A STUDY OF THE COMPOSITION AND DENSITY OF SLURRY FOR STABILITY OF TRENCHES

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By

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C O N T E N T S

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C E R T I F I C A T E

Certified that the dissertation entitled ," A STUDY OF THE COMPOSITION ~~OF~~ AND DENSITY OF SLURRY FOR STABILITY OF TRENCHES" which is being submitted by Mr C. N. P. TEM in partial fulfilment of the requirement for the award of the Degree of Master of Engineering in Water Resources Development of the University of Roorkee, Roorkee, is a record of student's own work carried out by him under my supervision and guidance as internal guide. The matter embodied in this dissertation has not been submitted for the award of any other degree or diploma.

This is further to certify that he has worked for a period of 6 months from 1st October, 1974 to 31st March,1975 for preparing this dissertation for Master of Engineering (WRD) of this University.

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A C K N O W L E D G E M E N T S

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CHAPTER - I

GENERAL

1.1 INTRODUCTION

The techniques of stabilization of excavations by clay suspensions have developed rapidly during the last 15 years. However, the mechanism by which the clay slurry stabilizes such excavations is not completely understood. The stability of circular excavations can be explained adequately by the hydrostatic pressure of the slurry and arching in the soil. With the application to rectangular excavations, in particular continuous trenches, the hydrostatic pressure of the slurry was found insufficient in certain cases, to stabilize the trenches according to classical earth pressure theory. Consequently other stabilizing mechanisms were proposed to account for this discrepancy[1].

Slurries are suspensions which are thixotropic. They have the property of becoming fluid on agitation and of settling to a gel when left undisturbed. Thixotropic suspensions can be made from a variety of materials but the cheapest and the one used very frequently universally is bentonite.

Bentonite slurry is a colloidal suspension formed by mixing powdered natural bentonite, and water, at an appropriate ratio. The bentonite and water must be agitated vigorously to ensure complete hydration of the bentonite particles.

1.2 USES AND ADVANTAGES OF SLURRY TRENCH TECHNIQUE[17]

This technique prevents any disturbance to the surrounding soil during work, thus avoiding the sort of damage to nearby structures which often results with the normal techniques of basement excavation. The diaphragm forms part of the final work providing peripheral basement retaining walls.

The system offers the advantage of noise reduction and freedom from vibrations. The construction of diaphragms in populated area has to be such as to give the least annoyance to local inhabitants through unnecessary noise and minimum hazard to adjacent structures by eliminating soil disturbance. Plate 1.1 shows an example of use of diaphragms in a built up area with no likelihood of danger to adjacent structures.

Slurry trench diaphragms solve the problem of constructing underground basement walls for deep excavations adjacent to exsisting shallow foundations. The slurry fills the excavation as work progresses and the costs of dewatering, sheeting or shoring are eliminated. Most of the excavated material may be reused for backfilling the trench after blending it with appropriate proportion of bentonite and suitable soils. This eliminates costs of dumping the excavated material from the work site and the necessity of borrowing special backfilling material.

The most remarkable advantage and use of slurry trench diaphragms is undoubtedly the impermeable cut off created at relatively cheap cost. The Wanapum Dam cut off[15] (to be referred to later) reduced the coefficient of permeability from 1,000,000 ft/yr to only 0.1 ft/year in addition to offering possibility of working in difficult soils without the need for dewatering and danger of caving. The total cost which amounted roughly to $ 7 / ft^2 of the cut off wall was much less than estimated cost for any comparable seepage barrier.

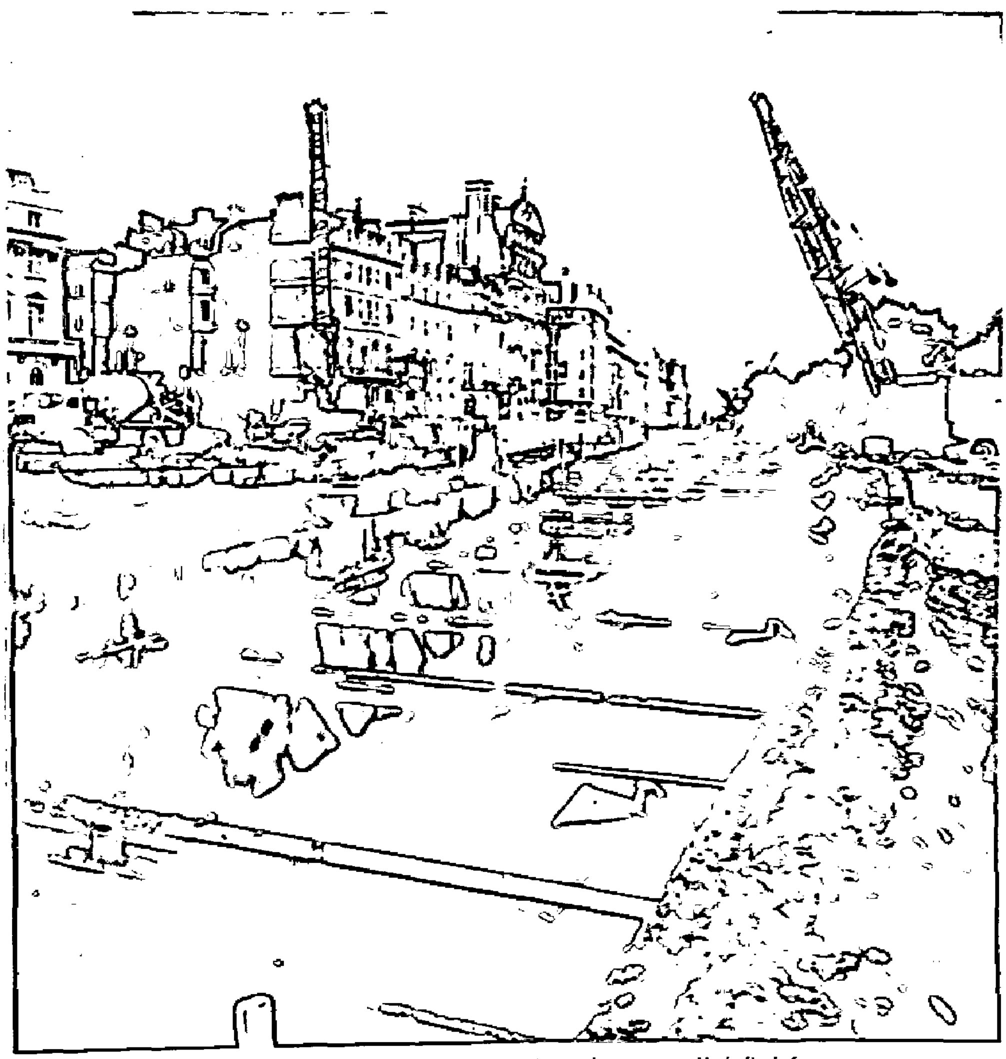


Figure 6. Example of use of diaphragms for underpasses at Hyde Park Corner

PLATE 1.1 USE OF DIAPHRAGM FOR UNDERPASSES

In Australia, at the Grahamstown Dam, due to the great depth to the impervious formation along the central 2,000 metres of the embankment no impervious core was provided during initial construction of the dam, as the cost of providing such a core would have been excessively high[4]. Subsurface investigation revealed that a vertical cut off would need to extend 30metres below the crest (maximum height of dam = 15.2 m) of the embankment, with a cutoff area of 43,500 square metres.

Comparison of alternatives for cutoffs were as follows:-

TABLE 1.1 COMPARISON OF ALTERNATIVE METHODS TO PROVIDE CUTOFF AT GRAHAMSTOWN DAM, AUSTRALIA

Method	Expected flow reduction %	Cost in A $\$$ /m^2 and Remarks.
Upstream blanket	50	Nil But dewatering was necessary.
Sheet Piling	70 - 90	35 - 60
Mixed in place piles	90 - 75	25 - 50
Membrane grouting	90 - 75	20 - 30
Alluvial grouting	90 - 95	150-200.Holes had to be closely spaced.
Slurry Trench (European Method)	90 - 75	35
Slurry Trench (USA Method)	90 - 75	20

Based on economy and achievement of impermeability the bentonite slurry trench method was adopted. At the time of construction, the permeability was reduced to the order of 10^{-5} to 10^{-6} cm/sec.

According to a survey conducted by L.B. Foster Company of Chicago, Illinois, it was revealed that more than half of all construction deaths which occurred in the United States in the first half of 1974, were the result of excavation failure and accidents involving trench cave-ins[16]. Though there are no records to indicate what the total fatalities resulting from trench accidents might be world wide, neither is there reason to believe that they occur with any less frequency elsewhere. The slurry trench technique is one of the most economical methods which can be used as a remedy to this hazard. Noise and disturbance reduction, and where applicable, water tight cutoffs for dams, are additional achievements of the method. This explains why this technique has been, in the past 15 years, widely adopted in various engineering projec ts.

1.3 DEVELOPMENT OF THE SLURRY TRENCH TECHNIQUE

The first experimental diaphragms in rectangular trenches were carried out in 1950 in Italy. The method was to enable constructing in-situ continuous reinforced concrete structures to unlimited depths. The structures had to be impermeable and able to carry vertical loads and capable of withstanding bending moments and shearing forces. The construction of such diaphragms had to be feasible in difficult soils, such as those with large boulders and it had to be possible to seal off the diaphragm in the underlying bedrock[17].

The slurry trench procedure was used under a permanent dam structure for the first time at Wanapum Dam in Columbia River, Washington State, U.S.A. in 1958[15]. The foundation materials at the site consisted of an erratic deposits of sands, gravels, and cobbles with some openwork gravel, extending to a depth of more than 80 ft (Fig. 1.1). The coefficient of permeability of the various layers ranged roughly between 100,000 and 3,000,000 ft/year and the average permeability was estimated roughly at 1,000,000 ft/year. Studies indicated that costs of installing a satisfactory sheet pile cutoff or of opening and dewatering a trench for a rolled earth cutoff through the pervious foundation soils would have been very high. A 10 ft wide foundation seepage barrier was constructed to a maximum depth of 80 ft by the slurry trench procedure. The excavation, made with a dragline, was kept continuously full of bentonite mud slurry, which prevented the walls from caving so that no sheeting or shoring had to be used. The excavated material (which consisted of a mixture of sand and gravel with bentonite slurry) was stock piled in windrows adjacent to the trench, and blended by means of dragline and dozers with a quantity of 15 to 20% of natural silt. This mixture was then dumped from one end of the trench displacing the bentonite slurry until the backfilling was completed.

Construction was carried out with a minimum trouble. Although heavy equipment worked very close to the edge of the trench, no cavings of the trench walls occurred, even though the trench was kept open for lengths of as much as 1000 ft before backfilling commenced. Air pressure tests made in boreholes drilled in the slurry cut off, indicated that the material had a coefficient of permeability of less than 0.1 ft/year.

There has been a continous improvement in the technique since then, at the same time the field of application has

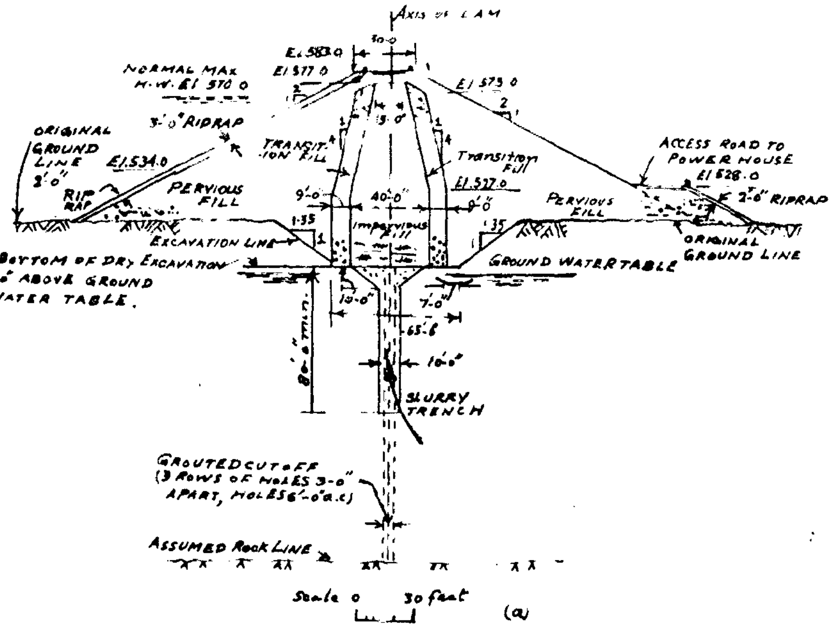


FIG 1-1 CUTOFF UNDER WANAPUM DAM

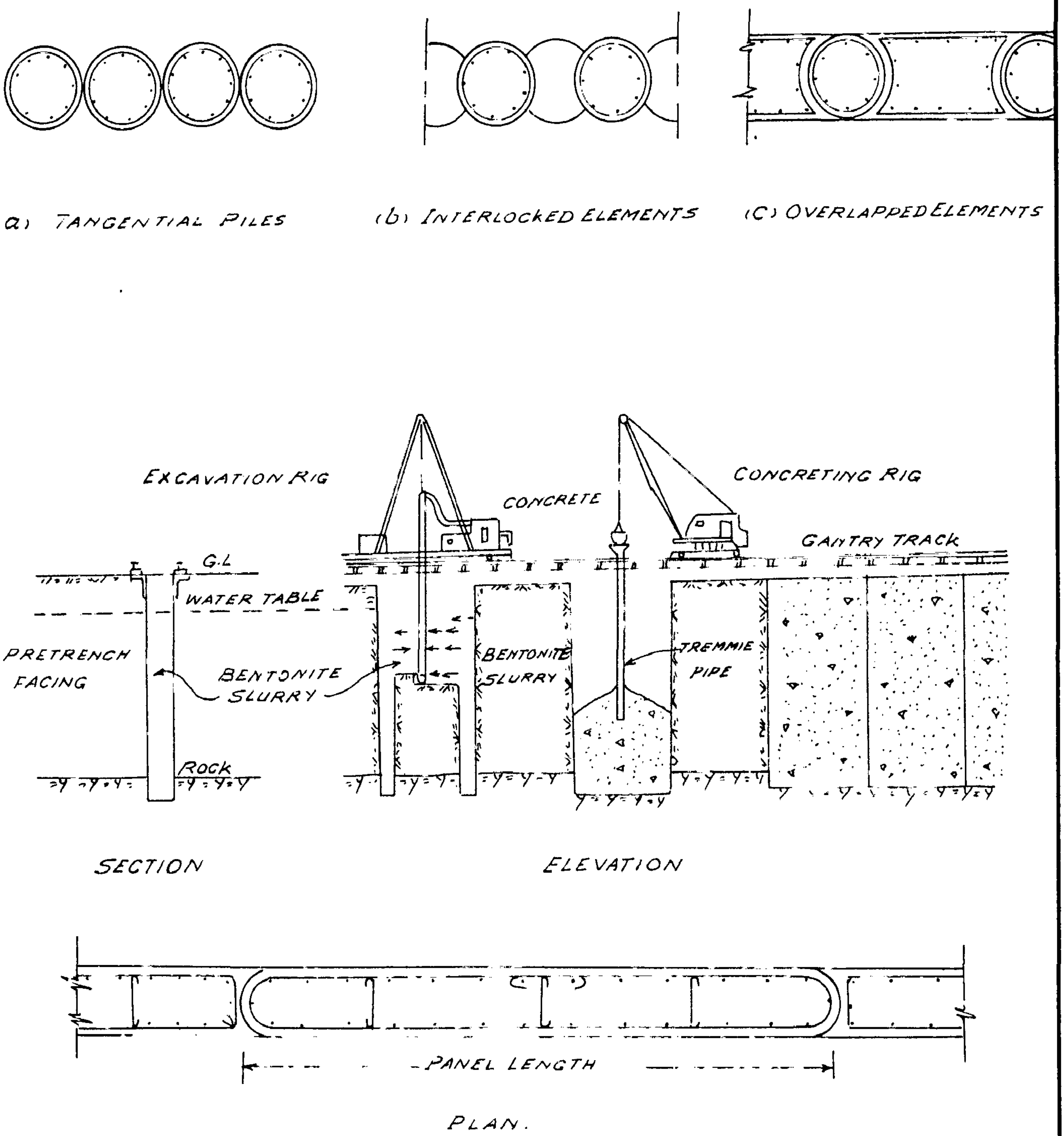
steadily widened.

Two methods of construc tion of slurry trenches are in practice - the European and American methods. In the former method, a rigid concrete diaphragm or wall is installed in section or panels by bentonite slurry trench process as shown in Figure 1.2 . This method can be used for cutoff and retaining walls for dams, river works, canals, locks, traffic underpasses, basements to buildings and other load bearing structures.

The American method is mostly used for construction of cutoffs for dams. The trench is excavated continuously in the presence of bentonite slurry without panelling (Fig. 1.3).

Apart from developments and improvements of the slurry trench method from the construction point of view research work on the composition and properties of bentonite slurries have been carried as outlined in Chapter Two.

Besides the Wanapum Dam, the slurry trench method has been used at the following places to provide impermeable cutoffs[2,6].



(d) METHOD OF PANELLED CONSTRUCTION (NOT TO SCALE)

FIG. 1.2 DEVELOPMENT OF SLURRY TRENCH CUT OFF IN EUROPE

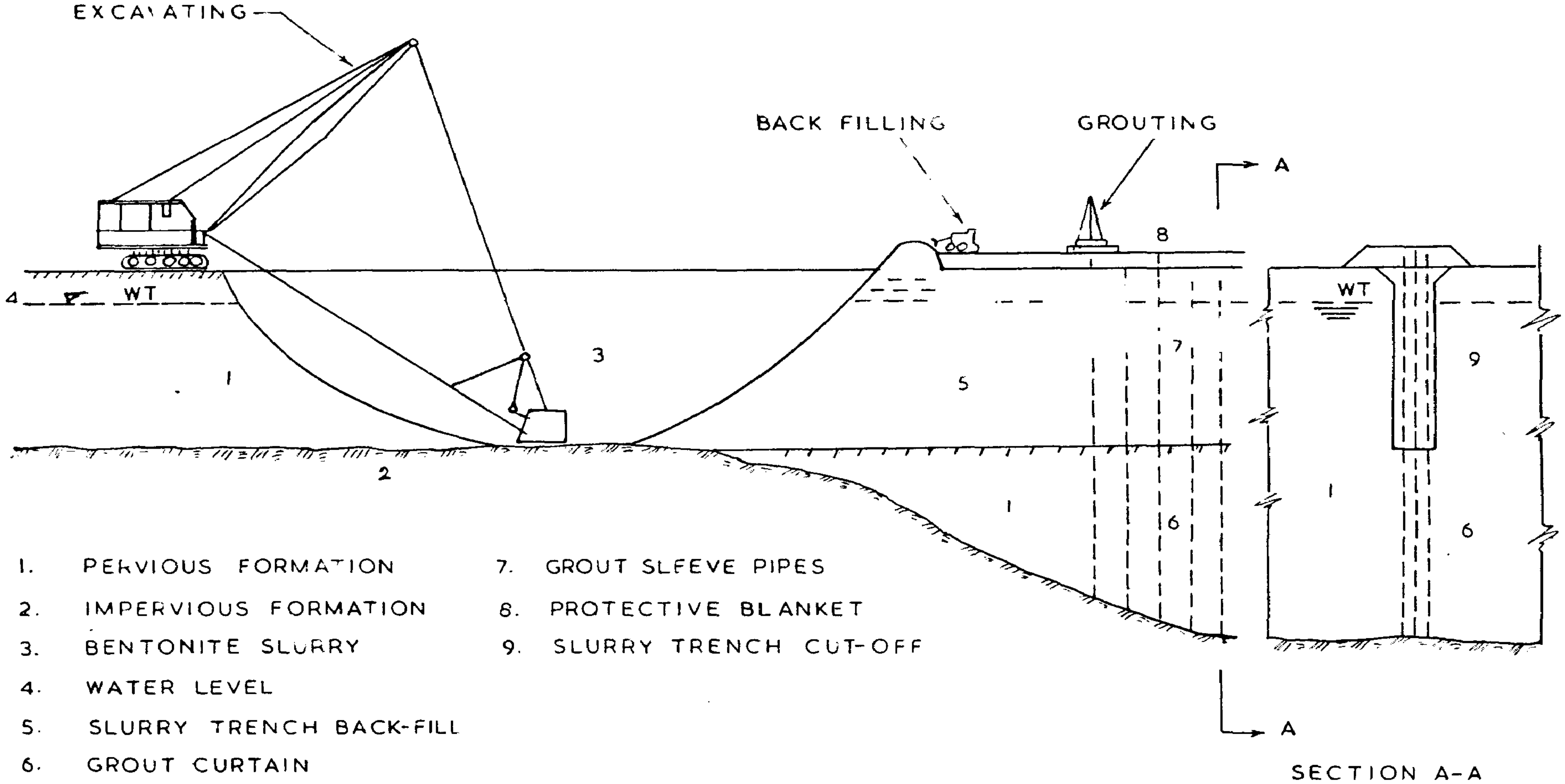


FIG 1·3 AMERICAN METHOD OF CONSTRUCTING SLURRY TRENCH CUT OFF WITH GROUT CURTAIN

TABLE 1.2 PLACES WHERE SLURRY TRENCH TECHNIQUE HAS BEEN UTILIZED

Sl. No.	Place	Maximum Depth of cut off m
1	Kenewick levees, McNary Dam Project, U.S.A.	6.7
2	Mangla Closure Dam, Pakistan	6.7
3	Duncan Lake Dam, Canada	18.3
4	West Point Dam, U.S.A.	30.5 (Grouting in sound rock below trench)
5	Saylorville Dam, U.S.A.	18.3
6	Brokopondo Project, Suriname River, South Africa	4.6
7	Wells Dam, U.S.A.	24.4
8	Yards Creek Lower Reservoir, U.S.A	12.2
9	Camanche Dam, California, U.S.A.	13.7
10	Bracis, Italy	10.0
11	Pietraporzio, Italy	18.0
12	Quero, Italy	19.0
13	Maria al Lago , Italy	42.0
14.	Isola Serafini, Italy	
	i. Power House	33.0
	ii. Levees	20.0
	iii. Dam	20.0
15.	Pierre Benite, France	28.0
16.	Tanughat, India	16.0
17.	Obra Dam, India	-
18.	Ukai Dam, India	20.0

1.4 SCOPE OF DISSERTATION

It is intended to compile and study the composition and properties of bentonite slurry from research work carried out by different engineers and scientists. The stability phenomenon of slurry trenches, its different factors and the effect of composition and properties of bentonite slurry have also been studied. This is outlined in Chapter Two.

An experimental work has also been carried out intended to find out the effect of replacing different proportions of bentonite by black cotton soil in four different slurry concentrations. Whereas bentonite is readily available in certain locations only, black cotton soil occurs virtually in all places at large. As found out in the experimental results, replacement of bentonite by black cotton soil in the slurry concentrations studied (1 to 4%) ,does in fact results in improvement of stabilizing capability in addition to the economical gain. Though the results obtained are interesting , yet it is the author's impression that further research work on the subjec t is necessary to help in understanding the problem fully and to enable,at least partially, in filling the gap that exists in understanding fully the phenomenon of stabilizing of slurry trenches by bentonite slurries. This gap exists due to the numerous variables involved and ample time is needed to study the effect of each and its variations. The work was mostly confined to dry sand through a few tests on saturated sand were also performed. The following is the range of variables:

Slurry Concentrations	1 % to 4%
Replacement of Bentonite by black cotton soil	0%, 25% , 50%, 75% and 100% for each concentration.

Flow tests were also performed to study slurry characteristics.

