage case.

$$\text{Resisting Force} = \sum \left[C' + \left(W \cos \alpha - \frac{b}{\cos \alpha} u \right) \tan \phi' \right]$$

it will be seen that the frictional force is reduced in the seepage case by $\sum \left(\frac{b}{\cos \alpha} u \right) \tan \phi'$. The same effect can be obtained by equating equation (51) to an equation using a modified angle of shearing resistance ϕ_m applied to the total pressure on the failure plane.

$$\begin{aligned} \text{Resisting Force} &= \sum \left[C' + \left(W \cos \alpha - \frac{b}{\cos \alpha} u \right) \tan \phi' \right] \\ &= \sum (C' + W \cos \alpha \tan \phi_m) \dots \dots \dots (52) \end{aligned}$$

$$\text{where } \sum \left(W \cos \alpha - \frac{b}{\cos \alpha} u \right) \tan \phi' = \sum W \cos \alpha \tan \phi_m.$$

and, therefore,

$$\tan \phi_m = \frac{\sum(W \cos \alpha - \frac{b}{\cos \alpha} u)}{\sum W \cos \alpha} \tan \phi'$$

$$\text{and } \phi_m = \tan^{-1} \left[\frac{\sum(W \cos \alpha - \frac{b}{\cos \alpha} u)}{\sum W \cos \alpha} \tan \phi' \right] \dots\dots (53)$$

using this value of ϕ_m instead of ϕ' the stability number can be read from the charts.

Although the stability number charts hold strictly for effective stress parameters they will also hold for equations (31), (34), (37) and (40) as these equations are based on the apparent intergranular rebound pressure.

As an example, apply equations (52) and (53) to the case using equation (40).

$$S = 1700 + \sigma'_{cr} \tan 15^\circ$$

Assuming the pore water pressure factor, $F_u =$

$$\frac{\sum(W \cos \alpha - \frac{b}{\cos \alpha} u)}{\sum W \cos \alpha} \text{ to have values of } 0.5, 0.75 \text{ and } 1 \text{ and}$$

using factors of safety $F_s = 1, 2$ and 3 the permissible heights in Table XVI were calculated. Where $F_u = 1$, $\phi_m = \phi'$, i.e. the dry case.

The safe heights for a factor of safety of 2 and for all three values of F_u are plotted in Fig. 79 which shows the effect of the pore water pressure on the permissible height of the wall for various slopes.

It will be seen that the effect of the pore water factor F_u is greater on the flatter slopes than on the steeper slopes. This is due to the fact that the frictional strength of the material plays a greater role in the stability of the flatter slopes than in the stability of the steep slopes. As an example; the permissible height at a slope of 30° and a factor of safety of 2 , for a dry wall, i.e. $F_u = 1$, would be 154 feet while if $F_u = 0.5$ the permissible height would only be 79 feet.

Before an accurate prediction of the stability of a slope can be made, it will first be necessary to know the value of F_u . If seepage occurs, the worst case is represented by $F_u = 0.5$ and this value should be used for determining ϕ_m the modified angle of shearing resistance unless the seepage conditions can be reasonably assessed.

TABLE XVI.

Safe heights calculated making allowance for pore water pressure, for slip circles passing through toe and strength according to equation (40)

Slope θ	$c = 1700 \text{ lb./sq.ft.}$			
	$F_u = 0.5, \phi_m = 7^\circ 40'$			
	c / γ_{tH}	$F_s = 1$	$F_s = 2$	$F_s = 3$
60°	0.150	94	47	31
45°	0.120	118	59	39
30°	0.900	157	79	52
22.5°	0.700	202	101	67
15°	0.400	354	177	118
		$F_u = 0.75, \phi_m = 11^\circ 18'$		
60°	0.130	109	54	36
45°	0.100	141	71	47
30°	0.063	224	112	75
22.5°	0.043	329	164	110
15°	0.013	1088	544	363
		$F_u = 1, \phi_m = \phi' = 15^\circ$		
60°	0.117	121	61	40
45°	0.082	173	86	58
30°	0.046	308	154	102
22.5°	0.025	566	283	188
15°	0	∞	Indeterminate	

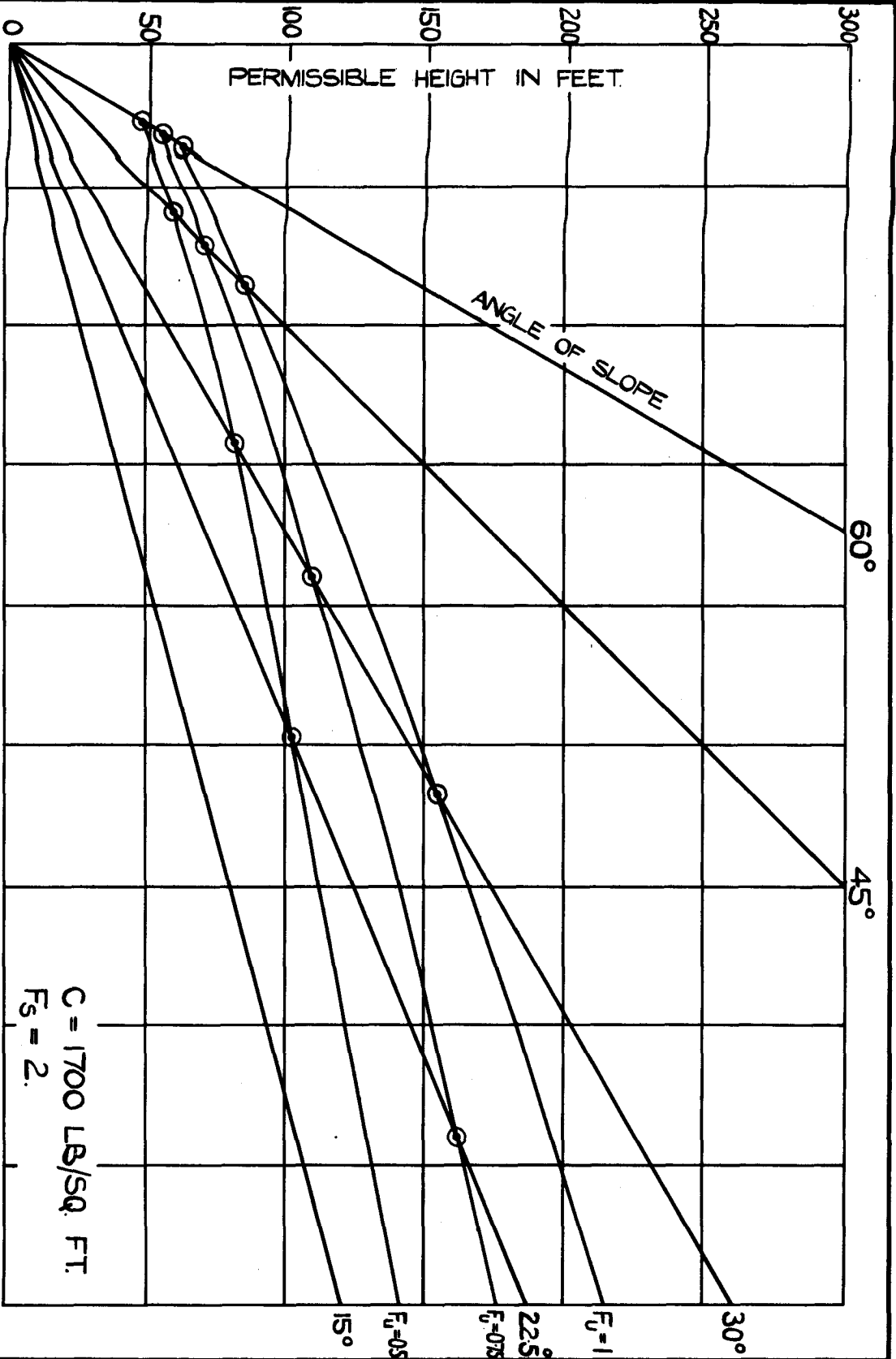


FIG. 79.

PERMISSIBLE HEIGHTS FOR VARIOUS PORE WATER FACTORS F_u .

FAILURE THROUGH THE FOUNDATION SOIL

In all the cases so far dealt with, the worst slip circle has been assumed to pass through the toe of the wall. It can be generally stated that this will occur if the shear strength of the foundation soil is greater than the shear strength of the slimes. However, if the shear strength of the foundation soil is the same or less than that of the slimes, the worst circle will pass through the foundation soil below the toe of the wall, as shown in Fig. 70. In these cases the resisting force will be a combination of the shear strengths of the slimes and of the foundation soil. A series of slip circles similar to those in Fig. 70 were drawn and analysed for the section shown in Fig. 70 where the embankment has a slope of 33° .

These circles were analysed to find the worst circles for the cases where the foundation shear strength (S_f) was equal to the shear strength of slimes (S_s) and where $S_f = \frac{1}{2} S_s$ and also for $S_f = 1000, 2000, 3000$ and 4000 lb./sq.ft. Assuming these values, the minimum shear strength of the slimes required to give a factor of safety of 1.5 was determined.

The results of these analyses are tabulated in Table XVII.

The results for the various foundation strengths are plotted in Fig. 80. From this graph, the values for slimes strengths of 1000, 2000, 3000 and 4000 lb./sq.ft. have been interpolated and plotted in Fig. 81.

The most striking fact in Fig. 81 is that a change in the strength of the slimes has a much smaller effect on the permissible height of the dam than the same change in the shear strength of the foundation soil would have. This is due to the fact that the worst circle in these cases would be very deep and the length of the failure arc passing through the slimes is very small compared to the length of the failure arc passing through the foundation soil. It can therefore be seen that the shear strength of the foundation soil plays a very important part in the stability of the slimes dams.

Where seepage flow occurs in the foundation soil, the stability analyses must take this factor into consideration. In all the analyses so far the slimes and the foundation soil have been treated as homogeneous materials with uniform shear characteristics.

TABLE XVII.

Minimum shear strength of slimes for a factor
of safety of 1.5 on a slope of 33°

Height of dam in ft.	Shear strength of slimes lb./sq.ft.					
	$S_f = S_s$	$S_f = \frac{1}{2}S_s$	$S_f = 1000$	2000	3000	4000
20	570	1,020	-	-	-	-
40	1,140	2,030	2,160	-	-	-
60	1,710	3,050	7,660	-	-	-
80	2,280	4,060	13,160	4,375	-	-
100	2,850	5,080	18,660	9,825	1,000	-
120	3,420	6,090	24,160	15,275	6,450	-
140	3,990	7,110	29,660	20,729	11,900	4,000
160	4,560	8,120	34,160	26,175	17,350	9,500
180	5,130	9,140	39,660	31,625	22,800	15,000
200	5,700	10,150	44,160	37,075	27,300	20,500

$F_s = 1.5$ PERMISSIBLE HEIGHT IN FEET.

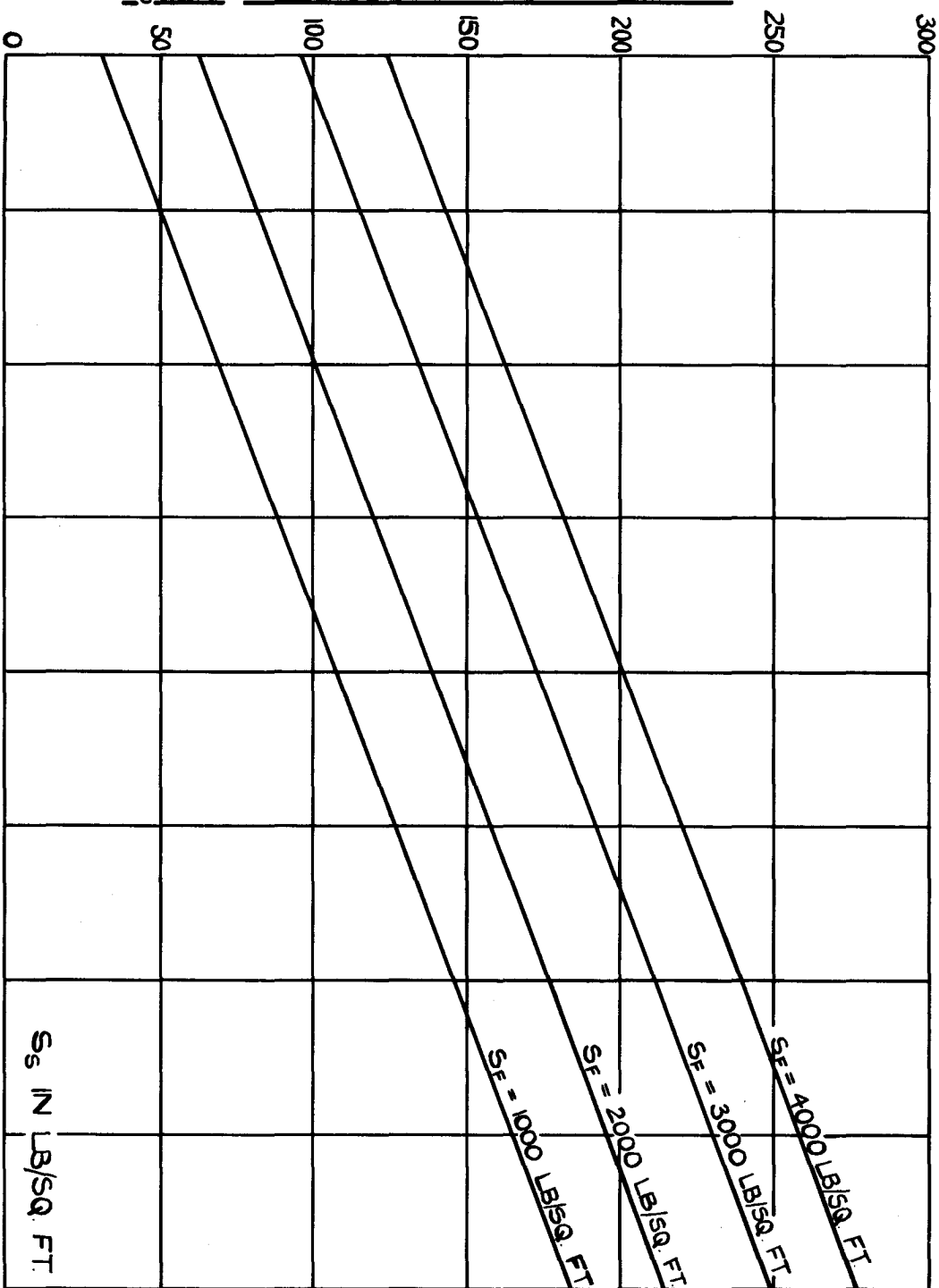
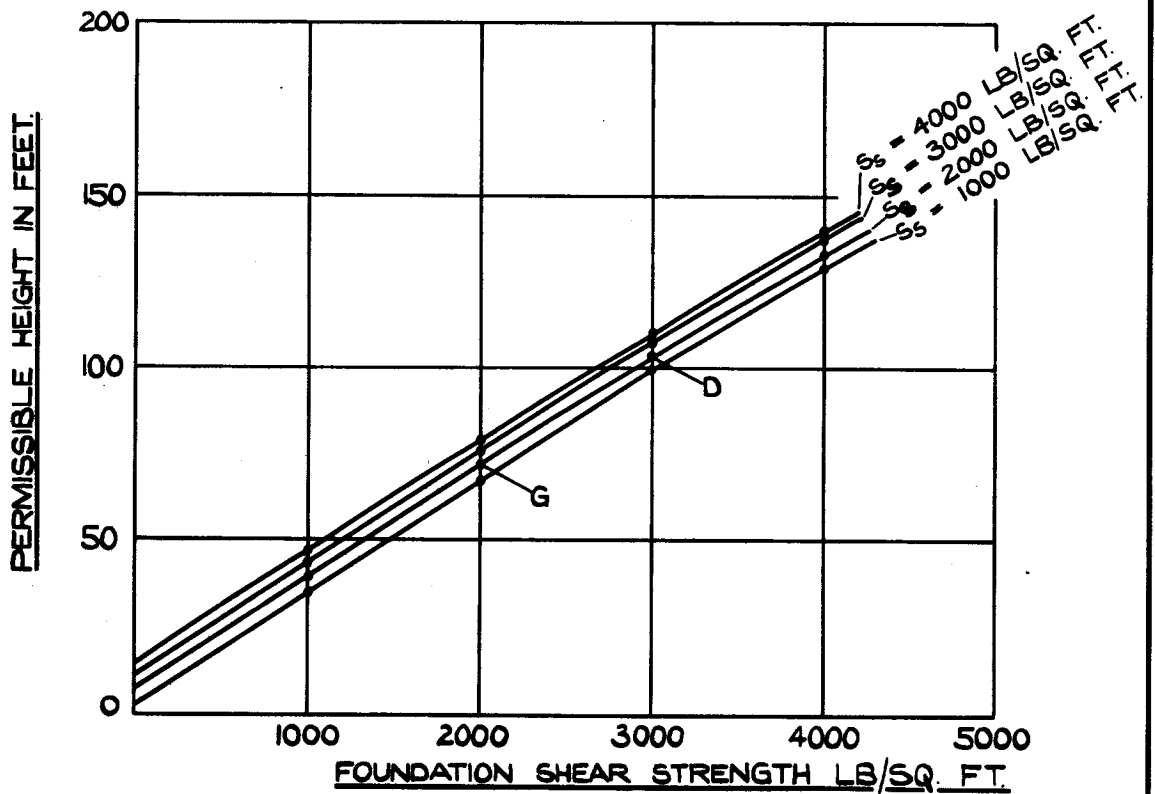


