GROUTING OF PERVIOUS SOILS

A Dissertation
submitted in partial fulfilment
of the requirements for the Degree
of

MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

By

S.G. GULABSINGANI

UNIVERSITY OF ROORKEE CENTRAL LIBRARY
Library Acc No. 105180
Date 27/12/68
ROORKEE

WATER RESOURCES DEVELOPMENT TRAINING CENTRE
UNIVERSITY OF ROORKEE
ROORKEE

September 1968

Dr K. C. Thomas,
B. Sc., M. S. (C. E.), D. Sc. (Engg.), M. I. E. (India),
M. Am., Soc. C. E., Member, Societie Des Ingenieor
Civil de France, Member Indian National Society
of Soil Mechanics & Foundation Engineering.

CERTIFICATE

Certified that the dissertation entitled "Grouting of Pervious Soils" which is being submitted by Sri S.G. Gulabsingani in partial fulfillment of the requirements to Degree of Master of Engineering in Water Resources Development of University of Roorkee is a record of student's work carried out by him under my supervision and guidance. The matter embodied in this has not been submitted for any other degree or diploma.

This is further to certify that he has worked for a period of 24 months from 1.10.1966 to Sept., 1968 in connection with the preparation of this dissertation.

(K. C. THOMAS)
DIRECTOR DAMS II,
Central Water & Power Commission,
New Delhi

New Delhi

Dated: 7/10/68

(Internal Guide by Virtue being professor WRDT upto 1.1.68)

UNIVERSITY OF ROORKEE
ROORKEE

REPORT ON THE DISSERTATION SUBMITTED BY SRI S.C.GULABSINGHANI FOR THE AWARD OF MASTER OF ENGINEERING DEGREE IN W.R.D.

The dissertation submitted by Sri S.C.Gulabsinghani is a review work on grouting of pervious soils. The dissertation has been compiled systematically and covers almost all the aspects including theoretical treatment available on the subject and practical applications. Although the subject is new, yet a comprehensive treatment of the subject has been presented. Almost all the available literature has been covered. The review is thus a good work for the practicing engineers. The utility of the work could have, however, been enhanced by inclusion of discussion on the difficulties encountered and the reasons of unsuccess in grouting operations on some of the dams of this country, about which mention has been made in the dissertation.

Sd/-B.N.Gupta
Research Officer(Soils)
I.I.R.I. ROORKEE
(External Examiner)

Dated -

.-.

UNIVERSITY OF ROORKEE
ROORKEE

No.Ex/ /E-139 Dated December ,1969.

Copy forwarded for information to :-

1. Prof.& Head of W.R.D.T.C.
2. Dr. K.C.Thomas, Director Dams, C.W.& P.C. New Delhi.

Asstt.Registrar(Exams)

S.C.Shah,
B.E.,S.S.E.(I),A.M.I.E.(India)

CERTIFICATE

Certified that the dissertation entitled " Grouting of Pervious soils " which is being submitted by Shri S.G.Gulabchandani in partial fulfilment of the requirements to the degree of Master of Engineering in Water Resources Development of University of Roorkee is a record of the student's work carried out by him under my supervision and guidance.

This is further to certify that he has worked for a period of 23 months from October 1966 to August 1968 in connection with the preparation of this dissertation.

Ukai.

Dated: 3rd Sept. 1968

(S.C.Shah)
Superintending Engineer (Civil)
Ukai Project, Ukai.

ACKNOWLEDGEMENTS

I take the opportunity to express my deep and sincere gratitudes to Dr K.C.Thomas who helped me in selection of subject, and for his invaluable guidance as well as assistance in advising on sources of information for preparation of this dissertation, inspite of his heavy engagements at the W.R.D.T.C.,Roorkee and at C.W.& P.C.,New Delhi.

My sincere thanks are also due to Sri S.G.Shah, Superintending Engineer (C), Ukai who inspite of heavy engagements at the project to be kind enough to become my external guide and for his invaluable suggestions in making this dissertation a success.

I am also thankful to Sri B.N.Gupta, Research Officer, U.P.Irrigation Research Institute,Roorkee, and Sri Vijendra Singh, Superintending Engineer, Design Directorate,Lucknow in collection of information on the Grouting Practice followed at Obra Dam and offered me to make use of the published literature.

I am also thankful to the authors of the published and unpublished literature and the Research Institutes whose references I have quoted.

I also gratefully acknowledge the help and encouragements received from all others in preparation of the dissertation.

S. Gulabsingani 7.10.1968

(S.G.GULABSINGANI)

SYNOPSIS

This dissertation presents the modern practice of grouting of pervious soils for impermeabilization and increase in structural strength. It traces the history of development from 1802 when it was first applied, to the present when this art has been more scientifically applied to form a seepage barrier. The importance of Geological investigation of dam sites is emphasized and various methods of field exploration, sampling and permeability tests are given which will help in drawing specifications and preparation of the estimates. Scope and objectives of the application of grouting, various types of grout ingredients, grout mixes formed by additions of one or more grout ingredients and their properties and rheological characteristics are also explained. The mechanism of grout penetration, effect of viscosity and shear strength granulometry of the grout and soil, injection pressures, consumption, zone of influence, efficiency of the grout curtain are all treated mathematically. It also gives the various types of laboratory tests performed to design a grout mix suitable for a job and procedure of field grout tests to find the suitability of the grout mix designed. Various processes of injection, their limitation and potentialities, field problems and how to solve, guide to grouting inspectors for successful operation, quality control and presentation of grouting data in various forms and charts are also explained. Different types of cutoff walls their limitations and potentialities, and cost analysis are illustrated by a practical example. Few examples of the earth dams where grout cutoff walls are provided are also given. In the end a conclusion is drawn that even though grouting process was applied 166 years ago, it has not yet developed sufficiently and still there is a great scope in further investigation to introduce cheap grout materials to make the process more economical and compatible with other methods of cutoff walls. It also requires to draw standard specifications to standardize the process.

"GROUTING OF PERVIOUS SOILS"

CONTENTS

" GROUTING OF PERVIOUS SOILS "

CHAPTER 1

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

1.1. GENERAL

The question of foundation treatment has not yet obtained the attention it deserves in this country as till recently before launching of Five Year Plans the projects actually constructed were so few compared with the size of the country that a solution of the best foundation was usually possible. With the launching of the Five Year Plans a large number of dams building projects covering almost every possible site, the question of foundation treatment gained increasing importance. With the present state of knowledge about the technique and practice of foundation treatment almost any site can be made suitable to build large dams upon. But before this can be confidently and economically done a thorough study of the Geology of the dam site, of the various alternative treatment possible, and the exercise of mature judgement and experience in selecting the most economical and feasible type of treatment and equipment is necessary. In this, a study of the foundation difficulties encountered in some of the successfully executed projects and how they were overcome will be of great value. Hence, before deciding upon any course of treatment, a full knowledge of the engineering geology of the dam site is essential. Vora[1] (1951) rightly remarked that the foundation treatment is "the art of remaking Geology of the dam site". Thus the art of grouting process cannot be said as a Civil Engineering job but it is more or less an art associating civil engineer, geologist, research officer, a mechanical engineer, at different stages of process.

The civil engineer after a thorough study of hydrology and consumptive use of water resources will design the most economical dam section consistent with the type of foundation and type of local materials available. The geologist will interprete the types of rock, types of alluvium deposits and their formations i.e. whether homogeneous or hetrogeneous and their characteristics to bear the load coming upon them safely with or without treatment. After knowing fully the geological condition of the site the civil engineer will decide upon the type of treatment needed; research officer will assist him in designing the grout mix consistent with geological formation and site condition prevailing. In case of rocky and coarse gravel deposits the treatment may vary from coarser grout of cement to finer grout of clay-cement, clay-chemicals or pure chemicals depending upon the nature and extent of the fissures in the rock, grain size of alluvial deposits and the basic requirement of foundation treatment i.e. for consolidation of foundations of gravity dams, or to have an impermeable curtain. In case of alluvial deposits the basic requirement is to have impermeable foundation and to improve the strength of foundation by improving the shear parameters 'c' and 'Ø' to arrest foundation failures. The mechanical engineer plays the role of selecting best equipments of drilling and grouting available indigeneously at a reasonable unit cost of treatment. With the modern practice of grout technique it is possible to inject and seal even finest fissures in the rock and very fine alluvial deposits to have a water-tight foundation but this will be a very expensive, the choice will depend upon the relative cost. We may not need a perfect water-tight foundation for all types of hydraulic structures. In some cases we may accept some leakage to a reasonable degree in the foundation provided that it does not endanger the stability of the structure built upon than to treat the foundation at very high and prohibitive cost compare to the overall cost of the project.

The scope of grouting of pervious soils in civil engineering is very wide. This is applicable to shaft sinking to seal the pores for water-tightness, tunnel driving through alluvial deposits, for stabilizing the soil for road and rail embankments, confine the area by open excavation below ground water table by providing a grout barrier alround the excavation pit and to provide a water-tight curtain below the hydraulic structure to reduce the seepage and resist the erosion of foundation material under differential head of water. Grundy[2] (1955) has classified two types of permeable media through which seepage takes place:

(i) permeable rocks like lime-stone, dolomite, volcanic breccia etc.

(ii) permeable alluvial deposits of coarse gravel, to fine sand and silt, glacial deposits etc.

The grouting process differs widely for both the types of permeable strata. The present dissertation is, therefore, confined on the subject of "grouting of pervious soils" for dam foundations.

1.2. DEVELOPMENT OF GROUTING PROCESS

1.2.1. Cement grouting: The injection or grout process has a fairly long history. Glossop[3] (1960) has traced the history of the injection of grout process from 1802 to 1960 in two articles. Charles Berigny (1772-1842) was the first French Engineer who in 1802 for the first time injected puddle clay and then an Italian pozzolana to repair scouring sluices at Dieppe port underneath of which mortar was washed out. He injected by means of a simple and ingeneous percussion pump of his own invention made of wood of size 8 cm internal diameter with 3 cm. diameter sheet metal nozzle at one end (this was lastly used in 1896 on the Nile Dam). This practice became the standard practice till 1850. In 1841, Collin a well-known soil engineer published his pioneer work on the use of injection for the repairs of hydraulic structures. He usefully sealed the cracks formed in the masonry on the downstream side at Grosbois on the canal de-Bourgogue in 1834. He found that the percussion pump of Berigny did not give good results on cracks of capillary dimensions. He therefore invented a cast iron injection pump with a piston operated by means of a lever.

In 1876 Thomas Howksley (1807-93) first used cement grout to seal water bearing fissures in rock unearth the foundation of a dam at Tunstall and shortly after this in 1882 Revmaux used cement grout to seal water bearing fissures in Colliny shafts. Although portland cement was patented in 1824, majority of injections schemes undertaken before 1876 utilized clay or hydraulic lime. Albert Francois though not an engineer developed new forms of grout pumps and made use of high pressures in rock grouting. This process was first applied systematically to seal fissures and strengthen rock beneath the foundation of dam at the New Croton project in the State of New York in 1893. The treatment was intended to reduce hydrostatic uplift beneath the base of large masonry dam but there was no attempt to form a cutoff. After New Corton the development of foundation treatment for dams progressed steadily in the United States during the first years of the Century and there are records of the use of grouting on 19 dams built between 1900-1930. The first case in which a grouted cutoff was incorporated in the design was at the Estacada dam on Clackamas river 25 miles S.E.of Portland, Oregan. The dam was built on a volcanic breccia which was highly permeable. Three row grout curtain was provided with holes at 6 ft. c/c and rows 6 ft. apart. Outer rows were grouted first and after taking percolation tests the middle row was grouted.