NOTATION

a = Half width of trench

C = Cohesive force in soil

C_u = Unit cohesion in soil (undrained test)

C_a = Shearing resistance of slurry saturated zone

E_p = Passive resistance of slurry to ~~resistance~~. displacement

e = void ratio

G = Specific Gravity of soil particles.

g = Gravitational acceleration

H = Depth of Slurry in Trench

mH = Height of water table above trench bottom

h_w = Depth of water table below top of trench

i_e = Critical pressure gradient

K_w = Permeability of soil to water

ℓ = Half thickness of slurry saturated zone

n = Porosity of soil

P_a = Active Earth pressure

P_s = Hydrostatic pressure of slurry

P_p = Total passive resistance of slurry

P_G = Negative gel pressure

p = Unit pressure

q = Unit surcharge

R = Resultant earth force

r = Radius of an equivalent pore passage

S_c = Slurry concentration

u = Pore fluid pressure

V = Mean velocity

W = Weight of failure wedge

θ = Inclination of failure wedge to horizontal

ϕ_D = Developed angle of friction

ϕ' = Angle of frictional resistance

γ = Bulk density of soil

γ_b = Submerged density of soil

γ_s = Density of slurry

γ_w = Density of water

η_w = Viscosity of water

η_{pL} = Plastic viscosity of slurry

τ_o = True yield stress

τ_f = Bingham yield stress

τ_m = Maximum gel strength

τ_s = Structural shear strength of slurry in trench.

CHAPTER II

REVIEW OF PROPERTIES AND RESEARCH WORK DONE ON AND SLURRY TRENCH TECHNIQUE

This chapter will deal with the following aspects

(i) Composition and Properties of Slurry Constituents.

(ii) Stability phenomenon of slurry trenches.

2.1 COMPOSITION AND PROPERTIES OF SLURRY CONSTITUENTS

2.1.1. Composition

A slurry fluid may consist of

(i) Clay

(ii) Bentonite powder

(iii) Water

(iv) Suspended cuttings

(v) Additives which may include sodium and or barium Sulphate.

The larger solid part of the fluid is bentonite powder, and its concentration is one on which stability and other properties of the slurry depend. Too low concentration causes caving and too dense concentration increases the plastic viscosity and the density and may float the excavating bucket of dragline or clam-shell. For the range of concentration studied, the lower the bentonite concentration (upto a certain limit) the less the plastic viscosity and the more the slurry saturated zone. This zone, as shown later, is the main contribution to the stabilizing effect of in dry sand trenches.

Thus depending on economy of availability of materials, construction convenience and the slurry properties required, different proportions of the bentonite powder can be replaced by chosen soil type. Likewise, the concentration, as well as additive proportion can be varied. The effects of these variations are discussed in para 2.2 which deals with experimental work carried out by different researchers.

Backfill Material[6]

The backfill for the trench should have sufficient fines to fill the voids of the coarser material. In order to limit consolidation, the percentage of fines (minus No. 200 sieve size) should not be excessive. About 10 to 25% of fines along with the entrapped bentonite should be sufficient.

The recommended gradation limits for backfill materials are as below :

Screen size or Number (U.S.A. Standard)	% passing by weight.
3" (76 mm)	80 -100
3/4" (19 mm)	40 - 100
No.4 (4.75 mm)	30 - 70
No. 30 (500 μ)	20 - 50
No.200 (75 μ)	10 - 25

2.1.2 Density

A study of available literature (Table 2.1) on field techniques of the slurry trench method reflects that the muds used in practice had densities of between 1.06 and 1.12 T/m^3 , and the actual values in any particular job depended more on the carrying capacity of the mud in relation to the particle size of the material being excavated than on the value related to the unit weight necessary to give support. On account of the inherent yield strength, the slurry manifests a higher density in the trench by keeping the cuttings in suspension. This increase in the density of the original suspension is one of the major factors in the stabilizing effect of the slurry and simultaneously economises the use of bentonite clay[9]. The degree of this sanding, i.e., the size of particles of muck remaining in the suspension, can be controlled in a wide range by changing the properties of suspension supplied to the trench. Generally speaking, sanding of the suspension depends on the following factors[11]: -

i. Structural shear strength (τ_o) of the suspension used.

ii. Method of excavation, and the type of equipment used

iii. Grain size distribution of the soil being excavated.

iv. The methods of handling the suspension (likelihood of separating it from muck etc).

The cuttings are held in suspension by the small yield strength of the slurry. By balancing the submerged weight against the surface drag, CARDWELL has suggested that the diameter, D, of a particle supported by a slurry is given by the equation

$$D = \frac{6\,\tau_o}{(\gamma - \gamma_s)\,g} \qquad \ldots(2.1)$$

where,

D = Equivalent diameter of particle in cm.

γ = Density of solid particle in gm/cm^3

γ_s = Density of slurry gm/cm^3

τ_o = Structural shear strength of slurry in dynes/cm^2

g = Acceleration due to gravity in cm/sec^2

Figures illustrating increase of slurry density in trench by suspension of cuttings are shown on Table 2.1

TABLE 2.1 SLURRY DENSITY INCREASE WITH SUSPENSION OF CUTTINGS

Project	Original Density of slurry T/m^3	Actual density of slurry observed in trench T/m^3
Wanapum	1.07 to 1.08	1.28
Pierre Benite	1.025	1.2 to 1.25
Ukai	1.04	1.08

Though some slurry rheological properties, like yield stress, do change with time, PIASKOWSKI's observations of the density of mud suspension in an experimental trench, revealed that the density, γ_s, of the slurry did not change with passage of time. The trench, 14.7 metres deep, was kept filled with slurry for a period of over 9 months. The observations are reproduced on table 2.2.

TABLE 2.2 VARIATION OF SLURRY DENSITY WITH TIME

Passage of time in days	Slurry density in T/m^3
16	1.310
149	1.297
247	1.291
261	1.291

Slurry density is very sensitive to fluctuations in ground water level. For higher ground water level denser slurries would be required for stability. If mH is the height of water table above the trench (filled with slurry to a height H) bottom, then the required density of the slurry for stability increases with m as shown in figure 2.1. The reason is undoubtedly that the wet soil material below the water table level (together with the water) is heavier than the same dry soil material alone. The water fills up all the pores hitherto occupied by weightless air and the total thrust of the trengh weight side is more. A cutoff under construction at Piere Benite in France had slips when an unexpected flood occurred. In situations where fluctuations of water table are likely to occur, either a high factor of safety in the selec tion of the slurry density has to be allowed or addition of suitable weighting material to overcome collapse of the trench can be substituted.

2.1.3. Thixotropy[6]

This is the property to undergo isothermal gel-to-sol-to-gel transformation upon agitation and subsequent rest. The structural strength of clayey materials which is lost in remoulding is in time slowly removered. The necessary time depends on moisture content, and particularly on the salts in solution in soil water. The gel-sol transformation can be repeated indefinitely without fatigue. Sodium bentonite in suspension has the most pronounced thixotropic properties and consistency for certain ranges of concentration. These properties are mainly due to the large amount of water adsorbed and retained in its structure. The strength of the thixotropic gel formed is dependent on the setting time, the concentration of bentonite in the suspension and the chemical composition of the suspending fluid. Figure 2.2 shows the effect of time on the shear strength of bentonite suspensions.

2.1.4 Flow Properties

Important flow properties for thixotropic muds are-

i. Yield stress and plastic viscosity
ii. Effect of clay concentration
iii. Gelation properties
iv. Yield gradient and limiting penetration
v. Effect of additives
vi. Activity Number.

The flow properties of bentonite may be attributed to its structure[5]. Its structure is such that addition of water or polar liquids to dry solid bentonite causes interlamellar swelling. The mineral possesses ion-exchange properties, and when the exchangeable base is sodium, bentonite swells in water to give thioxotropic suspensions showing anomalous flow properties at relatively low concentrations.

The characteristics swelling ability of bentonite is associated with the presence of like electrical charges on the critical surface, creating a repulsion between surfaces which are in close proximity . When the magnitude of this repulsion exceeds the attractive forces between atoms in neighbouring crystals, swelling takes place until a fresh equilibrium is established . The swelling takes place in one direction only, normal to the major dimensions of the crystals.

Bentonite suspension in water containing less than 1%

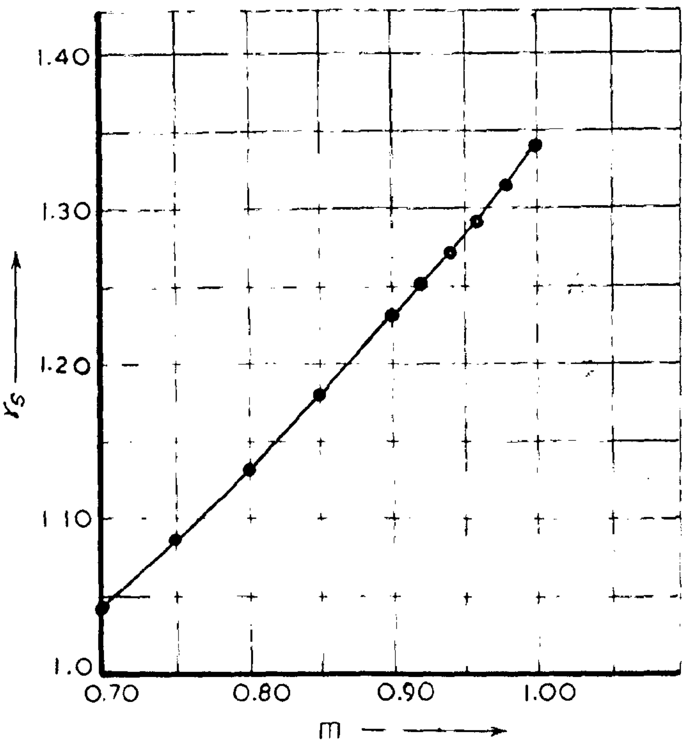


FIG 2·1 EFFECT OF RELATIVE WATER TABLE ON REQD DENSITY OF THE SLURRY

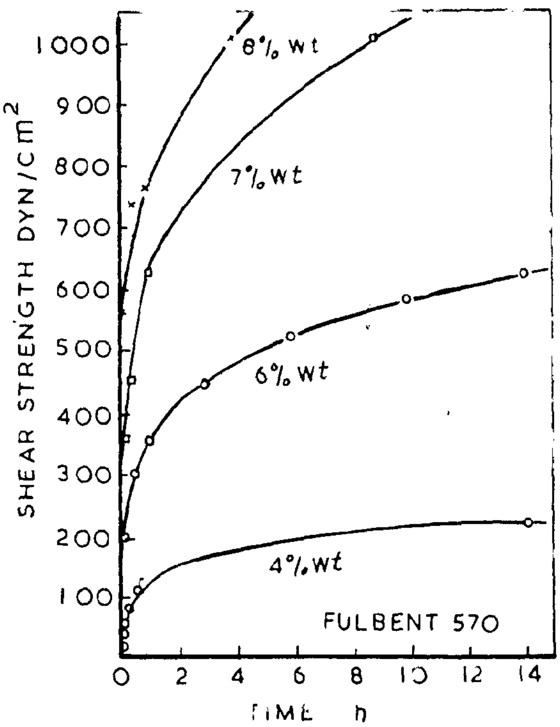


FIG 2·2 THIXOTROPIC GELATION OF BENTONITE SUSPENSIONS IN WATER solids are generally free flowing fluids showing no unusual flow properties. When the clay concentration is raised to 1 to 15% flow properties become anomalous and if a solid content of 50% is reached, stiff putty-like masses are formed. A liquid limit of 350-500 is observed.

Due to the anomaly in the flow property, bentonite suspensions cannot be characterized by one parameter describing viscosity, as can be done with Newtonian fluids such as water. Bentonite must be described by two parameters, the plastic viscosity η_{PL}, and the yield stress τ_f. Both of these factors increase almost exponentially with clay concentration in the suspension in the range 5 to 25 % solids. This anomality of flow exhibited by bentonite is of Bingham bodies. The flow curve no longer passes through the origin as it does ~~per~~ for a newtonian fluid, but makes an intercept τ_f with the shear stress axis as shown in Figure 2.3. Figure 2.4 illustrates the effect of bentonite concentration on the rheological properties of suspensions.

According to BINGHAM,[7] clay slurries start to flow when a certain shear stress τ_f (true shear stress) is reached. Thereafter the flow rate is directly proportional to the excess shear stress ($T - T_f$) . The rate of shear strain $\frac{d\Sigma s}{dt}$ (sec^{-1}) at any point in the material is given by

$$\frac{d\,\Sigma\,s}{dt} = \frac{1}{\eta_{PL}} (T - T_f) \qquad \ldots(2.2)$$

where η_{PL} is the plastic viscosity of the material.

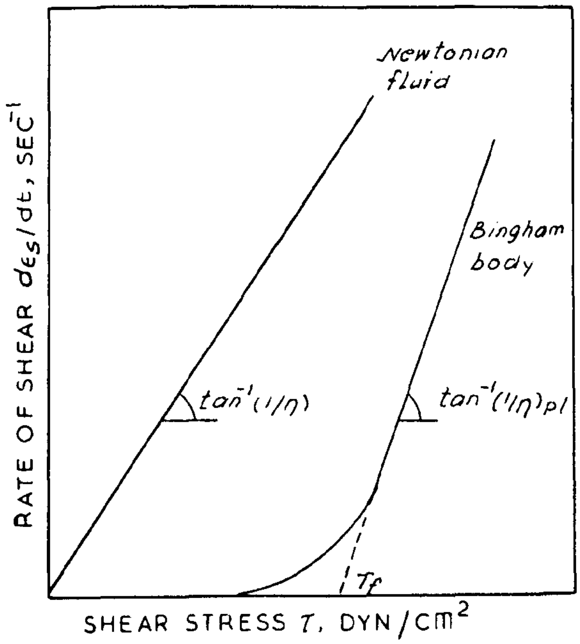


FIG 2·3 FLOW OF NEWTONIAN FLUIDS AND BINGHAM BODIES

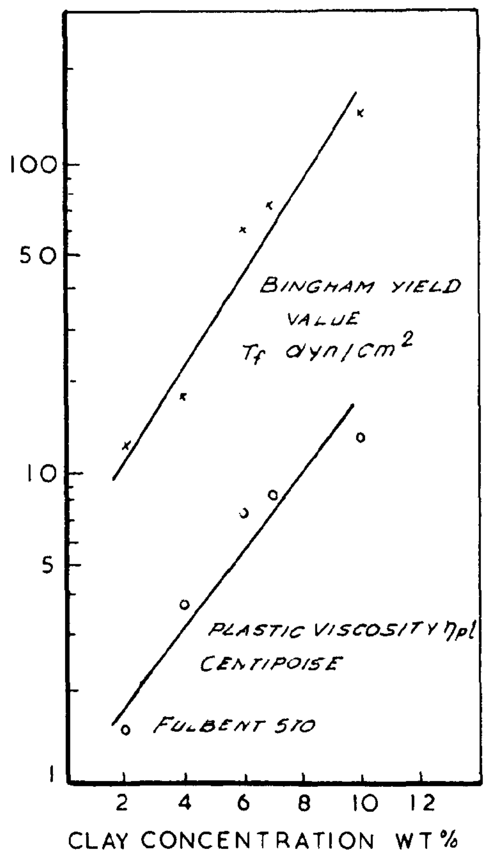


FIG 2·4 EFFECT OF BENTONITE CONCENTRATION ON THE RHEOLOGICAL PROPERTIES OF SUSPENSIONS

Clay slurries have the flow curves of the form shown in Figure 2.5. When the shear stress T_R reaches a certain value T_{R1}, shear failure occurs near the wall of the capillary (if flow is through capillaries) and the slurry moves forward as a plug. As the pressure gradient increases, the diameter of the solid plug becomes progressively smaller until the whole of the material in the capillary flows in a manner like a viscous liquid, and the rate of flow then increases linearly with the pressure gradient. If the intercept on the shear stress axis is T_{R1}, then from Figure 2.6 (a) ,

$$T_{R1} = \frac{R}{2} i_1 \qquad \ldots(2.3)$$

Where i_1 = pressure gradient $\frac{\Delta P}{\Delta L}$ at which flow commences and is called the yield gradient.

Flow of bentonite grout through glass capillary and sand were compared by determining the equivalent size of capillary corresponding to a particular sand bed. This was done by comparing Darcy's law for the flow of fluids through a porous body with Poiseuille's equation for capillary flow. This provided means of estimating the yield and flow properties of a grout in any sand from measurements made on the grout in a capillary viscometer, or a single sand bed of known water permeability.

<u>Effect of Additives on Flow Properties</u>

(Floculation and Peptization)

Bentonite dispersions are sensitive to the nature of electrolyte in which they are suspended. Addition of sodium chloride to a suspension of pure sodium bentonite in water causes at first a slight thinning, then a progressive thicknening. If the clay concentration is low it eventually coagulates and water separates at the top of the suspension. Progressively greater sensitivity towards flocullation is noted as the valency of the cation, added to the suspension, is increased.

A similar effect can be noticed when bentonite is added to salt solutions. Swelling does not take place so readily, particularly in calcium chloride or other solutions containing high valency cations, bentonite will completely fail to swell in strong solutions of many salts.

Whereas addition of sodium chloride causes thickening, addition of sodium salts of polyvalent anions or sodium poly phosphate causes a thinning of bentonite suspensions. The curves of figure 2.7 show the effect of suspensions treated with floculants (e.g. sodium chloride) and peptizers (e.g. sodium polyphosphate). While the viscosity is barely changed, the yield point is markedly influenced. Addition of flocullents to the suspension causes a reduction in the thickness of the electrical double layer existing at the surface of the particles, simultaneously reducing the repulsion between them, and allowing the increased development of links between positive edges and negative surfaces, thereby raising the yield point of the suspension. A further increase in the salt concentration decreases repulsion to the point where particles are able to move closely together causing collapse and coagulation of the dispersion. On the other hand the polyvalent anion of the paptizing agent is sorbed on the edges of the crystals, reducing and even

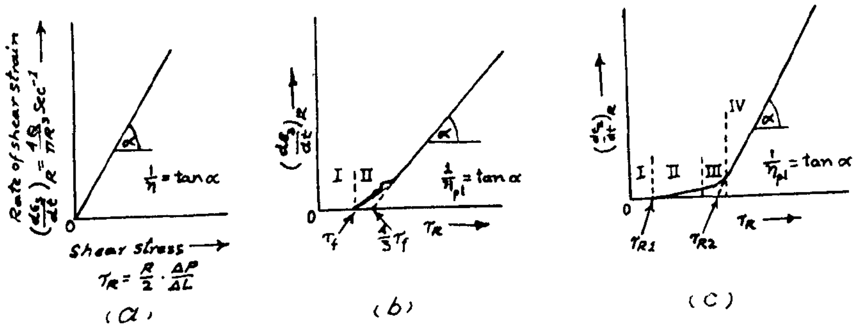


FIG. 2.6 Flow curves for viscous liquids and thin pastes in terms of rate of shear strain and shear stress at the wall of a circular capillary tube (a) Viscous fluid (b) Ideal Bingham material (c) clay paste.

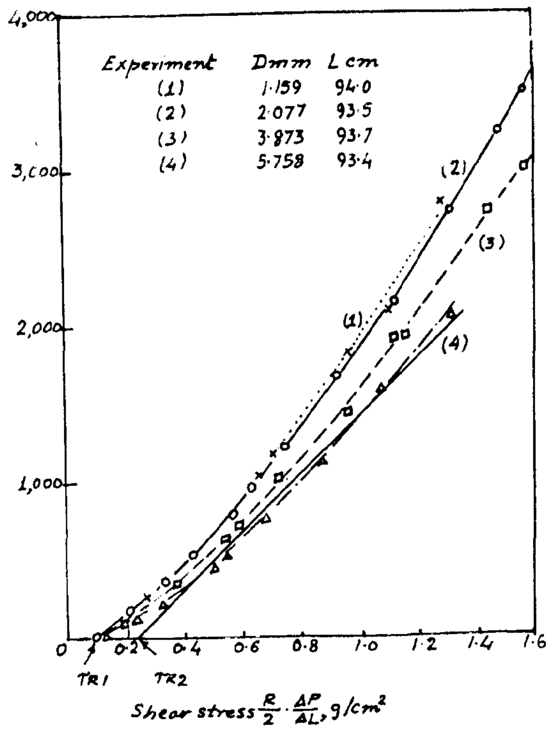


FIG. 2.5 Flow curves for 8 per cent bentonite grout in capillaries of different diameters.

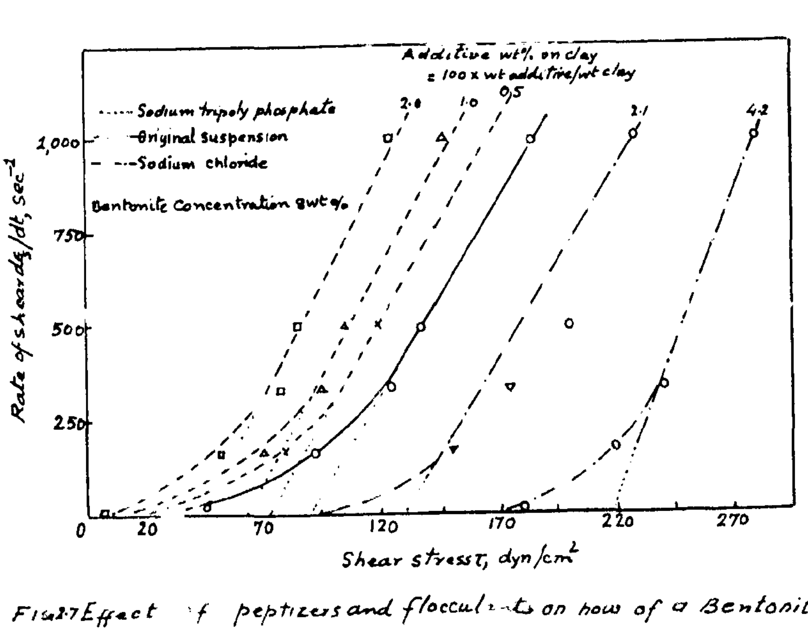


FIG. 2.7 Effect of peptizers and flocculants on flow of a Bentonite suspension

perhaps reversing positive charge, eliminating the forces giving mechanical structure to the suspension, and the yield point is reached without coagulation occurring . Thus the flocculating tendency of polyvalent cations may be offset by addition of peptizing agents giving a simple means of controlling the yield value under a variety of environmental conditions. The addition of peptizing agents to bentonite prior to contact with flocculating ions markedly increases their resistance to deterioration.

Relation between rheological characteristics of grouts and their capacity to permeate soils[13]

Mechanics of Permeation

The movement of fluid through the pores space of a soil is resisted by drag at the interface between the grains and the fluid. For true or Newtonian fluids , the drag is proportional to viscosity and shear rate - the shear rate being determined by means of flow velocity and the geometrical characteristics of the pore space.

Colloidal suspensions such as clays and cement pastes are semi-stable dispersions of particles and are non-newtonian fluids. They are more complex fluids than newtonian, and the resistance to movement is generally related to the rate of shear and the accompanying shear stress. Where shear stress is exhibited, there may remain a significant opposing drag even for a vanishingly small flow rates.

Experiments conducted showed that colloidal fluids with varying bentonite concentrations had their shear rate diminish with time from the instance of mixing. A 5% suspension had an initial shear rate of about 1000 sec^{-1} and movement ceased after 45 seconds when the radius of permeation was 5 cm. The shear strength then was 10 dynes / cm^2. With bentonite concentration of 7% , movement stopped after 15 seconds with radius of permeation of 2.5 cm. The limiting penetration corresponded to a grout with an average shear strength of 25 dynes/ cm^2.

It is thus evidenced that shear strength of bentonite fluid builds up with time and that the higher the shear strength the lower the shear rate. This is in agreement with G.K. JONES results of figure 2.4. Higher concentrations of bentonite also give rise to higher shear strength in a comparatively short time. This is an indication that the distance of permeation may be approximately taken as inversely proportional to the bentonite concentration (and hence the shear strength of the fluid) and the time elapse after mixing[7]. The type of fluid to be used for a particular soil may be decided on this guidance by considering the permeation required, the stabilizing force of the fluid, and the time of injection after mixing.