FIG. 80.

RELATIONSHIP BETWEEN PERMISSIBLE HEIGHT AND SHEAR STRENGTHS
OF SLIME AND THE FOUNDATION SOIL FOR A SLOPE OF 33° .



RELATIONSHIP BETWEEN HEIGHT, SHEAR STRENGTH
OF SLIMES AND SHEAR STRENGTH OF FOUNDATION
SOIL AGAINST FAILURE THROUGH THE FOUNDATION
FOR A FACTOR OF SAFETY OF 1.5 ON A SLOPE
OF 33°.

FIG. 81.

THE EFFECTS OF STRATIFICATION ON THE STABILITY OF SLIMES DAMS

When assuming the foundation soil to be homogeneous, the worst circle was found to pass very deeply into the foundation soil. In practice the foundation soil consists of layers of material of different shear strengths and these shear strengths must all be used in analysing a failure arc passing through the various layers. It should be realised that this variation in shear strengths will affect both the position of the worst circle and the worst factor of safety.

As an example, consider a ledge of hard rock at a depth less than the worst circle for a homogeneous soil. The failure arc will not pass through the hard rock and so the worst circle for failure through the foundation soil will be shifted up, as shown in Fig. 82; as a result, the factor of safety against failure through the foundation will be raised.

On the other hand there may be a layer of very soft material with a very low shear strength beneath the wall and in this case the failure surface will no longer be circular but will adjust its position to cause failure through the zone of weakness, as shown in Fig. 83. As the failure surface is contained in the weak stratum for a large part of its length, the factor of safety of this surface will be lower than for the worst circle. It is, therefore, necessary to know of the presence of such layers before the stability of a slimes dam can be predicted.

As has been stated earlier, the slimes itself is stratified, although each layer is of very small thickness. Since it is known that stratified materials are weaker in shear parallel to the stratifications, Taylor has suggested that the shear strength parallel to the stratifications should be used in the slip circle analysis. In slimes material it was found impossible to cut samples in this direction and it is felt that unless definite information on this effect is obtained, it can be handled by using an increased factor of safety.

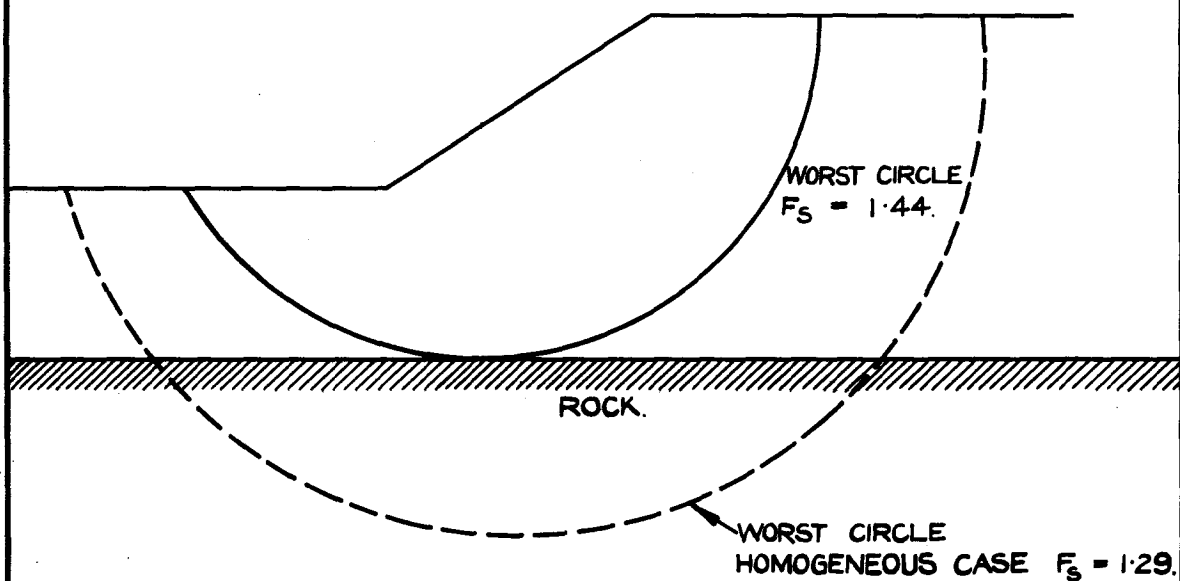


FIG. 82.

INFLUENCE OF A HARD STRATUM ON THE POSITION OF
THE WORST CIRCLE.

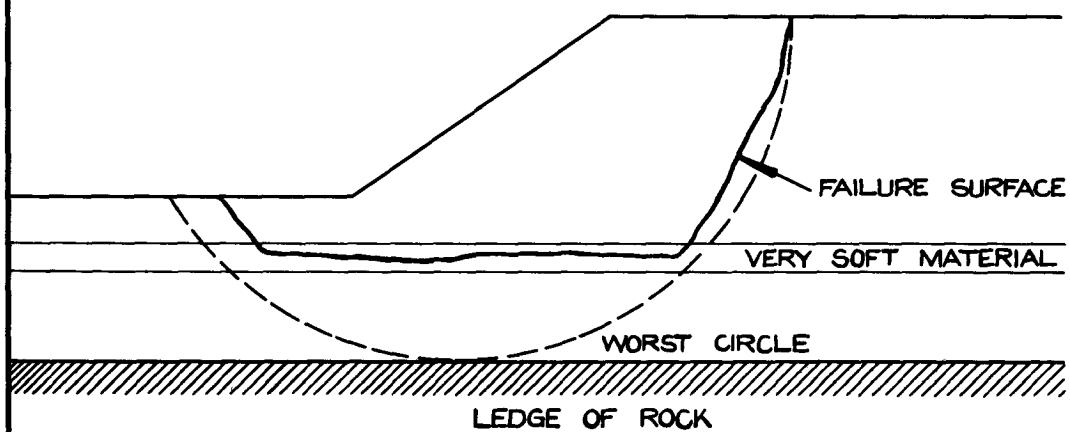


FIG. 83.

INFLUENCE OF A SOFT STRATUM ON THE POSITION
OF THE FAILURE SURFACE.

FLOW SLIDES

It is unfortunately impossible to analyse flow slides. Three factors are required to produce a flow slide viz.

- (a) the material must be in a very loose state
- (b) the pores must be filled with water
- (c) there must be some activating force to trigger the slide.

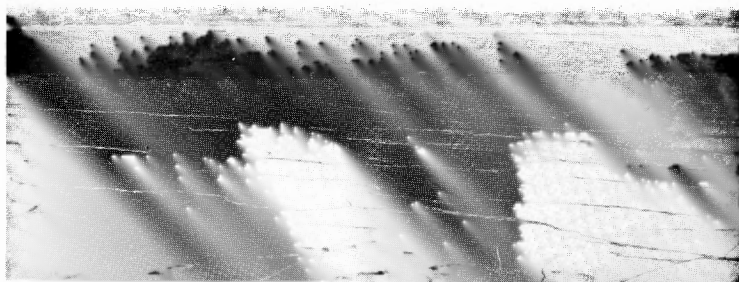
When the material is deposited the grains settle out and form a very loose open structure. If the material is subjected to a sudden shock the structure will collapse. If the material is dry, there will be a decrease in volume as the material settles into a more compact mass. However, if the pore spaces are full of water, the collapse of the grain structure will throw all the weight of the grains on to the pore fluid and the soil will be in a liquid state until the grains have settled out and been reconsolidated. It is while the material is in this state of liquefaction that flow slides occur and a suspension of solids and water flows out over the surrounding countryside. The activating force giving the necessary shock to liquefy the material can either be a normal slip failure or perhaps an earth tremor.

Two definite flow slides in slimes dams were recorded. The first is the famous Simmer and Jack failure of 1937 when slimes flowed for a considerable distance from the break, and engulfed a train, and the second, more recent one, at Grootvlei Mines in 1956 is illustrated in Fig. 84. Two possible sequences can be suggested for these failures:

- (a) a portion of the outer wall failed as a normal slip failure. The resulting shock caused the material inside the dam to liquefy and flow out through the breach made by the slip failure;
- (b) some shock liquefied material in the dam and possibly also some of the wall material. The pressure developed was sufficient to breach the outer wall and the suspension flowed out.



(a) Looking past one end of breach and showing the extent of the flow of slimes. (temporary Laundry in middle distance.)



(b) Extent of the crater from which the material flowed out.

Fig. 84.
Flow Slide, Grootvlei Mines, 1956.

No analysis of these slides is possible but the likelihood of their occurrence can be reduced by keeping the material as dry as possible and by making sure that normal slip failures will not occur.

THE EFFECT OF THE WIDTH OF THE WALLS ON STABILITY

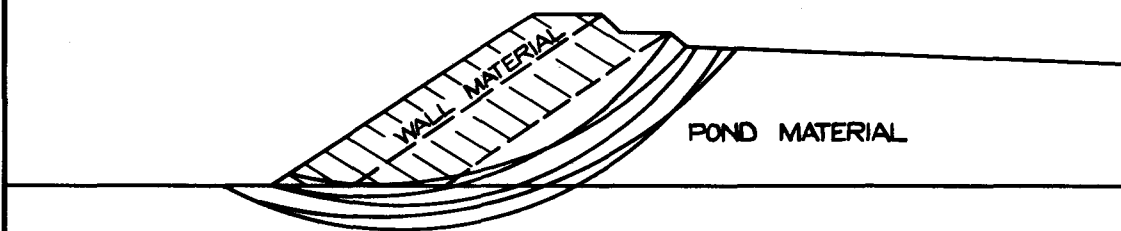
It has been shown in this chapter that the stability of the wall will, to a large degree, depend on the shear strength of slimes through which the failure surfaces pass and it follows that the stronger the slimes the more stable the dam will be. In the previous chapter, it was shown that the drier the material was kept, the greater would be its shear strength. As pointed out in the introduction to this report, the material on the day and night walls is subjected to regular drying out but the material on the floor of the dam may be inundated by the pond and receive no drying out at all.

Therefore, it is necessary to ensure that the worst failure surface will pass through the outer wall material and not through the pond material. Fig. 85(a) shows that where the walls have been kept narrow, most possible failure circles pass through very little wall material and will be contained mainly in the pond material. Fig. 85(b) shows that increasing the wall widths will result in almost all failure circles being contained in the wall material. In determining the correct wall width the positions of the worst failure arcs must be known. It can, however, be stated that a combined day and night wall width of 250 feet will be sufficient to ensure against this possibility in the majority of dams.