During this period grouting was generally adopted for the treatment of dam foundations in Great Britain, but the methods used were those which had been developed by the mining engineers in shaft sinking. But since about 1920 most of the important advance in the method have been made in France or Switzerland. The development of the systematic grouting in America can be said to have taken place between the year 1932-35, during the design of Hoover Dam. The U.S.B.R. made a close study of the grouting practice current then in order to draw the specifications for the dam. From this,day, the foundation treatment of dams by cement grouting has been generally accepted in the U.S.A. and has been used on the large dams built during the past 35 years on the Tennessee river, Missourie river and elsewhere. The standard practice was to provide consolidation or blanket grouting from shallow drilled holes over the whole area of foundation intended to seal the bed rock from fissures to improve load bearing capacity and to provide a deep curtain to form an impermeable zone. Hays[4] (1953) has recommended grunite blanket on rock foundations as shown in Figure No. 1.1 to act as a roofing for better grout travel. Higher grout pressures can be used to reduce the number of holes. The practice was used at Kenney dam in the British Columbia.

In recent years the term "Grouting" has come to replace the old term "Injection" named by Charles Beingny[21] in 1802 and the method is now being used more and more not only to seal fissures and prevent the flow of water but to strengthen the mass of jointed rock to enable it to carry heavy loads as in cases of the abutments of an arch dam. This is also being widely used in rock mechanics for roof bolting and anchoring.

These developments remain the basis of most of the modern practice in rock grouting, but recent advances in engineering geology and in soil mechanics have widened the scope of the process and made it more effective.

1.2.2. <u>Asphalt Grouting:</u> Cement grouting was generally used for treating fissures in rock for water-tightness, but where rock containing large fissures or caverns in which the flow of ground water is rapid the cement grout will be washed away. Many attempts were made to overcome this difficulty. G.N.Christain and American Engineer was the first to introduce asphalt grouting and used it with some measure of success in 1919-20 on the Halenbar dam, Tennessee. Asphalt was injected through a pipe heated by steam or electricity.

When the asphalt leaves the pipe and touches the water it congeals on the outside into nearly a solid form which is tough enough to resist the pressure of running water, but still soft enough to expand and seal the fissures under moderate pressure. It has been used in the U.S.A. on other sites and also recently this practice has been employed on the several sites in the U.S.S.R.

1.2.3. Alluvium Grouting: Although Berigny and others grouted coarse sand and shingle, the treatment of wide range of alluvial material was the last type of injection process to be perfected. During the first quarter of this century many attempts were made to consolidate beds of sand with cement grout. There were occasional successes but there were many failures and the method could not be relied upon. This was due to the fact that scientific study was not made at that time of the complex structure and of the geotechnical properties of such deposits, nor had the properties of grout been examined qualitatively. Application of cement grouting of sand in shaft sinking process was suggested as a substitute for the slow and relatively costly poetsch freezing process then in use. Portier (1905) pointed out that simple cement grouting had no chance of success in fine sand since the particles of cement were too large to enter the voids and would be filtered out at the point of injection, but inspite of this many attempts were made to grout non-cohesion soils with portland cement, there were successes in coarse grained sediments, and open structure, but failed in the finer grained and dense sediments of low porosity. Reliable methods of alluvium grouting eventually originated from two different sources known as "Two shot" grouting by Hugo Joosten, a Dutch mining engineer in 1925 and the development of modern method of "Single shot" grouting or Gutman's process which started about 1933 and is still in progress. The names associated with the second process are those of Terzaghi, Ischy, Mayer and Rodio. In Joosten's method the injection of concentrated solution of sodium silicate and of a strong electrolytic saline solution of calcium chloride was made to form a gel and seal the voids in the material. Injections were made with a pipe fitted with a point and perforated with small holes for a length of about 2 ft. at its lower end. The pipe was driven into the ground in stages and an injection of sodium silicate was made at each stage. This passed into the sand, driving the ground water ahead of it and sticking to the grains. Having reached the limit of the zone to be treated the pipe was withdrawn in a similar stage and an injection of calcium chloride was made at each stage. This passed through pores already occupied by sodium silicate partly displacing it and reacting with it to form a soft gel which bound the sand grains together at their point of contact making the whole mass homogeneous. Such mass could attain crushing strength of about 400 to 700 psi. The chemical reaction was almost instantaneous. The holes were kept usually at a distance less than 1 meter apart. This method was effective but costly. The complex nature of alluvial deposits led the development of grouting method known as "tube-a-Manchette" by a French Engineer Ischy in 1933 who used this method at Bou Hanifia dam, Algeria. This permits the grout of different properties to be injected into the ground in any order and at any interval of time from the same bore hole. In 1939, A.Mayer, carried out experiments of single fluid silicate grout with a controlled gelling time and a satisfactory mixture was developed using commercial sodium silicate with density of 1.33 diluted by 50% to which was added commercial hydrocloric acid with a density of 1.15 in a ratio of 75 cc/liter and 15 gr. of copper sulphate. Since this mixture was very expensive attempts were made to add clay and eventually it was found that by the addition of suitable additives clay grout could be prepared which was thixotropic and stable.

Maag in 1938 published his theory of injection into a granular material. His theory did not contribute very much to grouting practice, since alluvial deposits are invariably complex in structure. However, it drew attention to the relation between grouting pressure, viscosity and the rate of penetration. Since then the properties of grout were studied from this angle and new and more economical fluids were introduced using mixture of portland cement and clay, with additives to improve their properties and render them thixotropic. Also a number of organic grouts of low viscosity were introduced. The complex structure of alluvial deposits were examined, methods were developed for measuring permeability of soils, the relation between the viscosity and the grain size of the grout, pressure of grouting and the rate of injection were investigated. Experiments were made with different methods of injection and tube-a-Manchette process by Ischy was the most advanced method. The work on the properties of grouts and on the physics of the injection are still being actively pursued.

The first dam on which alluvial grouting was used on large scale was on the construction of dam at Genissiat in 1936. The site was underlain by a buried channel 25 m.deep filled with alluvium. To drive the sheet piles to the full depth was impossible because of presence of boulders to large extent and a grout curtain was used to seal the lower part of the channel. This curtain consisted of 3 rows of grout holes, the outer rows were injected with clay grout and the inner row with a sodium silicate grout. This method gave good results. Work on the project was delayed by 1939-45 war, but when resumed 6 years later it was found that the grouting was till effective. This observation encouraged French Engineers to adopt grouted cutoff as a permanent features in the construction of dams. Further grouting of alluvial deposits at the dam site of Serre-Poncon on the river Durance in 1951 has established grouting as the best solution for the formation of an impermeable cutoff beneath a dam built on alluvial deposits.

1.2.4. <u>Developments in India:</u> The C.B.I[5] (India) has compiled the statistics of dams over 50 ft. height constructed in India in pre-independence period. According to these statistics there are about 118 dams, 41 masonry, 69 earth dam and 8 composite dams. The first masonry dam constructed was Khadakwasla (1870-1879) 130 ft.high on the river Mutha for irrigation and domestic water supply. The dam was founded on hard Deccan trap and no foundation treatment was provided. After the invention of cement grout by Hawksby in 1876 and foundation treatment of the New Croton project in 1893 in the State of New York, Indian Engineers used grouting process for repairing cracks developed in the dams for water-tightness built in the early part of this century. Examples are: Periyar dam (1887-1897) 176 ft. high in Madras State, Kodayar dam (1895-1906) 152 ft. high in Mysore State, Tansa dam (1886-1922) in Bombay State. For the dams built thereafter, the cement grouting was used for foundation treatment of masonry dams to reduce uplift. Examples are Thokarwadi Dam (1916-1922), Radhanagri (1918-1951), Auda Dam (1927-1934) and Thumbrapani Dam (1938-1946).

After Independence a large number of dam building projects covering almost every possible site are undertaken which necessitated a systematic design of their foundation treatment on cost considerations and safety against failures. Thorough study of geological conditions and behaviour of the sub-surface strata were made critically and the degree of treatment required, use of grouting ingradients, and methods of injections were studied. The project laboratories played a key role in designing the mixes consistent with the type of foundation for basic requirements of water-tightness and strengthened foundations, examples are Tungabhadra, Mayurkashi, Bhakra, Hirakud, Rihand, Kotah Barrage, Gandhi Sagar etc.

The masonry weirs and barrages constructed on permeable foundation in North India in the early part of the century showed floor failures by piping due to under-seepage. Study of such failures led the famous Khosla's theory[6] (1934) of founding weir and barrages on permeable foundations. This mainly consists of a solid floor with 3 rows of piles at Upstream, at Downstream and below the crest. Subsequent construction of almost all weirs and barrages on permeable foundations has been undertaken according to the Khosla's theory, and the results are very satisfactory.

The earth dam constructed earlier were founded on rocky foundations with a cutoff in the centre filled with puddle clay or were provided a core wall in the centre in masonry or in lime mortar. Hardly any attempts were made to build earth dam on alluvial deposits. The Panchet hill dam (one of the Damodar Valley Corporations Dams) built in 1955 was provided sheet pile cutoff for impermeabilization.

The first attempt to provide grout curtain in alluvial deposits was made at Kotah barrage in 1958 where the foundation of earth dam portion in the river bed consisted of mixed sand, gravel, cobble and boulder with permeability of about 1.8×10^{-1} cm/sec. The sheet piles could not be driven due to presence of boulders, nor backfilled cutoff could be provided due to high leakage met during excavation. Therefore foundation was treated by clay-cement-chemicals grout for forming a permanent cutoff. Since then research was carried out at the P.W.D. concrete Research Laboratory, Chepauk, Madras State of clay injection at the proposed Ramapada Sagar Dam in the Madras State. Much research work on the clay-cement-chemical grout is carried out by the C.W. & P.R. Station, Poona and individual contribution is made by Mistry (1965), Datya and Vinayaka (1965) and other project laboratories who have carried out intensive laboratory testing on various grout ingradients to evolve optimum grout mix for the Ukai Dam and Girna Dam for the treatment of foundations of alluvial deposits to provide a permanent grouted cutoff.