2.1.5 Yield Gradient and Limiting Penetration[13]

The last article discusses the factors governing the movement of a fluid through the pore spaces of a soil. Thus if a suspension which has a definite strength is to penetrate a soil formation under constant pressure, the opposing drag due to the corresponding shear stress acting at the growing area of the surface wetted by suspension ultimately becomes equal to the whole of the applied pressure so that none is available to maintain the viscous flow.

The numerical value of the opposing drag can be estimated by using a model in which the void passages between granules are regarded as replaced by cylindrical tubes. Raffle and Greenwood[13] deduce that an extra pressure gradient of $\frac{2\tau_f}{r}$ must be applied at all parts of the advancing fluid to overcome the shear strength during penetration, where τ_f is the Bingham yield stress for the fluid and 'r' is the effective radius of an average pore passage. As a first approximation, they assess the value of 'r' from the corresponding Kozeny equation.

$$\frac{dp}{dx} = \frac{8\,\eta_w\,\bar{v}}{r^2\,n} = \frac{\gamma_w\,g\,\bar{v}}{K_w} \qquad \ldots(2.4)$$

where, dp/dx = pressure gradient in dynes/cm^3

η_w = Viscosity of water in poise

γ_w = Density of water in gm/cm^3

K_w = Permeability of soil to water in cm/sec.

n = porosity of soil formation

$\bar{v}$ = mean flow velocity per unit area in soil in cm/sec.

g = gravitational acceleration in cm/sec^2.

For an average temperature of 20°C , when η_w = 0.01 poise,

$$r^2 = \frac{8.2\,K_w}{n} \times 10^{-5} \text{ Sq. cm.} \qquad \ldots(2.5)$$

Thus for soils of known permeabilities and porosity, the corresponding values of 'r' can be obtained and hence estimate the limiting value of the pressure gradient $2\tau_f / r$ for known values of the Bingham's yield stress T_f .

In case of permeation from the sides of an excavated trench, the slurry cannot penetrate farther than a distance given by

$$X_L = \frac{p\ g\ r}{2\ \tau_f} \qquad \ldots(2.6)$$

(The gravitational constant g will appear if τ_f is in dynes/cm^2 and p is in gm/cm^2).

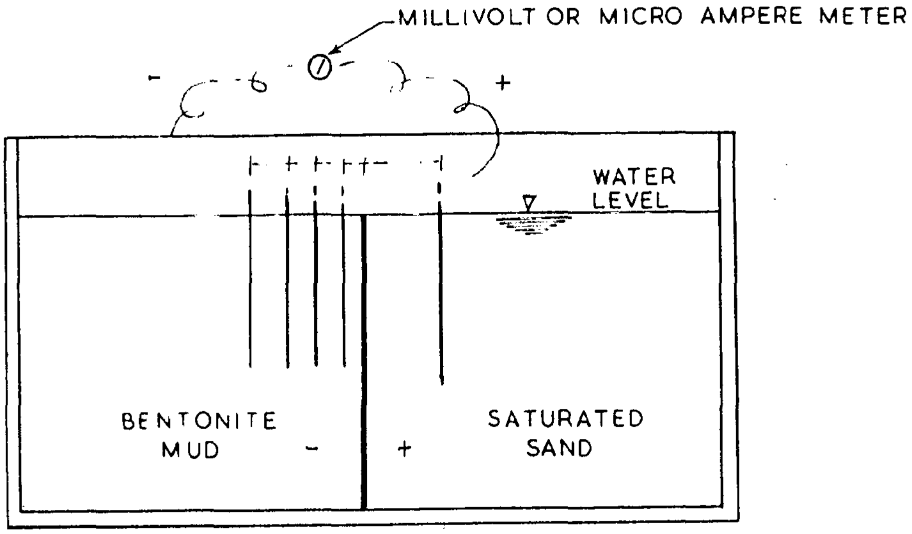
2.1.6 Electrical Properties

M.C. Veder[18] carried out an experiment to determine the current conducting properties of the area of contact between the bentonite mud and cohesionless material and the effect on the interface lining. The experiment was carried out in a tank shown in sketch of figure 2.8. The tank was filled with cohesionless material and a trench was excavated under bentonite mud at constant level. A brass plate electrode was inserted into the bentonite mud and brass rod electrodes into the sand away from the bentonite lining. The electrodes were connected by a 0.8 mm copper wire and the natural existing current was meas ured. Then a direct current was introduced into the circuit and was regularly changed every 4.5 hours.

The results indicated -

i. The natural existing current helped in the formation of the lining whose thickness reached 2.1 cm. The wall collapsed when the level of the mud was lowered by 10cm.

ii. When no current was introduced but the natural current, the thickness of the lining was 1.5 cm as normal and the vertical face collapsed when the level of the mud was lowered by 6 cm.



ELECTRODE IN NATURAL SAND POLARITY +

ELECTRODE IN BENTONITE MUD POLARITY —

MEASUREMENTS FOR MUD ELECTRODE AT 1 CM FROM LINING

TIME	mV	μA
1st Reading at start	32·50	20·00
3 Minute Reading	10 50	19·00
10 Minute Reading	9·60	19 80
15 Minute Reading	9 50	20·00

Note - Measurements at distances of 3, 5 and 8 cms. gave practically the same results

FIG 2·8 ARRANGEMENT OF ELECTRODES FOR MEASURING THE SPONTANEOUS ELECTRIC POTENTIAL AND ELECTRIC CURRENT (VEDER, 1961)

iii. Introduction of an opposing current gradually weekened the lining, especially in the area close to the electrodes. This caused the face of the cohesionless material to collapse although the level of the bentonite mud had not been lowered. However, it led to formation of solid lining of bentonite on the central electrode.

The investigation indicates that the formation of bentonite lining on the vertical face of the cohesionless material can be altered by introducing between the two faces a direct electric current. Formation of the lining and stability will be more favourable if the polarity of the introduced current is the same as that already existing. Opposite polarity weakens the lining and stability of the vertical face of the soil. This offers a method of strengthening stability of slurry trenches if of course, found within economies of scale.

2.2 STABILITY PHENOMENON OF SLURRY TRENCHES

The stability phenomenon of slurry trenches has been studied by different engineers. M.C. Veder, and W.K. Elson have carried out experimental work independently on the subject while Nash and Jones, Mongenstern and Elson have made theoretical approaches. The theoretical approaches are all practically the same, and are based on equating the total disturbing and stabilizing forces on a sliding wedge at the trench face for two dimensional equilibrium.

2.2.1 Experimental Work by Veder and Elson.

2.2.1.1. M.C. Veder

This experiment has briefly been explained on para 2.1.5

the results of which indicated existence of a distinct polarity between the two phases of bentonite and cohesionless material. This polarity (which gives rise to existence of an electro-motive force between the two phases) has a great influence on the formation of a filter cake on the wall of the excavation. Bentonite particles are deposited at the interface by electro-phoresis - a process in which colloids suspended in a medium migrate under the influence of an electric field to cathode or anode, according to the sign of their charge. As will be explained later, the filter cake thus formed has some influence on the stability

2.2.1.2. W.K. Elson[1]

Tests were carried out on small trenches to find out the factors which contribute to stability of the trenches.

The test trench was constructed in a sectional pressed tank with valves for draining the tank . One end of the tank was sealed with a bulkhead fitted with perspex windows to enable complete inspec tion of the trench .

Pressure gauges with sensitivity of 0.01 lb/sq.in(0.00067 kg/cm^2) were used to measure the pressure at the bottom of the trench which is the critical one for failure. Dial micrometer gauges calibrated in 0.001 inch (0.025 mm) per division measured the changes in levels of the sand and slurry surfaces.

The tank was filled with sand in 7.5 cm layers placed under water. Each shovelful of sand was placed in a regular pattern to reduce the effect of segregation to a minimum. A minimum depth of 15 cms of water was maintained above the sand. Each layer of sand was tamped.

After filling the tank with slurry the partition between the sand and the slurry was removed slowly to allow sufficient time for the slurry to penetrate into the sand and stabilize the face of the trench. The surface of the slurry was covered with polythene sheet to reduce evaporation. The slurry was then allowed to gel for a period of 4 hours.

Before testing, the slurry level was adjusted so that it was flush with the sand. The water level in the sand was raised at increments of one inch, depending on the stability of the system. At each increment, readings of the slurry, sand and water levels and pressure in the trench were taken at 15 minutes intervals. The water level was raised by a further increment when the readings reached a sensibly constant value. Failure was taken as the water level at which a steady creep occurred which caused a wedge of soil to slip into the trench. If water pressure alone was insufficient to cause failure, a surcharge was applied to the surface of the sand. After failure, the shape of the failure zone, at the surface was measured and sand samples were taken for determining the bulk density. The flow and the thixotropic properties of the slurry were determined with a rotational viscometer.

The angle of shearing resistance ϕ' was determined by carrying out a series of drained triaxial tests on saturated samples of sand at the various void ratios. The mean angle of shearing resistance and permeability were determined.

The results were analysed using the measured slip surface and compared with the results obtained from Coulomb's wedge theory.

These results are reproduced in para 2.2.4.1 after discussing the factors which affect stability of trenches on para 2.2.2 below.

2.2.2 Factors Which Affect Stabilization of Trenches

2.2.2.1 Trench Dimensions

Width

This is a function of the head on the cutoff and the gradation of the backfill materials and adjacent foundation materials. It will also depend on the type of trenching equipment used, as draglines, clamshells, back hoes or trenching machines. For deep trenches the dragline is the most suitable tool, while a trenching machine is the most economical and suitable equipment for excavating narrow, shallow trenches upto 2m width. The effect of the trench width, 2a, on the trench stability is indicated in equation 2.28 on para 2.2.3 .

Depth

This is usually that required to give a positive cutoff and extends to the same depth as the pervious zone. As depth increases, the depth of excavation and cleaning the bottom increase and a grouted cutoff may be combined with the slurry trench to give the most economical cutoff beyond the limiting depth of the slurry trench operations. The limiting depth depends on the equipment used, generally 27m for dragline and 30 m for clam-shell. In the stability point of view the trench depth increases the hydrostatic force of the slurry, the shearing resistance zone, and the activating force composed of the soil and water (if saturated). Equation 2.28 summarizes these effects.

2.2.2.2 Density of Slurry γ^*

The ability of slurry to support the adjacent sand mass which, may be saturated; is attributed partly to the higher density of the slurry than that of water [6]. The slurry exerts sufficient pressure on the walls of the trench to support them in their vertical position. The mud may be considered non-penetrating fluid and support of the excavation is achieved by the action of the hydrostatic pressure of the slurry on the impermeable face of the excavation. As discussed in para 2.1.2, the slurry density is increased by the suspended cuttings and that hardly any change of the achieved slurry density in the trench takes place with time.

The density of the slurry must be properly controlled. If the density is too low, the slurry will not prevent caving of the trench walls. If the slurry is too dense, the dragline bucket or clam - shell used for excavating will tend to float- high density will also increase the tendency of the backfill material to segreggate. The upper and lower density before placing are, in general 1.44 and 1.04 T/m^3 respectively. The final value after setting is higher due to the cuttings from the excavated material.

2.2.2.3 Slurry Properties (other than density)

Quite a number of factors contributing to trench stability come up as a result of the slurry properties discussed in paras 2.1.3, (thixotropy), 2.1.4 (flow properties), 2.1.5 (yield gradient) and 2.1.6 (electrical properties).

(a) Formation of Filter Cake

This has been discussed in Veder's experimental work, and methods of strengthening and accelerating it by electrodes to create an electro-motive force. The impervious filter cake formed at the trench side forms a 'membrane' on which the hydrostatic pressure of the slurry can act. Laboratory measurements of the permeability of filter cakes from bentonite muds give values of 2.3×10^{-9} cm/sec, with a porosity of 88%.

The mud slurry will continue to exert hydrostatic pressure against the filter cake if it is stable with respec t to time. Mud suspensions have stability, a property which enables them to preserve their original rheological characteristics. Lack of stability is revealed by separation of the constituents, the solid particles falling to the bottom and leaving a variable depth of clear water above them. Bentonite suspensions, being thixotropic in nature, exhibit excellent stability so far as separation of constituents is concerned and there is no sedimentation of suspended particles.

The filter cake formed functions as a vertical memberane exhibiting a resistance to deformation. This bentonite cake(along with the resistance of the slurry saturated zone) is able to withstand the pressure of a wedge of retained soil when the level of the slurry in the trench is lowered. Triaxial tests carried out by Veder on cylindrical samples of sand, on which a cake of bentonite had formed indicated that the resistance to deformation of the filter cake was equivalent to an increase in ambient pressure of 74 gm/cm^2.

(b) Shearing Resistance

Depending on fluidity of the slurry, it will permeate the trench sides of coarse granular cohesionless soils to form a slurry saturated zone. This zone forms to an appreciable thickness before the formation of the filter cake. The slurry saturated zone will have a shearing resistance on account of cohesive strength of the slurry which improves the physical properties of the zone by gelling in soil pores under quiescent conditions[7,9,17].

On account of the heterogeneity of natural soil formations, it is difficult to estimate the depth of penetration of slurry into trench sides. At the same time, the extent of improvement in the physical properties of slurry saturated zone is another factor which cannot be precisely evaluated. However, if the properties of the trench soil and the slurry are known to a fair degree of accuracy , the following approach can be used to estimate approximately the shearing resistance of the slurry saturated zone.

Thickness of Saturated Zone

From equation (2.6) para 2.1.5, it is shown that the limiting depth of penetration of a slurry, having a yield strength of τ_f , is given by

$$X_L = \frac{P\, g\, r}{2\, \tau_f}$$

where p and r are as defined earlier. Thus for given values of the shear strength τ_f , soil permeability K_w , and pressure differential, the corresponding values of limiting depth of penetration can be calculated. The effective radius of an average pore pressure 'r' is calculated by equation (2.5) from the known viscosity of water of 0.01 poise at 20°C. Table 2.3 shows calculated values of limiting depths of penetration for the values of the variables shown.

TABLE 2.3 $n = 0.30$, $\eta_w = 0.01$ poise

τ_f dynes/cm^2	Limiting depth of penetration in cm					
	p = 0.25 kg/cm^2			p = 0.5 kg/cm^2		
	K_w=1 cm/sec	K_w=10^{-1} cm/sec	K_w=10^{-2} cm/sec	K_w =1 cm/sec	K_w=10^{-1} cm/sec	K_w=10^{-2} cm/sec
80	26	8.1	2.6	52	16.2	5.2
60	35	10.9	3.5	70	21.8	7.0
40	52	16.2	5.2	104	32.4	10.4
20	104	32.5	10.4	208	65.0	20.8
10	208	65.0	20.8	416	130.0	41.5

Table 2.3 clearly shows that -

(i) For the same soil and pressure differential the depth of penetration is inversely proportional to the shear strength of the slurry.

(ii) For the same slurry and pressure differential the depth of penetration decreases with the reduction in the soil permeability.

(iii) For the same slurry and soil, the depth of penetration is directly proportional to the pressure differential.

Thus in order to increase the depth of penetration-

i. Slurry of low shear strength must be used

ii. Porous soil must be used

iii. A high pressure differential must be maintained.

It is not always possible to maintain all these requirements in the field and compromise has to be achieved after taking other stabilizing factors into consideration.

The increase in the shearing resistance of the slurry saturated zone can be evaluated by triaxial tests on slurry saturated specimens simulating the field conditions, and then to study the effect of gelation of slurry in soil pores.

According to Elson, for known gel-strength (T_m) of the slurry, thickness of the slurry saturated zone (2ℓ), and the effective radius of soil pore (r), the shearing resistance of the slurry saturated zone, C_a, can be determined by the following procedure.

Consider a tube of radius r, and length x, full of gelled slurry. Then at the commencement of plug flow, the equation for equilibrium of forces from Fig. 2.9 is:-

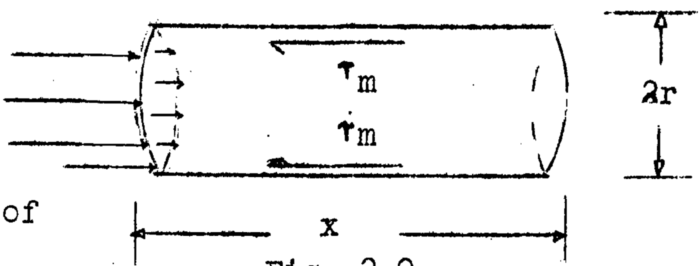


Fig. 2.9

$$\pi r^2 p = 2 \pi r \tau_m x$$

where, p is the pressure necessary to displace the slurry.

$$\text{Thus } p = \frac{2 \tau_m x}{r} \quad (\tau_m \text{ in gm/cm}^2) \qquad \ldots(2.7)$$

or in terms of the critical pressure gradient i_c.,

$$i_c = \frac{2 \tau_m}{r} \qquad \ldots(2.8)$$

Equation (2.8) represents the maximum pressure gradient that can be resisted by a capillary filled with a gelled slurry, neglecting the effect of creep.

Assuming that the pressure of the slurry in the voids of the soil does not affect the angle of shearing resistance ϕ', then the increase in shear strength, C_a, is given by

$$C_a = - u \tan \phi' \qquad \ldots(2.9)$$

where 'u' is the negative pore pressure and ϕ' is the angle of internal friction in terms of effective stresses, substituting from equation (2.7)

$$C_a = - \frac{2 \tau_m \ell}{r} \tan \phi'$$

where 'ℓ' is the minimum distance to the permeable boundary of the mass of soil saturated with slurry (Fig. 2.10)

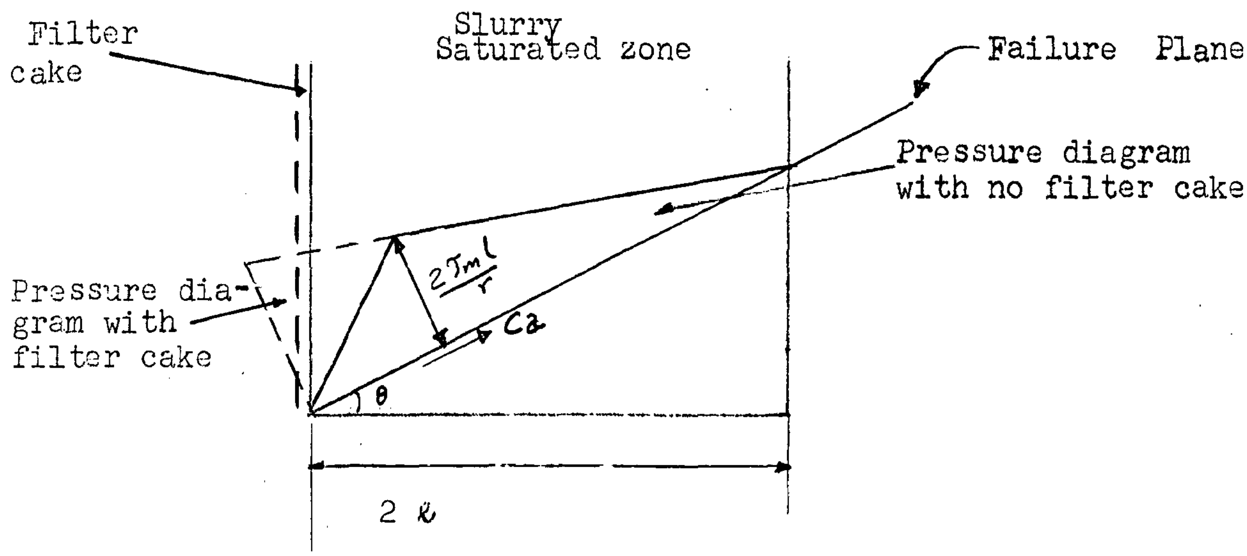


Fig. 2.10 - Pore Pressure Diagram in Slurry Saturated Zone

Consider a vertical zone of soil of thickness 2ℓ, saturated with slurry. If the failure plane passes through it at an angle θ to the horizontal, then the pore pressure diagram in the failure plane can be plotted as in Figure 2.10. The shearing resistance of the zone along the rupture surface is given by

$$C_a = (\text{Area of Pore pressure diagram}) \times \tan \phi'$$

$$= \left[\frac{2 \tau_m \ell}{r} \cdot \frac{2\ell}{2 \cos \theta} \right] \times \tan \phi'$$

$$= \frac{2 \tau_m \ell^2}{r \cos \theta} \cdot \tan \phi' \qquad \text{...(2.10)}$$

$$= P_a \tan \phi'$$

where $P_a = \frac{2\tau_m \ell^2}{r \cos\theta}$ = magnitude of negative pore pressure.

$(\tau_m$ in gm/cm$^2)$ ...(2.10a)

If a filter cake has formed on one boundary, assuming it has a very large strength compared with the original slurry, then,

$$C_a = \frac{4\tau_m}{r} \cdot \frac{\ell^2}{\cos\theta} \cdot \tan\phi' \qquad ...(2.10b)$$

This increased shear resistance of the slurry saturated zone will act along the direction of the rupture surface.

(c) Passive Resistance of the Slurry [8]

Morgenstern suggests that the problem of the stability of slurry trenches should be regarded as one in which the bank material tends to fall in and extrudes into the slurry, but the finite strength of the bentomite mud aids the resistance to this movement.

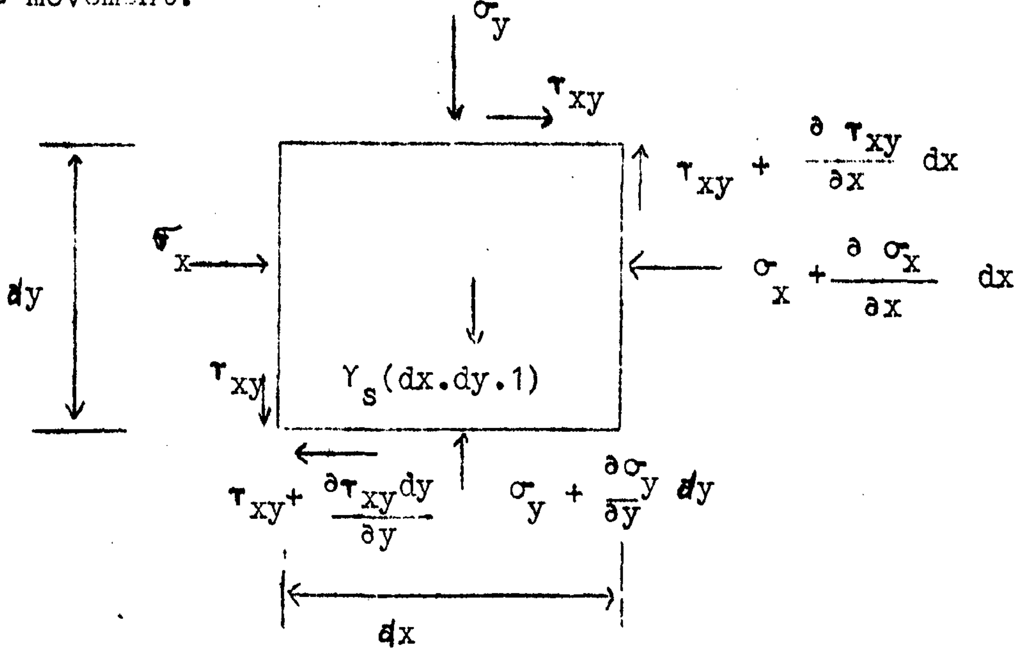


Fig. 2.11 -Equilibrium of a Two Dimensional Element

Considering an infinitesimal element in the clay slurry (Fig.2.11), the equations of equilibrium become :-

$$\left.\begin{aligned} \frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} &= 0 \\ \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{xy}}{\partial x} - \gamma_s &= 0 \end{aligned}\right\} \qquad \ldots(2.11)$$

where γ_s is the density of the slurry. Plastic flow of the slurry will occur when the stresses satisfy the yield criterion.

$$(\sigma_x - \sigma_y)^2 + 4\,\tau_{xy}^2 = 4\,\tau_s^2 \qquad \ldots(2.12)$$

where τ_s is the shear strength of the bentonite slurry treated as a purely cohesive material.