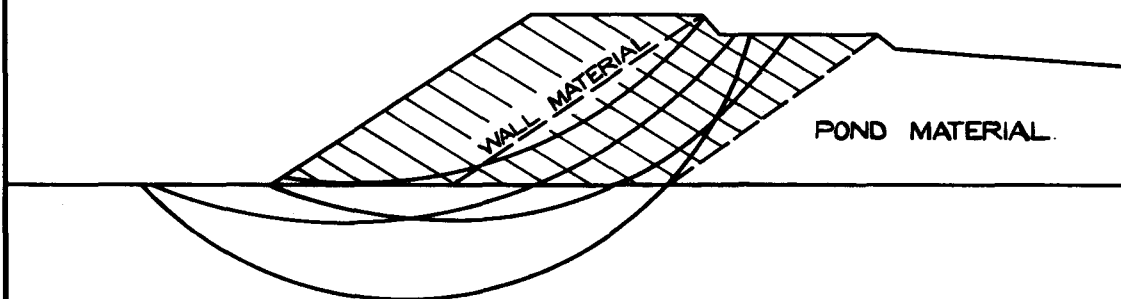
CONCLUSIONS

The conclusions to be drawn from the research into the stability analyses of slimes dams can be summarised as follows:

1. The stability of the slimes dam depends on the shear strength of the slimes.



(a)



(b)

FIG. 85.

INFLUENCE OF WALL THICKNESS ON THE STABILITY
OF THE DAM.

2. The stability of the slimes dam depends to an equal degree on the shear strength of the foundation soil.
3. Seepage flow through the wall or foundation soil will reduce the stability of the dam.
4. The flatter the slope of the outer wall for given shear strengths of slimes and foundation soil, the higher the dam can be built for a given factor of safety.
5. There are certain factors influencing the stability of slimes dams which cannot be analysed numerically.
6. It is impossible to draw up a generalised rule for calculating the stability of slimes dams but general principles can be laid down.

RECOMMENDATIONS

As has just been stated, it is impossible to derive one simple formula which will allow prediction of the stability of any slimes dam. It is essential that each slimes dam should be analysed with regard to its own peculiar circumstances. However, certain general principles for stability analyses can be stated but they should not be treated as hard and fast rules.

1. Factor of safety

The factor of safety is the ratio of Resisting Force to Actuating Force on any slip circle and the factor of safety for a slope is the lowest factor of safety determined for the worst slip circle passing through that slope. A factor of safety of one means that the embankment is on the point of failure.

It is necessary to determine what factor of safety should be used in the design of slimes dams. In earth dam work where all factors are known and, where the construction is rigidly controlled, factors of safety as low as 1.3 are accepted for the worst conditions. In slimes dam work there are many variables which cannot be evaluated and it is therefore

recommended that, where full knowledge is available of shear strengths and seepage flow, a factor of safety of 1.5 should be used, and that this factor should be increased to 2 or 3 as more uncertainty enters the picture.

2. Prediction of stability

Determining the stability of an existing slope is relatively simple. Once in-situ shear strengths have been measured, the factor of safety can be determined.

Predicting the stability of a future dam is not as easy. It is possible to measure the strength of the foundation soil and its shear characteristics. It is then necessary to predict the shear strength of the slimes and the seepage conditions in the wall and foundation soil. Where no information is available on shear strength, the material should be treated as purely frictional and equations (44), (45) and (46) should be used. Finally, a suitable factor of safety must be chosen. The dam can then be analysed to determine the safe slope and height to which it can be built. As foundation conditions vary on a dam site it may be necessary to analyse various parts of the dam to determine the stability for each set of conditions. The dam must always be analysed under the worst conditions likely to arise.

Example: It is predicted that the slimes material will have a shear strength of 2000 lb./sq.ft. treated as apparent cohesion, and that there will be no seepage flow through the dam or foundation. It is desired to build the outer walls at a slope of 33° and to use a factor of safety of 1.5. In one area the foundation shear strength is 2000 lb./sq.ft. and in another area 3000 lb./sq.ft. To what heights can the dam be built in these areas?

For failure through the toe, Fig. 72, point B, gives a safe height of 78 feet.

For failure through the foundation where $S_f = 2000$ lb./sq.ft., Fig. 81, point A, gives a safe height of 71 feet.

In this area failure through the foundation is the limiting factor and the safe height will be 71 feet.

For failure through the foundation where $S_f = 3000 \text{ lb./sq.ft.}$, Fig. 81, point D, gives a safe height of 103 feet.

In this area failure through the toe is the limiting criterion and the safe height will be 78 feet.

It is further suggested that, in view of the many factors involved, the services of an expert in the field of soil mechanics should be employed for stability analyses. In-situ tests should be performed periodically to check the assumed design conditions.

3. Ensuring and increasing stability of a dam

The stability of a wall can be ensured by the following steps:

- (a) Keeping it as dry as possible by removing surface water.
- (b) Removing seepage forces by providing adequate underdraining to remove all seepage flow from the possible zone of failure.
- (c) Carrying out a site investigation before locating the dam and in this way building the walls on the strongest foundation soil.
- (d) Ensuring that the worst circles pass through the strongest material by building the walls of sufficient width.
- (e) Keeping the slopes flat enough to ensure a reasonable factor of safety for the height of dam envisaged.
- (f) As a corollary to (e) where an existing wall shows signs of distress, reducing the average slope by stepping back or building a buttress against the toe.
- (g) If adequate drainage has been provided under (b) no difficulties with erosion at the toe will be experienced.

CHAPTER VII.

C O N C L U S I O N S
A N D R E C O M M E N D A T I O N S

Slimes dams are among the largest man-made structures in the world. Their structural performance, as has been shown in the earlier chapters of this report, is governed by a wide range of factors, some of which can be evaluated and some which cannot. This research project has tried to investigate and assess all the factors affecting the stability of slimes dams and, wherever possible, to present a quantitative analysis of them.

As in every research project, a considerable amount of thought, time and energy was spent on exploring avenues which either proved to have little effect on the problem, or which indicated further investigation, too complex to allow any satisfactory results to be obtained.

Nevertheless the project has revealed the mechanism of gain in strength of slimes and the various factors causing failures. No revolutionary suggestions are made, but the recommendations are rather aimed at using the best existing methods of design and construction and suggesting possible improvements in these methods.

A study was made of the available literature on slimes dam building in Canada and the United States of America and a report on this survey was circulated. (26) Slimes dams in these countries are built on the principle of hydraulic fill dams where the suspension of material is discharged around the periphery and flows down a beach towards a penstock in the centre of the dam. The coarser particles settle out first near the edges of the dam while the finer particles are carried further towards its centre. This method is suitable for use where there is a good proportion of coarse material in the suspension, and has been used successfully both at Premier Mine and in the ash disposal dam at SASOL. However, it is considered that the slimes from the gold mines is too fine for this method of building and it was reported at one meeting of the Steering Committee on the Construction of Slimes Dams that this method had been tried at Rand Leases and found to be very costly.

While many of the problems experienced and the techniques used in North America are similar to those in South African practice, unfortunately no solutions to

these problems were mentioned in the literature. One point of interest was that the use of sand to fill in erosion due to seepage or run-off had been found to have little effect, as the sand was soon washed out again, and it was, therefore, common practice to fill badly eroded places with bags of wood pulp.

Some of the factors investigated affect only one aspect of the stability of a slimes dam, whereas other factors may influence several aspects. The conclusions to be drawn from the various sections of this report are summarised below.

SUMMARY OF CONCLUSIONS

Surface erosion

- (a) Crust formation will occur on the outer walls of slimes dams where the pyrite content is greater than 0.7 per cent.
- (b) In cases where the pyrite content is less than 0.7 per cent, promising results were obtained with the use of additives as crust-forming agents.
- (c) It may be possible to find plants which will grow on slimes dams and which have root systems which will bind the slimes. No systematic work on this appears to have been done in South Africa.

Seepage erosion

- (a) It has been shown in this report that seepage flow is probably the most important factor in the whole problem of the stability of slimes dams.
- (b) Seepage flow will occur on slimes dams where the drainage of the foundation soil is insufficient to cope with the water seeping down from the pond on the dam.
- (c) The size of the zone of seepage flow will depend on:
 - (i) the level and extent of water in the pond;

- (ii) the ratio of the horizontal to vertical permeability of the slimes;
 - (iii) the ratio of the permeabilities of the slimes and foundation soil;
 - (iv) the ratio between the height of the dam and the depth of the foundation soil.
- (d) Seepage flow can be controlled by the use of a properly-designed system of underdrainage.

Overall stability

- (a) The overall stability of the dam is largely dependent on the shear strength developed in the slimes.
- (b) The slimes is a purely frictional material with no true cohesion.
- (c) Slimes which has been over-consolidated will have a higher shear strength than normally consolidated slimes.
- (d) On a slimes dam the material exposed to the air will be dried out, and this desiccation will cause over-consolidation with a resultant increase in shear strength.
- (e) As the slimes is continually wet and dried in the process of building, the shear strength of the slimes will be subject to cyclic variations. The extent of these variations will depend on the amount of wetting and drying which occurs in each cycle.
- (f) Seepage flow through the walls of the dam will reduce the shear strength of the slimes.
- (g) Where an accurate assessment of all the factors governing overall stability can be made, it is relatively easy to carry out stability analyses.
- (h) The development of increased shear strength due to over-consolidation will allow the walls to remain stable at greater slopes than in the purely frictional case for the same factor of safety.

Bearing these conclusions in mind, the following recommendations are made.

SUMMARY OF RECOMMENDATIONS

As has been pointed out earlier in this report the complexity of the factors involved warrants a separate analysis for each dam and the recommendations listed below should be taken as general principles.

Surface erosion

- (a) Where possible, the pyrites content of the slimes should be maintained above 0.7 per cent to ensure the formation of a hard crust.
- (b) When a run-off channel shows signs of developing into a deep gulley, this development should be arrested.

Seepage erosion

- (c) Before building a new dam, a site survey should be carried out to determine the drainage conditions of the dam site including permeabilities, depth to water table and related drainage features of the area considered.
- (d) If the drainage conditions on the site vary, the penstock should be located over a well-drained area.
- (e) If it is unavoidable to locate the penstocks over areas where the drainage is inadequate, it will be necessary to provide improved drainage, for instance by one of the drainage systems suggested in this report, to control the seepage water.
- (f) The extent of seepage flow can be reduced by keeping the water-level in the pond as low as possible. This can be assisted by the use of a bypass valve for the disposal of stormwater, provided that it does not lead to undue pollution of water courses.

- (g) Water should be removed from the wall areas of the dam as soon as possible.
- (h) Existing walls with seepage problems can be treated by the addition of an under-drained buttress.
- (h) Existing walls with seepage problems can be treated by the addition of an under-drained buttress.

Overall stability

- (i) In all stability analyses the worst conditions likely to occur during the life of the dam must be envisaged and these conditions should be used for the stability analysis.
- (i) In all stability analyses the worst conditions likely to occur during the life of the dam must be envisaged and these conditions should be used for the stability analysis.
- (j) A site investigation should be carried out before the building of a slimes dam to determine the shear strength of the foundation soils to a depth at least equal to the final height of the dam. These shear strengths will allow the dam to be adequately designed against failure through the foundation.
- (j) A site investigation should be carried out before the building of a slimes dam to determine the shear strength of the foundation soils to a depth at least equal to the final height of the dam. These shear strengths will allow the dam to be adequately designed against failure through the foundation.
- (k) In the absence of any other information, slimes should be treated as a purely frictional material with an angle of internal friction of 35° and the maximum permissible slopes can be calculated from equations (44), (45) and (46).
- (k) In the absence of any other information, slimes should be treated as a purely frictional material with an angle of internal friction of 35° and the maximum permissible slopes can be calculated from equations (44), (45) and (46).
- (l) If allowance is to be made for the added strength due to over-consolidation, relationships of the type shown in Figures 72 to 81 should be established.
- (l) If allowance is to be made for the added strength due to over-consolidation, relationships of the type shown in Figures 72 to 81 should be established.
- (m) In order to gain the maximum shear strength from the material it should be kept as dry as possible and all seepage flow should be eliminated.
- (m) In order to gain the maximum shear strength from the material it should be kept as dry as possible and all seepage flow should be eliminated.
- (n) Where the design of a new slimes dam is based on the prediction that over-consolidation will occur and give increased shear strengths, in-situ tests should be carried out periodically to ensure that the design strengths are being obtained. If the practical conditions are not the same as those predicted in the design stage, the design will have to be modified.
- (n) Where the design of a new slimes dam is based on the prediction that over-consolidation will occur and give increased shear strengths, in-situ tests should be carried out periodically to ensure that the design strengths are being obtained. If the practical conditions are not the same as those predicted in the design stage, the design will have to be modified.
- (o) It is suggested that no more than 2 inches depth of slimes should be deposited at a time and that two weeks or longer should elapse between consecutive depositions of slimes.
- (o) It is suggested that no more than 2 inches depth of slimes should be deposited at a time and that two weeks or longer should elapse between consecutive depositions of slimes.

- (p) It is further suggested that slimes dams be worked in sections so that after a period of several months of deposition, the wall area will be "rested" for several months to allow thorough desiccation to occur.
- (q) Existing dams showing signs of distress can be treated by the addition of a low buttress which will have the effect of reducing the overall slope.

In this report an endeavour has been made to analyse the causes and mechanisms of stability failures in slimes dams and also the mechanisms responsible for the gain in strength of material deposited in slimes dams. In every case of failure reported to this Institute during the course of the investigation, the main cause was either seepage flow or a weak stratum in the foundation soil, often assisted by the presence of erosion gulleys.

The conclusions of this report have been proved theoretically, in the laboratory and have been observed to apply to the field experience. The recommendations have all been proved theoretically and in the laboratory and to a lesser extent in the field. Final proof of the usefulness of these recommendations can only come with the application of these recommendations to full scale slimes dams as in the case of the experimental wall at St. Helena. It is outside the scope of this project to carry out such full scale tests and it, therefore, remains for individual Groups or Mines to conduct such full scale tests by applying the results of this project to their own specific problems.

Once this step has been taken it remains for the Group or Mine to compile a report stating the nature of the problem, its causes, the solution applied to the problem and the degree of success obtained. A certain amount of this information may already be available in the files of various Mining Companies, and a library of information on the practical application of these recommendations can thus be built up, which together with this report should form the basis for future slimes dam design.

APPENDIX 1.

EXPERIMENTAL WALL, ST. HELENA SLIMES DAM

The reasons for building an experimental wall at the slimes dam of the St. Helena Gold Mining Company Limited have already been briefly stated (27) and this appendix forms a complete report on that full-scale experiment, up to April, 1959.