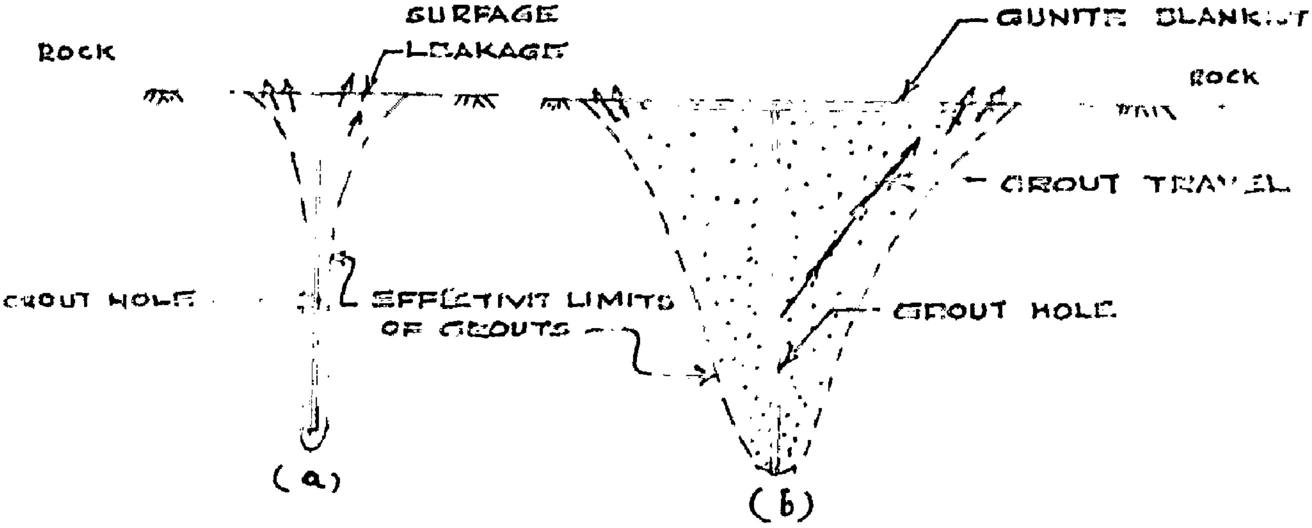


FIG:- 1·1

INFLUENCE OF SURFACE GUNITE BLANKET TO OBTAIN
A WIDER GROUTED ZONE AT CRACKED SURFACES
(a) WITHOUT SEALING ROCK SURFACE.
(b) WITH SURFACE SEALING.

CHAPTER - 2

GEOLOGY OF DAM FOUNDATION

2.1. GENERAL

The geological investigation of the dam site should include determination of kind of bed rock, position of beds, direction and spacing of joints, faults, shear zone, depth of weathering in rock, forms of rock channel, depth and character of filling in it, permeability of material, and ability of rock to withstand hydraulic and dynamic pressures. It is desirable to have the dam foundation rest on solid rock as far as possible. If in the foundation of dam we come across stretches of weak foundation channel filling etc. it is important to determine its nature, whether gravel, sand, clay, boulders or a mixture of these and the relative form and distribution of each kind of material, as it has a bearing on the permeability of the foundation material. This chapter deals with the geological formations and the characteristics of pervious soils of alluvial deposits.

2.2. GEOLOGICAL FORMATION OF ALLUVIAL DEPOSITS

In the strict sense of the word "alluvial" applies only to water borne sediments. However, in grouting practice its use has been extended to cover any uncemented deposits. Such deposits may be formed of rock debries of all sizes, with no cohesion, and are rarely, homogeneous. The formation of any sediments involves atleast three stages: The disintegration of original rock, by weahtering or otherwise; the transport of the debris so formed by wind, water, ice or in any otherway; and finally deposition or accumulation. During transport a certain amount of sorting always occurs, and the material is seggregated according to particle size and particle density. As a result most sediments are built up of superimposed layers, or lenses differing in grain size distribution, and in porosity, some being dense and some having a relative open structure. Thus they exhibit the phenomena of stratification or bedding, which control their principal geotechnique properties. Stratification is not always well developed, thus certain deposits, notably glacial and land slide debris, consistent of block of mass particles of differing sizes, with no trace of sorting or stratification; whereas loess.. and wind blown sand are found in relatively homogeneous deposits. Sediment deposits under water, whether by rivers, in lakes or in the sea, are always stratified and hetrogeneous, with alternate layers of sand and gravel deposits formed by variations in current velocity during deposits. Each such bed will have its own well defined structure, depending upon the distribution of grain size and the degree of compaction within it, and this structure will determine its porosity, permeability and other properties; such structures exist not only in relatively coarse alluvial material but also in fine sand deposits which on casual inspection appear and are often considered to be homogeneous. The presence of stratification in most fine sand is demonstrated by the fact that their permeability in a vertical direction is many times less than that in horizontal direction.

Ischy and Glossop[7] (1962) have grouped these deposits into two main groups viz;

(1) Cohesive soils like clay and silt.

(2) Non-cohesive soils like sand and clay.

2.3. CHARACTERISTICS OF ALLUVIAL DEPOSITS

2.3.1. Terminology: According to the terminology given by the I.S. Specification No. I.S. 1498-1959 the soils are grouped as below:

Clay: Clay is an aggregate of microscopic and sub-microscopic particles, derived from the chemical decomposition and disintegration of rock constituents. It is plastic within a moderate and wide range of water-content.

Silt: A fine grained soil with little or no plasticity.

Sand & Gravel: Cohesionless aggregates of rounded or sub-rounded, angular, sub-angular, flaky or flat fragments of more or less unaltered rock or mineral. Particles from 0.06 mm. upto 2.00 mm. are referred to as sand, those of size greater than 2 mm~~, to 60~~ mm. as gravel and above 60 mm. size as boulders. Indian specifications have given 4 divisions of soils as under:

1. Coarse grained soil: In these soils more than half of the material is smaller than 75 micron. These are divided into 2 sub-divisions:
   a) Gravelly soils: In which more than half of the coarse grains are larger than 4760 microns.
   b) Sandy soil: In which more than half of the coarse grains are smaller than 4760 micons.

2. Fine grained inorganic soil: In these soils more than half of the material is smaller than 75 microns. These are divided into 2 sub-divisions:
   a) Inorganic silt & clay: With low to medium compressibility having a liquid limit between 0-50.

b) Inorganic silt & clay: With high compressibility having a liquid limit above 50.

3. Silt and clays: These soils have high organic contents. These are divided into 2 sub-divisions:

a) Organic silts.

b) Organic clays.

4. Peat: This is a dark, spongy and fibrous soil of vegetable origin.

2.3.2. Properties:

2.3.2.1. Mechanical analysis: The range of particle size fractions as per different standards is given in table No.2.1.

Table No.2.1

Designation of soil fraction.		Soil particle fraction Dia. in mm A.S.T.M. standard.	Int.Soc.of Soil Science standard.	I.S.-1498-1959
Boulders		-	-	60 - 200
Gravel:	Fine			2 - 6
	Medium	> 2	-	6 - 20
	Coarse			20 - 60
Sand:	Fine			.06 - .2
	Medium	.42 - .074	.2 - .02	.20 - .6
	Coarse			.6 - 2
Silt:	Fine			.002 - .006
	Medium	.074 - .005	.02 - .002	.006 - .02
	Coarse			.02 - .06
Clay		< .005	< .002	< .002
Colloids		< .001	-	-

2.3.2.2. Permeability: Different authors have given different range of permeability for various soil types. Indian Standard Specifications have divided the soils in three groups in the range of permeability. Impervious soils with permeability coefficient of .0001 cm/sec. or less, semi-pervious soils with permeability coefficient in the range of .0001 cm/sec. to 0.10 cm/sec. and pervious soil with permeability coefficient greater than 0.1 cm/sec. Terzaghi and Peck[8] (1948) have given the relative values of permeability of soil as given in Table No.2.2.

Table No.2.2

Relative values of soil permeability adopted by Terzaghi and Peck

Degree of permeability.	Range of coeff. of permeability cm/sec.	Soil texture.
High.	10^{-1} and over	Gravel and coarse sand.
Medium.	10^{-1} to 10^{-3}	Sand and fine sand.
Low.	10^{-3} to 10^{-5}	Very fine sand, silty sand, loose silt, loose rock flour.
Very low.	10^{-5} to 10^{-7}	Dense silt, dense loess, clayey silt, clay.
Impervious.	10^{-7} and less.	Homogeneous clay.

2.3.2.3. Plasticity: Jumikies[9] have given the relation of plasticity of soil and soil characteristics as given in table No.2.3.

Table No.2.3

Soil Characteristics

Plasticity index.	L.L.	Soil characteristics.	Soil Type.	Cohesionless.
0	20	Non-plastic.	Sand.	Non-cohesive.
< 7	25	Low plastic.	Silt.	Partly cohesive.
7-17	40	Medium plastic.	Silty clay (clayey soil)	Cohesive.
> 17	70	High plastic.	Clay.	Cohesive.

2.3.2.4. Consistency of clays: Karol[10] has given the consistency of clays as given in table No.2.4.

Table No.2.4

Consistency of clays

Descriptive term.	No.of blows per foot (standard penetration test)	Unconfined compressive strength (t.s.f.)
Very soft.	0 - 2	0 - 0.25
Soft.	2 - 4	0.25 - 0.5
Medium	4 - 8	0.5 - 1.00
Stiff.	8 -15	1.0 - 2.0
Very stiff.	15 -30	2.0 - 4.0
Hard.	30 and over.	4.00 and over.

2.4. EXPLORATION OF SUB SURFACE:

2.4.1. Purpose of Exploration: Whenever grouting is contemplated for pervious soils, very careful exploration is necessary to determine the following:

(1) The grain size distribution of the foundation material so as to decide its groutability with different types of grout materials.

(2) The permeability of the strata in order to decide the extent of zone requiring treatment.

2.4.2. Extent of Exploration: Terzaghi and Peck[8] (1948) state that the extent of exploration of site depends upon the magnitude of the project for which investigations are to be undertaken. For example for small projects it is not necessary to carry out elaborate investigations. Some exploratory borings with a few classifications of sub soil will be quite sufficient compensated by liberal factor of safety in design and construction. For large scale construction projects extensive subsoil investigations are likely to be justified for the safe and economical design of the structure. Clark[11] (1963) regards the expenditure on the investigation as an insurance premium to safeguard against failure due to faulty foundations.

2.4.3. Methods of Soil Explorations:

2.4.3.1. The following methods are generally used for soil exploration:

(i) Boring
(ii) Sampling
(iii) Sounding
(iv) Field permeability tests, and
(v) Geophysical explorations.

2.4.3.2. Exploratory boring: The most common procedure in making sub surface investigations is to drill holes on the chosen site to extract samples for further examination and studies. The information on the quality of the sub surface material may be obtained not only from the samples but also from the observations of the resistence to advancing of the hole. There is no universal rig for making bore holes capable of taking every type of samples in all types of sub surface material. The choice of boring rig is based on the following:

(i) Economics - for small jobs, less costly rigs may be used.
(ii) Geological conditions - i.e. sub surface is soft or hard.
(iii) Type of terrain, accessibility, topography and availability of drilling equipments.

2.4.3.3. <u>Drilling methods and type of equipments:</u> The following boring rigs are generally in use for exploratory drilling:

(1) Auger drills

(2) Wash boring

(3) Rotary drills

(4) The cable drill or percussion drilling.