Subjec t to the appropriate boundary conditions, the stresses are statically determinate, and using equations (2.11) and (2.12), Morgenstern has shown that for a trench width '2a', the horizontal stress is

$$\sigma_x = \gamma_s\, y + \tau_s \frac{y}{a} + \frac{\pi\, \tau_s}{2} \qquad \ldots(2.13)$$

The total passive resistance of the slurry is

$$\begin{aligned} P_p &= \int_0^H \sigma_x \, dy \\ &= \gamma_s \frac{H^2}{2} + \tau_s \frac{H^2}{2a} + 2\pi \, \tau_s \frac{H}{2} \end{aligned} \qquad \ldots(2.14)$$

where H is the height of the slurry filled trench. The first term on the right hand side of equation (2.14) corresponds to the hydrostatic force exerted by the slurry and the other terms represent the passive resistance of the slurry on account of shear strength τ_s. Thus the passive resistance, E_p, is given by

$$E_p = \tau_s \frac{H^2}{2a} + \pi\, \tau_s \frac{H}{2} \qquad \ldots(2.15)$$

It is difficult to conceive as to what magnitudes the shear strength (τ_s) can develop in a practical slurry trench under conditions of considerable disturbance caused by passage of tools up and down. By assuming average values of τ_f, τ_s, τ_m, γ, ϕ' and G., T.C. Gupta[3] has calculated the magnitudes of the passive resistance E_p, hydrostatic force P_s, and the magnitude of the other forces contributing to stability of trenches. These values are shown in para 2.2.4.2 Table 2.4. Likewise Elson's results on Table 2.5 offer a good comparison of these forces.

(d) Electro - Osmotic Forces

The term electro-osmosis is used to describe the electro-kinetic phenomenon of a liquid moving through a porous medium relative to a fixed solid under the influence of an electric field. As shown by Veder, the bentonite suspension is electro= negative relative to the soil and there exists an electric potential between the two 'phases'. Therefore the electro-osmotic forces come into play due to movement of the slurry towards soil.

The equation of the motion of the fluid can be given in the form[14]

$$V_e = \frac{E\,\varepsilon\,\zeta}{4\pi\eta L} \qquad \ldots(2.16)$$

where,

V_e = The fluid velocity

E = Electrical potential

ε = Dielectric constant of the fluid

ζ = Electro-kinetic potential (or zeta potential)

η = Fluid viscosity

L = Distance between the electrodes.

The application of this equation to evaluate the effect of electro-osmotic forces on the trench face is rather difficult because it is not possible to determine ε and ξ to any reliable accuracy. At the same time the magnitude of the electric potential E , existing between the two phases (the slurry and the soil) is small and variable being influenced by the ionic concentration in ground water and the water used for making the slurry. Any venture to evaluate electro-osmotic forces is, therefore, not very meaningful. Their presence may be taken as a positive factor in the mechanism of stabilization, where the quantitative effect can be ignored. This assumption is confirmed by a model experiment which was carried out by Nash and Jones[10] in 1963. In this, potential difference between saturated sand and bentonite mud was reduced to zero but no decrease in stability could be detected.

From the foregoing paras, the factors affecting stability of slurry trenches can be summarised to

a) Hydrostatic pressure exerted in the trench sides

$$P_s = \frac{1}{2} \gamma_s H^2$$

b) The passive resistance of the slurry

$$E_p = \frac{\tau_s H^2}{2a} + \frac{\pi \tau_s H}{2}$$

c) The resistance to deformation of the filter cake and

d) The shearing resistance of slurry saturated zone

$$C_a = \frac{2 \tau_m \ell^2}{r \cos \theta}$$

The quantitative contribution of these factors to stability is shown in Tables 2.4 - (Elson's experimental results) and 2.5 (analysis of a particular full scale field slurry trench) in para 2.2.4.

2.2.3 Stability Analysis of Slurry Trenches

The theoretical procedure outlined below is based on Elson's approach as given by T.C.Gupta[2a]. The stability of long, vertical trenches in different types of soils has been discussed. The common method of estimating the disturbing and stabilizing forces, and equating them for a translating equilibrium of the coulomb's failure wedge has been adopted. No attempt is made to satisfy the moment equations as the estimation of forces is itself approximate and the points of their application are not precisely known. The main advantages of the method are its simplicity of application to field problems and ease of calculations.

2.2.3.1 Disturbing and Stabilizing Forces

The disturbing forces which cause a slide in the vertical face of a long continuous trench are

a. The weight W of the sliding wedge, moist or saturated.

b. Surcharge unit load q, if any, on the sliding wedge.

The stabilizing forces which resist the slide are-

a. Developed frictional forces in the soil mass at the failure plane.

b. Developed cohesive forces in the soil mass along the failure plane.

c. Hydrostatic pressure of the mud slurry.

d. Passive resistance of the slurry

e. Shearing resistance of the slurry saturated zone

f. Resistance to deformation of the filter cake

g. Electro-osmotic forces

Out of these resisting forces, the effect of the first five can be numerically evaluated, while the effect of the last two cannot be evaluated reliably. However, their contribution towards the overall stability of the trench side is comparatively small, and therefore, their effect in numerical calculations can be ignored. Their effect can be taken to mean existence of some positive factors which improve the calculated value of the factor of safety.

2.2.3.2 Trenches in Clay[10]

This case will be met with in cities where concrete diaphragms are required to be construc ted for basements of buildings, traffic under passages etc. On account of the fineness of the soil pores , no penetration of slurry into the trench sides is likely to take place. Under undrained conditions, the angle of frictional resistance (ϕ') will be practically zero so that no frictional resistance is developed at the failure plane. The only resisting forces will be the cohesion in clay, the hydrostatic pressure of the slurry and its passive resistance.

The total passive resistance of the slurry is given by

$$P_p = P_s + E_p$$

which from para 2.2.2 is given by

$$P_p = \frac{1}{2} \gamma_s H^2 + \frac{\tau_s H^2}{2a} + \frac{\pi \tau_s H}{2}$$

Consider the equilibrium of a soil wedge under the forces W,C, R and P_p, as shown in Figure 2.12(a). The wedge is at the verge of sliding. Under undrained conditions,

$\phi' = 0$, hence $\phi_D = 0$, so that $\theta_{cr} = 45^o$

The force polygon is shown inFigure 2.12(b).

Then, with the defined notation ,

$$W = \frac{1}{2} \gamma H^2$$

$$P_s = \frac{1}{2} \gamma_s H^2$$

$$P_p = \frac{\tau_s H^2}{2a} + \frac{\pi \tau_s H}{2}$$

$$C = \sqrt{2} \; H C_u$$

From Figure 2.12(b), resolving horizontally and vertically and eliminating R, we obtain,

$$C = \frac{W - P_p}{\sqrt{2}} \qquad \ldots(2.17)$$

If F is the factor of safety , the developed cohesion

$$C = \sqrt{2} \; H \frac{C_u}{F} \qquad \ldots(2.18)$$

From Equations (2.17) and (2.18) we obtain

$$F = \frac{2H - C_u}{W - P_p}$$

$$= \frac{2 H C_u}{\frac{1}{2} \gamma H^2 - \frac{1}{2} \gamma_s H^2 - \frac{\tau_s H^2}{2a} - \frac{\pi \tau_s H}{2}}$$

$$= \frac{4 C_u}{H(\gamma - \gamma_s) - \tau_s \left(\frac{H}{a} + \pi \right)} \qquad \ldots(2.19)$$

Equation (2.19) shows that the factor of safety depends on the shear strength and densities of both the soil and the slurry, and the trench dimensions.

2.2.3.3. Trenches in Dry Cohesionless Sand[10]

In this case, the frictional resistance on account of the normal component of the dry weight of the soil wedge is mobilized at the failure plane, and the resulting factor of safety, after combining the stabilizing effect of hydrostatic pressure of the slurry, is quite large even if the effect of all other factors is ignored. For these conditions, the force polygon gets simplified as shown in figure 2.13(b), where ϕ_D is the developed angle of friction.

$$W = \frac{1}{2} \gamma H^2 \tan (90 - \theta)$$

$$P_s = \frac{1}{2} \gamma_s H^2$$

Thus,
$$\frac{W}{P_s} = \frac{\gamma \tan (90 - \theta)}{\gamma_s} \qquad \ldots(2.20)$$

Also from the force polygon, under conditions of equilibrium,

$$\frac{W}{P_s} = \tan \left[90 - (\theta - \phi_D) \right] \qquad \ldots(2.21)$$

From equations (2.20) and (2.21)

$$\frac{\gamma}{\gamma_s} = \frac{\tan \theta}{\tan (\theta - \phi_D)}$$

Since ϕ_D is maximum when $\theta = (45 + \frac{\phi_D}{2})$ for the most critical plane

$$\frac{\gamma}{\gamma_s} = \frac{\tan (45 + \frac{\phi_D}{2})}{\tan (45 - \frac{\phi_D}{2})}$$

or by trignometry

$$\tan \phi_D = \frac{\gamma - \gamma_s}{2 \sqrt{\gamma \cdot \gamma_s}} \qquad \ldots(2.22)$$

If we define the factor of safety as $F = \dfrac{\tan \phi'}{\tan \phi_D}$

where ϕ' is the effective angle of internal friction for the sand, then,

$$F = \frac{2\,(\gamma.\gamma_s)^{1/2}.\tan\phi'}{\gamma - \gamma_s} \qquad ...(2.23)$$

Thus for dry cohesionless sands, the factor of safety is independent of depth.

2.2.3.4 Trenches in Saturated Cohesionless Sands[1,9]

This is the case of perhaps more general occurrence in river valley projects.

As shown in figure 2.14 , a wedge of soil inclined at an angle θ to the horizontal is assumed to be on the verge of sliding. The water level in the soil is mH and the sand is assumed to be saturated upto the top by capillary or otherwise. The effect of capillary forces on the shear strength of soil has been neglected, being small and on safer side. The wedge is separated from the slurry by a thin impermeable membrane.

In figure 2.14, T denotes the tangential force acting along the base of the sliding wedge, N , denotes the reaction normal to the base, and P_a denotes the horizontal force required to stop the wedge from sliding.

If the wedge is in horizontal equilibrium

$$P_a + T\cos\theta = N\sin\theta \qquad ...(2.24)$$

If it is in vertical equilibrium,

$$W = T\sin\theta + N\cos\theta \qquad ...(2.25)$$

The tangential force T is given by

$$T = N'\tan\phi' = (N - u)\tan\phi' \qquad ...(2.26)$$

where N' is the effective force normal to the slip plane and u denotes the net neutral force on the slip surface, figure 2.14(b).

From equations (2.24),(2.25) and (2.26),

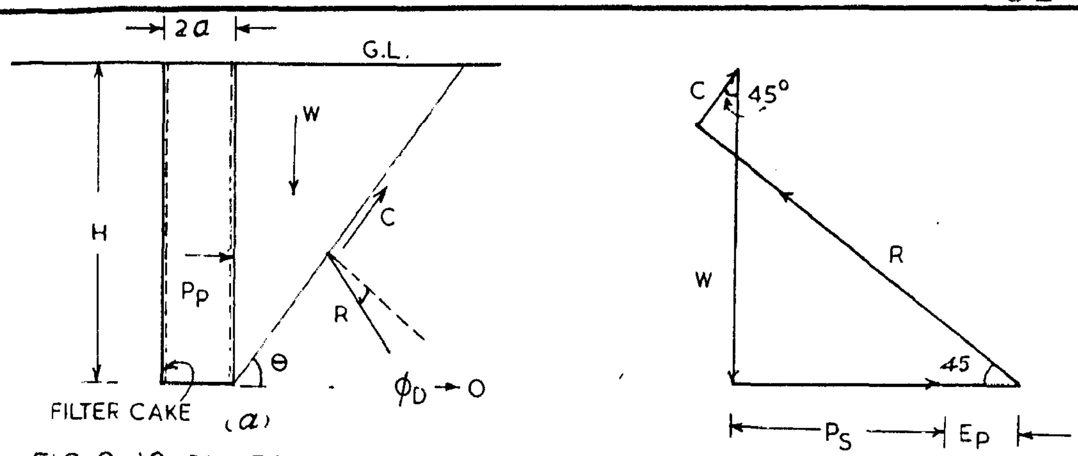


FIG 2·12 SLURRY TRENCHES IN CLAY

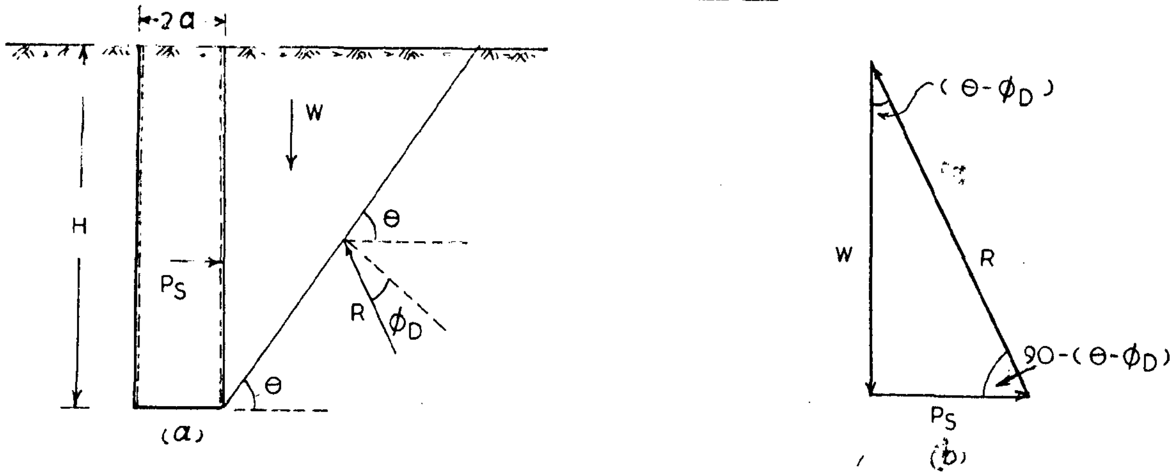


FIG 2·13 SLURRY TRENCH IN DRY COHESIONLESS SAND

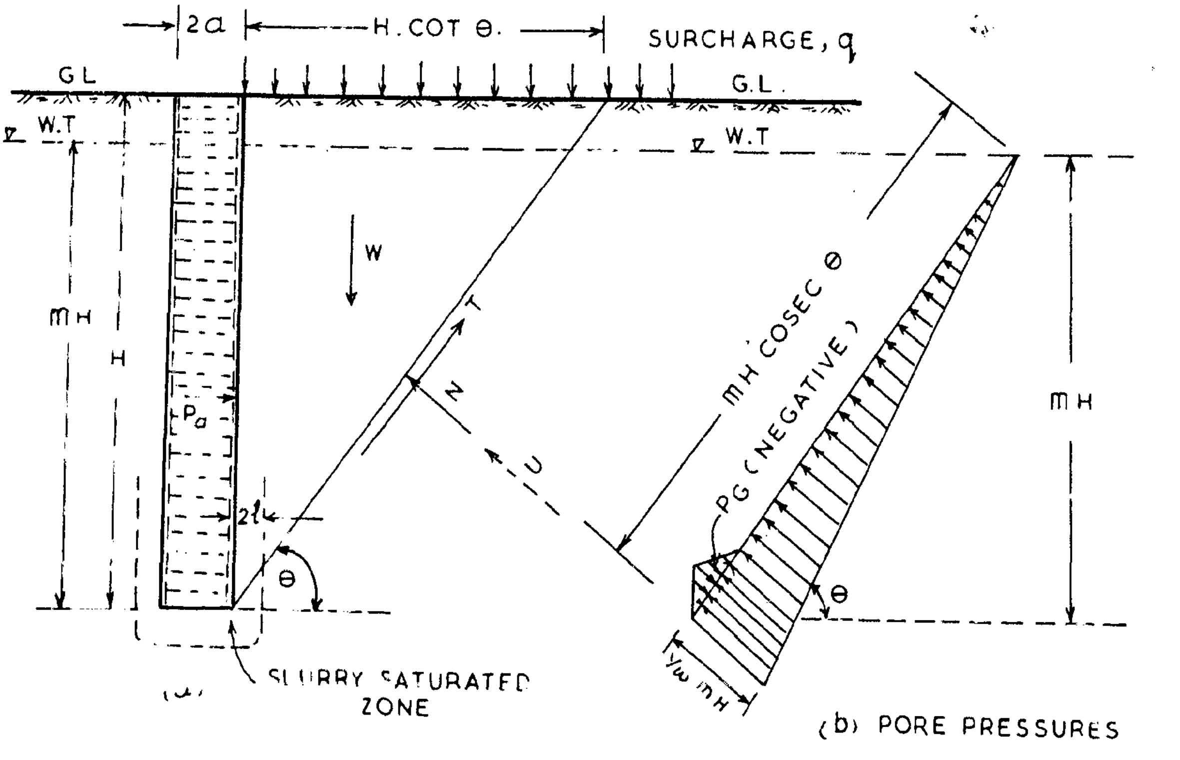


FIG 2·14 SLURRY TRENCH IN SATURATED COHESIONLESS SAND

$$P_a = \frac{W (\sin\theta - \cos\theta \tan\phi') + U \tan\phi'}{\cos\theta + \sin\theta \tan\phi'} \qquad \ldots(2.27)$$

Now $W = \cot\theta \left(\frac{1}{2} \gamma H^2 + q H \right)$

$$u = \frac{1}{2} \gamma_w (mH)^2 \operatorname{cosec}\theta - P_G$$

where, $P_G = \dfrac{2 \tau_m \ell^2}{r \cos\theta}$ (equation 2.10a)

The total passive resistance offered by the slurry is given by

$$P_p = P_s + E_p$$
$$= \frac{1}{2} \gamma_s H^2 + \left(\tau_s H^2 + \frac{\pi \tau_s H}{2}\right)$$

If for stability $P_a = P_p$, then

$$\frac{1}{2} \gamma_s H^2 + \left[\frac{\tau_s H^2}{2a} + \frac{\pi \tau_s H}{2} \right]$$
$$= \frac{\cot\theta \left(\frac{1}{2}\gamma H^2 + q H\right) (\sin\theta - \cos\theta \tan\phi') + \left(\frac{1}{2}\gamma_w m^2 H^2 \operatorname{cosec}\theta - P_G\right) . \tan\phi'}{\cos\theta + \sin\theta \tan\phi'}$$

...(2.28)

The two terms on the left hand side, P_s and E_p respectively, have been discussed. The first term on the right hand side represents the active earth pressure of the sliding wedge of the soil (including the effect of saturation) and the second term represents the stabilizing effect of the slurry saturated zone in horizontal direction.

2.2.4 Quantitative Effect of Factors Affecting Stability of Slurry Trenches

The quantitative contribution of the factors affecting stability of trenches discussed in the foregoing pages will be demonstrated by two methods - one involving the results of W.K. Elson's experimental work outlined in para 2.2.1.2 and the second from calculations based on the application of the stability analysis theory discussed in para 2.2.3 on a field trench [3].

2.2.4.1 Elson's Results

$\phi' = 33.1^{\circ}$ $\quad H = 36.6''$

$mH = 34.2''$ $\quad \gamma = 0.0745\ \text{lbs/in}^3$

$\gamma_w = 0.0362\ \text{lb/in}^3$, $\quad q = 0$

$\gamma_s = 0.037\ \text{lbs/in}^3$ $\quad e = 0.545$,

$\tau_m = 0.41\ \text{lbf/ft}^2$ $\quad \tau_s = 0.41\ \text{lbs /ft}^2$

$a = 2''$ $\quad \tau = 0.0019''$,

$\ell = 1''$

The calculations are summarised in Table 2.4.

2.2.4.2 Stability of Full Scale Field Trench

T.C. Gupta[3] has carried out calculations of the magnitudes of stabilizing forces and their percentage of the disturbing force for the Pierre Benite Trench. These calculations are reproduced on Table 2.5.

TABLE 2.4 RESULTS OF ELSON'S EXPERIMENTAL WORK ON STABILITY OF TRENCHES

Width of Trench in	Force	7% Slurry				4% Slurry			
		Hansen		Coulomb		Hansen		Coulomb	
		1	2	1	2	1	2	1	2
4	Hydrostatic	77.4	77.3	80.4	80.2	80.2	77.1	84.0	85.3
	Passive Slurry resistance	18.4	17.5	19.1	18.3	3.6	3.7	3.8	4.1
	Shearing resistance of slurry saturated zone	3.9	6.8	3.3	5.4	13.6	18.7	12.7	16.0
	Unaccounted	0.3	-1.6	-2.8	-3.9	2.7	0.5	-0.5	-5.4
	Disturbing	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
2	Hydrostatic	64.1		70.0		76.5		81.4	
	Passive slurry resistance	27.1		30.0		6.0		6.4	
	Shearing resistance of slurry saturated zone	4.9		4.4		14.5		13.4	
	Unaccounted	3.9		-4.4		3.0		-1.2	
	Disturbing	100.0		100.0		100.0		100.0	
1	Hydrostatic					67.5		71.5	
	Passive slurry resistance					18.5		19.8	
	Shearing resistance of slurry saturated zone					12.7		11.3	
	Unaccounted					1.3		-2.3	
	Disturbing					100		100	

N.B. (a) Forces are expressed as % of the disturbing force.
(b) Negative sign indicates that the stabilizing force exceeds the disturbing force.

TABLE 2.5 STABILITY OF PIERRE BENITE TRENCH

Data Permeability of soil 1.0 to 0,1 cm/sec.

Slurry concentration 4% Bentonite (Fb -2-5)

Slurry Density γ_s : i. Initial = 1.025 T/m^3
ii. In trench=1.2 to 1.25 T/m^3

Bulk Saturated density of fill γ = 1.85 T/m^3

Assumptions: $\phi' = 30^o$, $G = 2.65$

$\tau_f = 0.05$ gm/cm^2 , $\tau_m = 0.75$ gm/cm^2

$\tau_s = \tau_M = 0.75$ gm/cm^2

Trench Dimensions L = 9 to 25 metres , H = 3.5 metre
2a = 0.6 metre mH = 3.5 metre

Nature of force	Magnitude T/m	% of Disturbing force.
Total disturbing	7.85	100
Stabilizing forces		
i. Hydrostatic (P_s)	6.77	86.2
ii. Passive resistance of slurry (E_p)	0.18	2.3
iii. Shearing resistance of slurry saturated zone in horizontal direction	0.67	8.5
Total stabilizing forces	7.62	97.0

Tables 2.4 and 2.5 indicate at a glance the quantitative contribution of the major three stabilizing forcesof hydrostatic force, the passive resistance of the slurry and the shearing resistance of the slurry saturated zone. The unaccounted forces in Table 2.4 can be considered as the 3% (of the disturbing force) shortage in Table 2.5.

CHAPTER III

EXPERIMENTAL SETUP , PROCEDURE AND RESULTS

The setup of this experiment was a simplified version of Elson's experiment discussed in para 2.2.1.2..

3.1 APPARATUS

This consisted of a mixer for the slurry, stirrer, a prismatic container fitted with a capillary tube for flow test, and a box in which the loading tests were carried. Loading device, containers like basins , and buckets, measuring cylinders, weighing apparatus, a stop watch and sieving apparatus formed the supplementary apparatus.

3.1.1. Mixer

This is a cylindrical container, about 0.75 litre in capacity in which a stirrer driven by an electric motor could be inserted.

3.1.2 Loading Test Box

This measured 38.6 x 26.0 x 20 cm internally and was divided into two halves by a 20 x 20 cm plate sliding through grooves in the walls and floor of the box. The box walls and floor as well as the partition plate were made from perspex plates to facilitate viewing of the sand and slurry inside. The box walls and floor plates were joined by chloroform and adequately stiffened by steel angles. The joints were made watertight by smearing grease along them. A wooden plank was placed under the bottom of the box to prevent sagging or bending of the floor plate. A perspex plate was also provided to be placed on the sand surface to distribute the load uniformly on the sand surface. A sketch of the box is shown in Figure 3.1(a) and its photograph in plate 3.1.