On the site plans of the slimes dams, Fig. 86, it will be seen that the first section was built on the eastern half of the site and the second section built later to cover the remainder of the area of the first dam. When the western wall of the first section was about 15 feet high, a slip failure occurred at the point marked A. As this was shortly to become an internal wall, the wall was stepped back round the breach, as shown in Fig. 87(a) and no further precautions were taken.

The second section of the dam was started and when the north-western wall was about 12 feet high it began to show signs of seepage failure as shown in Fig. 87(b) and (c). A site survey was performed and a number of hand-auger holes were put down along the whole of the north-west wall and along the internal wall at A. It was found that both points A and B were located where the walls crossed a water course, and from the soil profiles obtained, which are shown in Fig. 88, it will be seen that the problem areas were underlain by pockets of heavy black clay. To the north of B the depth of sand increased and no problems were evident while to the south of B there was more sand but the drainage was bad and the area was marshy.

It was originally intended to stabilize the wall at B by building a 50-foot wide slimes buttress, with under-drainage, which would later become the outer wall at this point. The scheme as finally carried out is that the buttress runs the whole length of the north-west wall, and in the portion marked "experimental section" in Fig. 88(b) it is under-drained by a blanket 50 feet wide.

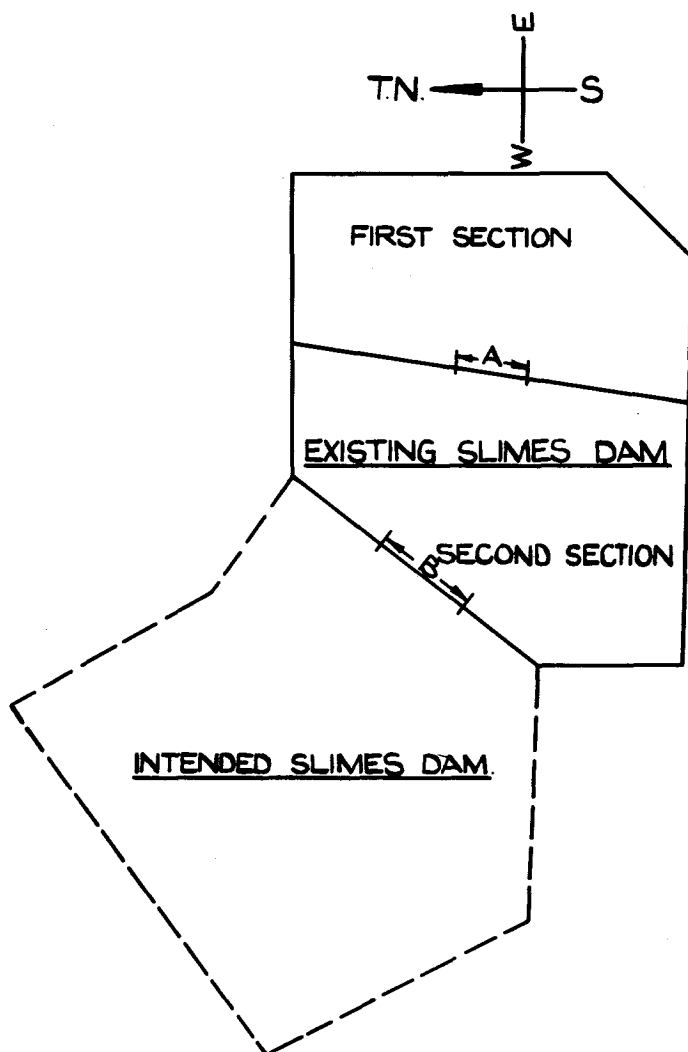
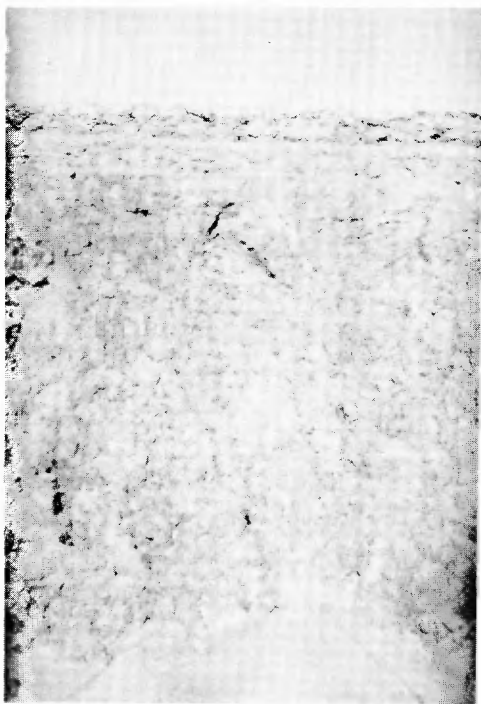


FIG. 86.

SITE PLAN, ST. HELENA SLIMES DAMS.



(a) Wall stepped back
round slip failure
on internal wall at
point A (see Fig.86)

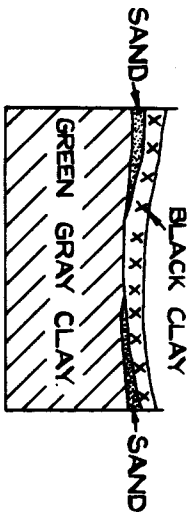


(b)

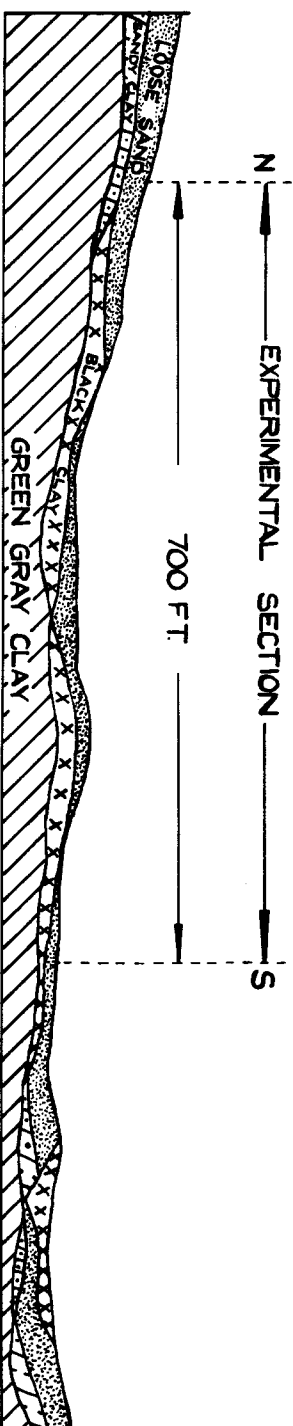
(c)

Signs of distress due to seepage flow
at point B on north west wall (see Fig.86)

Fig. 87.
St. Helena slimes dam, 1954.



(a) POINT OF FAILURE IN INNER WALL.



(b) WEAK POINT IN NORTH - WEST WALL.

FIG. 88.

SOIL PROFILES ST. HELENA SLIMES DAM.

HORIZONTAL SCALE 1" = 150' VERTICAL SCALE 1" = 15'

Design of drain and filter layers

It was assumed that a blanket drain of a depth of one foot, with filter layers, would be adequate for dealing with the seepage water from the dam. Initially there would be a heavy discharge of water into the drain as the slimes was deposited on it but as soon as a relatively thin layer of slimes covered the drain the rate of flow of seepage water would be reduced. It was decided that this drain as well as the collection trench at the toe of the wall be filled with $1\frac{1}{2}$ inch stone. The water in this collection trench would be drawn off through pipes at its lowest points.

The most important factor to be considered was the protection of the drainage layer against clogging by the ingress of slimes. This was done under the dam as economically as possible, by making use of locally available materials to construct a number of filter layers between the slimes and the coarse rock. The grading analyses for these materials together with that of the slimes is given in Fig. 89.

According to equation (1) (Chapter II), the criterion to prevent clogging of a filter layer is that

$$\frac{D_{15} \text{ (coarse material)}}{D_{85} \text{ (fine material)}} < 4 \text{ to } 5$$

From Fig. 89:

D_{85} of slimes = 0.11 mm.

D_{15} of crusher sand = 0.30 mm.

D_{15} of Vaal River sand = 0.40 mm.

Therefore, both these materials would be suitable for use as filters with the slimes but as crusher sand was cheaper, its suitability was considered first.

From Fig. 89:

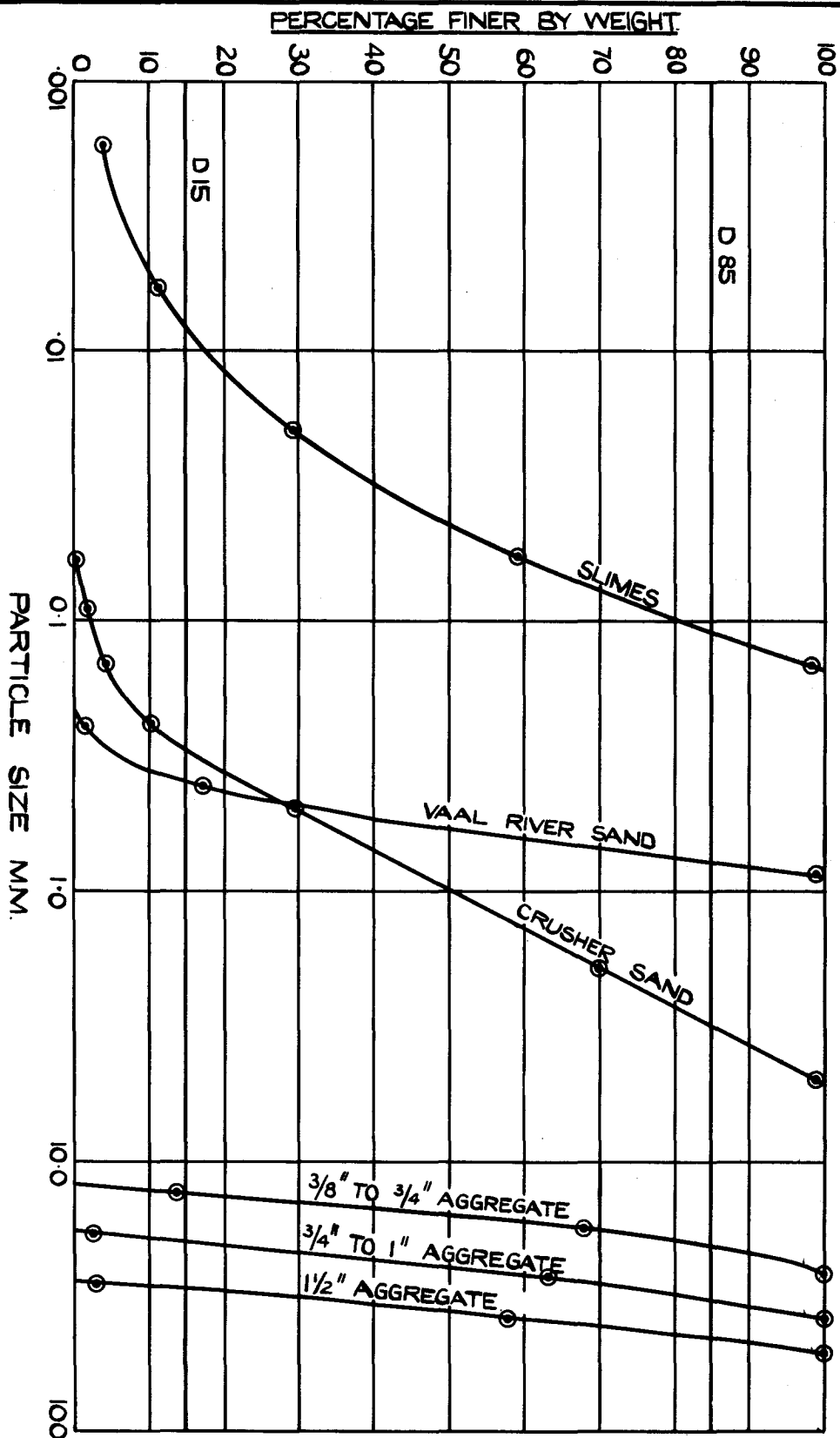
D_{85} crusher sand = 3.1 mm.

D_{15} of $\frac{3}{8}$ in. - $\frac{3}{4}$ in. aggregate = 10.3 mm.

Therefore the $\frac{3}{8}$ in. - $\frac{3}{4}$ in. aggregate was suitable to hold the crusher sand.

FIG. 89.

PARTICLE SIZE DISTRIBUTION CURVES OF POSSIBLE FILTER MATERIALS.
ST. HELENA SLIMES DAM.



In turn

D_{85} of $\frac{3}{8}$ in. - $\frac{3}{4}$ in aggregate = 20.5 mm.

D_{15} of $1\frac{1}{2}$ in. aggregate = 30 mm.

and thus the $\frac{3}{8}$ in. - $\frac{3}{4}$ in. aggregate could go directly onto the $1\frac{1}{2}$ in. rock in the drain.

As a result of these calculations it was decided to place a layer of $\frac{3}{8}$ in. - $\frac{3}{4}$ in. aggregate on the $1\frac{1}{2}$ in. rock and then a layer of crusher sand, between the $\frac{3}{8}$ in. - $\frac{3}{4}$ in. aggregate and the slimes, to form a filter. A model of this filter was built in the laboratory and it was discovered during tests that the fines were washed out of the crusher sand which then ceased to function as a filter and the whole drain became clogged with slimes.

Research carried out since that time gave rise to equation (2) where it is stated that for a material to be a filter within itself, it should have a uniformity coefficient of

$$D_{85}/D_{15} < 4 \text{ or } 5.$$

and for crusher sand, $D_{85}/D_{15} = \frac{3.1}{0.30} = 10.$

Therefore, crusher sand is not a filter within itself and was unsuitable for the filter layer under the slimes. It was shown that if the crusher sand was screened into a coarse and a fine sand, these two fractions would each be suitable as filter materials and could be used to form two layers between the slimes and the $\frac{3}{8}$ in. - $\frac{3}{4}$ in. aggregate. However, the cost of screening was considered to be too high and it was decided to use Vaal River sand under the slimes.

D_{85} of Vaal River sand = 0.68 mm.

D_{15} of $\frac{3}{8}$ in. - $\frac{3}{4}$ in. aggregate = 10.3 mm.