2.4.3.31. <u>Auger drills:</u> Auger drilling is very fast compared to other methods of drilling. The boring is made by turning the auger into the soil for a short distance, withdrawing the auger and removing the soil which clings to it for examination. The auger is again inserted into the hole and turned further. There are various types of auger and holes can be drilled from 4" to 3 ft. in diameter; upto 20 ft. deep drilling there is no difficulty but beyond that it is usually difficult to withdraw it. Usually tripods and hoists are used when drilling over 10 ft. is contemplated. Casing is sometimes required when drilling is carried out below water table, and through non-cohesive materials. Generally it is difficult to drill through (1) fluid soils such as super saturated sands (2) boulders and hard strata such as firmly cemented sand.

2.4.3.32. <u>Wash boring:</u> This is a cheaper method of drilling holes for explorations. First about 10 ft. deep hole is drilled by auger and casing generally of 2½" pipe diameter is lowered in the hole by derrick and a wash pipe of 1" diameter in length of 5 - 10 ft. is inserted in the casing. A hose pipe connection is made through a swiveal head to the top of the wash pipe and the lower end of the pipe is fitted with a chopping bit, provided with water ports so that the wash water can be pumped down the wash pipe and forced out of the ports. The water rises in annular space between the wash pipe and the casing. It is collected in collecting tanks from where water is recirculated. The hole is advanced by the churning and washing and additional casing is driven as needed. While drilling progresses the column of wash out cutting can be observed. Whenever change in colour of wash out cutting is observed, the wash out is turned off and a sample is taken, otherwise for every 5 ft. of interval irrespective of change in wash out colour one sample is to be taken. The elevation of water table can also be determined.

2.4.3.33. Rotary drilling: In the rotary rig, motor is connected to a drill head which actuates drill rod with a bit at the end. The motor is oil driven or electrical. The drill rod is rotated by means of a gear and the bit is forced into the ground by a hydraulic or screw jack. Thus the bit progresses in the depth and the material acted upon is cut, chipped and ground. The drill rod in rotary drills generally is thick-walled hollow pipe. Different types of bits are used to obtain cores if required for examination.

2.4.3.34. Cable tool rig: In the cable tool rig a "string of tools" is used. This consists of a chopping bit screwed into a stem which in turn is attached to a set of two jaws held by a rope or a cable. The chopping bit may be a heavy bar 4 ft. or more long, working on the bottom of the hole. It acts by either with the string of tool in a hammering or percussion action. The energy required for this action is provided in several ways. The jaws serves as a hammer to drive the sampler after the hole is made or may be used to jerk the bit, loose if it sticks at the bottom of the hole.

In this method the hole usually is fully or partly filled with water. The slurry formed at the bottom of the hole is periodically bailed out by means of a bailor or sand pumps.

2.4.3.35. Choice of size of hole: Krynine and Judd[12] (1957) have given the standard sizes of casings, rods, core barrles and size of holes as given in table No.2.5. According to them, it is advisable to use a large diameter of core as economy will permit. The larger the diameter the more accurate are the geological observations on jointing and fracturing. Also less core loss is apt to occur if fractured rocks are drilled with large diamond bits. Very small diameters such as Ex size are to be avoided when possible. Often a small diameter core will give deceptive information on the extent of fracturing, as many widely spaced fractures may be missed entirely. In extremely deep holes, it may be necessary to resort to Ex size holes because of the weight of the drill rods. For drilling deep holes, the size of the hole commonly is reduced with depth e.g. the hole may start with NX size be reduced to BX at about 50 ft. depth, be further reduced to AX size at about 150 ft. depth and finally be drilled with EX size bits beyond 250 ft. depth. This reduction in size permits easier handling of casing when it must be carried to great depths.

Table No.2.5

Standard size in inches, for casing, core and holes.

Size symbols.	Casing O.D	Approximate dia of core hole.	Approximate dia. of core.
EX	1 13/16	1½	7/8
AX	2 1/4	1 7/8	1 3/16
BX	2 7/8	2 3/8	1 5/8
NX	3½	3	2 1/3

2.4.3.4. Sampling:

2.4.3.41. Samples extracted from the ground are of two types:

(i) Undisturbed, and

(ii) Disturbed.

Undisturbed samples are required in all cases where properties of rock or soil in situ are to be studied. Disturbed samples are obtained to study properties of soil as a constructional material. Samplers are divided into 2 basic groups:

(1) Drive samplers which are forced into the ground by pushing or jacking (static method) or hammering (dynamic method), and

(2) Core sampler. Cores are used mostly in rocks but can be applied to hard cohesion soil type also. These are obtained either by dry core drilling or wet core drilling.

There are various types of drive samplers depending upon the nature and cohesiveness of the strata. The following are commonly used:

(1) Ball valve sampler

(2) Bailor small diameter

(3) Bailor large diameter

(4) Sampling spoon

(5) Open drive sampler and piston sampler

(6) Bishop type sampler.

2.4.3.42. Procedure of sampling: After the hole is drilled by auger or wash water it should be cleaned by spoon or auger and the sampling spoon attached at the lower end of the wash pipe is driven in the soil. The blow counts should also be counted for penetration of 1 ft. in soil. This furnishes a vital information regarding relative deneness of strata with little extra efforts. The description and limitation of some of the samplers is given as under.

2.4.3.43. Ball Valve sampler: It consists of a large tube open at lower end and having sharp cutting edge. At the top there is a ball valve, which lets out air or water when the sampler, penetrates into any strata. Thus the sampler holds the sample by atmospheric pressure and cohesion of the material collected. sufficiently good samples can be obtained in medium sand with some silt i.e. material with at least a trace of cohesion. Natural stratification is not much disturbed thus there is no mixing of layers. But sampling is generally not possible in case of clean sand without any trace of silt i.e. cohesionless sand. Sampling is also not possible in gravel or conglomerate, as penetration of sampler is rather difficult. In general it can be said that samples are reliable whenever they can be obtained in respect of their grain size distribution and stratification as a smooth side tube is driven continuously into the strata.

2.4.3.44. Bailor: Diameter of the sample depends upon the internal diameter of casing pipe in which it works. Bailor is a tube having sharp hardened cutting edge at lower end. Just above the bottom there is a hinged flap which opens inside the tube. When the bailor penetrates into the strata, displaced air and water finds its way out from openings near upper end. Flap at the bottom does not allow the material to fall when bailor is lifted up. Large diameter bailor operated by a winch is driven into strata by the impact of its free fall. Material is pushed inside the bailor and retained by a flap. To get more reliable results, sampling by large diameter bailor should be done only for one ft. depth at a time. Small diameter bailor is not heavy enough to penetrate into the strata by its free fall and may be hydraulically driven or by mannual hammering. In case of samples taken by bailor, there are two possibilities which may lead to erroneous results. The first one is the washing of finer materials and second one is the mixing of layers. Due to washing of fine materials from the sample, an impression is created that sand is coarser than what it actually is. When alluvium contain some silt, it gets washed away during the operation of the bailor. Being very fine its setting time is more and so it remains suspended for long time in the water column. Sample obtained by bailor appears to be clean sand when actually it is not so. Thus bailor fails to record presence of silt in alluvium. This is very serious drawback because presence or absence of only 5% silt considerably influences permeability of the alluvium. To correct this defect to a certain extent, after one lift, time interval of atleast 1 minute should be kept for next bailor operation so that fine silt will settle and will be collected in subsequent operation. In river deposits, fine and coarse layers are found in close proximity. In some cases the layers alternate at intervals of only a few inches. In case of bailor sample all coarse and fine layers are mixed, thus it gives an idea of particle size of mixed sample. The top few inches of the material collected by the bailor consists of loose material falling during the withdrawal of bailor followed by driving of the casing. This completely disturbs sequence of stratified layers.

2.4.3.45. Core drilling: This method is successfully adopted both in case of slightly cohesive soils and hard strata. In case of soft material the core barrel is nearly pressed in the strata to get the sample. Thus there is not even slightest impact as may be in the case of bailor or ball valve sampler. This gives most suitable samples, as there is less disturbance of layers and there is no washing of fines.

If the strata is hard where no other sampling method is workable core drilling is the only method. In case of wet drilling, water under pressure is forced through core barrel and it comes out from the casing. This water circulation assists in drilling and keeps the drill bit cool. Wet method is only suitable for hard and compact strata such as rock or compact conglomerate. In case of compact clay, indurated silt and soft conglomerate this circulation is likely to wash the material. In such cases dry drilling where there is no circulation of water, is adopted. Recoveries can be observed accurately except in non-cohesive soils and slushy material where there is possibility of loss of sample. Observations of recovery are reliable only in case of rock or hard conglomerate. Percentage recovery of samples in case of fissured and disintegrated rock or soft conglomerate is generally poor.

2.4.3.46. Bishop type sampler: This sampler requires bore hole of minimum 8" diameter and the procedure for sampling is very complicated and hence it is not convenient for routine exploration. The sample is ideally suited for non-cohesive fine sand. It is not suitable where hard layers such as conglomerate alternates with loose sand.

2.4.3.47. Limitations and reliability of sampling: The most reliable sample can obviously be obtained from open pit and it is therefore essential to excavate at least one open pit for a job where grouting is contemplated. When practical difficulties arise in excavating a deep open pit allowance must be made for the error in sampling and considerable variations can arise in the estimated cost of grouting and the actual cost. In some cases, it may be possible to confirm reliability of sampling by comparison of the results of samples from open excavation upto a part of the depth with the sample from bore holes provided it is confirmed that the geological formation for the deposits of the soils are not changed at greater depths.

When ball valve sampler or cores can be obtained the results of sampling can be relied upon. On the other hand when bailor is used, the conflicting effects of mixing of soils and washing of fines make it difficult to judge whether the sampling is liable to err on the finer or the coarser side. Large diameter bailor should be used for determining the proportion of material above one inch size. The results of large diameter bailor which are not driven by separate monkeys cannot be relied upon for determining the grain size distribution of material finer than one inch. This is because of the predominent pumping action and the consequent mixing and the washing away of fines. When the natural alluvium is free from silt the error due to washing of fines is not significant and the sampling is liable to err on the finer side. This is, perhaps the reason that grouting in the field has been found easier in many of the major jobs in Europe.

When hard layers such as conglomerate are interspersed with loose sand, large diameter bailor is necessary to break the material. The alternative method core drilling is not effective in recovering the loose sand.

At Aswan High Dam where sub-surface strata consisted primary of non-cohesive sandy gravel of 220 meter deep, it was difficult to obtain undisturbed samples. During exploration undisturbed samples were obtained by freezing process. During execution sampling was carried out by following methods:

(I) Where strata was slightly cohesive, samples were collected by bailor or tube. In the bailor a device similar to the core catcher or the segmented spring in the DensIon sampler U.S.B.R. was used. In the lower partially cemented strata and in grouted zones samples were obtained by rotary drilling.

(II) Boundary of different zones of overburden such as sand, gravel was ascertained by analysing the material from the drilling mud. The affluent was washed over a sieve of 0.1 mm. size and the analysis carried out by drying the material retained. The percentage of fines below 0.1 mm. could not therefore be detected. The strata varies from fine sand of 0.1 mm. size to coarse gravel over 3.5 mm. sizes. having permeability of 10^{-1} cm/sec. for coarser gravel and 5 to 10^{-3} cm/sec. for finer sand.