3.1.3 Flow Test Model

This is also a perspex prismatic box 10 x 10 cm in cross section and 44 cm in height. It was fitted with a capillary tube of 0.195 cm internal radius and 61.3 cm long (Another tube of the same radius and 63.2 cm long was substituted later).. A scale was fitted beside the walls of the box to record slurry levels above the capillary tube. A sketch of it is shown in Figure 3.1(b) and a photograph in plate 3.2.

3.2 MATERIALS

Commercial bentonite, black cotton soil, fine sand and water were used. The commercial bentonite had the problem of segregation for pure bentonite slurries of lesser concentration than 4% . Virtually all the slurries between 1 and 4% concentration with replacement of bentonite by black cotton soil displayed the segregation problem.

The black cotton soil was ground and sieved through 75 μ (0.075mm) sieve size.

3.3 PROCEDURE

3.3.1. Mixing

Slurries of 1% to 4% concentration were made. For each concentration, bentonite was replaced by black cotton soil in stages of 25% of the bentonite portion by weight. This resulted in 20 different slurry concentrations.

The slurry concentration, SC, was expressed as -

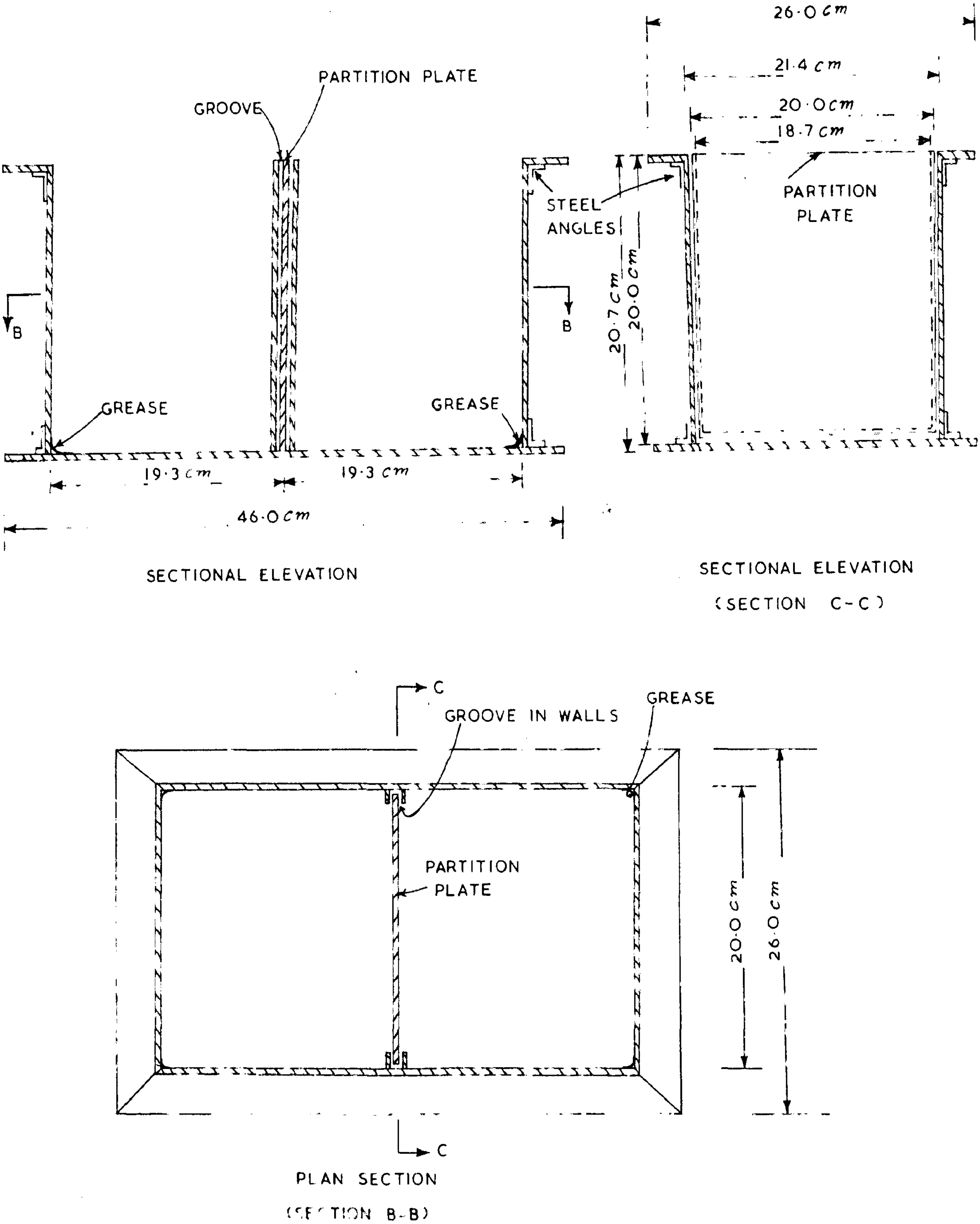


FIG. 3·1(a) LOADING BOX SKETCH

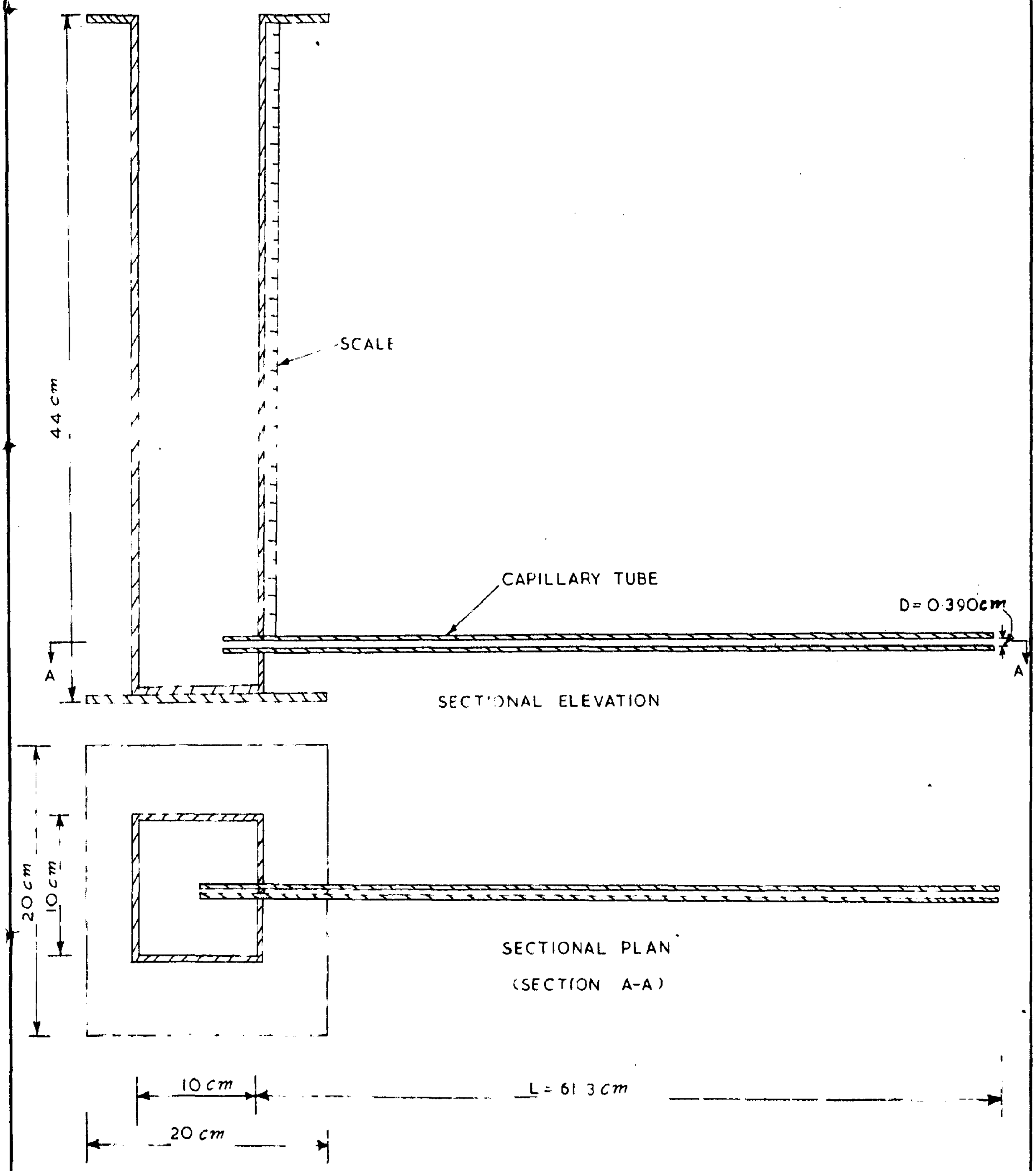


FIG 3·1(b) FLOW TEST MODEL SKETCH

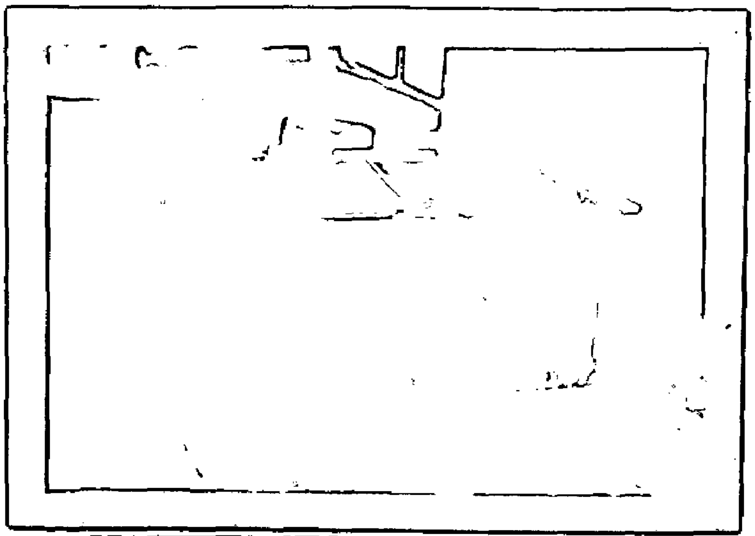


PLATE 3.1 LOADING TEST BOX

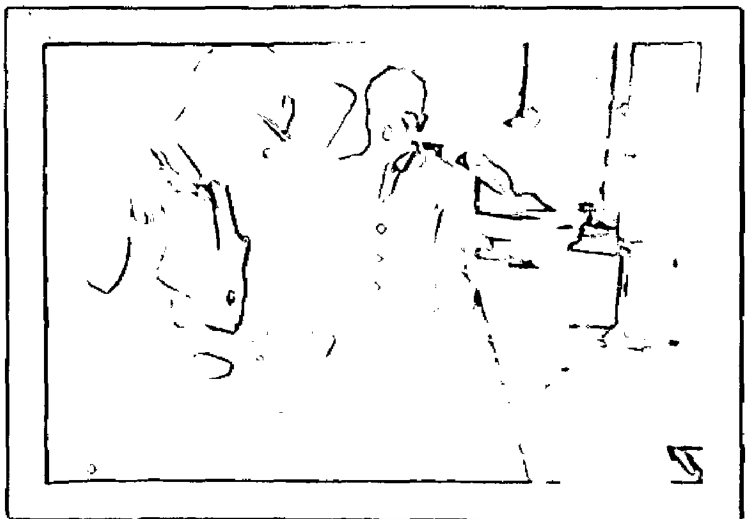


PLATE 3.2 FLOW TEST MODEL AND OPERATION

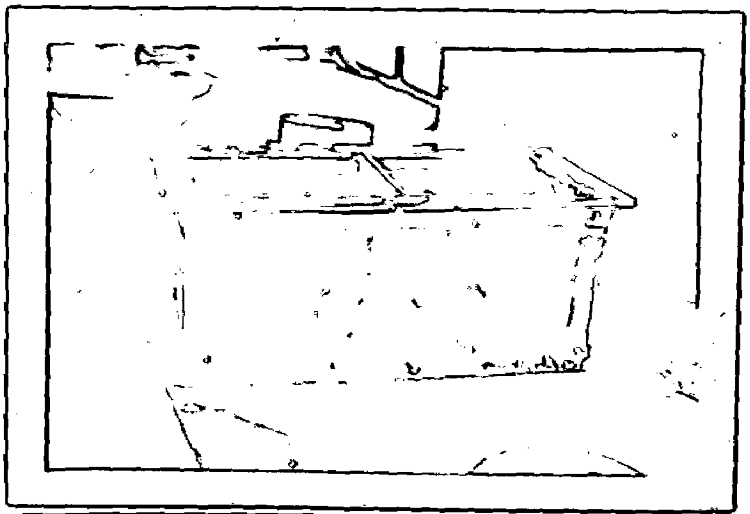


PLATE 3.1 LOADING TEST BOX

PLATE 3.2 FLOW TEST MODEL AND OPERATION

$$SC = \frac{\text{Weight of solid portion(Bentonite/bentonite plus black cotton soil)}}{\text{Weight of solid portion plus weight of water}} \quad ...(3.1)$$

In order to have sufficient slurry quantity the amount of water for each slurry was taken as 10 kg (10,000 cc). The requisite amounts of water and bentonite/black cotton soil for each slurry concentration were calculated from equation (3.1) above and the quantities obtained by weighing and measuring. Small portions of the constituents of each slurry concentration were mixed in the mixer, each portion being mixed for one minute. All the portions were discharged into a bucket for further agitation with the stirring rod.

3.3.2 Slurry Density

The slurry density was obtained by dividing the total weight by the total volume taken, taking the specific gravity of solid bentonite and black cotton soil as 2.55. A sample calculation of this is shown on para 3.5.

3.3.3 Flow Test

Portion of the slurry from the bucket was tested for flow properties in the flow test model. The model was filled with slurry and the capillary tube was opened to allow the slurry to run freely. The tube was then closed, and the level of the slurry was noted. Quantities of slurry were collected in a measuring cylinder for a known period of time. (Plate 3.2). The final slurry level was noted The slurry was kept in agitation throughout to prevent segregation. The level for no flow was noted. Flow results and calculations are shown in Table 3.1. Graphs of Rate of Shear strain Versus shear stress are shown in Figure 3.2(a),(b),(c) and (d).

3.3.4 Loading Test

A loading test was carried out for each slurry concentration. With the partition plate in position, one half of the loading test box was filled with dry fine sand (gradation curve shown in Fig. 3.3) . The top of the sand was levelled off and the loading plate was placed on the sand surface. Slurry was then poured into the other half of the box and agitated. The partition plate was then slowly removed. After pulling up a small portion of the plate the slurry was allowed to saturate the sand. The slurry which flowed into the sand was replenished to the level of the sand. Further withdrawal of the plate was continued until all of it was removed. If there was no failure, the loading plate was loaded till failure occurred.

Two methods of loading were used . The first involved placing known loads on the loading plate until failure was achieved. Plate 3.3 shows this operation. For reasons explained later, this method of loading was found unsuitable and unsafe.

The other method was the adoption of the triaxial loading machine. The cell unit was removed leaving the raising floor on which the wooden plank could be placed. The loading test box was placed on the wooden plank and sand poured into the box as explained above. A small cylindrical wooden piece, and frictionless balls were used to make contact between the loading plate and the calibrated proving ring of the machine. Slurry was poured into the other half as before, the machine was levelled for loading and the partition plate was removed as explained before. This set up is shown in plate 3.4. The slurry was allowed to penetrate the sand for 15 minutes. The depth of the slurry saturated zone was measured. The floor of the machine was raised by turning the handle manually and slowly as shown in Plate 3.4 . This raised the wooden plank and the box and compressed the sand against the loading plate. This compression was thus communicated to the proving ring and the load could be recorded. The sand was loaded to failure and the failure load was noted. The weights of the loading plate, and the wooden cylindrical piece were added to the recorded failure load. The loading test results on dry sand and calculations are shown on Table 3.3.

The same loading procedure was repeated for saturated sand. These results are shown on Table 3.4.

3.3.5 Sand Density

As the sand was fine its density could be easily determined by a 25 ml specific gravity bottle. A physical balance was used for weighing.

3.3.6 Sand Sieving

This was done in order to obtain the effective radius of pore passage. The sieving results are tabulated on Table 3.2. and the grain size distribution curve is shown on Figure 3.3.

3.4 RESULTS AND CALCULATIONS

All the observed results and calculations are summarized in para 3.5 Flow test results and calculations are shown on

PLATE 3.3 STATIC LOADING

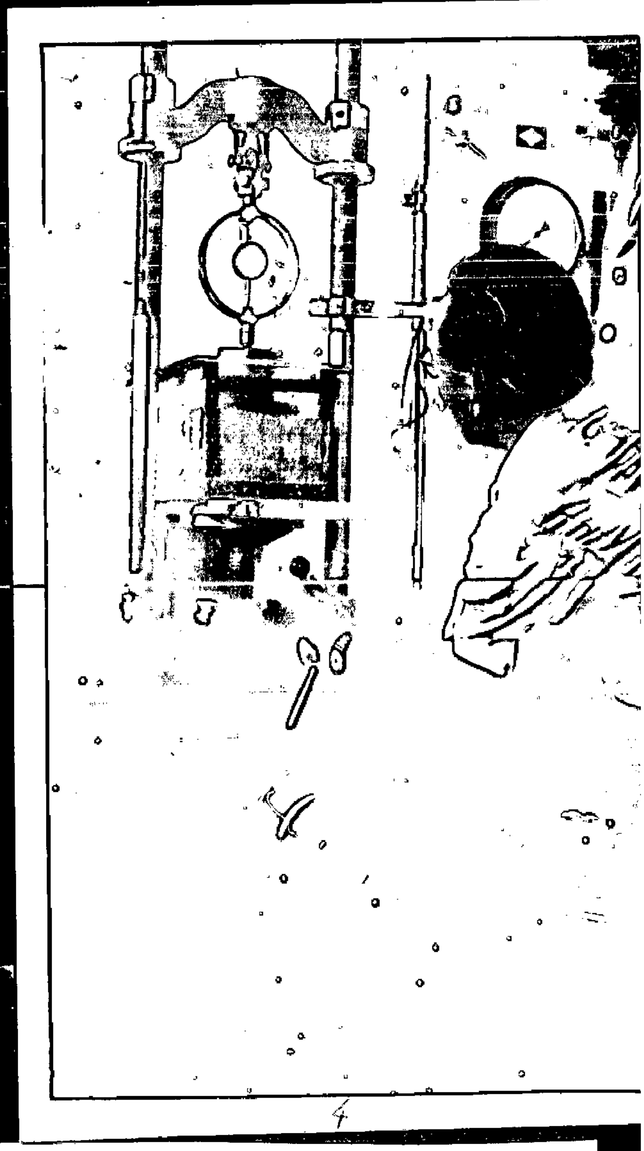


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PLATE 3.4 LOADING BY TRIAXIAL

Table 3.1, and plotted on Figures 3.2(a), (b), (c) and (d). From these the plastic viscosity η_{PL} has been determined. Sieve analysis is dealt with on para 3.5.2, and para 3.5.3 deals with the results and calculations of stability. All the results for dry sand loading are summarised on Table 3.3, and on saturated sand on table 3.4 . Figure 3.4 shows the relation between the failure surcharge and the percentage replacement of bentonite by black cotton soil.

3.5 CALCULATIONS

3.5.1 Flow Test Computations

Length of tube L = 61.30 cm.

Radius R = 0.195 cm.

Δh = difference in slurry levels in cm.

Q = rate of flow in cc/second.

$$\text{Hydraulic gradient } i = \frac{\Delta h}{\Delta L} = \frac{\Delta h}{61.30} = 0.0164 \ h.$$

$$\text{Rate of shear strain} = \frac{d\gamma}{dt} = \frac{4Q}{\pi R^3} = \frac{4Q}{\pi (0.195)^3} = 171.3Q \ (\text{Sec}^{-1})$$

$$\text{Shear stress } \tau_R = \frac{R}{2} i_1 = \frac{R}{2} \frac{\Delta P}{\Delta L} = \cancel{\frac{0.195}{2} \frac{\Delta h \ \gamma_s}{\Delta L} \times 981 \text{ dynes/cm}^2}$$

$$= \frac{0.195}{2} \frac{\Delta h \ \gamma_s}{61.30} \times 981 \text{ dynes/cm}^2$$

$$= \frac{0.195 \ \Delta h \ \gamma_s}{2 \times 61.3} \text{ gm/cm}^2$$

$$= 0.0016 \ \gamma_s \ \Delta h \text{ gm/cm}^2$$

The density of the slurry, γ_s, is calculated from the mixture materials, taking the dry specific gravity (G_s) of clay as 2.55.

e.g. For 1% pure bentonite, slurry,

Weight Taken	Volume
Water 10,000 gm	10,000 cc
Bentonite 101 gm	$V = \frac{W}{G_s} = \frac{101}{2.55} = 39.6$ cc
Total weight=10,101.00gm	Total volume=10,039.60 cc

$$\gamma_s = \frac{\text{Total weight}}{\text{Total volume}} = \frac{10,101.00}{10,039.60} \text{ gm/cm}^3$$

$$= 1.005 \text{ gm/cm}^3$$

TABLE 3.1 - FLOW TEST RESULTS AND CALCULATIONS

Initial level h_1 (cm)	Final level h_2 (cm)	Time in sec. t	Volume in cc V	Mean head $\Delta h = \frac{(h_1+h_2)}{2}$ (cm)	Volume per cc $Q=\frac{V}{t}$	$dE_s/dt' = 171.3Q$	$T_R = 0.0016\gamma_s\Delta h$ gm/cm^2
1	2	3	4	5	6	7	8

1% Pure Bentonite Slurry , γ_s = 1.005 gm/cm^3

31.00	26.70	34.30	350	28.85	10.20	1750	0.0464
26.70	21.20	46.90	445	23.95	9.50	1630	0.0384
21.20	18.90	18.90	170	20.05	9.00	1540	0.0322
18.90	16.30	24.90	210	17.60	8.45	1450	0.0282
16.30	13.80	31.10	240	15.05	7.70	1320	0.0242
13.80	10.40	34.10	230	12.10	6.75	1157	0.0194
10.40	7.80	40.70	220	9.10	5.40	925	0.0146
7.80	5.10	51.40	210	6.45	4.10	702	0.0104
5.10	3.20	56.00	160	4.15	2.86	490	0.0067

Head for no flow = 0.00 cm , T_{R1} = 0 gm/cm^2

- -

1% Slurry with 25% Replacement of Bentonite by black cotton soil

γ_s = 1.005 gm/cm^3

34.70	31.20	35.90	290	32.95	11.20	1920	0.0530
31.20	27.10	31.40	380	29.15	12.10	2075	0.0468
27.10	23.20	32.90	325	25.15	9.80	1680	0.0404
23.20	19.50	33.30	300	21.35	9.00	1540	0.0342
19.50	16.20	31.00	260	17.85	8.40	1440	0.0286
16.20	12.90	34.30	264	15.55	7.70	1320	0.0250
12.90	10.10	33.10	222	11.50	6.70	1150	0.0185
10.10	7.70	37.00	206	8.90	5.57	955	0.0143
7.70	5.30	44.20	190	6.50	4.30	737	0.0104
5.30	3.70	43.30	137	4.50	3.16	541	0.0072
3.70	2.40	46.80	100	3.05	2.14	367	0.0049