Therefore, the Vaal River sand could not be placed directly on the aggregate. As only the coarse particles were required, it was decided to place a layer of crusher sand to act as the filter between the Vaal River sand and the $\frac{3}{8}$ in. - $\frac{3}{4}$ in. aggregate. This would mean that the finer particles of the crusher sand might be washed into the succeeding layers, but the relative thicknesses of the layers were arranged so that the quantity of fines washing through would be very small.

In addition, the thicknesses of the filter layers were chosen to give sufficient protection to the layer below and to prevent breaks occurring in the layers due to disturbance during construction. The top layer of sand was made 6 inches thick as a protection against washing out when the first slimes was placed on it.

A clay seal was placed over the front of the drain to prevent slimes being washed in by run-off from the face of the dam. The final design of the drain is shown in Fig. 90. By the time the blanket drain and filter were laid it had already been necessary to put a rock buttress against the existing wall which was in imminent danger of failing. Two views of the completed blanket drain before the slimes was put on to it, are shown in Fig. 91.

The first slimes was placed on the drain from the dam side and was very carefully run over the drain so that there would be no disturbance of the upper filter layer of sand. It was immediately apparent that the filters were successful because as soon as the suspension ran on to the drain the solids were left behind and clear effluent was discharged from the outlets. Two views of the slimes deposited on top of the sand are given in Fig. 92. The small walls were not built until the slimes had reached a minimum depth of one foot and then great care was exercised to see that the top filter layer was not damaged during the digging. From there on the dam was built in the normal way but at a faster rate.

Clear water has been discharged at a steady rate since deposition of slimes was commenced in October, 1956. It was found early on that there was a point in the collection channel which was at a lower level than the outlet points and an additional outlet point was put in at this level. Views of two of the outlet points are shown in Fig. 93.

The buttress had reached a height of about 30 feet by March, 1959, having overtaken the existing wall and thus itself become the outer wall. The wall was completely stable and there were no signs of distress, as can be seen from Fig. 94(a), except at the southern end where there is no under-drainage and it has been necessary to tip a rock buttress to stabilize this section. Fig. 94(a) should be compared with Fig. 87(b) and (c) which cover the same section of wall before treatment.

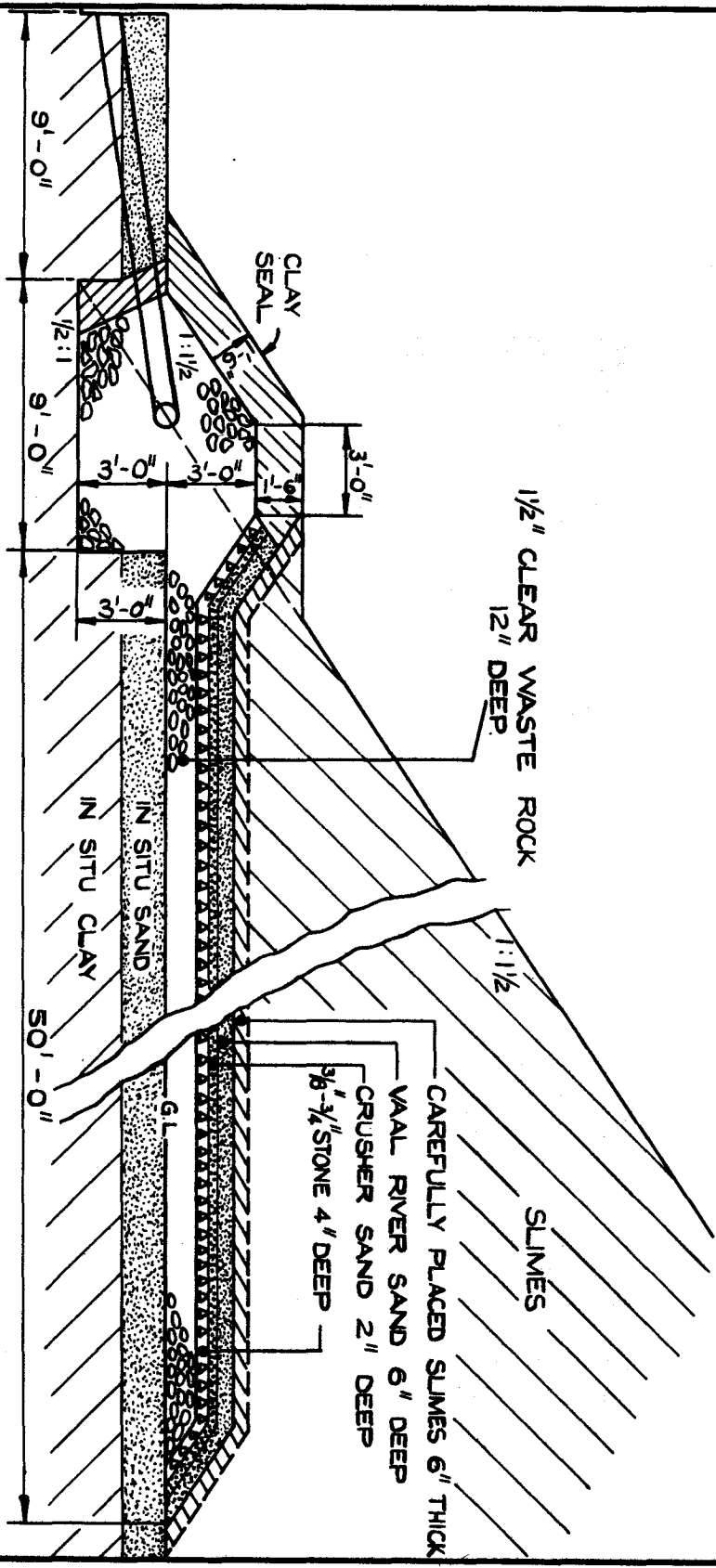
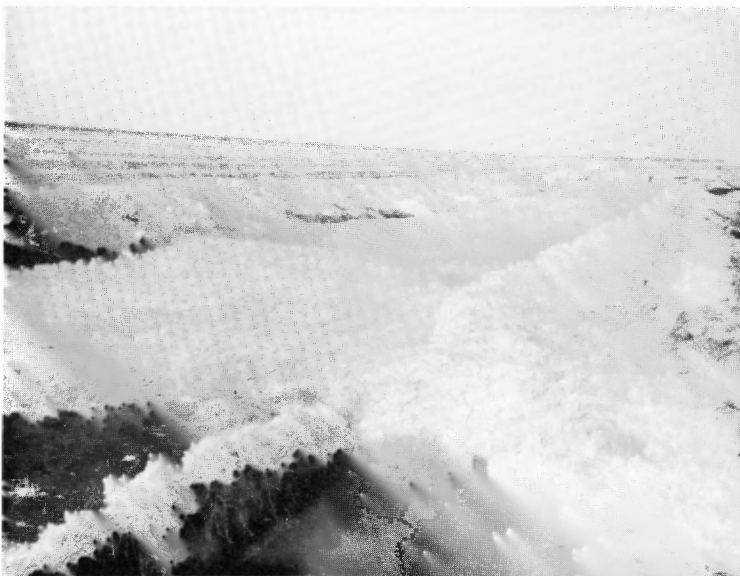


FIG. 90.

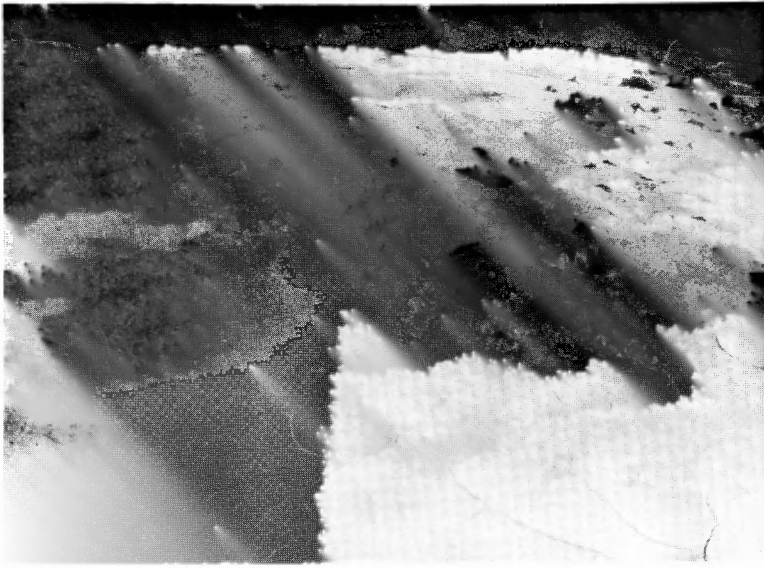
BLANKET DRAIN WITH FILTER LAYERS, ST. HELENA SLIMES DAM.



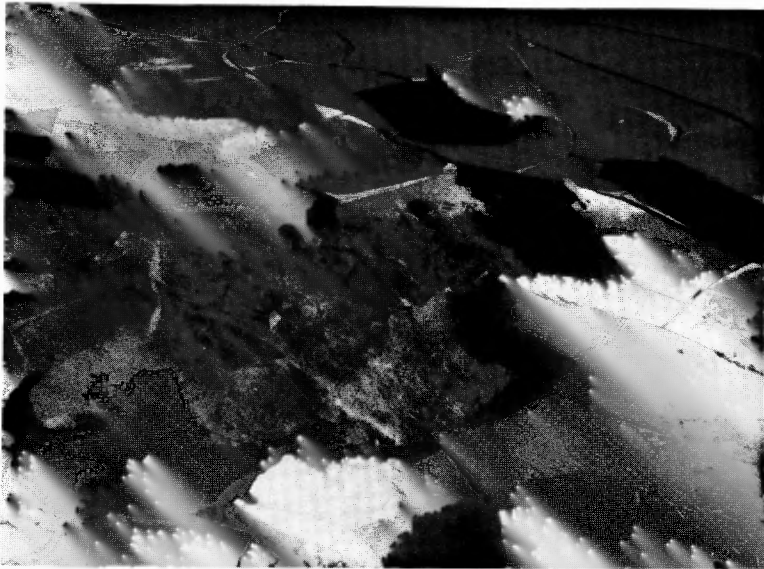
(a) Looking North from existing wall.



(b) Looking South toward existing wall
with rock buttress and showing clay
seal over toe of drain.



(a) Limit of the spread of slimes
at one deposition.



(b) Slimes removed to show clean sand underneath.

Fig. 92.

Initial deposition of slimes on blanket
drain , St. Helena.



Fig. 93.
Drain outlets discharging clear water.



(a) Buttress wall, 1958.



(b) Top of buttress wall showing
tops of piezometers.

Fig. 94.
Buttress wall St. Helena slimes dam.

Further emphasis is given to this aspect of the work by the fact that the new dam was started in 1957 and by the time its eastern wall, about 100 feet away from the experimental section, had reached a height of 12 feet, it was in danger of failing and had to be stabilized with a rock buttress.

In order to assess the efficacy of the under-drainage in eliminating seepage flow through the walls, it was decided to install piezometers in both the under-drained and undrained sections of the buttress near the southern end of the under-drains. A very simple type of piezometer consisting of a porous porcelain cylinder, acting as a filter, attached to a length of $5/8$ inch plastic hose as shown in Fig. 95, was installed. The piezometer was installed in a 4 inch hole put down by hand-auger to the top of the filter layers or to the foundation soil where there were no under-drains. After connecting the hose to the filter candle, it was threaded through a waterpipe, which was used to push the filter candle to the bottom of the hole. The hole was filled for about 3 feet with a thick slimes slurry while the pipe was left in place, and the pipe was then withdrawn, leaving behind the filter candle with thick slimes slurry tamped into a compact mass. About 5 feet of hose were allowed to project from the hole, and this length was fastened to a jumper rod and tied in a loop to prevent evaporation. As the dam rises, the hose and jumper rod are buried and whenever it is necessary, a new length of hose is added and new supporting rods are put in. Fig. 94(b) shows the top of the wall with the tops of the piezometers visible. Two piezometers were installed in the undrained area and three on the under-drained section.

The water level in the piezometers is read by means of an electric dipper. Sets of readings have been taken at regular 3-monthly intervals since the piezometers were installed in November, 1957, and three of these sets of readings are shown plotted on the cross-section of the dam in Fig. 96. At piezometer points 1 and 2 over the under-drains no seepage water has been recorded. The interpolated phreatic surfaces in Fig. 96 show the effect of the drain in keeping the seepage zone out of the toe area.

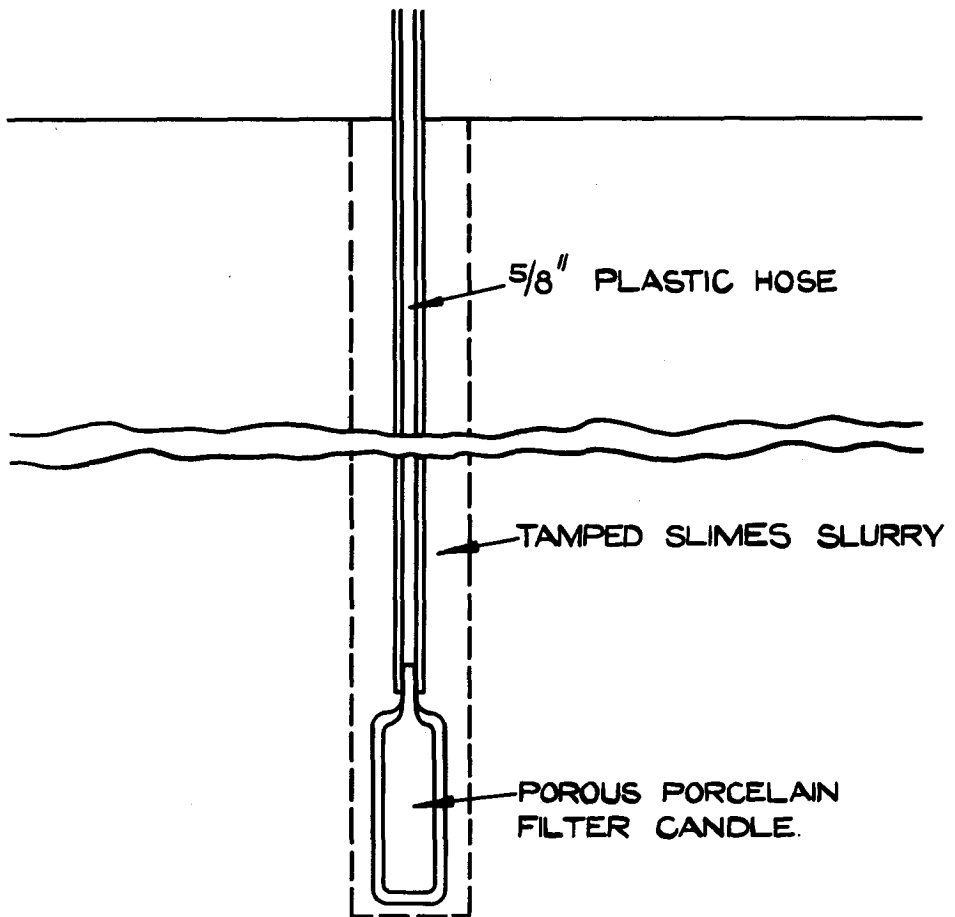


FIG. 95.