For Ukai project in Gujarat sampling was done mostly by large diameter bailor of 22 inch diameter size. Characteristics of deposition showed depth of overburden of about 15 meters in river bed and about 30 meters on the right bank. The overburden consists of coarse sand mixed with gravels.

The river bed sand is coarser varying from 0.6 mm. to 2.5 mm. size while shoal sand is medium varying from 0.3 mm. to 1.00 mm size. Grain size curve for Ukai Dam project is given in figure No.2.1. The permeability varies from 2 x 10^{-2} cm/sec. to 10^{-1} cm/sec. For Girna and Mulla Dam projects in Maharashtra sampling was done by bailor and ball valve sampler and also rotary drill. Overburden depth varies from 15 to 20 meters having a permeability in the range of $10^{-1}$to 10^{-2} cm/sec.

2.4.3.43. Soundings: Sub surface soundings are for exploring layers of soil with an erratic structure. They are also used to make sure that the sub soil does not contain exceptionally soft spots located between drill holes and to get information or the relative density of soil with little or no cohesion. Experience shows that erratic soil profiles are far more common than regular ones. For this reason many different procedures have been developed. They are divided into two large groups, static and dynamic. In the static method the sounding rod is pushed into the ground by static pressure. The dynamic methods consist of a driving the rod by the impact of a drop of hammer. According to Terzaghi and Peck[8] (1949) no sounding method is equally suitable under all the soil conditions that may be encountered in the field. This explains why many different methods have come into existence. The method must be chosen in accordance with the type of information called for by the project.

2.4.3.5. Field permeability:

2.4.3.51. Field permeability tests are generally taken before and after grouting to evaluate the efficiency of the grouting. Field permeability measurements fall broadly into the following two categories:

(1) Local or point permeability measurements.

(2) Average permeability measurements.

2.4.3.52. Point permeability measurement: Point permeability measurements are based on measurement of seepage from a pocket of known dimension. For a reliable interpretation the test pocket must be of defined shape and size. The size of the pocket should be small as compared to the depth of the soil mass. In practice none of the above conditions are realised. In a saturated structure when the length of the pocket is less than 1/15th thickness of the structure the formula customarily used can be considered as reasonably accurate. As all natural formations are hetrogeneous and anisotropic the numerical validity of point permeability measurement is open to question. The point permeability measurement can therefore be considered only as indications of relative permeability. More severe errors can arise due to defective procedure. Errors caused by defective procedure consist of the following:

(1) Error due to silting caused by sediment brought by the water used for the test or the sediment thrown into suspension by disturbances.

(2) Leakage along casing pipe.

(3) Choking of the perforations in the pipe used for testing.

(4) Packing of soil near the sides of the bore hole and plastering of the bore holes due to up and down movement of the casing pipe. The plastering effect can be very serious when the soil contains layer of grout or soft clay.

Errors in point permeability measurements can be minimized by using casing pipe with large perforation or perferably slots and avoiding use of driving shoe. Holes should be flushed by surging and thorough water jetting. Alternatively air jetting can be used but this may give rise to leakage along the casing pipe. If test pockets are formed of gravel poured into open holes after withdrawal of the casing pipe a film of sediment is liable to be formed at the top of the gravel pocket. This procedure is open to objection as the sides of the holes can get plastered by up and down movement of the casing pipe. It is therefore desirable to drive a perforated pipe through the gravel pocket. When side collaps does not take place an ideal procedure would be to stop the casing pipe at the top of the test pocket. The test section can then be drilled by a rotary tool or by wash boring.

For reliable comparison it is obligatory that identical procedure is followed for test carried out prior to and after grouting. It is advisable to conduct permeability test in portion when results of the test can be verified by sampling from open excavation. If the comparison is satisfactory the suitability of the procedure would be confirmed. Another method would be to compare results of the point permeability test with the average permeability value determined by suitable test. While comparing results the agreement can be considered satisfactory, if the permeability measured by point permeability method and the average permeability method lies within the same range of index.

2.4.3.53. Average permeability measurements: These measurements are commonly carried out by pumping from a large diameter hole and observing the depressed surface by a system of piezometers installed in the vicinity of the pumping well. For reliable measurements of average permeability steady condition must be attained. Flow can be considered as steady for all practical purposes if the velocity corresponding to rate of fall of the water surface is a small fraction. When marked stratification is observed interpretation of the pumping test is complicated and it will be necessary to observe piezometric level for the different strata individually and the seepage from the different layers would have to be measured by running a velocity traverse through the bore hole. In the interpretation of the test, allowance should be made for infiltration from free bodies of water. It is necessary, therefore, to install piezometers in two mutually perpendicular directions. The principal system of piezometers being on the line of shortest distance between the bore hole and the free body of the water.

The field permeability test (average permeability method) was carried out at Mangla Dam in 1960. As described by Skempton and Cattin[13] (1963) overburden consisted of about 15 meter deep in river bed and about 25 meters deep on river banks mostly of boulders and gravel mixed with sand. Bed rock is of middle Siwalik feebly cemented sand stone. Pumping test in test plot before grouting showed a permeability of 4×10^{-1} cm/sec. and after grouting it was reduced to 5×10^{-5} cm/sec.

Permeability test on Aswan High Dam was carried out by point permeability test by La Franc Method. Pumping out test in bore holes was also done. As this test was carried out in a short pocket the interpretation was similar to the Le franc test. The average permeability for coarse gravel was in the order of 10^{-1} cm/sec. whereas for fine sand it was 5×10^{-3} cm/sec. After grouting the result of pumping tests are not yet available but criteria is lead to achieve efficiency of grout curtain with permeability less than 5×10^{-4} cm/sec. Piezometers are installed on both the sides of the grout curtain to observe the uplift and permeability of the grout curtain.

At Ukai Project in 1961 field permeability tests were carried out to determine the groutability of the sheared rock zone on the right bank between Ch.9650 to Ch.9950. The test is described by Desai[14] (1965). The rock met with is highly sheared and jointed by minerals like lime, clomphate and calcitic vanes. The drilling recovery was usually in a powdered form coming out with wash water and a few pieces of zeolitic bassalt. in the core barrel. The depth of strata extended down to more than 70 meters below ground level. The results indicated range of permeability of $2\text{-}6 \times 10^{-2}$ cm/sec. For economic reasons the alignment off the right bank was shifted towards upstream to avoid continuous sheared rock zone, where only sheared stringers were observed of the same characteristics. The permeability test results after grouting in 1967 show a water loss of less than 1 lugeon which corresponds to permeability of about 10^{-5} cm/sec.

2.4.3.6. <u>Geophysical Exploration</u>: Geophysical exploration is a form of field investigation in which physical measurements normally are made at the ground surface by using special instruments to secure sub surface information. It is a blend of physics and geology because the physical measurements are interpreted in terms of sub-surface, geological conditions. These methods are appropriate for a rapid though approximate solution of certain geotechnical problems in area, such as the depth to bed rock at a dam site.

2.4.4. Logging:

2.4.41. The term logging in its broad sense means recording the earth crust material along a single direction usually along a vertical line starting at the ground surface. Such records are represented in the form of a log sheet. There are two methods of logging:

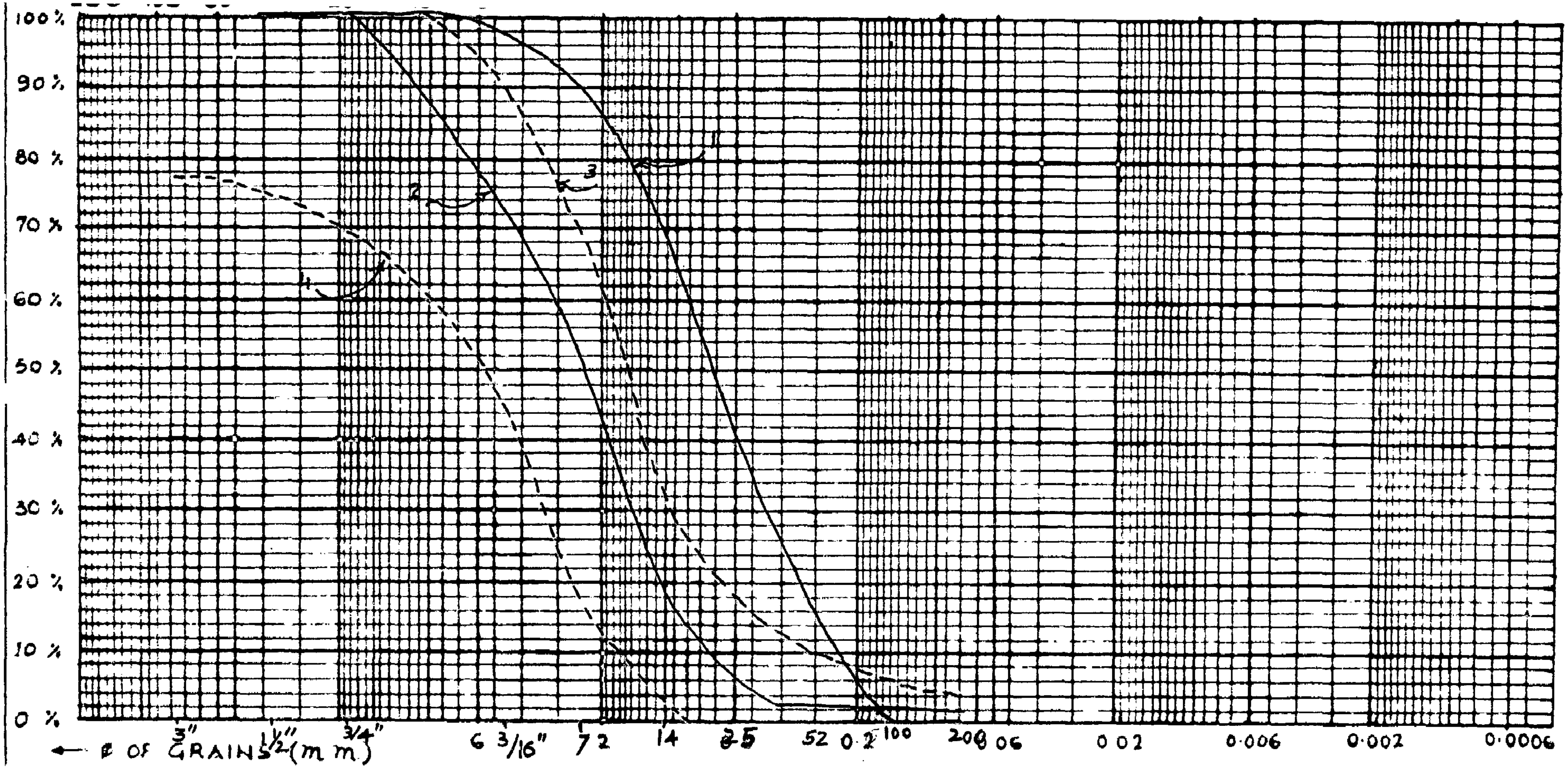
(i) Direct logging such as obtained from boring in soil and rock, and

(ii) Indirect logging such as obtained by electrical and radioactivity.