Head for no flow = 0.00cm, T_{R1} = 0 gm/cm^2

- -

1	2	3	4	5	6	7	8
1% Slurry with 50% Replacement of bentonite by Black cotton soil							
γ_s = 1.005 gm/cm^3							
31.70	29.10	19.60	205	30.40	10.43	1790	0.0490
29.10	26.10	25.00	252	27.60	10.10	1730	0.0444
26.10	23.30	23.00	223	24.70	9.70	1660	0.0397
23.30	20.40	25.50	233	21.85	9.15	1570	0.0352
20.40	17.00	31.30	270	18.70	8.64	1480	0.0300
17.00	13.80	33.40	264	15.40	7.90	1353	0.0248
13.80	11.30	29.40	201	12.55	6.85	1172	0.0202
11.30	8.70	36.50	215	10.50	5.90	1010	0.0169
8.70	6.30	39.10	187	7.50	4.78	810	0.0120
6.30	4.50	43.90	157	5.40	3.58	615	0.0087
4.50	3.00	45.70	118	3.75	2.58	441	0.0060

Head for no flow = 0.00 cm , T_{R1} = 0 gm/cm2

1% Slurry with 75% replacement of Bentonite by black cotton soil							
δ γ_s = 1,005 gm/cm^3							
35.50	31.70	26.50	302	33.60	11.90	2040	0.0540
31.70	27.70	30.90	329	29.70	11.65	2000	0.0477
27.70	23.60	32.90	325	25.65	9.90	1690	0.0412
23.60	20.20	30.50	275	21.90	9.00	1540	0.0352
20.20	16.30	37.70	315	18.25	8.36	1435	0.0295
16.30	13.00	34.50	262	14.65	7.60	1300	0.0236
13.00	9.90	35.90	245	11.45	6.65	1140	0.0184
9.90	7.40	36.50	210	8.65	5.75	985	0.0139
7.40	5.30	35.60	162	6.35	4.55	780	0.0102
5.30	3.60	38.10	135	4.45	3.54	606	0.0072
3.60	2.40	41.50	105	3.00	2.53	434	0.0048

Head for no flow = 0.00 cm , T_{R1} = 0 gm /cm2

1	2	3	4	5	6	7	8
1% Slurry with 100% black cotton soil, γ_s = 1.005 gm/cm^3							
37.10	32.80	29.30	237	34.95	8.10	1390	0.0560
32.80	29.00	28.00	307	30.90	10.98	1880	0.0496
29.00	25.10	30.80	315	27.05	10.22	1755	0.0435
25.10	21.30	33.50	315	23.20	9.40	1610	0.0373
21.30	17.80	32.00	275	19.55	8.60	1472	0.0314
17.80	14.90	30.10	240	16.35	7.97	1365	0.0263
14.90	12.00	31.90	235	13.45	7.38	1262	0.0216
12.00	9.30	35.20	230	10.65	6.54	1120	0.0171
9.30	6.90	36.00	194	7.95	5.40	925	0.0128
6.90	5.20	33.10	142	6.05	4.30	736	0.0097
5.20	3.70	38.50	125	4.45	3.25	556	0.0072
3.70	2.50	46.50	95	2.10	2.04	350	0.0034
Head for no flow = 0.00 cm, T_{R1} = 0 gm/cm^2							
2% slurry (100% Bentonite) γ_s = 1.013 gm/cm^3							
36.00	32.60	23.80	270	34.30	11.34	1945	0.0555
32.60	28.50	30.60	305	30.55	9.98	1710	0.0491
28.50	24.80	28.80	290	26.60	10.08	1730	0.0430
24.80	21.30	29.50	280	23.05	9.50	1630	0.0370
21.30	18.00	30.50	270	19.65	8.86	1520	0.0318
18.00	15.10	29.50	230	16.55	7.80	1338	0.0268
15.10	12.80	26.50	180	13.95	6.80	1165	0.0226
12.80	10.50	30.90	185	11.65	6.00	1030	0.0189
10.50	8.50	32.10	165	9.50	5.15	883	0.0154
8.50	6.70	34.80	147	7.60	4.21	725	0.0123
6.70	5.30	34.90	118	6.00	2.96	507	0.0097
5.30	4.20	33.40	90	4.75	2.69	460	0.0077
4.20	3.20	39.00	78	3.70	2.00	353	0.0060
Head for no flow = 0.00 cm , T_{R1} = 0 gm/c^2							

1	2	3	4	5	6	7	8
2% Slurry with 25% Replacement of Bentonite by Black Cotton Soil							
γ_s = 1.013 gm/cm^3							
36.30	33.20	21.70	250	34.75	11.50	1970	0.0561
33.20	29.50	27.00	293	31.35	10.84	1860	0.0506
29.50	25.10	34.70	355	27.30	10.18	1740	0.0491
25.10	21.50	30.00	286	23.30	9.54	1630	0.0378
21.50	18.10	31.00	275	19.80	8.90	1523	0.0320
18.10	14.70	33.10	265	16.40	8.00	1370	0.0265
14.70	11.90	33.15	233	13.30	7.05	1210	0.0216
11.90	9.20	36.10	214	10.55	5.92	1015	0.0171
9.20	7.20	33.70	163	8.20	4.84	827	0.0133
7.20	5.50	34.00	133	6.35	3.91	670	0.0103
5.50	4.00	37.50	114	4.75	3.04	520	0.0077
4.00	2.90	45.70	100	3.45	2.19	376	0.0056
Head for no flow = 0.00 cm , T_{R1} = 0 gm/cm^2							
2% Slurry with 50% replacement of Bentonite by black cotton soil							
γ_s = 1.013 gm/cm^3							
37.70	34.10	25.60	295	35.90	11.50	1970	0.0580
34.10	30.30	27.70	305	32.20	11.00	1885	0.0520
30.30	26.30	31.50	326	28.30	10.30	1770	0.0458
26.30	23.20	26.10	250	24.75	9.60	1645	0.0400
23.20	20.20	26.50	242	21.70	9.10	1560	0.0352
20.20	17.10	29.00	250	18.65	8.60	1472	0.0302
17.10	14.20	30.10	237	15.65	7.90	1352	0.0256
14.20	11.50	31.50	220	12.85	7.00	1200	0.0208
11.50	9.20	33.00	195	10.35	5.90	1010	0.0168
9.20	7.20	34.20	168	8.20	4.90	840	0.0133
7.20	5.50	33.60	135	6.35	4.00	685	0.0103
4.10	3.10	35.80	83	3.60	2.30	394	0.0058
Head for no flow = 0.00 cm T_{R1} = 0 gm/cm^2							

1	2	3	4	5	6	7	8
2% Slurry with 75% Replacement of Bentonite by black cotton soil							
$\gamma_s = 1.013\ gm/cm^3$							
37.30	33.50	28.20	328	35.40	11.63	2000	0.0573
33.50	29.40	29.50	324	31.45	11.00	1885	0.0510
29.40	25.30	33.50	344	27.35	10.27	1760	0.0443
25.30	21.70	31.50	297	23.50	9.42	1612	0.0380
21.70	18.60	27.60	245	20.15	8.90	1525	0.0326
18.60	16.00	26.10	219	17.30	8.40	1440	0.0280
16.00	13.10	31.30	240	14.55	7.66	1312	0.0236
13.10	10.30	32.90	220	11.70	6.69	1142	0.0189
10.30	7.80	35.30	196	9.05	5.55	950	0.0146
7.80	6.20	30.90	140	7.00	4.50	770	0.0113
6.20	4.40	40.60	142	5.30	3.48	596	0.0086
4.40	3.00	41.10	110	3.50	2.68	460	0.0057
Head for no flow = 00 cm		T_{R1} = 0 gm/cm^2					
2% Slurry with 100% black cotton soil $\gamma_s = 1.013\ gm/cm^3$							
37.80	33.80	25.30	300	35.80	11.87	2035	0.0580
33.80	29.90	28.70	320	31.85	11.12	1910	0.0516
29.90	26.10	29.50	304	28.00	10.30	1765	0.0453
26.10	21.60	38.30	365	24.35	9.53	1630	0.0394
21.60	18.60	28.40	250	20.10	8.80	1510	0.0326
18.60	15.50	31.00	250	17.05	8.07	1382	0.0276
15.50	12.60	30.00	230	14.05	7.66	1312	0.0228
12.60	10.00	31.00	200	11.30	6.45	1103	0.0183
10.00	7.60	36.10	197	8.80	5.45	958	0.0142
7.60	5.50	35.50	158	6.55	4.45	767	0.0106
5.50	4.10	36.60	125	4.80	3.42	585	0.0078
4.10	2.60	50.40	115	3.35	2.28	390	0.0054
Head for no flow = 0.00 cm ,		T_{R1} = 0 gm/cm2					

1	2	3	4	5	6	7	8
3% Slurry Pure Bentonite , γ_s = 1.018 gm/cm^3							
35.90	32.00	27.90	319	33.95	11.65	2000	0.0552
32.00	27.90	30.10	335	29.95	11.10	1900	0.0487
27.90	24.00	31.10	312	26.96	10.00	1713	0.0439
24.00	20.60	31.30	279	22.30	8.91	1530	0.0363
20.60	17.30	33.50	262	18.95	7.81	1340	0.0309
17.30	14.50	34.20	230	15.90	6.71	1150	0.0260
14.50	11.80	39.30	225	13.15	5.72	980	0.0214
11.80	9.50	38.00	185	10.65	4.86	834	0.0173
9.50	7.80	37.00	150	8.65	4.05	695	0.0141
7.80	6.20	39.60	130	7.00	3.28	561	0.0114
6.20	4.70	47.00	118	5.45	2.51	430	0.0089
Head for no flow = 0.00 cm , T_{R1} = 0 gm/cm^2							
3% Slurry with 25% Replacement of Bentonite by black cotton soil							
γ_s = 1.018 gm/cm^3							
37.80	34.20	25.50	295	36.00	11.58	1990	0.0585
34.20	30.20	28.60	315	32.20	11.00	1888	0.0525
30.20	26.40	29.90	310	28.30	10.37	1778	0.0460
26.40	21.40	41.40	405	23.90	9.80	1678	0.0389
21.40	18.20	28.10	250	19.80	8.90	1526	0.0322
18.20	15.30	29.50	235	16.75	7.96	1363	0.0273
15.30	12.70	31.00	215	14.00	6.94	1188	0.0228
12.70	10.40	30.80	185	11.55	6.00	1030	0.0188
10.40	8.40	32.60	169	9.40	5.19	890	0.0153
8.40	6.30	41.00	170	7.35	4.15	711	0.0120
6.30	4.40	50.60	155	5.35	3.06	525	0.0087
4.40	2.70	71.50	132	3.55	1.85	318	0.0058
Head for no flow = 0.00 cm , T_{R1} = 0 gm / cm^2							

1	2	3	4	5	6	7	8
3% Slurry with 50% Replacement of bentonite by black cotton soil							
γ_s = 1.018 gm/cm^3							
36.60	32.50	29.40	335	34.55	11.40	1955	0.0561
32.50	28.50	30.30	330	30.50	10.90	1870	0.0496
28.50	24.40	32.80	325	26.45	9. 1	1700	0.0430
24.40	20.50	34.10	320	22.45	9.40	1670	0.0366
20.50	16.70	34.80	295	18.60	8.50	1454	0.0303
16.70	13.40	36.80	270	15.05	7.34	1255	0.0245
13.40	10.70	34.90	220	12.05	6.30	1080	0.0196
10.70	8.20	37.70	200	9.45	5.30	909	0.0154
8.20	6.20	39.60	167	7.20	4.21	723	0.0117
6.20	4.50	44.80	141	5.35	3.14	538	0.0087
4.50	3.00	54.30	120	3.75	2.21	379	0.0061

Head for no flow = 0.00 cm , T_{R1} = 0 gm / cm^2

1	2	3	4	5	6	7	8
3% Slurry with 75 replacement of bentonite by black cotton soil							
γ_s = 1.018 gm/cm^3							
38.10	33.90	30.50	365	36.00	11.96	2050	0.0585
33.90	29.10	35.10	385	31.50	10.98	1890	0.0513
29.10	24.70	35.20	355	26.90	10.10	1730	0.0438
24.70	20.90	33.50	310	22.80	9.25	1585	0.0371
20.90	17.30	33.70	228	19.10	8.55	1465	0.0310
17.30	13.80	36.10	283	15.55	7.85	1345	0.0253
13.80	10.70	35.40	244	12.25	6.90	1182	0.0200
10.70	8.20	36.00	205	9.45	5.70	978	0.0154
8.20	5.90	39.70	180	7.05	4.56	783	0.0115
5.90	4.00	44.80	150	4.95	3.34	572	0.0081
4.00	2.90	34.90	85	3.45	2.44	419	0.0056

Head for no flow = 0.00 cm T_{R1} = 0 gm / cm^2

3% Slurry with Pure Black Cotton Soil $\gamma_s = 1.018$ gm/cm^3

1	2	3	4	5	6	7	8
35.70	32.10	25.00	284	33.90	11.35	1945	0.0551
32.10	27.90	30.60	330	30.00	10.80	1850	0.0489
27.90	24.20	29.80	295	26.05	9.90	1698	0.0425
34.20	20.50	32.00	295	22.35	9.22	1580	0.0364
20.50	16.90	34.00	290	18.70	8.55	1465	0.0304
16.90	13.40	34.30	274	15.15	8.00	1370	0.0247
13.40	10.50	33.00	230	11.95	6.96	1193	0.0195
10.50	8.10	32.80	190	9.30	5.80	995	0.0151
8.10	6.00	33.90	162	7.05	4.78	820	0.0115
6.00	4.50	32.80	120	5.25	3.66	626	0.0086

Head for no flow = 0.00 cm , $T_{R1} = 0$ gm /cm^2

- -

4% slurry Pure Bentonite $\gamma_s = 1.024$ gm/cm^3

38.30	34.80	29.00	288	36.55	9.93	1700	0.0600
34.80	31.30	31.70	284	33.05	8.96	1540	0.0541
31.30	28.10	33.90	266	29.70	7.85	1345	0.0486
28.10	25.30	33.70	223	26.70	6.63	1136	0.0438
25.30	21.30	56.10	329	23.30	5.86	1080	0.0382
21.30	18.60	43.90	225	19.95	5.13	880	0.0327
18.60	15.60	55.50	238	17.10	4.30	735	0.0280
15.60	13.00	61.30	215	14.30	3.44	590	0.0234
13.00	10.70	70.10	190	11.85	2.71	465	0.0194
10.70	8.90	72.40	153	9.80	2.12	364	0.0161

Head for no flow = 1.30 cm , $T_{R1} \pm \frac{R}{2} i_1 = \frac{R}{2} \frac{\Delta h}{\Delta L} \gamma_s$

$$T_{R1} = \frac{R \times 1.3 \times 1.024}{2 \times 61.30} = 0.00212 \text{ gm/cm}^2$$

- -

1	2	3	4	5	6	7	8

4% slurry with 25% Replacement of bentonite by black cotton soil

γ_s = 1.024 gm/cm^3

37.70	33.20	32.10	378	35.45	11.80	2020	0.0581
33.20	28.60	35.20	370	30.90	10.50	1800	0.0506
28.60	24.60	35.30	325	26.60	9.20	1578	0.0435
24.60	21.10	35.30	284	22.85	8.05	1380	0.0374
21.10	18.20	33.70	240	19.65	7.11	1220	0.0322
18.20	15.20	38.20	235	16.70	6.15	1053	0.0274
15.20	12.70	39.80	204	13.95	5.11	878	0.0228
12.70	10.20	46.70	205	11.45	4.40	755	0.0188
10.20	7.80	56.80	197	9.00	3.48	596	0.0148
7.80	5.90	61.40	158	6.85	2.59	445	0.0112

Head for no flow = 0.60 cm, $T_{R1} = \frac{0.195 \times 0.60}{2 \times 61.30} \times 1.024$ gm/cm^2

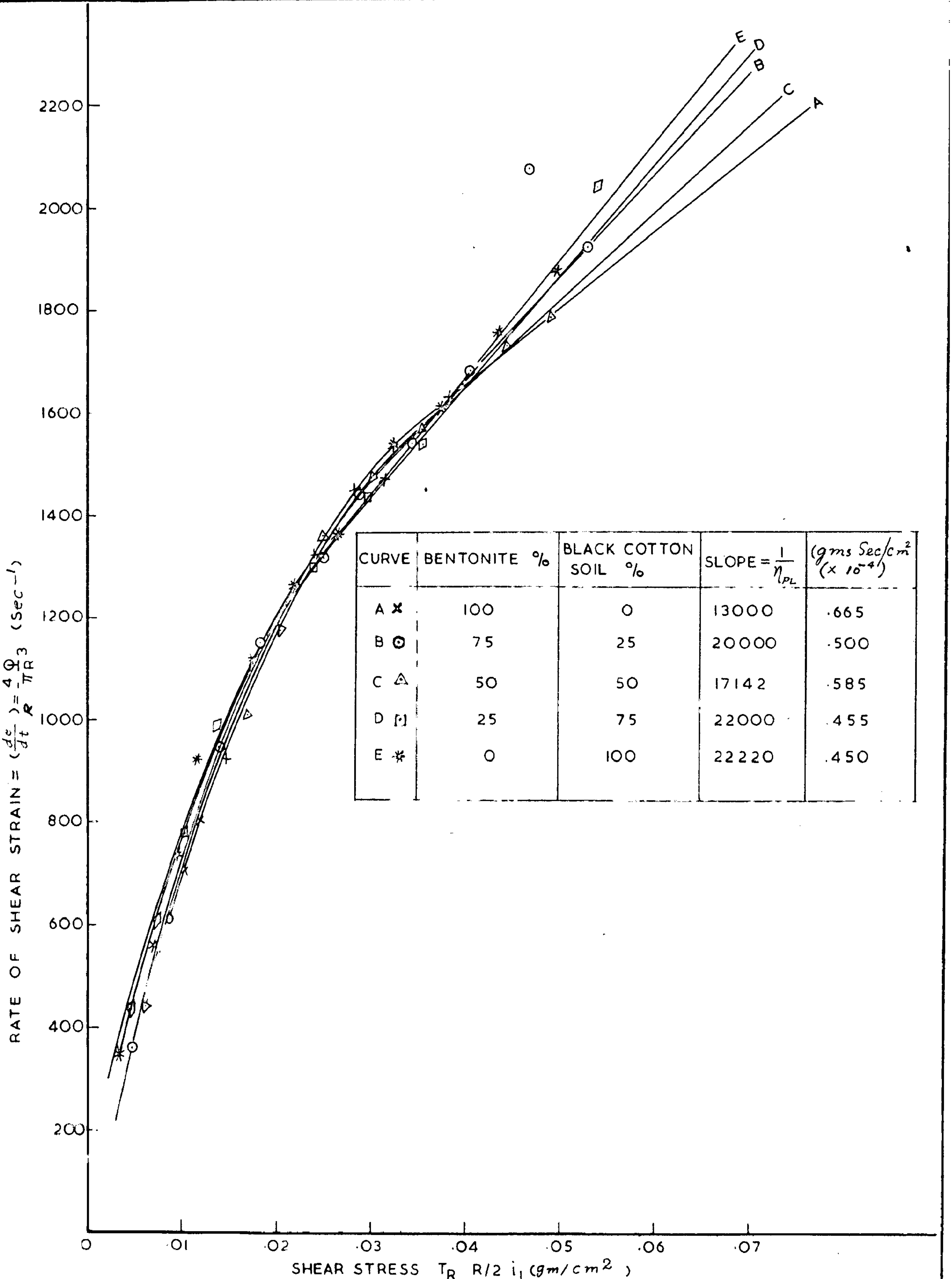
$= 0.000975$ gm/cm^2

4% Slurry with 50% Replacement. γ_s = 1.024gm/cm^3

37.80	33.20	35.00	398	35.50	11.75	2020	0.0560
33.20	29.00	32.10	341	31.10	10.60	1820	0.0490
29.00	24.20	40.10	390	26.60	9.70	1663	0.0420
24.20	20.20	36.00	319	22.20	8.85	1520	0.0361
20.20	16.60	37.00	287	18.40	7.75	1330	0.0291
16.60	13.20	40.50	266	14.90	6.55	1122	0.0236
13.20	10.20	44.10	240	11.70	5.43	930	0.0185
10.20	7.40	49.40	214	8.80	4.34	743	0.0139
7.40	5.40	49.00	159	6.40	3.25	557	0.0101
5.40	3.80	55.00	125	4.60	2.28	391	0.0073
3.80	3.00	45.00	70	3.40	1.56	267	0.0054

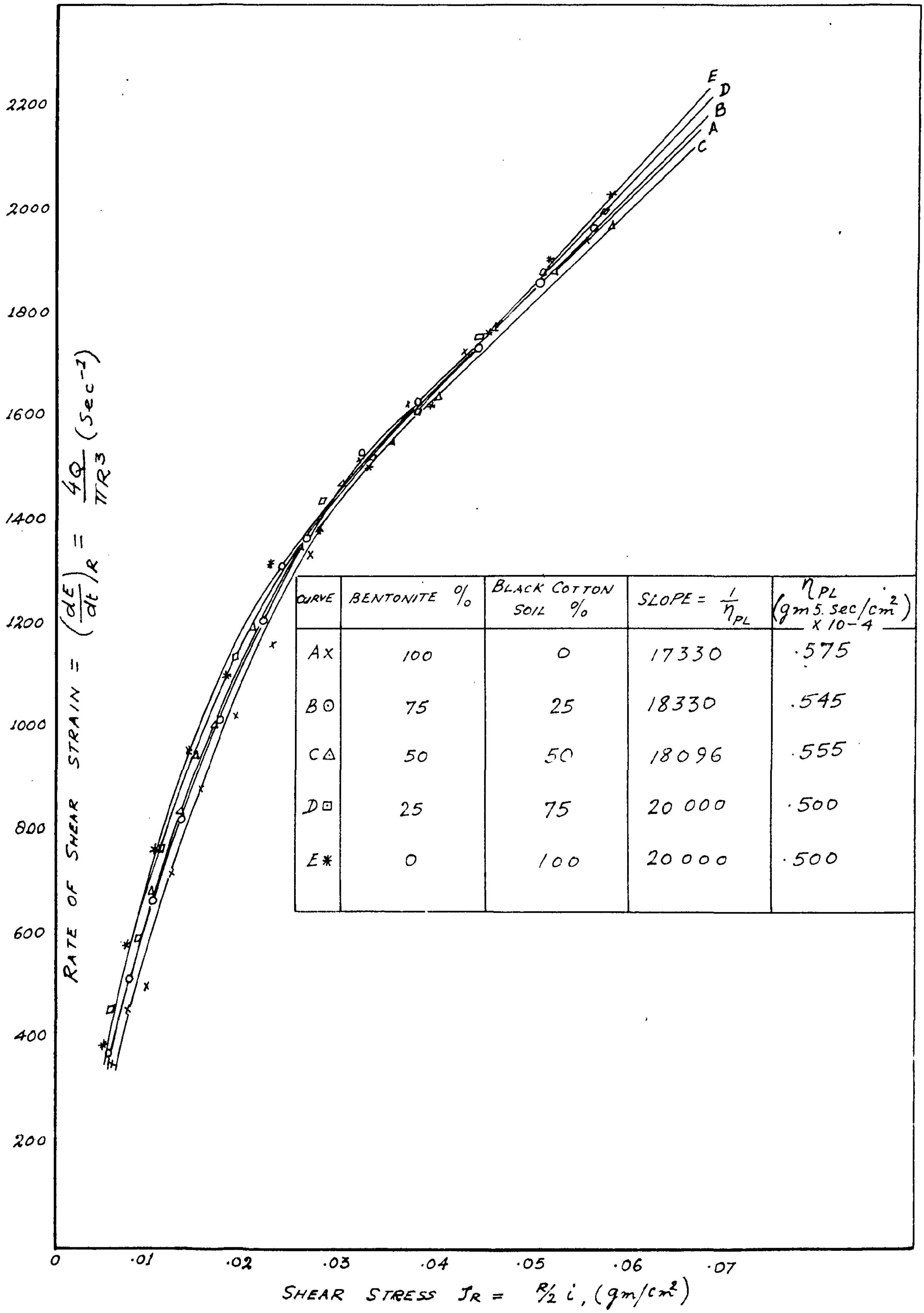
Head for no flow = 0.00 cm T_{R1} = 0 gm/cm^2