PIEZOMETER USED ON SLIMES DAMS

PIEZOMETERS 1, 2, 3 ——— DRAINED SECTION PHREATIC SURFACE.
 PIEZOMETERS 4, 5 - - - - - UNDRAINED SECTION PHREATIC SURFACE.

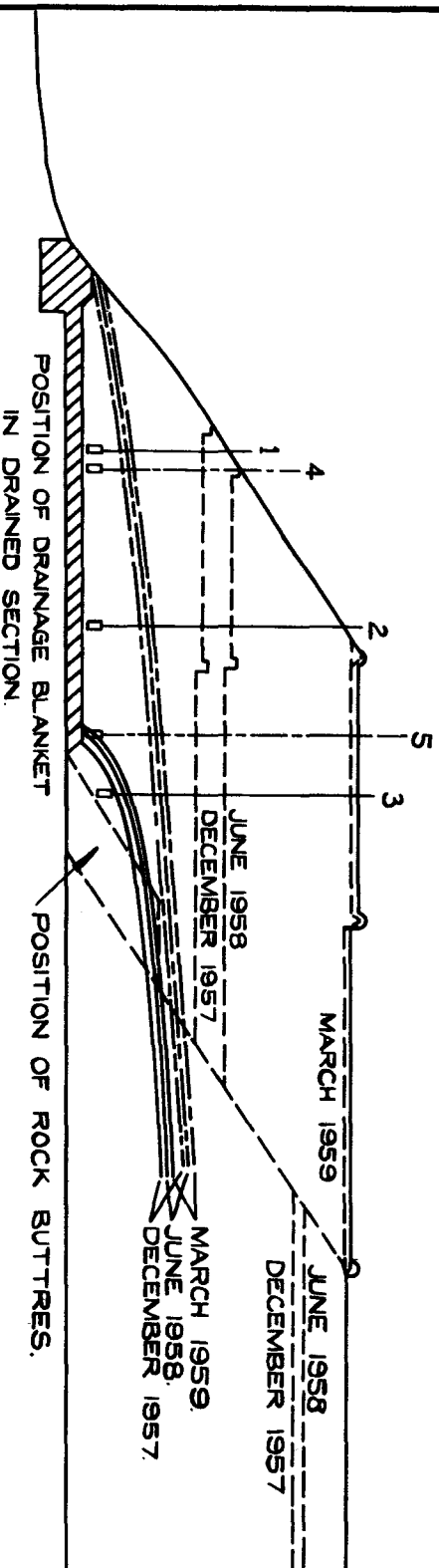


FIG. 96.

PHREATIC LINES FOR DRAINED AND UNDRAINED SECTIONS OF THE BUTTRESS
WALL AT ST. HELENA SLIMES DAM.

SCALE: 1" = 20'

It is hoped to continue these observations until the dam is much higher and also to carry out in-situ tests to compare the shear strengths of the drained and undrained sections of the wall. The pond which was lying behind this wall has been shifted to the northern wall of the dam, now that the main penstock is in operation. This should result in decreased seepage flow to the drains.

APPENDIX. 2.

SAMPLING AND IN-SITU TESTING OF SLIMES AT THE SLIMES
DAM OF EAST GEDULD GOLD MINING COMPANY LTD.

In order to obtain a better understanding of the behaviour of slimes on a slimes dam, it was decided to carry out field tests at East Geduld slimes dam. The sampling and testing procedure was based on a 4-foot cycle: a 2-foot length of undisturbed sample would be taken, then an in-situ vane shear test would be performed 1 foot below the bottom of the hole and then a sampler 2 foot long would clear out the material disturbed during the vane test and a further foot below it. Miniature samples were taken from the bottom of this cleaning sampler for moisture content determinations by means of a piston sampler. The apparatus used in the sampling and testing was the same as that used in Durban by Collins. (28)

As it was impossible to get a drilling machine on to the top of the dam, the drilling was carried out by the "shoestring" method illustrated in Fig. 97. Two earth anchors were screwed into the ground, one on either side of the hole, and snatch blocks were attached to the tops of these anchors. A wire cable was attached to the top of the drill rods and threaded through these pulleys and over a third pulley hanging from the shear legs. When the winch was tightened the cable pulling against the earth anchors forced the sampler into the ground.

The location of the holes is shown in Fig. 98. Initially in hole 1 an open drive sampler was used. This consisted of a 30-inch length of thin-walled seamless tubing, sharpened at one end and connected to drill rods at the other. Satisfactory samples were obtained for a short depth and then the samples dropped out of the tube and could not be recovered. A piston sampler as shown in Fig. 99 was used thereafter with very satisfactory results. The sampling was done on the floating piston principle. The piston was slipped forward almost to the front of the sampler and the collet and spring were then put in position. The piston was then tapped forward to lock the piston in the forward position. In this locked position the sampler was put down to the bottom

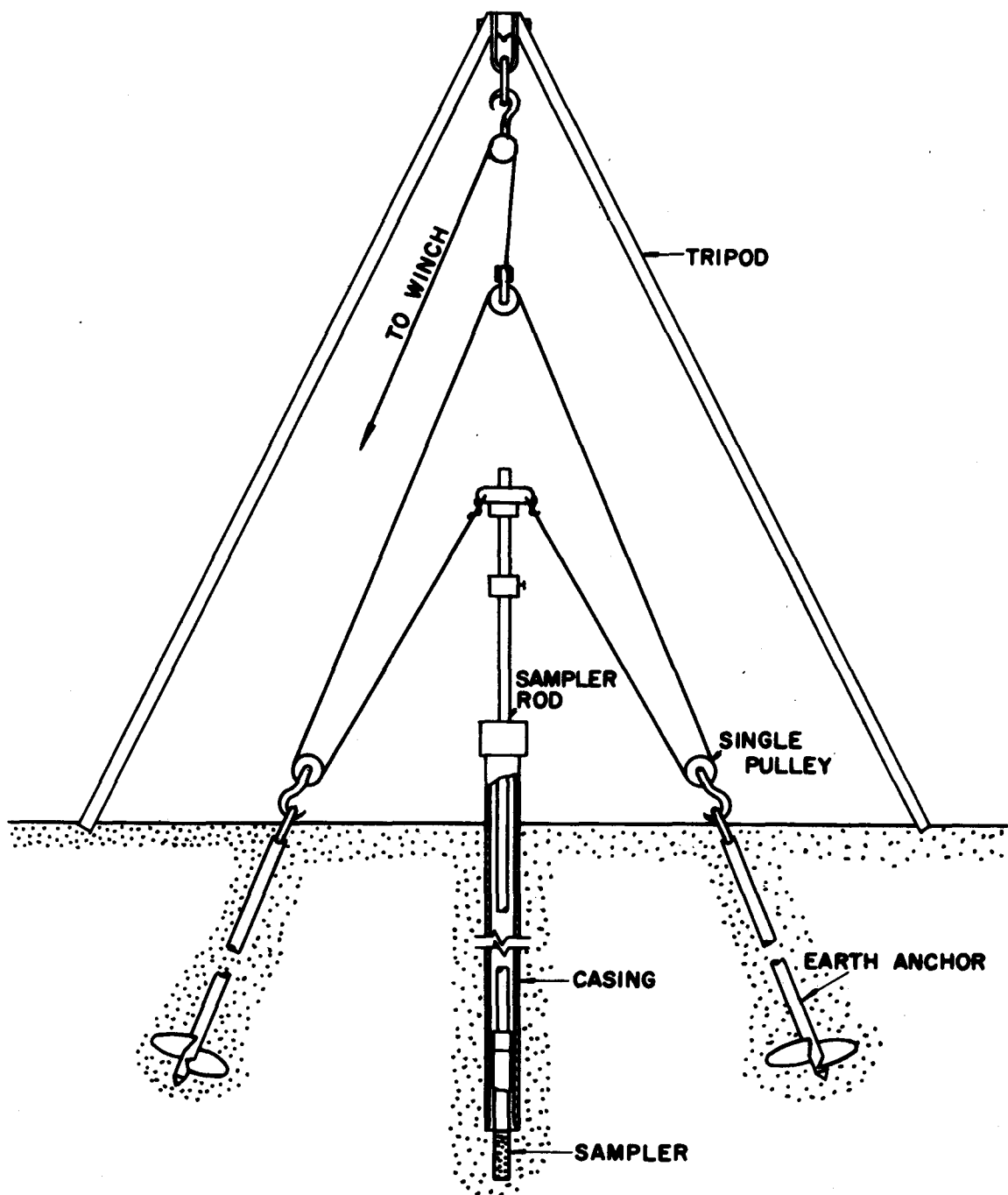


FIGURE 97.

SHOESTRING METHOD OF SAMPLING.

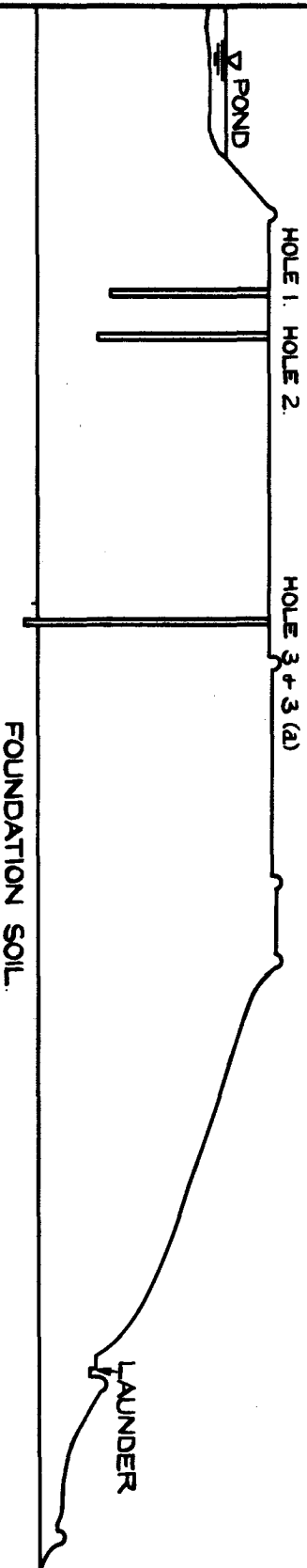


FIG. 98.

LOCATION OF BOREHOLES EAST GEDULD SLIMES DAM.

SCALE: 1" = 40'

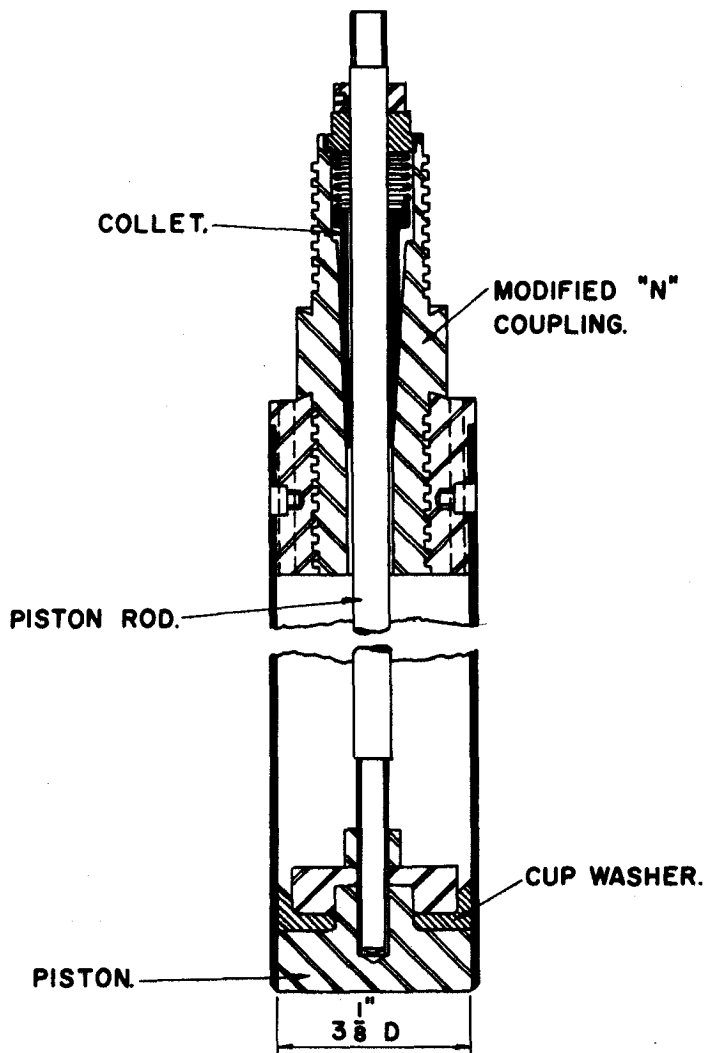


FIGURE 99.

THREE-INCH THINWALL PISTON SAMPLER.

of the hole and the rods were connected up to the shoe-string apparatus. The thrust was sufficient to unlock the piston which remained at the level of the bottom of the hole (floated) while the sampling tube penetrated into the slimes. When the sampler was extracted the piston tended to be pulled down but the collet jammed and locked the piston in the up position and the sample was retained in the tube. The undisturbed samples were sealed in wax and taken to the laboratories for testing.