2.4.4.2. Direct Logging: Logs of bore holes may be made in written form or plotted graphically to a scale. The character of log depends on the materials of the bore hole, on the method of drilling, and finally on the purpose for which the bore hole is being drilled. For example, the logs of shallow bore holes drilled in soft material may be much simpler than the logs of drilling in rock. All field observations should be recorded in the form. Separate log sheets should be maintained for separate holes. The water table, if found should be recorded as actual depth or elevation and the date of such measurement. If no water has been found, this should be clearly stated on the log. A sample form of log sheet is given in Fig. No.2.2.

Recent development by the Corps of Engineers is the bore hole camera. This camera is small enough to be lowered into NX size hole having a diameter of 3". By means of this camera a continuous undistorted cylinderical colour picture of dry or water filled borings can be obtained. Now-a-days, T.V.Cameras are perfected in U.S.A. where full details of bore hole can be obtained by sitting in offices by closed circuit television operation. Bore hole camera offers a great saving in cost and time involved as compared with the method of sinking larger diameter calyx holes. But this method has not found any encouragement in India because of the availability of instruments.

2.4.4.3. <u>Indirect Logging:</u> These are often used in oil industries. These are electrical loggings and radioactive loggings. No samples are taken in either method. In Civil Engineering these methods are rarely used.



SYMB	BORE	SAMPLE	DEPTH	CLASSIFICATION	% BOULD	% GRAV	% SAND	% SILT	% CLAY		ϕ 60%	ϕ 10%	U	REMARKS
		1 & 2	————	GRAINSIZE CURVE OF SHOAL SAND										
		3 & 4	- - - - -	GRAINSIZE CURVE OF RIVER SAND.										

FIG NO 2.1

GRAINSIZE CURVE OF SAND AVAILABLE IN RIVER BED AND SHOALS AT UKAI PROJECT.

PROJECT DESIGNS DIVISION, SURAT

DRG No DD/MIS/22

FIG 2.2 BORE HOLE RECORD

LOCATION PROJECT

CO ORDINATES.... COLLAR ELEVATION.. RIVER:

BEARING OF HOLE. ANGLE WITH HORIZONTAL TOTAL DEPTH

STARTED... COMPLETED

GROUND WATER TABLE ELEVATION....

NOTE.... TYPE OF BIT INDICATED W- WASH BORING H. HAYSTILLITE S SHOT C CUTTER D. DIAMOND

ELEVATION.	FORMATION	LOG	GROUT	CORE		CASING				SIZE OF HOLE	BIT USED	PERCOLATION				REMARKS
					%		2X	1X	AX			INTER-VAL	Lb IN	COLLAR	LOSS G.P.M.	
	5															
	10															
	15															
	20															
	25															
	30															
	35															
	40															
	45															
	50															
	55															
	60															
	65															
	70															
	75															
	80															

CHAPTER - 3.

PLANNING OF GROUTING

3.1. GENERAL

Grouting essentially consists of drilling a number of holes to various depths in the area to be treated and forcing through them, under requisite pressures a workable grout that will enter and fill up the crevices and seams in the neighbouring rock and fill up pores in the alluvium to set rapidly into an impervious mass capable of resisting the stresses imposed. The main points of investigations, therefore, are the choice of grouting materials, the size, depth, location of holes, method of treatment, pressures to be used and the equipment employed. Cement grout in the past was more frequently used but now with the modern concept of grouting other media such as clay, bentonite, asphalt, chemicals resins etc. are also used in grouting in addition to cement.

3.2. SCOPE

With modern technique of grouting process it can almost be used for a number of purposes in construction. Some of the applications of grout are listed below:

i) Foundation treatment of dams and hydraulic structures.

ii) Repairs to dams and hydraulic structures.

iii) Joint grouting for concrete dam.

iv) Contact grouting behind penstock liners.

v) Control of ground water movement for opcn oxcavation for buildings, powerhouses etc.

vi) Repairs to buildings.

vii) Repairs to bridges.

viii) Strengthening machinery foundations against variations.

ix) Repairs to pipe lines and sewers.

x) Stabilization of soil for roads and pavements.

xi) Stabilization of soil for rail and road embankments.

xii) Linking reservoirs and canals for control of seepage.

xiii) Control of ground water movement for shaft sinking and mining.

xiv) Tunnel driving.

3.3. OBJECTIVES

Many grouting techniques are employed for number of purposes of which commonly known are:

(1) Filling of cracks, fissures and cavities in ground formations.

(2) Consolidation and strengthening of the foundations.

(3) Reduction of seepage.

There are many cases where grouting of foundations beneath structures is undertaken to fill voids in the over-burden or in rock caused by defects due to solution or other causes and in which the strength requirements of the grout are so nominal that the necessary strength is easily obtained with one of a number of different types of grouts. The main essence of the operation is to place a large volume of grout under relatively low pressure.

Grouting for consolidation of foundation generally in rock is usually done to assure uniform deformation over the loaded area. When grouting is undertaken to increase the structural strength of the foundation the strength of the grout becomes important. The area required to be consolidated is covered by a pattern of holes. Washing of seams and joints by alternate flushing with air and water is carried out in the hole to remove soft materials from these seams and joints whenever possible. This process is also known as jetting. Hundred percent removal of the clay or sand is impracticable. Grundy[2] (1955) is of the opinion that natural silt and clay in fissures are best left in for curtain grouting to be compacted by the pressure of the injection. When the primary purpose of grouting is to reduce the seepage through soil or rock the ability of the grout to penetrate the formation and when set its resistance to erosion, are important. Grout curtains are usually provided on the upstream of dam axis for water-tightness and to reduce the uplift pressure on the structure. Casagrande[15] (1961) in his first Rankin's lecture delivered at the Institute of Engineers, London, has brought out that a single line of grout curtain alone does not contribute towards the checking of uplift unless followed by a line of drainage holes downstream of grout curtain. In his view 100% cutoff is not possible and even 1% of the opening in the cutoff may reduce the efficiency of grout curtain to only 29 to 30%. But Mayer[16] (1961) in the discussion has refuted Casagrande and brought out that grouting not only improves the consolidation of rock and im permeability by sealing fissures but also affects the reduction of uplift pressure.

Grouting of pervious soils is usually done to arrest or reduce seepage, to strengthen the material against erosion and to increase the load bearing capacity or reduce settlement under existing loads, or both of these functions. Grouting is also done to increase shearing resistance for stability against lateral movement.

Experience of grouting in alluvial soils has shown that highly pervious foundations respond very well to treatment by grouting. If the average permeability exceeds 10^{-1} cm/sec. the grout curtain can be achieved with low cost materials. On the other hand, if the permeability is of the order of 10^{-2} cm/sec. considerable amount of treatment with costly materials may be necessary. For average permeability of 10^{-5} cm/sec. grouting is liable to be ineffective unless very expensive chemicals are used.

In soil mechanics terminology, to strengthen or improve the mechanical properties of a cohesionless soil involves one or both of the following changes in soil properties:

(a) increase the effective angle of internal friction, and

(b) increase the effective cohesion of the grains.

The angle of internal friction can be effected by changing the relative density or porosity by altering the particle size distribution by the introduction of void filling particles with or without adhesive or cohesive properties. The increase in cohesion requires the introduction of a cohesive material. To improve the above mechanical properties of soil Schiffman and Wilson (1958) have proposed two types of grout, first the "void filling" grout and secondly the "adhesive" type of grout.

3.4. DESIRABILITY OF GROUTING

One of the most difficult problem facing the engineer at a given dam site is to decide whether or not it is necessary to grout which is a subject of considerable controversy among experienced engineers. There are no general or defined rules which can be adopted so that grouting must be done for the foundation of dam over a certain height.

The desirability of grouting depends primarily on the height of dam, the permeability of the foundation material and the value of the water which might be lost from the reservoir as a result of controlled seepage through the foundation. It depends to a lesser degree on the zoning of the dam. In cases where the results of the field investigations show that the foundation is very pervious and where the water in the reservoir is very valuable, grouting obviously should be done. In the more general case when a reasonable quantity of water loss from the reservoir due to foundation seepage is not important, and when the results of preliminary exploration indicate that the foundation is not excessively pervious, the internal erosion of foundation material due to high differential water head will have to be safeguarded. To improve the structural strength of foundation, grouting of foundation will have to be generally planned.

3.5. GROUTING MATERIALS

3.5.1. Materials for injection must be capable of mixing with water into a fluid capable of being pumped under pressure, setting without contraction after injection and of resisting erosion during the period of setting. They must have a grain size small enough to penetrate the finest fissures in rock and soil. Grouting materials commonly used are:

(1) Cement

(2) Clay

(3) Bentonite

(4) Admixtures

(5) Chemicals

2.5.2. Cement:

2.5.2.1. The following cements are used in grouting operations:

(i) Portland Cement: It is universally used, generally alone and with admixtures. Indian standard specification No. I. S. 269-1951 gives 3 types of portland cement:

(a) ordinary portland cement.

(b) Rapid hardening portland cement, and

(c) Low heat portland cement.

For most of the works ordinary portland cement is commonly used. Rapid hardening portland cement is used where high early strengths are required. Low heat portland cement has nearly the same physical properties of an ordinary portland cement except it is finer evolves less heat of hydration and the development of strength is slightly slower (see table No. 3.1).

(ii) Slag cement: Blast furnace slag cement has specific surface of 4000 to 6000 sq. cm /gr. blaine and setting time of more than 100 minutes. Sodium hydroxide may be used as a diflocculating agent. This product exhibits a superior sulphate resistance and because of its fineness is often a better material for grouting medium and fine sands. But its cost is prohibitive.

(iii) Resin Gypsum cement: Certain resin-gypsum cements of proprietory make are available for extremely fast or controlled set. These are expensive for normal use and mostly are used in oil well industries. Initial set is achieved in 30 to 90 minutes and compressive strength within 10 to 15 minutes after initial set achieved is of the order of 100 kg/cm^2.

Further advantage is that it has a linear expansion of about 0.3% at time of initial set. The full strength of approximately 250 kg/cm^2 is attained in 3 to 30 hours after initial set.

3.5.2.2. <u>Properties of Cement</u>: The physical properties of portland cement as per I.S. 269-1951 are given in Table No.3.1

Table No. 3.1

Physical properties of Portland cement.