1	2	3	4	5	6	7	8
4% Slurry with 75 % Replacement of bentonite by black cotton soil							
γ_s = 1.024 gm/cm^3							
38.40	34.00	32.30	355	36.20	11.00	1885	0.0571
34.00	29.70	32.60	344	31.85	10.50	1800	0.0504
29.70	25.60	33.50	330	27.65	9.85	1686	0.0436
25.60	21.10	39.30	365	23.35	9.30	1592	0.0369
21.10	17.60	34.50	280	19.35	8.15	1400	0.0306
17.60	14.20	39.00	275	15.90	7.05	1210	0.0251
14.20	11.50	39.60	220	12.85	5.55	952	0.0203
11.50	9.30	36.20	171	10.40	4.72	810	0.0164
9.30	7.50	43.00	160	8.40	3.72	637	0.0133
7.50	6.00	45.70	110	6.75	2.40	411	0.0107
Head for no flow = 0.00 cm , T_{R1} = 0 gm /cm^2							
4% Slurry with pure black cotton soil, γ_s = 1.024 gm/cm^3							
38.60	34.20	33.30	370	36.40	11.20	1920	0.0575
34.20	29.80	33.90	355	32.00	10.50	1800	0.0505
29.80	25.50	36.40	357	27.65	9.76	1675	0.0436
25.50	21.40	37.00	333	23.45	9.00	1542	0.0370
21.40	17.00	42.70	350	19.20	8.20	1405	0.0303
17.00	13.50	38.20	280	15.25	7.31	1257	0.0241
13.50	10.40	40.50	250	11.95	6.16	1057	0.0189
10.40	7.80	4.170	207	9.10	4.96	850	0.0144
7.80	5.60	46.20	180	6.70	3.90	668	0.0106
5.60	4.00	49.00	135	4.80	2.76	473	0.0076
4.00	2.60	55.80	105	3.30	1.88	322	0.0052
Head for no flow = 0.00 cm. T_{R1} = 0 gm/cm^2							



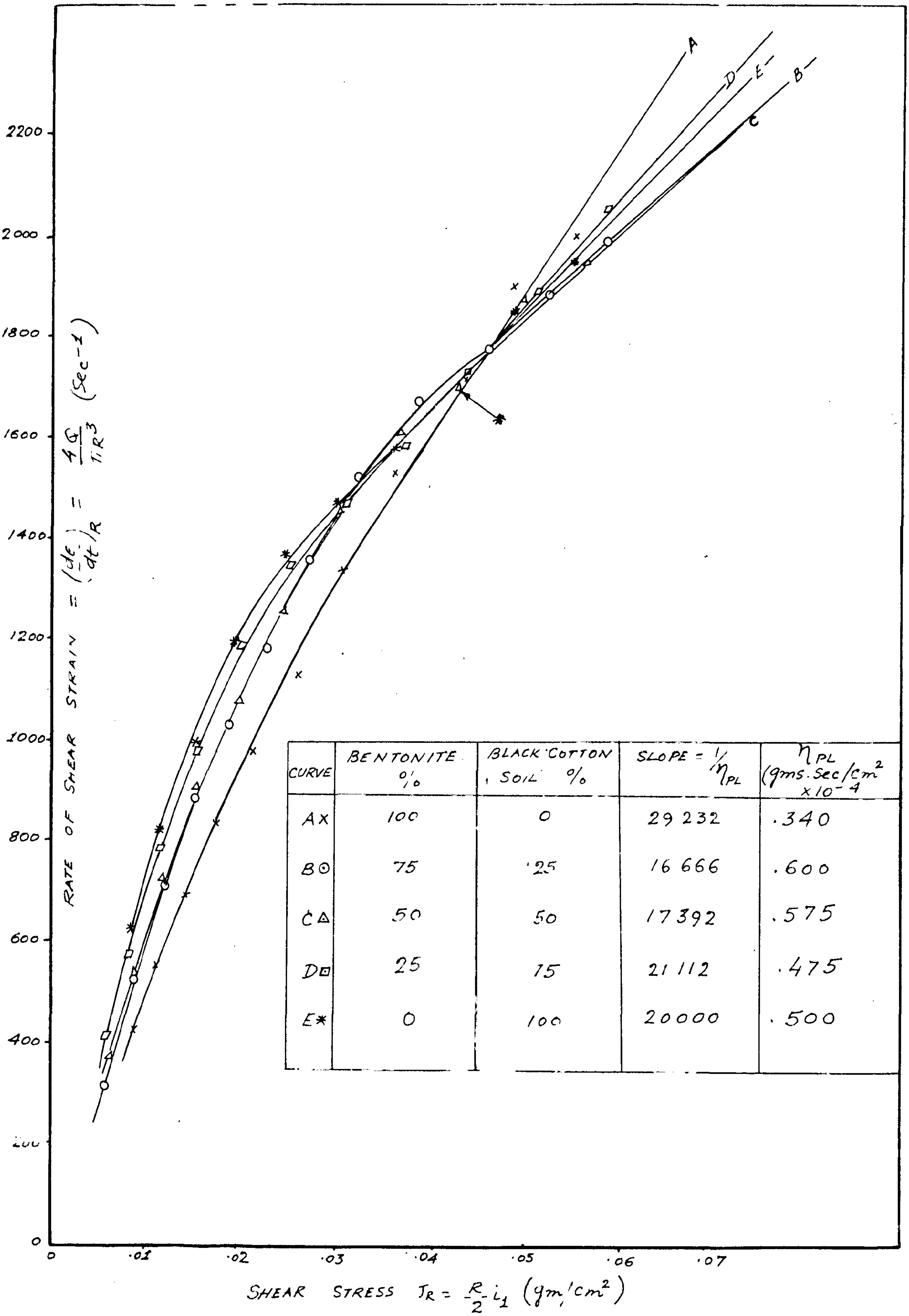
CURVE	BENTONITE %	BLACK COTTON SOIL %	SLOPE = $\frac{1}{\eta_{PL}}$	(gms Sec/cm^2) (× 10^{-4})
A ×	100	0	13000	·665
B ⊙	75	25	20000	·500
C △	50	50	17142	·585
D [·]	25	75	22000	·455
E ✳	0	100	22220	·450

FIG.3 2(a) FLOW RESULTS FOR 1% SLURRY



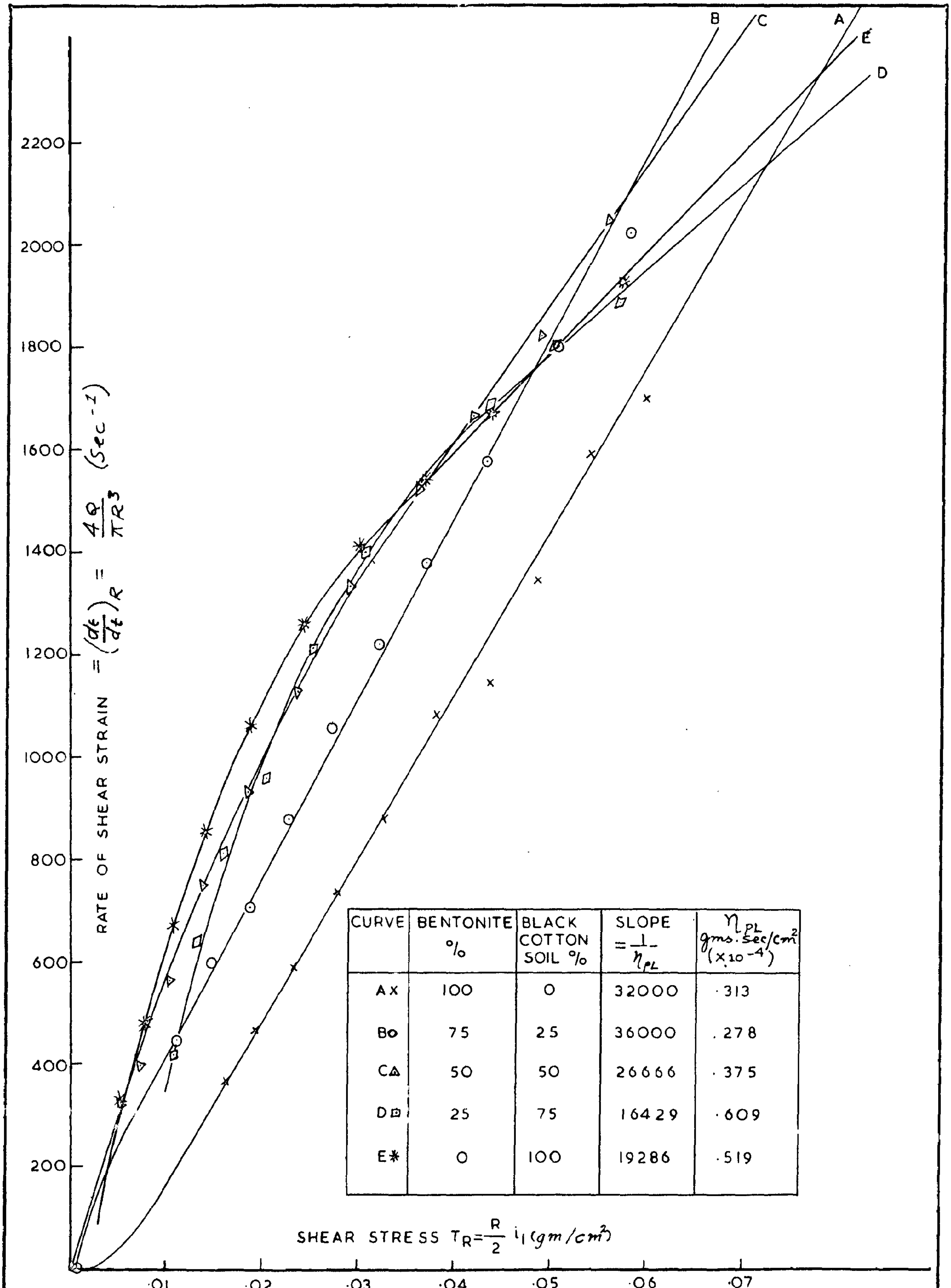
CURVE	BENTONITE %	BLACK COTTON SOIL %	SLOPE = $\frac{1}{\eta_{PL}}$	η_{PL} ($gm\,s.\,sec/cm^2$) $\times 10^{-4}$
A ×	100	0	17330	·575
B ⊙	75	25	18330	·545
C △	50	50	18096	·555
D ⊡	25	75	20 000	·500
E ✱	0	100	20000	·500

FIG 3.2(b) FLOW RESULTS FOR 2 % SLURRY



CURVE	BENTONITE %	BLACK COTTON SOIL %	SLOPE = $1/\eta_{PL}$	η_{PL} (gms. Sec/cm^2 $\times 10^{-4}$)
A ×	100	0	29 232	.340
B ⊙	75	25	16 666	.600
C △	50	50	17392	.575
D □	25	75	21 112	.475
E ✱	0	100	20000	.500

FIG 3.2(c) FLOW RESULTS FOR 3% SLURRY



CURVE	BENTONITE %	BLACK COTTON SOIL %	SLOPE $= \frac{1}{\eta_{PL}}$	η_{PL} gms. Sec/cm² (×10⁻⁴)
A ×	100	0	32000	·313
B o	75	25	36000	.278
C △	50	50	26666	·375
D ◻	25	75	16429	·609
E ✱	0	100	19286	·519

FIG. 3·2(d) FLOW RESULTS FOR 4 % SLURRY

3.5.2 Sieve Analysis of Sand

TABLE 3.2 SIEVE ANALYSIS RESULTS FOR THE SAND

Weight of Sand Taken 990.0 gm.

S. No.	Sieve No.	Particle size D (mm)	Weight of empty sieve (gm)	Weight of sieve and sand retained (gm)	Weight of sand retained (gm)	% retained	Cumulative % retained	Cumulative % finer
1	2.36	2.360	511.0	518.0	7.0	0.76	0.76	99.24
2	1.00	1.000	455.0	458.0	3.0	0.33	1.09	98.91
3	850μ	0.850	385.0	386.0	1.0	0.11	1.20	98.80
4	600μ	0.600	448.0	449.0	1.0	0.11	1.31	98.69
5	420μ	0.420	367.0	373.0	6.0	0.65	1.96	98.04
6	300μ	0.300	439.0	512.8	73.8	7.95	8.41	91.59
7	225μ	0.225	320.0	1000.6	680.6	68.74	77.15	22.85
8	150μ	0.150	408.0	420.0	12.0	1.30	78.45	21.55
9	75 μ	0.075	376.0	559.6	183.6	18.54	96.99	3.01
10	Pass	-	398.0	420.0	22.0 (passing 75μ)			

3.5.3 Stability Results and Calculations

Stabilizing effect of slurry saturated zone

From equation 2.28 the stabilizing effect of the slurry saturated zone, P_G is given by

$$P_s+E_p = \frac{\cot\theta(\frac{1}{2}\ \gamma H^2+ qH)(\sin\theta-\cot\theta\tan\phi')+(\frac{1}{2}\gamma_w m^2H^2\operatorname{cosec}\theta\tan\phi'-P_G)\tan\phi'}{\cos\theta + \sin\theta\tan\phi'}$$

where the variables are as defined earlier.

$$P_G = \frac{2\,\tau_m\,\ell^2}{r\cos\theta} \quad \text{(Equation 2.10 (a))}$$

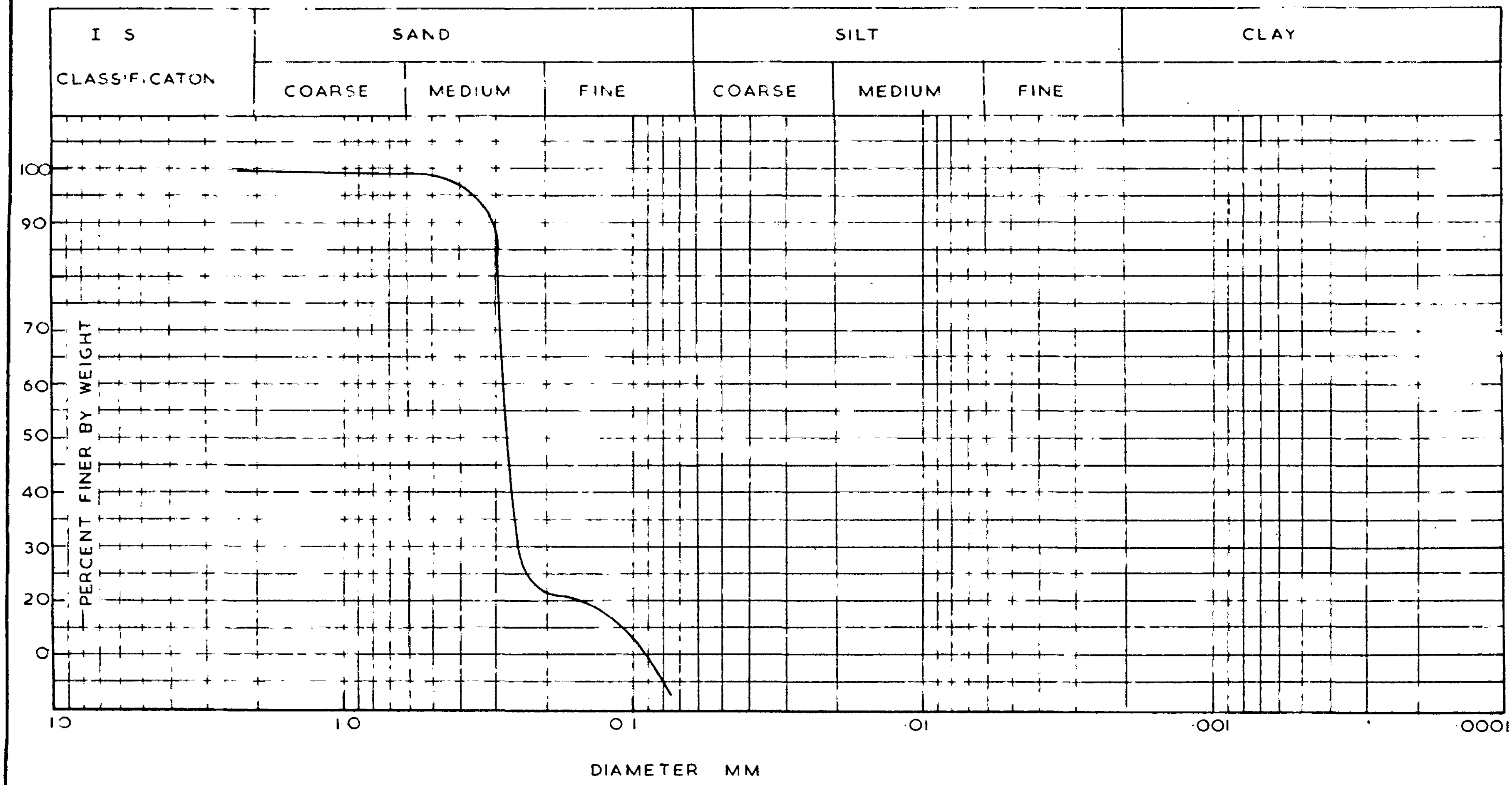


FIG. 3·3 GRAIN SIZE DISTRIBUTION FOR SAND

Effective Diameter $= D_{15} = 0.11$ mm

Effective pore radius $= \frac{D_{15}}{2} = 0.055$ mm

$= 0.0055$ cm

r being the effective pore radius of the sand = $\frac{D_{15}}{2}$

Also $P_s + = \frac{1}{2} \gamma_s H^2$

$$E_p = \frac{\tau_s H^2}{2a} + \frac{\pi \tau_s H}{2}$$

and substituting above

$$\frac{1}{2}\gamma_s H^2 + \frac{\tau_s H^2}{2a} + \pi \tau_s H$$

$$= \frac{\cot\theta(\frac{1}{2}\gamma H^2+qH)(\sin\theta-\cot\theta\sin\phi')+(\frac{1}{2}\gamma_w m^2H^2\operatorname{cosec}\theta\tan\phi'-P_G)\tan\phi'}{\cos\theta + \sin\theta \tan\phi'}$$

τ_s , the structural shear strength of the slurry in the trench is sufficiently small to be neglect~~ing,~~ed thus,

$$\frac{1}{2}\gamma_s H^2$$

$$= \frac{\cot\theta(\frac{1}{2}\gamma H^2+qH)(\sin\theta-\cot\theta\sin\phi')+(\frac{1}{2}\gamma_w m^2H^2\operatorname{cosec}\theta\tan\phi'-P_G)\tan\phi'}{\cos\theta + \sin\theta \tan\phi'}$$

Taking ϕ' for the sand as 30°, $\theta = 45 + \frac{\phi'}{2} = 60^\circ$. For the loading tests carried out all the variables except P_G are known (For dry sand m = 0) . For each loading test, P_G is calculated as follows -

Sample Calculation for P_G

For 4% Pure bentonite slurry,

$\gamma_s = 1.024$ gm/cm^3

H = 14.7 cm,

Failure load = 45.74 kg.

Area of Loading plate = 316 sq.cm.

$$\therefore \quad q' = \frac{45.74}{316} = 0.145 \ \text{kg/cm}^2 = 145 \ \text{gm/cm}^2$$

$2\ell = 3$ cm , $\ell = 1.50$ cm

For dry sand $m = 0$, $\gamma = 1.578$ gm/cm^3

$$\therefore \quad \frac{1}{2} \times 1.024 \times (14.7)^2$$

$$= \frac{\cot 60^\circ \left[\frac{1}{2} \times 1.578 \times (14.7)^2 + 145 \times 14.7\right]\left[\sin 60^\circ - \cos 60^\circ \tan 30^\circ\right] + \left[0 - P_G\right] \tan 30^\circ}{\cos 60^\circ + \sin 60^\circ \tan 30^\circ}$$

$$\therefore 110.9 = \frac{0.5774(170+2134)(0.866-0.5 \times 0.5774) + (-P_G \times 0.5774)}{0.5 + 0.866 \times 0.5774}$$

$$= 770 - 0.5774 \, P_G$$

$$\therefore \quad P_G = \frac{770 - 110.9}{0.5774} = 1140 \ \text{gm/cm}$$

From sieve analysis results (Fig. 3.3), $r = \frac{D_{15}}{2} = 0.0055$ cm.

$$\therefore \quad 1140 = \frac{2\,\tau_m \times (1.50)^2}{r \cos\theta} = \frac{2\,\tau_m \times 2.25}{0.0055 \times 0.5}$$

$$\therefore \quad \tau_m = \frac{1140 \times 0.005 \times 0.5}{2 \times 2.25} \ \text{gm/cm}^2$$

$$= 0.696 \ \text{gm/cm}^2$$

say $= 0.7$ gm/cm^2

All the calculations and results for dry sand are summarised on Table 3.3 . Results for loading on saturated sand are shown on Table 3.4 . Fig. 3.4 shows a plotting of the loading results for dry sand.