As was mentioned earlier, a miniature piston sampler (29) was used to obtain samples from the bottom of the 3-inch tube used for cleaning the hole. This sampler was set to give a sample of standard dimensions and, therefore, having determined its moisture content, it was also possible to calculate porosity, void ratio, dry density, total density, degree of saturation, etc. These results are given in Tables XVIII, XIX and XX for holes 1, 2 and 3 respectively. Consolidation test results are given in Tables I, II, III and Fig. 44 in Chapter IV.

The in-situ shear strengths were measured with the vane shear apparatus; the vane itself is illustrated in Fig. 100. The vane, attached to drill rods, was lowered to the bottom of the hole and was then forced into the soil to a depth of one foot below the bottom of the hole. The vane was then rotated in the soil and the torque necessary to turn the vane was measured. From this torque the shear strength was calculated by the following formula

$$S = \frac{M}{D^2 \left(\frac{H}{2} + \frac{D}{6} \right)} \dots\dots\dots (54)$$

where M is the maximum torque and H and D are the height and diameter of the vane respectively.

In practice the speed of rotation of the vane has a small effect on the value of S, but it has been found by Carlson (30) that for a rate of rotation of 0.1 degree/sec. the values of shear strength agree with those obtained in the laboratory in the consolidated undrained triaxial test. Once the maximum torque had been recorded for calculating the undisturbed shear strength, the vane was rotated rapidly for 12 turns to remould the soil and the shear test was then repeated to determine the remoulded shear strength. The results of

TABLE XVIII.

Soil properties determined from miniature samples

Hole 1 East Geduld Slimes Dam

Depth	Moisture content %	Dry density γ_d lb./cu.ft.	Total Density γ_t lb./cu. ft.	Natural voids ratio e
0'0"	16.9	96	112	0.76
6'0"	21.8	94	115	0.78
10'6"	22.8	95	117	0.77
14'0"	22.5	102	125	0.65
16'0"	39.1	85	119	0.97
18'0"	46.6	77	113	1.18
22'0"	17.4	91	107	0.84
26'0"	11.3	87	97	0.93
33'0"	30.5	86	112	0.95
36'6"	23.2	94	116	0.78
40'0"	31.6	80	105	1.09

TABLE XIX.

Soil properties determined from miniature samples
Hole 2 East Geduld Slimes Dam

Depth	Moisture content %	Dry density γ_d lb./cu.ft	Total density γ_t lb./cu.ft.	Natural voids ratio e
1'8"	34.5	83	111	1.03
6'9"	22.3	92	112	0.83
10'6"	27.3	93	118	0.81
15'3"	40.2	78	110	1.14
19'6"	26.7	78	99	1.14
22'4"	33.5	87	116	0.94
26'2"	37.8	19	109	1.12
30'0"	37.2	78	107	1.15
33'0"	28.5	93	120	0.79
38'0"	30.0	87	113	0.93
40'0"	27.3	89	114	0.87
43'6"	41.2	67	104	0.89

TABLE XX.

Soil properties determined from miniature samples
Hole 3 & 3(a) East Geduld Slimes Dam

Depth	Moisture content %	Dry density γ_d lb./cu.ft.	Total density γ_t lb./cu.ft.	Natural void ratio e
2'0"	23.8	89	111	0.88
7'0"	33.7	83	111	1.02
12'0"	31.6	89	117	0.89
17'0"	35.4	82	110	1.06
20'7"	24.7	83	103	1.03
22'0"	10.1	87	996	0.92
26'10"	11.3	86	996	0.94
32'0"	28.8	85	110	0.97
37'0"	9.3	87	95	0.92
41'0"	13.4	91	103	0.85
43'7"	15.9	93	108	0.81
44'0"	25.2	88	110	0.91
47'0"	23.8	92	114	0.82
51'9"	28.2	92	118	0.82
52'2"	29.1	92	119	0.83
55'10"	26.2	94	119	0.78

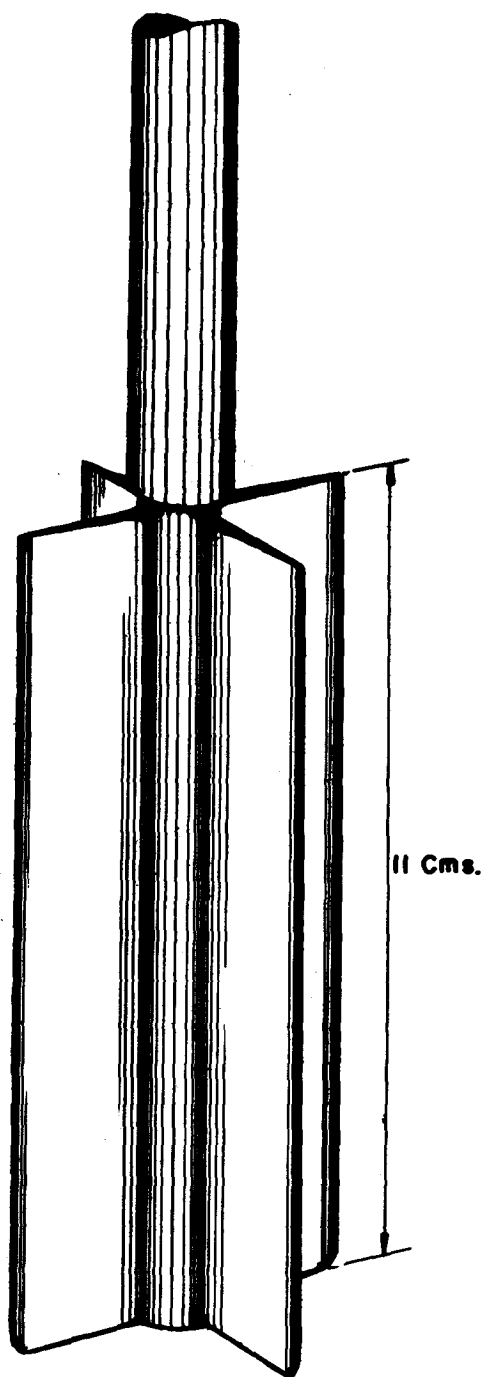


FIGURE 100 VANE SHEAR APPARATUS.

a typical test are plotted in Fig. 101 and other results are given in Tables VII, VIII and IX and Fig. 61 in Chapter V.

Near the surface the holes were drilled easily and the sides remained standing. As the depth increased there was a tendency for the holes to collapse and thus they could not be kept clean. Hole 1 was cased with 4-inch diameter flush-coupled casing. As the casing was in 10-foot lengths and it was only possible to allow 2 feet of casing to project above the surface, the hole had to be advanced 8 feet before new casing could be added. In this way hole 1 was taken down to a depth of 40 feet and no further progress could be made due to the water and slimes filling the bottom of the hole. In hole 2 the same difficulty was experienced and this hole reached a depth of 43 ft. 6 in. In hole 3 the casing was kept filled with water to counteract the upthrust of seepage water on the slimes at the bottom of the hole. The casing was driven into the slimes by means of a drop weight of 140 lb. When a depth of 49 feet was reached the 4-inch casing could be driven no further, owing to side friction on the casing and also to curvature of the hole. The casing was extracted and the hole was enlarged to take 6-inch flush-coupled casing. The 6-inch casing was taken to a depth of 30 feet where it could be driven no further. The hole was then continued using 4-inch casing inside the 6-inch casing. After a depth of 50 feet had been reached, sampling and testing was recommended: this was designated hole 3(a) as it was a deflection of hole 3. The material was very soft and it is considered that the results obtained are not very reliable owing to the amount of disturbance which took place. This hole penetrated the foundation soil which was lateritic.

At a later date 8 holes were put down by hand-auger and jetting and 8 piezometers of the type installed at St. Helena slimes dam (Appendix 1) were installed, in two rows of 4 each approximately 800 feet apart. The one set was placed at the same place at which the sampling had been done and where there was no weeping, the second set was placed where there was evidence of seepage flow. The phreatic surfaces for the two sets of points are shown in Fig. 102. It will be seen that the phreatic surface in the zone of weeping was much higher than in the zone where there was no weeping. Unfortunately these piezometers were covered with slime soon after their installation and were not found again.

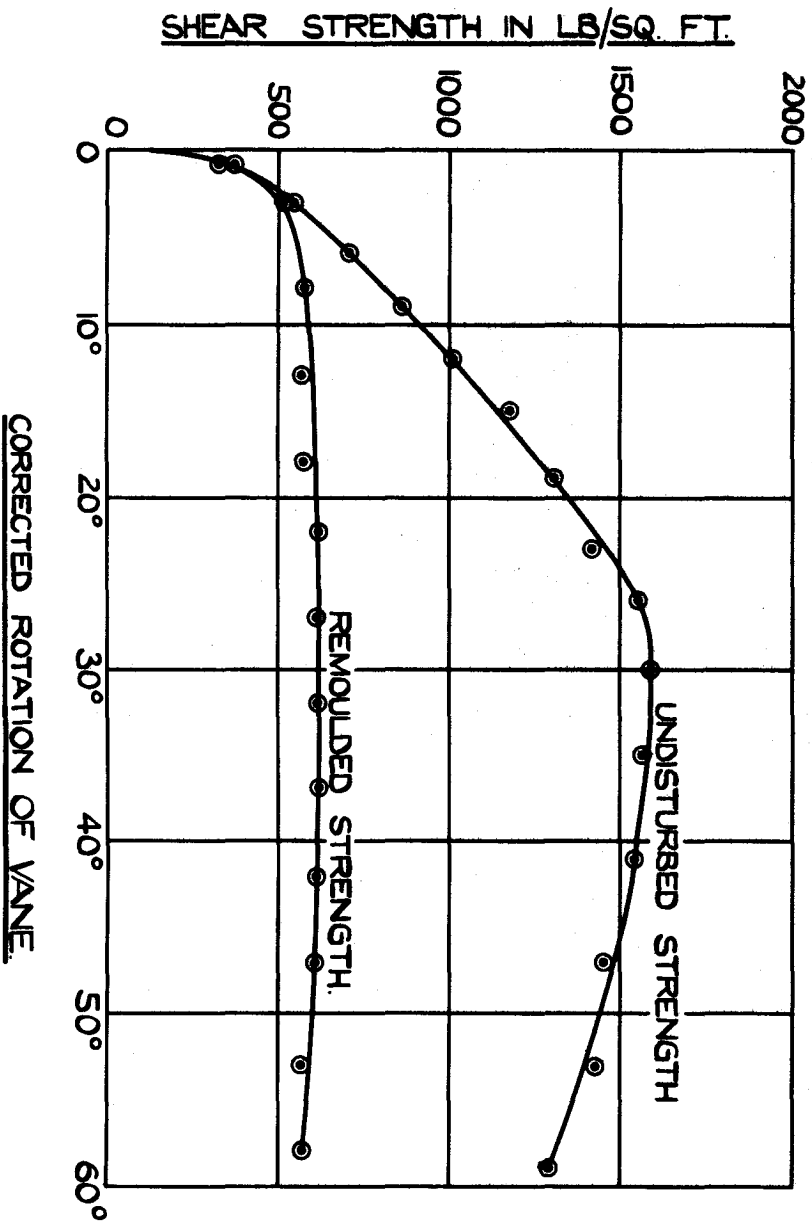


FIG. 101.

IN-SITU VANE SHEAR TEST RESULTS AT DEPTH 21' 6" IN HOLE 2.

ZONE WHERE THERE IS NO WEeping 1, 2, 3 & 4 -----
 ZONE WHERE WEeping OCCURS 5, 6, 7 & 8 _____

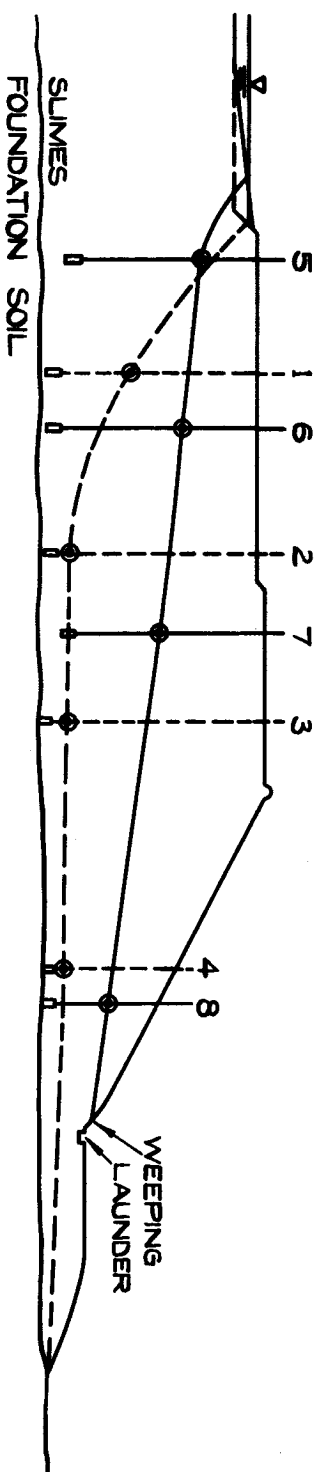


FIG. 102.

PHREATIC SURFACES IN EAST GEDULD SLIMES DAM.

APPENDIX 3.

LABORATORY DESICCATION TEST

The laboratory desiccation tests were carried out in large containers into which a slurry of slimes was placed and allowed to dry out. During the test the negative pore water pressures were measured by an apparatus similar to the pore water piezometers used by the Building Research Station (31) in earth dams.

The circuit diagram for the apparatus is shown in Fig. 103. The piezometer head consisted of a hollow plastic body with a sintered glass disc on the front and two leads into the cavity behind this disc. The reservoir was filled with air-free water and the taps leading into the reservoir were closed. Taps B, C, E and G were closed and taps A, D, F, H and K were opened; air was then pumped into the balloon in the reservoir and in this way air-free water was pumped through the piezometer head to force out any air that might be in the system. The direction of flow was then reversed by opening taps G, F, D, B and K and closing taps H, E, C and A. The vacuum gauge was filled with air-free water by closing taps B, D, F and G while opening taps A, C, E, H and K and after a while reversing the flow by closing taps H, F, D and A and opening taps G, E, C, B and K. The apparatus was then ready for the test and all the taps were closed.