	Ordinary portland cement.	Rapid hardening portland cement.	Low heat portland cement.
<u>Fineness</u>			
(a) Residue on B.S. No.170 sieve.	10%	5%	-
(b) Blaines specific surface.	2250 sq.cm/gr	3250 sq.cm/gr	3200 sq.cm/gr
<u>Setting time</u>			
(a) Initial not less than	30 m	30 m	60 m
(b) Final not more than	600 m	600 m	600 m
<u>Compressive strength</u>			
1 day not less than	-	1600 psi (112.5 kg/cm^2)	-
3 day.	1600 psi (112.5 kg/cm^2)	3500 psi (246 kg/cm^2)	-
7 day.	2500 psi (175.8 kg/cm^2)	-	1600 psi (112.5 kg/cm^2)

The specific gravity of dry cement is 3.15. The characteristics of various physical properties are as below:

(I) <u>Setting time</u>: After addition of water in cement hydration takes place. C3A(tricalcium alluminate - 3CaO Al203) and

C3S(Tricalcium silicate - 3 Cao Sio2) compounds set first and forms a gel. The increase in the C3A compound causes flash set but addition of gypsum delays the formation of calcium alluminate hydrate and thus C3S compound sets first. The C2S (2Cao Sio2) compound stiffens more gradually. The setting time can be altered by addition of additives depending upon the nature of requirement of the job. For accelerating the development of strength as in cold temperature or where early strength is required in running water and for repair works, calcium chloride (CacL2) is added in the cement in proportion of 1 to 2% by weight of cement. The addition of 1% of Cacl2 causes rise in temperature of $11^{o}F$. An excess of Cacl2 may cause flash setting. This is added in solution to the mixing water. Sodium chloride is similar to calcium chloride but is of a lower intensity, effect is also more variable and there is a loss of strength at 7 days. Retarders are used to delay in the setting time. These are sugar, carbohydrates, soluble zink salts, soluble borates, clay, bentonite lignosulphuric acids and their salts etc.

(II) Fineness: Finer cement leads to a stronger reaction with alkali reactive aggregates. It exhibits a higher shrinkage and a greater proneness to cracking. However, fine cement bleeds less than a coarser one. An increase in fineness, increases the amount of gypsum required for retardation as in a finer cement more grains of C3A become available for early hydration. Fineness of cement also improves the workability. Troxell and Davis[17] (1956) have given that the most active part of a cement is the material finer than 10 or 15 microns. D85 of an ordinary portland cement is less than

40 microns. At Boulder Dam screened cement passing 100% through 100 mesh (U.S.) and 95 to 98% through 200 mesh(U.S.) was tried. Normal setting cement was used. Both rapid hardening and slow setting cement were tried but proved failure.

At Aswan High Dam cement used is generally fine of blaine's specific surface of 3000 sq.cm/gr.

King and Bush[18] (1961) have recommended use of the fine cement for grouting of medium and fine sand, Datye[19] (1963) has also advocated to use preferably fine cement exceeding specific area of 3500 sq.cm/gr. Blaine and setting time exceeding 100 minutes. Mistry and Purohit[20] (1964) have also preferred use of finer cement with specific surface of 3250 sq.cm/gr. Blaine. This cement is available in India from M/S Sone Valley Portland Cement Compnay, Japla(Bihar).

(III) Bleeding: Bleeding is a water gain in a static condition, in a form of seggregation in which some of the water in the mix tends to rise to the surface. This is caused by the inability of the solid constituents of the mix to hold all of the mixing water when they settle downwards. It is a special case of sedimentation. Bleeding can be expressed quantitatively and can be determined. Tendency to bleed depends on the properties of cement. Bleeding is decreased by increasing the fineness of the cement and is also affected by certain chemical factors, there is less bleeding when the cement has a high Alkali content, a high C_3A compound or when calcium chloride ($CaCl_2$) is added. A high temperature within the normal range increases the rate of bleeding, but the total bleeding capacity is not affected. A reduction in the bleeding is also obtained by addition of pozzolans and allumінium powder. King and Bush[18](1961) have worked out the percentage bleeding of neat cement grout with different water cement ratios after 4 hours mixing. The results are given in Table No.3.2. The bleeding in the grout mix is kept minimum to avoid voids in the cracks or poxes when the grout sets in.

Table No.3.2

Bleeding of neat cement grout

W/C ratio by weight.	percentage of bleeding.
0.4	0
0.55	2
0.65	10
0.75	12.5
0.90	20.0
1.0	25.0
1.15	30.0
1.5	40.0
1.85	50.0
2.25	60.0
2.70	70.0
3.12	80.0
3.60	90.0
4.0 (extrapolated)	(99)

3.5.3. Clay: Clays occur in natural deposits. From the grout engineers point of view there is no bad clay, simply some are better than others and it is a matter of economics to select clay which will fulfill his requirements. Broadly clays that can be considered as a basic material for grouting purpose are divided into two main groups viz; natural clays and prepared clays. Natural clays are those which occur in natural deposits and require little or no treatment before being used as a grout base. Prepared clays are those obtained from natural clays after some treatment to condition them to permit the necessary characteristics for employment as a base material. Clay deposits of massive and alluvial class are better than glacial deposits which tend to contain high proportion of sand and silt and are more erratic in structure. The properties of clay vary from place to place and that each one should be individually investigated for suitability for a particular job. As a first guide in selection, particle size distribution and liquid limit test provide useful means of eliminating the poorer samples. In most clays all the particles under 2 microns consist of clay minerals of type montmorillonite, illite and keolinite groups which are the product of weathering of the unstable constituents of parent rocks. High conteat of these minerals are advantageous. Montmorillonite have the capacity to swell and take water molecules directly into their space lattice. The following properties of clay should be investigated for its suitability as a grout material:

(1) <u>Mechanical properties</u>: The particle size distribution of natural clay is determined by a standard sieve analysis and by hydrometer tests. The natural clay deposits contain silt and sand in varying proportion. Clay when used as a

grout base should as far as possible be very fine and coarser fraction of sand and silt should be removed by treatment. Clay content should be 50 to 60% of size less than 2 microns and clay and silt should form 98 to 99% of the total volume. Normally a clay with liquid limit of less than 60% may not be considered unless the removal of the coarser fraction can be accomplished economically which improves the liquid limit.

(2) Chemical properties: Clays are divided into the following groups:

Keolinite

Montmorillonite

Illite

Of these first two groups are the most commonly found whereas 3rd group is not too well known. The Keolinite clay is characterized by sillica allumina ratio varying from 1.5 to 3, and by a compact and rigid structure. The montmorillonite clay has a silica allumina ratio varying from 3 to 5, and is characterized by a loose structure. Because of their different type of structures and composition the physical and chemical properties of these two groups are quite opposite.

The keolinites take very little water and have a low base exchange capacity. On the other hand, montmorillonite takes more water and is more active. Activity of the clay is a function of surface area of the particles in a given mass of solid. According to Grim[21] (1962) highly active clays would have relatively higher water holding capacity, high thixotropy, low permeability and low resistance to shear, cohesion largely responsible for strength.

At Kotah barrage black cotton soil available locally was used after treatment in settling tanks.

At Ukai project clay was imported from a distance of about 60 km. available near village Tadkeshwar. This contains about 50 to 60% clay, 35 to 40% silt and 10 to 15% sand, liquid limit of about 80% and plasticity index of 43%. This clay was used after treatment in hydrycyclone to eliminate sand particles. The composition of treated clay ranged between 98 to 99% of clay and fine silt.

At Mangla Dam, local clay was used with clay content of 30% of size less than 2 microns and liquid limit of 50%.

At Girna Dam local black cotton soil predominantly having montomorillonite clay mineral was used. Clay was treated in hydrocyclone to eliminate sand fraction. Final clay contents was 60% and fine sand fraction was under 5% with liquid limit of 70 to 80%, and plasticity index 40-50%.

3.5.4. Bentonites: Bentonite, a geological rock formation is one of the well known ultra fine clay, mainly composed of the montmorillonite group of clay minerals. The colour varies from white to light green or light blue. When a dried bentonite is immersed in water, it increases its volume or swells. Swelling of montmorillonite is due to the expansion of the lattice as the water molecules enter between its unit layer. There are two types of bentonite:

(i) Sodium based montmorillonite and

(ii) Calcium based montmorillonite.

Sodium based montømorillite exhibits high swelling colloidal and thixotropic properties than calcium based montomorillonite. But the study of replacing powers of the common ions reveals that in general the replacement of Na^{+} by Ca^{++} is much easier and quicker than that of Ca^{++} by Na^{+}. The rate of cation exchange reaction in montømorillonite is slower with respect to that in Keolinite in which it is almost instantaneous. In montømorillonite it may vary from a few minutes to about an hour, depending upon a number of factors. Grim[21] (1962) has given that the activity of sodium montømorillonite bentonite is about 7.2. whereas for calcium montømorillonite bentonite it is only 1.5. Krynine and Judd[12] (1957) have given free swelling capacity of Na montmorillonite bentonite between 1400 to 2000% and permeability of 10^{-7} cm/sec. to fully impermeable and for Ca montømorillonite bentonite between 50 to 150% and permeability of greater than 2×10^{-7} cm/sec. Mukherjee[22] (1965) has carried out laboratory tests on inactive montmorillonite bentonite available in Bihar. Their inactivity is due to base of Ca and Mg ions in exchangeable positions. The physical properties of Bihar bentonites are clay 52.0%, silt 11.0%, sand 36.0%, L.L. 92.0%, P.I. 38.0% and permeability of 3.5 to 10^{-5} cm/sec. Physico chemical properties indicated that the bentonite in its natural state is of low grade quality and unsuitable for soil stabilization. Number of reagents were tried for sedimentation of bentonite and it was found that with 1% standard soap solution the finer particles remain in suspension for longer periods and the concentration of suspended particles is much higher. Calcium chloride ($CaCl_2$) was added for stability. The laboratory results indicated that the activated bentonite has nearly the same properties of commercial bentonite.

The physical properties of commercial bentonite show clay content (less than 2 microns) 70 to 80%, silt (between 2 to 20 microns) of 10 to 20% and fine sand 5 to 10% with liquid limit between 200 to 500% specific gravity of about 2.35, blaine fineness of about 4000 to 5000 sq. cm/gr. Like a clay it will not develop a stable gel without the assistance of additives such as cement, silicates or other suitable chemicals. It is also used as a lubricating agent to improve the penetrability and workability of grout but delays its setting time.

Since its colloidal properties are far superior to other clays, it is commercially advantageous to prepare and market bentonite in a similar manner to cements. In this way the products can be graded for intended uses and the quality controlled and maintained within close limits than is usually possible with natural clays. Bentonite ores are usually mined as a soft rock, crushed, dried and grinded to a fine powder.

3.5.5. <u>Admixtures:</u>

3.5.5.1. Admixtures are of the following principal categories:

1. Relatively water insoluble, finely divided granular materials:
   a) Inert
   b) Chemically active.

2. Water-soluble compounds:

   a) Water-reducing agents

   b) Retarders and accelerators.

3. Protective colloids.

4. Gas producing agents.

3.5.5.2. Finely divided granular materials:

a) Inert: A wide variety of finely divided granular materials are available as admixtures to grouts. These are employed in relatively large quantities, upto 50% by weight of cementing materials. These are bentonite and other clays, silt, rock flour, lime stone dust, and silica gels. The setting times of grouts are affected by such materials, the rate of setting generally being retarded. The use of bentonite clay or silica gels, which swell upon absorption of water, serves to stiffen the mix without true setting, and can result in various phenomena such as thixotropy. When added in finely divided form it keeps the solids in suspension in grouts of high water content and produce grouts of high water retentivity. The advantage of such finer materials is that they can fill cracks too fine to be filled with normal neat cement grout.

b) Chemically active: The chemically active granular materials generally employed in relatively large quantities and in finely divided form include pozzolans, blast furnace slag and natural cement. The pozzolans in themselves possess no cementitious value whereas natural cement and water quenched blast furnace slags are cementitious. A pozzolan is a finely divided siliceous material which, when used as an admixture in concrete or grouts, chemically react with calcium hydroxide to produce cementitious compounds. The pozzolanic reaction proceeds at a slower rate than normal cement hydration, but continues for a much longer period so long as moisture, pozzolan, and calcium hydroxide are present. Pozzolans may be employed in their natural or raw state, or after calcination to a desired "activation" temperature. The effect of calcination are:

(1) To reduce the rate of reaction of pozzolan with lime, and

(2) To reduce the mixing water requirement.