TABLE 3.3 SUMMARY OF RESULTS (Dry Sand Loading) γ = 1.578 gm/cm^3, ϕ' = 35^0

Slurry Concentration (%)	Slurry density γ_s (gm/cm^3)	Bentonite (%)	Black cotton soil (%)	Depth of slurry H (cm)	Double width of slurry saturated zone 2ℓ cm	Plastic viscosity η_{PL} gm-sec/cm^2 (x10^{-3})	Total Failure load (kg)	Failure surcharge (kg/cm^2)	Hydrostatic Pres. of slurry $\frac{1}{2}\gamma_s H^2$ (kg/cm)	$P_G \tan\phi$ (kg/cm)	P_G (kg/cm)	τ_m (kg/cm^2) (x10^{-3})
1	1.005	100	0	16.5	-	0.0665	40.24	0.127	0.136	0.635	1.100	-
		75	25	15.0	-	0.0500	41.24	0.130	0.113	0.595	1.028	-
		50	50	14.8	-	0.0585	64.24	0.204	0.111	0.950	1.650	-
		25	75	15.0	-	0.0485	98.24	0.298	0.113	1.440	2.495	-
		0	100	16.0	-	0.0450	67.24	0.181	0.128	0.910	1.572	-
2	1.013	100	0	15.0	-	0.0575	49.24	0.140	0.115	0.645	1.117	-
		75	25	15.0	-	0.0545	82.00	0.260	0.115	1.250	2.160	-
		50	50	15.4	-	0.0555	90.24	0.286	0.120	1.410	2.440	-
		25	75	14.3	-	0.0500	64.24	0.203	0.104	0.936	1.620	-
		0	100	14.8	-	0.0500	75.70	0.240	0.112	1.720	2.970	-
3	1.018	100	0	14.5	5.3	0.0340	79.24	0.250	0.107	1.158	2.000	0.394
		75	25	10.9	8.5	0.0600	82.24	0.276	0.105	1.270	2.210	0.168
		50	50	13.1	13.5	0.0575	119.24	0.374	0.088	1.600	2.760	0.084
		25	75	14.0	13.9	0.0475	76.24	0.242	0.100	1.080	1.870	0.054
		0	100	14.0	10.0	0.0500	37.24	0.118	0.100	0.505	0.873	0.048
4	1.024	100	0	14.6	3.0	0.0313	45.74	0.145	0.110	0.659	1.133	0.693
		75	25	14.6	3.0	0.0278	86.24	0.115	0.110	0.507	0.870	0.536
		50	50	13.5	6.0	0.0375	104.24	0.330	0.093	1.440	2.485	0.380
		25	75	14.5	14.5	0.0609	83.04	0.262	0.108	1.210	2.100	0.055
		0	100	15.0	10.5	0.0519	79.24	0.250	0.115	1.200	2.070	0.104

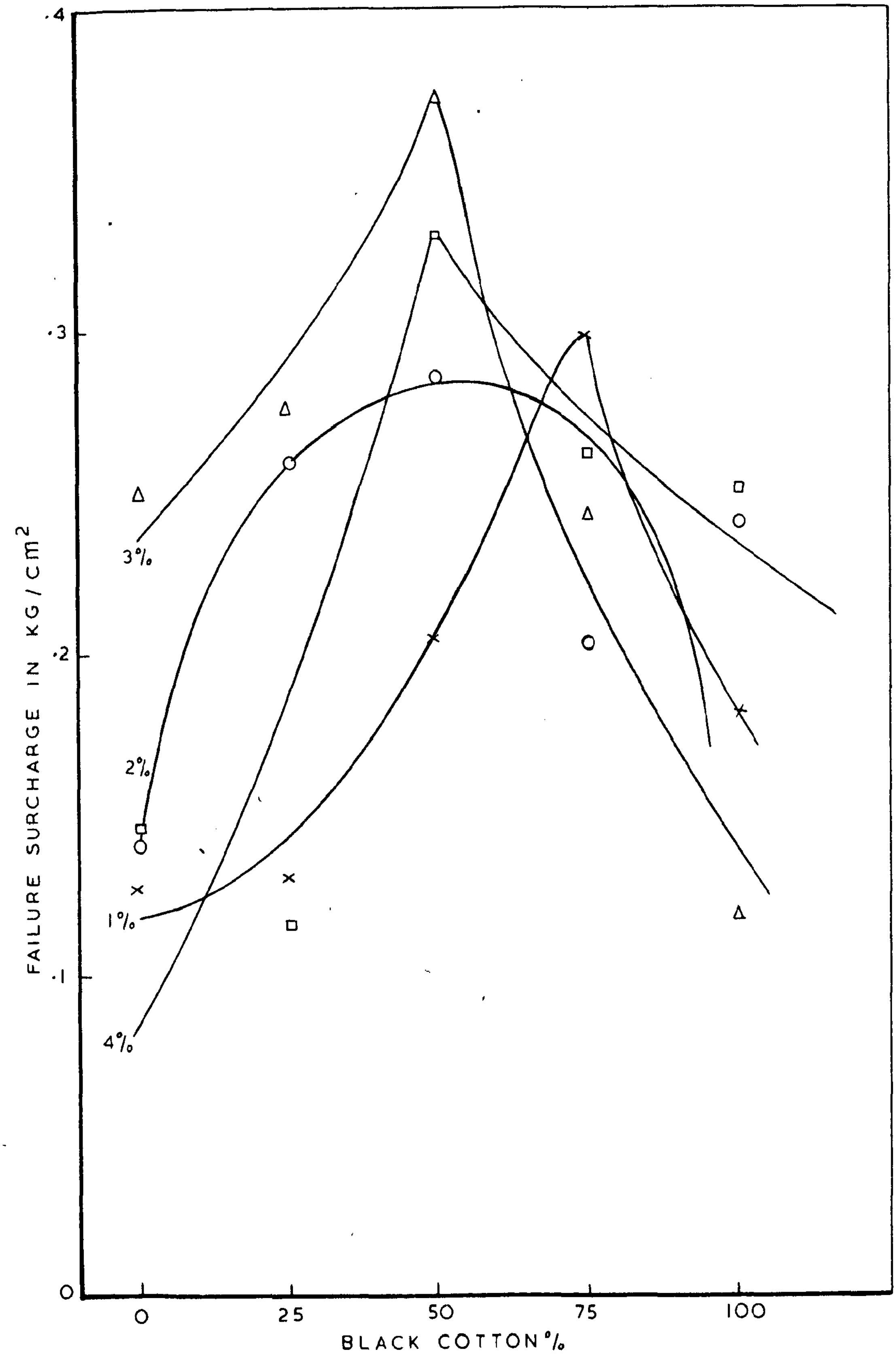


FIG.3·4 LOADING RESULTS FOR DRY SAND

TABLE 3.4 LOADING RESULTS FOR SATURATED SAND

Slurry concentration %	Slurry density gm/cm3 γ_s	Bentonite %	Black cotton soil %	Depth of slurry H cm	Remarks
1	2	3	4	5	6
1	1.005	100	0	15.0	No load . Failure occurred as soon as partition was removed
		25	75	14.0	No load , Failure occurred before partition was completely removed i.e. failure at partial removal of partition
2	1.013	100	0	14.5	No load.Failure at partial removal of partition.
3	1.018	100	0	14.5	No load. Failure occurred immediately after complete removal of partition.
4.	1.024	100	0	14.5	No load but just stable.

3.6 PROBLEMS ENCOUNTERED

A few problems were encountered in this study. The slurry made from the commercial bentonite and black cotton soil could not be kept free of segregation. Only the 3% and the 4% pure bentonite slurries did not show much segregation. This segregation disrupted the the uniformity of the slurry and subsequently the shear strength of the slurry already created. In this respect it would be difficult to achieve the expected maximum gel strength, τ_m , of the slurry.

In order to avoid the segregation the slurry had to be kept in agitation throughout and the shear strength utilized is that under disturbed conditions.

Efforts to determine the shear strength of the slurry with a shear vane apparatus failed mainly due to the small amount of the shear strength as compared to the magnitude which could be determined by the available apparatus.

Initially trials to obtain failure load by placing different loads on the loading plate did not result either to success or practicability. The loading method as shown on plate 3.3 was found to be very unsafe. The loads could slip which was very likely at failure as the loading plate was tilted by the flow of sand into the slurry. At one occasion the loading box was greatly damaged and time had to be allowed for repair work. Fortunately no person or other apparatus was nearby otherwise greater damage would have occurred. It was also difficult to accurately record the failure load due to difficulty in gradual variation of the loading operation. At times it was necessary to repeat the loading due either to unreliable results or the necessity of checking the failure load. This method of loading was unsuitable for this checking as the slurry could not be recovered after failure. The sand was flown into the slurry by the sudden skidding of the weights and no undisturbed slurry could be recovered. The axial loading unit of the triaxial machine, however, was a remedy to all these pitfalls.

CHAPTER IV

DISCUSSION OF RESULTS

1. Bentonite replacement by black cotton soil in bentonite slurries increases the fluidity of the slurry. This is obvious from the flow test results in Table 3.1 which show that for the same slurry concentration and mean head, more flow per unit time, Q, is obtained for the slurries where bentonite has been replaced by black cotton soil. Likewise from the loading test results, table 3.3, for same slurry concentration, the rate of permeation into the sand is more for the slurries where bentonite has been replaced by black cotton soil. The increase in the fluidity of the slurry is due to the lowering of the plastic viscosity, η_{PL}, of the slurry (Figures 3.2(a), (b), (c) and (d)).

2. Decrease in plastic viscosity of the slurry with subsequent increase of permeation rate into soil means increase of the slurry saturated zone. This will increase the magnitude of the pore pressure, P_G, (equation 2.10(a)), resulting in more stabilizing force of the trench (equation (2.28)). This is confirmed by the results of table 3.3. which show that more replacement of bentonite by black cotton soil means more slurry saturated zone and more surcharge that can be sustained.

3. For each slurry concentration, there is a particular proportion of bentonite which can be replaced to give a peak sustained surcharge. This proportion varies between 50% and 75%. (Table 3.3 and Figure 3.4).

4. For dry sand and for the slurry concentration range studied (1% to 4%), the major stabilizing force is that due to the shearing resistance of the slurry saturated zone, $P_G \tan \phi$. As shown on table 3.3, the stabilizing force due to P_G, is between 5 and 10 times the hydrostatic force.

In order to visualize the magnitude of this contribution in prototype (field) trenches the following calculations are necessary.

From equation (2.10(a)),

$$P_G = \frac{2 \tau_m \ell^2}{r \cos \theta}$$

Assuming that for the model and prototype, the same sand, and slurry are used, and that the inclination of the failure wedge to the horizontal is the same, then,

$$P_G \propto \ell^2 \qquad \ldots(4.1)$$

Further, from equation (2.6) the depth, 2ℓ, penetrated by the slurry is given by

$$2\ell = \frac{pgr}{2\tau_f}$$

$$\text{i.e. } 2\ell \propto p \qquad \ldots(4.2)$$

where p is the applied pressure differential

$\therefore \ell \propto (\gamma_s H - \gamma_w m H)$, for saturated sand, and

$\ell \propto \gamma_s H$

i.e. $\ell \propto H$ for dry sand (m=0) ...(4.2a)

$\therefore P_G \propto \ell^2$

$\propto H^2$ (from equations (4.1) and 4.2a) ...(4.3)

Thus if

P_{Gm} = Negative gel pressure in model

P_{Gp} = Negative gel pressure in prototype (field)

H_m = depth of slurry in model

H_p = depth of slurry in prototype.

Then,

$$\frac{P_{GP}}{P_{Gm}} = \frac{H_P^2}{H_m^2}$$

$$\therefore \quad P_{GP} = \left(\frac{H_P}{H_m}\right)^2 P_{Gm} \qquad \ldots(4.4)$$

From equation (2.28), if q_p is the unit surcharge in the prototype then,

$$\frac{1}{2}\gamma_s H_p^2 = \frac{\cot\theta\,(\frac{1}{2}\gamma H_p^2 + q_p H_p)(\sin\theta - \cos\theta \sin\phi') - P_{GP}\tan\phi'}{\cos\theta + \sin\theta\tan\phi'}$$

(Since m = 0 for dry sand)

$$\frac{1}{2}\gamma_s H_p^2 = 0.334\,(\frac{1}{2}\gamma H_p^2 + q_p H_p) - 0.5773\,P_{GP} \qquad \ldots(4.5)$$

for $\phi' = 30^\circ$ and $\theta = 60^\circ$

From a known value of H_p, using the values of H_m, γ, γ_s and P_{Gm} from table 3.3, values of P_{Gp} can be calculated from equation (4.4) and subsequently values of the first term on the right hand side of equation (4.5) Then

(i) Total disturbing force = $0.334\,(\frac{1}{2}\gamma H_p^2 + q_p H_p)$

(ii) Stabilizing forces are

(a) Hydrostatic pressure = $\frac{1}{2}\gamma_s H_p^2$

(b) Horizontal component of $P_{GP} = P_{GP}\tan\phi' = 0.5773 P_{Gp}$.

neglecting other forces which are small in comparison. Calculations based on a 10 meter deep trench are summarised in Table 4.1

Table 4.1 shows that for the full scale trench, the stabilizing force due to the slurry saturated zone is still predominant. However, the neglecting of the stabilizing forces due to the filter cake, the pressure resistance of the slurry and the other small stabilizing forces will have the effect of making $P_G \tan \phi$ appear slightly larger than it should actually be. From table 2.4 and 2.5 these neglected forces account for less than 10-20% of the disturbing force. Even with this allowance the contribution of the slurry saturated zone in the full scale trench is still more than 60%. This would mean that the slurry saturated zone contribution appears to be more than three times that of the hydrostatic force, in the particular conditions assumed.

5. For the few tests carried on saturated sand, stability was only achieved for 4% pure bentonite slurry. This seems to indicate that denser slurries are required for stability of trenches in saturated soils. Experiments carried out by RAJU and SASTRY[12] on saturated sand showed that denser slurries (3% and above) are required for stability of trenches in saturated sands. The trench was 50 cm deep and the sand was saturated from the bottom until failure occurred. The sand was saturated after the filter cake and the slurry saturated zone had formed. All the trenches failed when the water level in the sand was between 39 and 48 cm above bottom except for 7% pure bentonite slurry. The sand used had D_{10} of 0.3 mm. There was no surcharge. These

TABLE 4.1 APPLICATION OF LOADING RESULTS TO FULL SCALE TRENCH.

H_p = 10.0 metres γ = 1.578 gm/cm^3 $\frac{1}{2}\ \gamma\ H_p^2$ = 789 kg/cm

Slurry Conc. %	Bento-nite %	Black Cotton soil %	Slurry Density γ_s (gm/cm^3)	Depth of slurry in model H_m (cm)	P_{GM} (kg/cm)	P_{GP} (kg/cm)	Disturbing force $0.334(\frac{1}{2}\gamma H_p^2 + q_p H_p)$ (kg/cm) (9+10)	Stabilizing Forces. Hydrostatic pres. of slurry (kg/cm)	Stabilizing Forces. $P_G \tan\phi$ ±.5773P_G (kg/cm)	% of Disturbing Force Hydrostatic Pres. of slurry	% of Disturbing Force $P_G \tan\phi$ ±
1	2	3	4	5	6	7	8	9	10	11	12
1	100	0	1.005	16.5	1.100	4025	2852.5	502.5	2350	17.60	82.40
	75	25		15.0	1.028	4560	2843.0		2640	17.65	82.35
	50	50		14.8	1.650	7500	4832.5		4330	10.40	89.60
	25	75		15.0	2.490	11000	6802.5		6300	7.40	92.60
	0	100		16.0	1.572	4850	3302.5		2800	15.20	84.60
2	100	0	1.013	15.0	1.117	4950	3356.5	506.5	2850	15.20	84.60
	75	25		15.0	2.160	9600	6006.5		5500	8.45	91.55
	50	50		15.4	2.440	10300	6406.5		5900	7.90	92.10
	25	75		14.3	1.620	8000	5106.5		4600	9.90	90.10
	0	100		14.8	2.970	13600	6306.3		7800	8.05	91.95
3	100	0	1.018	14.5	2.000	9500	5989.0	509.0	5480	8.50	91.50
	75	25		14.4	2.210	10700	6609.0		6190	7.70	92.30
	50	50		13.1	2.760	16200	6609.0		6300	5.20	94.80
	25	75		14.8	1.870	8560	5449.0		4940	9.35	90.65
	0	100		14.0	0.873	4450	3059.0		2550	16.65	83.35
4	100	0	1.024	14.6	1.133	5300	3562.0	512.0	3050	14.40	85.60
	75	25		14.6	0.876	4100	2862.0		2350	17.90	82.10
	50	50		13.5	2.485	13600	8342.0		7800	6.15	93.85
	25	75		14.5	2.100	10000	6285.0		5773	8.15	91.85
	0	100		15.0	2.070	9300	5812.0		5300	8.80	91.20

results reveal less stabilizing effect of slurry saturated zone than that in the present series. (The 4% pure bentonite test was just stable). One factor would be that the sand used by Raju and Sastry being coarser (the one used here had D_{15} of 0.055 mm) would, from equation(2.10a), decrease the magnitude of the negative pure pressure P_G, which is as shown is inversely proportional to r and $r = D_{15}/2$ (or $D_{10}/2$). $P_G \tan \phi'$ is the contribution of the slurry saturated zone to stability. However, the main aspect seems to be the nature of shear strength in the slurry saturated zone and the mode of loading. The penetration of slurry creates large capillary forces leading to a large apparent cohesion in the dry sand. This apparent cohesion is lost on saturation leading to loss of strength, even in cases where the slurry has been allowed to penetrate, as by Raju and Sastry, under dry conditions. Actually in the field, below water table the sand is already saturated at the time of excavation, and the penetration itself is much less than for dry sands. Thus, the large stabilising force of slurry saturated zone, can only be relied upon above the water table.

The equilibrium achieved for the 4% pure bentonite slurry can be viewed from Rankine's Earth pressure theory approach. The activating force, P_a, is made up of

(a) Lateral pressure due to submerged sand (of submerged density γ') and (b) lateral pressure due to water (there was no surcharge)

i.e. $P_a = \frac{1}{2} K_a \gamma' H^2 + \frac{1}{2} \gamma_w H^2$, $K_a = \frac{1}{3}$ for $\phi' = 30^\circ$,

$\gamma_{sat} = 1.96$ gm/cm^3

$$P_a = \frac{1}{6}(1.960-\gamma_w) + \frac{1}{2}\gamma_w H^2 = \frac{1}{6} x\ 0.96H^2 + \frac{1}{2} H^2 \quad (\gamma_w = 1.00\, gm/cm^3)$$

$$= 0.66\ H^2$$

H was 14.5 cm.

$$\therefore P_a = 0.66\ (14.5)^2$$

$$= 140\ gm/cm.$$

Hydrostatic pressure of the slurry $= \frac{1}{2}\ \gamma_s H^2$

$$= \frac{1}{2} \times 1.024 \times (14.5)^2\ gm/cm$$

$$= 108\ gm/cm.$$

All other stabilizing forces sum up to

(140- 108) gm/cm = 32 gm/cm.

The hydrostatic pressure of the slurry contributes a resistance, 77.3% of the disturbing force.

This indicates that the hydrostatic pressure of the slurry provides the major contribution to stability in saturated sands. The permeation of the saturated sand by the slurry is not effected unless a substantial differential head between the slurry and the water level in the sand is maintained (equation 2.6). This limits the depth of the slurry saturated zone, further as remarked earlier, capillary tensions cannot be sustained in saturated sand.

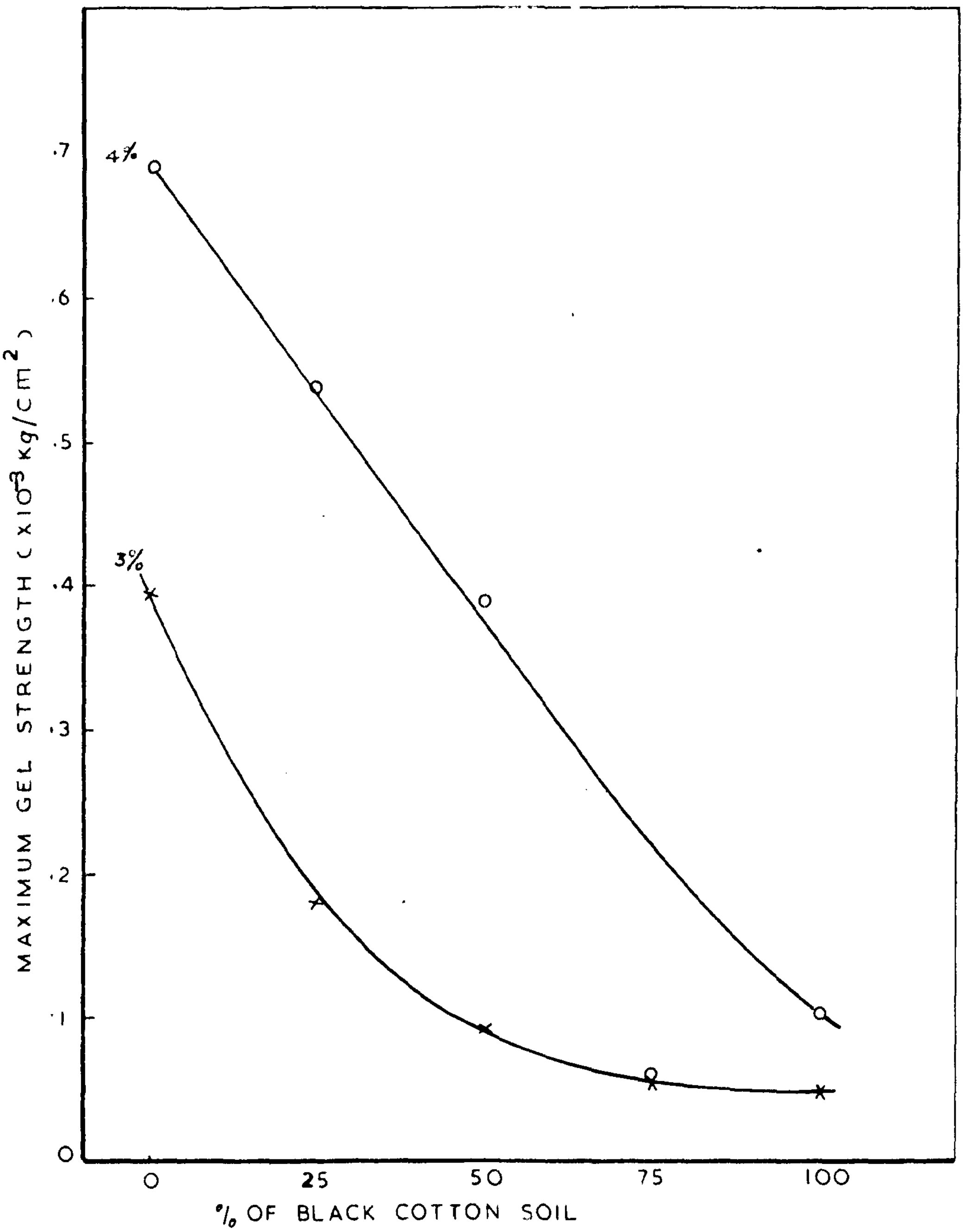


FIG. 3·5 RELATION BETWEEN Jm AND % OF BLACK COTTON SOIL

CHAPTER V

CONCLUSIONS

From the observed results and the discussions of Chapter IV, the following conclusions can be drawn.

1. Lean bentonite slurries have less plastic viscosity than dense ones. This reduced plastic viscosity increases their fluidity and their ability to flow and permeate soil pores. Replacement of bentonite by black cotton soil also reduces the plastic viscosity of the slurry and thus increasesthe permeation rate. The reduction of plastic viscosity with increase in percentage of black cotton soil does not,however show a regular trend. This is probably due to the limited accuracy of apparatus used.

2. In stability of slurry trenches the three main stabilizing factors are the hydrostatic pressure of the slurry, the shearing resistance of the slurry saturated zone and the passive resistance of the slurry. In the lean slurries studied, the structural shear strength of the slurry is very small. This will also make the contribution of the passive resistance of the slurry to stability small. Thus in the slurries studied the first two were the main stabilizing factors.

3. In dry sand trenches and for the sand size used, the major stabilizing force is that due to the shearing resistance of the slurry saturated zone. The rate of penetration and the depth of the slurry saturated zone can be increased by

replacing some proportion of bentonite by black cotton soil.

4. For each slurry concentration there is a particular proportion of bentonite which can be replaced by black cotton soil to achieve optimum stabilizing effect.

5. For saturated sands the rate of permeation of bentonite slurry into the sand is greatly reduced . In this case, unless the permeation rate can be increased by creating a sufficient pressure differential, the effect of the slurry saturated zone is not significant. Stability will depend mainly on the hydrostatic pressure of the slurry. The effect of using bentonite slurries with black cotton soil as admixture for stability of saturated sand trenches will thus need maintaining a pressure differential to effect formation of a sizable slurry saturated zone. Even so due to full saturation of pores the effect would be relatively less. This needs further exploratory work.

Scope for Further Work

1. The results of loading tests on dry sand indicated the importance of the slurry saturated zone formed by permeation of slurry into the sand to improve the physical properties of the zone by gelling in soil pores under quiescent conditions. Further study is necessary to investigate the extent to which the shear strength of the zone is improved by the slurry. Triaxial shear tests on specimens sampled from the zone would give qualitative results.

2. The effect of additives as sodium carbonate need also to be thoroughly studied.

3. There are many variables controlling the stabilizing effect. Some of these, like the structural shear strength of the slurry, resistance to deformation of the filter cake and the electro-osmotic forces, though small, have been neglected. A qualitative contribution of these factors needs investigation and refined apparatus is needed. This will also enable more control of the variables.

4. The results obtained are based on commercial bentonite, which as discussed earlier had, among other things, the problem of segregation. Usage of better quality bentonite may have some effect on the results.

5. The range of slurry concentration studied is only from 1% to 4% . A wider range, preferably upto 8% is suggested.

6. Most river valley projects involve sites of saturated sand deposits. The few tests carried on saturated sand indicate that the properties of the slurry required for stability (e.g. density, black cotton soil etc) are different from those of slurries for stabilizing dry sand. A thorough exploratory work is needed in this field which in most cases is of more practical application to the hydraulic engineering profession.

As a concluding note, it is worth mentioning that the experimental work has been planned and carried out with the purpose of investigating the usefulness of black cotton soil as an admixture to bentonite slurry in a qualitative way. This aim has been fulfilled to a certain extent. The tests were mainly conducted on dry sand and more work is necessary on other types of soils which occur in fields of engineering practice. All the variables and factors likely to affect the study should, where possible, be evaluated as accurately as possible with their quantitative and qualitative contribution fully determined.

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