The pore water pressure in the soil was transmitted to the water in the apparatus through the sintered glass disc in the piezometer head. The measurement was done by opening taps C and D and taking a reading and then closing these taps and opening taps E and F and taking another reading. If the two readings were the same the apparatus was functioning satisfactorily. If the two readings were not the same, it meant that air had entered one of the leads. Although every precaution had been taken to eliminate this form of trouble, it did occur frequently, necessitating the process of flushing described in the previous paragraph to remove the air from the system. This process had to be carried out as quickly as possible to prevent too much water being pumped into the soil through the piezometer head.

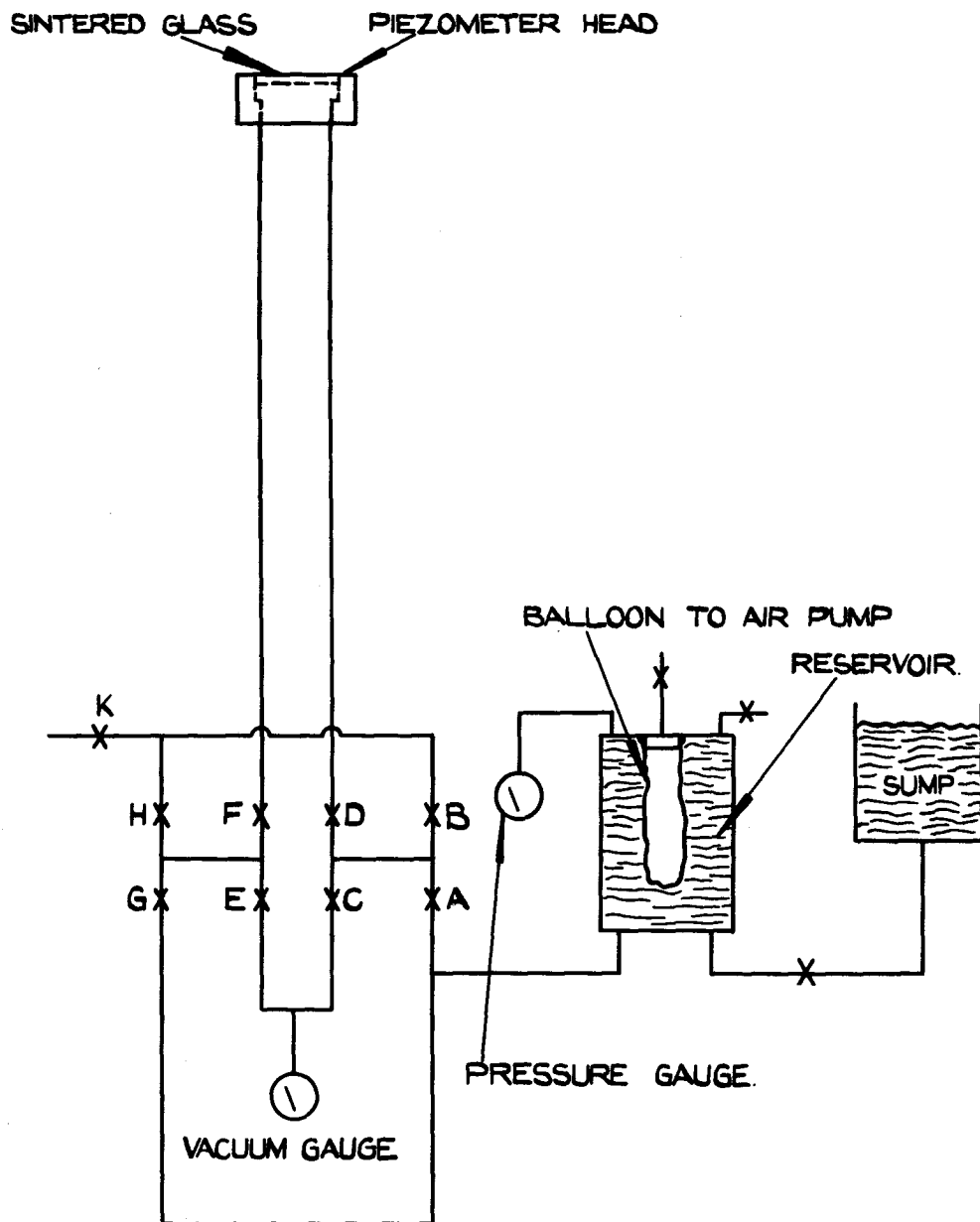


FIG. 103.

CIRCUIT DIAGRAM FOR NEGATIVE PORE WATER PRESSURE MEASUREMENT.

Eventually the stage was reached where the vacuum gauge reached its limit and the tests were terminated. Samples were cut for consolidation tests.

In the multiple tests, 4 piezometer heads, each with its own gauge, were connected into the system so that each one could be flushed independently while the remaining gauges were isolated.

Results of these tests are given in Table IV and Figures 46, 47, 48 and 49 in Chapter IV.

APPENDIX 4.

MODIFIED TRIAXIAL COMPRESSION TEST FOR OVER-CONSOLIDATED
SLIMES

The triaxial compression tests carried out on the slimes were treated as standard tests as far as possible but there were some special features in these tests. The tests were carried out as explained below on $1\frac{1}{2}$ inch diameter samples in a standard triaxial machine as illustrated in Fig. 104.

The pore pressure gauge (L) was fitted to the machine base plate (N) under water to prevent air bubbles being trapped in the leads. The burette (M) was filled with clean water and the tap (K) was opened to allow the water from the burette to flush out any air trapped in the base plate (N) and the tap was then closed. The rubber membrane (B) which fitted round the sample, was slipped over the base plate and sealed with a rubber sealing ring (C). The tap (K) was then opened and water from the burette was run in to fill the bottom of the membrane to a depth of $\frac{1}{2}$ inch. The $1\frac{1}{2}$ inch diameter split mould (D) was then fitted outside the membrane which was turned over the top of the mould and pulled taut and smoothed on the inside to remove any wrinkles. The porous disc (A) was placed on top of the base plate and a $1\frac{1}{2}$ inch diameter disc of filter paper was placed on the porous disc. This filter paper was placed very carefully to avoid trapping any air bubbles underneath it. Four strips of filter paper $\frac{1}{2}$ inch wide and 4 inches long were then placed down the sides of the membrane with the tops hanging $\frac{1}{2}$ inch over the top of the mould. These strips were wet and smoothed against the membrane so that no air was trapped between the filter paper strips and the membrane.

A thick slurry of slimes was well mixed with water. This slurry was then poured slowly into the mould taking care to see that there was always a layer of water above the slurry in the mould and agitating the slurry in the mould to release any trapped air bubbles. This was continued until the slurry was level with the top of the mould. A few minutes were allowed for the slurry to settle out and excess water collected on top of the sample. The ends of the filter paper strips were folded on to the

- A POROUS DISC.
- B RUBBER MEMBRANE.
- C RUBBER SEALING RING
- D SPLIT MOULD.
- E UPPER CAP.
- F SEALING RING.
- G PLASTIC CELL AND COVER.
- H PIPE FROM CELL TO PRESSURE RESERVOIR.
- K TAP TO BURETTE.
- L PORE WATER PRESSURE GAUGE.
- M BURETTE.

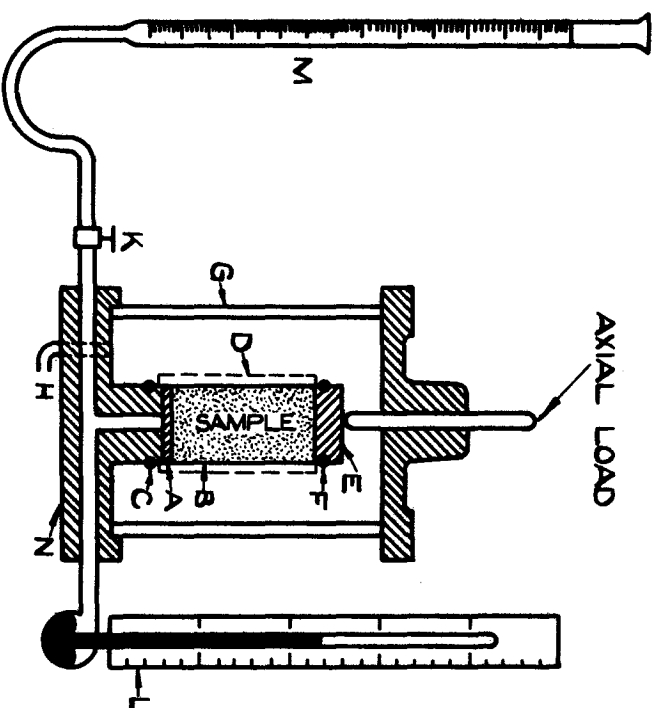


FIG. 104.

DIAGRAMMATIC SKETCH OF TRIAXIAL COMPRESSION MACHINE.

top of the sample and a $1\frac{1}{2}$ inch diameter disc of filter paper was placed on top of this. The top of the rubber membrane was then slipped off the top of the sample mould and the upper cap (E) was placed in position inside the membrane; the water above the sample was smoothed past the sides of the cap to remove any air bubbles that were trapped there. The sealing ring (F) was then placed round the membrane and upper cap. The sample was then ready for consolidation and the plastic cell and cover were placed in position. The filter paper discs and strips were used as drains to assist in the consolidation of the sample and by equalizing the pore water pressure throughout the sample during shear testing.

The water level in the burette (M) was read and the cell was filled with water from the pressure reservoir (H); the pressure in the cell was then taken to a consolidating pressure of 5 lb./sq.in. and the tap (K) to the burette was opened. The sample mould could not be removed before the test as the sample of slurry would have collapsed. After two hours sufficient consolidation had occurred to allow the sample to stand on its own. At this stage the burette tap (K) was closed, the cell pressure was released and the cell was then emptied. The cell cover was removed and then the split mould was taken off the sample. The cell cover was replaced and filled and the cell pressure then set at the desired consolidating pressure for the test. The burette reading was taken and the burette tap (K) was opened. Burette readings were taken regularly and a consolidation curve was drawn. When this curve showed full consolidation the sample was ready for the next step. In practice the samples were left to consolidate for between 18 and 24 hours although 90 per cent of the consolidation was found to occur in the first hour.

If a normally consolidated sample was required, the burette tap was closed and the axial load was applied in the normal way. The pore water pressure was read during the test on the simplified pore water pressure gauge (H) as suggested by Collins (32).

If it was to be a rebounded sample, the burette tap was left open and the cell pressure was reduced to the rebound pressure. Again 18 to 24 hours were allowed for full rebound to occur. The samples were then tested in the normal way with pore pressure measurement.

In the first tests, difficulty was experienced with unexpectedly high pore water pressure in the samples. It was then found that some of the rubber membranes had punctured after being kept under pressure for 2 or 3 days and others, while not punctured, were allowing a large amount of leakage. This was overcome by giving the membranes 2 coats of Bridgeport water repellant before use and the results were entirely satisfactory.

The results of these tests are given in Tables VI, X, XI and XII and in Figures 59, 60, 64 and 65 in Chapter V.

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GLOSSARY OF TECHNICAL TERMS

actuating forces: forces tending to cause a slip failure.

angle of internal friction: the tangent of the angle of internal friction is the coefficient relating the frictional forces developed within soil to the intergranular pressure within the soil.

angle of shearing resistance: the tangent of the angle of shearing resistance is the coefficient relating the increase in shearing resistance of a soil to the increase in pressure in that soil.

anisotropic: not having the same physical properties in all directions, as opposed to isotropic.

blanket drain: a layer of free draining material placed under an embankment to collect seepage water.

breakaway: a term used by slimes dam builders to describe a shear failure of the wall.

cohesion: that part of the shear strength of soils due to the attraction between particles.

consolidation: the process by which a soil is inelastically compressed causing a decrease in the voids in the soil and an increase in intergranular pressures.

day wall: the outer wall of the slimes dam which is built during the daytime.

delivery point: the place on the slimes dam where the slimes pumped from the reduction plant is discharged from the pipes on to the top of the dam.

dry density: the unit weight of a soil with all water replaced by air.

factor of safety: ratio of the forces resisting failure to the forces tending to cause failure.

filter layers: graded layers of material between a fine material and a coarse material arranged to prevent the fines being washed into the voids of the coarse material.

floor of dam: the area of the top of a slimes dam contained within the outer walls of the dam.

flow slide: a failure of a slimes dam where the material in the dam liquefies and flows out over the surrounding area.

foundation soil: the soil on which the dam is built.

freeboard: the difference in elevation between the top of the outer wall and the top of the water in the pond.

ground water level: the water table in the foundation soil outside the dam before the dam was built.

homogeneous: consisting of the same material throughout.

intergranular pressure: that part of the total pressure at any point in the soil which is carried by the grain structure at that point.

isotropic: having the same physical properties in all directions.

negative pore water pressure: the tension in the water in the voids of a non-saturated soil.

night wall: the inner portion of the outer wall between the day wall and the floor of the dam.

normally consolidated: the state when the intergranular pressure is the greatest that has ever obtained in that sample.

normal hydrostatic pressure: the pressure in the water due only to the height of the free water level above the point considered.

overburden pressure: pressure due to the weight of the column of soil above the point considered.

overconsolidated: having previously been subjected to a higher intergranular pressure than the intergranular pressure at the time considered.

penstock: the overflow weir and pipes used to remove excess water from a slimes dam.

permeability: the rate at which water flows through a soil when the pressure gradient is unity.

phreatic surface or line: the upper flow line, or boundary, of seepage water flowing through a mass of soil.

piezometer: a gauge or standpipe used for measuring fluid pressure.

pond: the pool of water round a penstock on a slimes dam.

pore water pressure: that part of the total pressure at any point in the soil which is carried by the pore water at that point.

preconsolidation pressure: the greatest intergranular pressure which has ever obtained in a soil sample.

pressure deficiency: the tension in the water lenticals in a non-saturated soil.

rebounced material: material which has been consolidated at higher pressure than the pressure under which it obtains at the time considered.

resisting forces: the forces in the soil which tend to resist a slip failure.

slip circle: the circular arc used to represent the failure plane in stability analyses.

submerged density: the reduced density of the material due to its being submerged under water.

total density: the unit weight of the soil mass including both the soil grains and the water in the pores.

virgin consolidation curve: the relationship between void ratio and the logarithm of intergranular pressure for a soil consolidated from zero intergranular pressure.

void ratio: the ratio of the volume of the voids in a soil to the volume of solids in the soil.