An addition of pozzolans in a cement based grout reduces its bleeding and seggregation. It also resists the erosion of grout to the percolating action of low PH waters and sulphate water. The amount of pozzolans employed in a grout mixture be as high as 50%, by weight of the cementing material, depending on the nature and fineness of the pozzolan, the richness of the mix and the desired properties of fresh and hardened grout. For a given pozzolan the optimum amount can only be determined by experiments.

3.5.5.3. Water soluble compounds: This group of admixtures are chemical agents and are employed in relatively minute quantities, from 0.01 to 2% by weight of portland cement or when pozzolans are used by weight of total materials. This group includes surface active agents (water reducing, air entraining, and dispersing agents), retarders and accelerators. Such agents may be employed either alone or in combination in the grout mix

(1) Water reducing agents: Surface active agents, when employed in grouts, have a capacity to improve the fluidity

in grouts and reduces water content of the mix for a given consistency. Water reducing compounds widely utilized in grouts, mortars and concrete are of two basic categories:

(1) Calcium, sodium or ammonium salts of lignosulphonates.

(2) Carbohydrates.

When incorporated in cementitious mixtures, these compounds exert side effects other than mere increase in fluidity and workability or reduction in water requirement at given consistency. In some cases these side effects are desirable, in other cases undesirable. For example, many water reducing agents act as retarders, that is, the rate of stiffening is decreased, and the setting and hardening functions are delayed. In warm weather this retardation may be a desirable feature, but at low temperatures, it may be objectionable. An additional property claimed is that of dispersion, whereby the solid particles in the mixture are separated from each other, but causes objectionable bleeding. For correction of this condition may require other admixtures. Nevertheless when these agents are employed in cement based grouts of given consistency the unit water content is substantially reduced which increases strength, water tightness, and durability.

(2) Retarders or accelerators: The use of retarders decreases the water cement ratio which will reduce the bleeding and improve the strength and water-tightness. This is advantageous in hot weather. In cold weather, accelerators, chiefly calcium chloride, are used.

3.5.5.4. Protective colloids: Protective colloids, thickeners or chemical suspending agents, when used in very small amounts, are effective in reducing bleeding and seggregation, and can even replace natural materials such as clay or bentonite to produce very lean and economical mixes. They have the advantage of being quite predictable in their performance compared to locally accuring clays have no real grain size, and appear to give a lubricating effect to the grout that enables it to penetrate fine voids much better.

3.5.5.5. Gas Producing agent: When finely divided aluminium powder is added in the cement, minute hydrogen gas bubbles are generated by the reaction of the alkalies in the cement, which causes expansion of the grout throughout the mass. The presence of the many small bubbles gives a certain cohesiveness to the mix and helps reduce bleeding, while expansion during the plastic stage maintains contact and improves the bond of the grout with the walls and particularly the ceilings of the fissures into which it is forced. Aluminium powder is added in the order of 0.01% by weight of cement which amounts to a few grams and difficult to batch. Therefore more reliable procedure is to add the powder with a chemically inert material, the combination of which can be batched.

3.5.5.6. Lubricants or Primars: The use of lubricating chemical grout to prime the hole before the injection of neat cement grout has been a fairly recent developments and has proven most effective in the field for the grouting of fine sands and rock fissures. Under this system a batch or two of chrome-lignin or silicate-aluminate chemical grout is first pumped down the hole, followed by a high water content cement grout. It has been found that this considerably reduces pumping pressures for the cement grout and assures much better take.

3.5.5.7. Practical Limitations on use of admixtures in grouts

Admixtures of acceptable quality, properly used, may in some cases effect significant economics, and in other cases may make possible construction otherwise difficult or impractical. Recent developments indicate that admixtures intelligently used contributed to desirable properties of cement grouts. Inasmuch as admixtures contribute significant effects, and since their effects may vary greatly with the ehemical and physical properties of other ingredients of the grout mixture they should not be used indiscriminately. Rather, careful consideration should be given to the design of grout mix containing admixtures, and tests to determine their effect should be conducted prior to their use in major construction projects.

3.5.6. Chemicals:

3.5.61. Chemicals used in a grout as a base materials are of two types:

1) Inorganic chemicals
2) Organic chemicals

Inorganic chemicals in common use are:

i) Sodium silicate (Na_2Sio_3)
ii) Lignosulphite (or ligno-sulphonate).

Organic chemicals are synthetic resins. Products commonly used are AM.9 of the American Cyanamid Co's patent which is a mixture of acrylamide and one of its methyl derivates and 2nd product is of soletanche patent process which is a mixture of phenol and formaldehyde. There are other chemicals but these are mostly in the laboratory stage. The properties of chemicals used as a grout base materials are as below.

3.5.62. Sodium Silicate: Sodium silicate produced commercially is available in dry lumps and is available in various grades. The Alkali silica ratio for products may range from 1.6 to 4.0 and the specific gravity from 20° to 50°. Be (Baume). The silica in the soluble sodium silicate may either be in crystalloidal or as a colloidal silicate, the latter being in micelle form. The amount of silica presents as ionic micelle increases rapidly as the ratio exceeds 3. It is probable that the colloidal silicate is the one that produces better polymerization and stronger gel. The silicate ion affects alkalinity (PH) but the micelle has a greater influence. For ratio of 3 and high, the PH reaches a maximum at high concentrations. It should be noted that, by dilution, PH is effected by a standard silicate. A disturbing factor in the true appreciation of PH is the influence of NaOH, sometimes used to assist dissolution of sodium silicate after crushing and steaming. The excess NaoH even if it is ½% can indicate a high PH in the original solution and still influence the PH of diluted silicate. The tests carried at Girna and Mula dams confirm that the primary need is to ensure production of good sodium silicate of ratio of NaO: Sio2 over 3.3 having the necessary structural colloidal silica in ionic micelle form. It should be noted that variation in the gelling and strength development depends upon the correct chemical and structural composition of the silicate, though to a lesser extent the precipitant should also satisfy some minimum requirements. The present standard specifications viz; Indian Standard, A.S.T.M. etc. do not specify the grade of silicate as a grouting agent. The practice followed in France and Switzerland specify the grade of sodium silicate for grouting purpose with silica ratio of 1:3.3 ± 0.1 (38^{o}Be). Another positive index of the manufacture of the proper grade of silicate has been a definite trend in the relation between the gel time and gel strength. For quality control it is therefore necessary to stipulate a minimum gel stength with a standard reagent as well as the desired range of silica ratio.

Sodium silicate used at Ukai Project as a grouting agent was specified as below:

Appearance:	Colourless, clear and not turbid or with glassy incrustation.
Density:	Between 37^{o} and 42^{o} Be. SP:Gr 1.36 to 1.37
Alkalinity ratio:	$\frac{Na_2O}{SiO_2}$ between 2 to 4

Cao + Mgo3 to be less than 2%.

Sodium silicate at concentration less than 1% is an agent of deflocculation whereas at concentration greater than 1% is an agent of flocculation.

3.5.63. <u>Ligno-Sulphite</u>: It is a residual product formed during the production of cellulose from wood pulp by the bisulphite process. The residue is concentrated and is produced in the form of a syrupy liquid or powder. Soluble ligno sulphite mixed with a bicromate become firm gelatinous masses.

3.5.64. AM-9: AM-9 chemical grout is a patent of the American cyanamid Co, it is a mixture of two organic monomers. Acrylamide and its Methyl derivates.

Appearance:	White powder, friable lumps.
Bulk density:	35 lbs.per cu.ft.(approximately)
Stability:	Excellent when stored dry in original containers at temperatures below 85^{o}F.
Solubility:	Very soluble in water, ethanol, methanol, Insoluble in gasoline, kerosene and oils. AM-9 dissolves endothermically in water, that is the solution gets colder.

There are many catalyst and mixtures of catalysts which may be used to gel AM-9. For normal use, however, the catalyst, system is composed of catalyst DMAPN, AP(ammonium per sulphate) and KFe (Ptasium ferri cyanide).

DMAPN - This is a liquid somewhat caustic, chemical used as an activator for the reaction. The density of catalyst DMAPH, between 32^{o}F and 104^{o}F is about 0.96 gram/c.c. It should be used very carefully.

Ammonium persulphate (AP): Ammonium persulphate is a granular material and a very strong oxiding agent. It is the initiator that triggers the reaction and is therefore the last material to be added. Gel formation begins with its addition. Generally it dissolves in water and added as a 5 to 20% solution to the AM-9 solution through a separate pump or by gravity.

Potasium Ferricyanide (KFe) - Potassium Ferricyanide is a nontoxic, reddish granular material which is used to control the reaction. It behaves as an inhibiter in very small quantities and must be used cautiously.

3.5.65. Stabilizing agents: The following electrolytes are commonly used to stablize clay grouts:

Potassium nitrate	KNo_3
Potassium carbonate	K_2Co_3
Sodium Aluminate	$NaAlO_2$
Sodium silicate	Na_2Sio_3
Lithium carbonates.	$Li_2\ Co_3$
Sodium hydroxide.	NaoH

Quantities used are very small and are expressed either in grams per liter of grout (average 1 to 10 g/lit.) or preferably as a percentage of the clay by weight (0.25 to 5%).

Electrolytes are not the only material used to stabilize clay grouts, but bentonite is also used. In very small concentration it makes extremely stable and thixotropic grouts which are ideal for sand foundation grouting. It is added in a small quantity (1 to 10% of the clay weight), it gives stability, rigidity and thixotropy. Bentonite is very expensive, and may be used economically.

3.5.66. Flocculating agents:- Electrolytes used as flocculating agents are usually salts of the sulphuric and hydro chloric acids. The most commonly used are:

Aluminium sulphate	$AL_2\ (SO_4)_3$
Sodium sulphate	$Na_2\ So_4$
Calcium chloride	$CaCL_2$
Copper sulphate	$CuSo_4$
Ferrous sulphate	$FeSO_4$

Flocculents are used in even smaller quantities than stabilizers, proportions are expressed in grams per litre of grout or in percentage of the clay weight. The action of flocculent depends on the stabilizer used in the suspension. Therefore, each flocculent should be tested with different stabilized grouts. This may lead, in practice to a considerable amount of testing, which very often can not be afforded. When sodium silicate is the stabilizer, HCL, NaAto2 and other chemical grouting reagents are used.

3.5.67. Deflocculating agents: The clay ingredients used in the clay based grouts may not be completely dispersed even under the action of a high speed mixer. The minute clods that may remain unbroken act as individual particles of greater size and impair the fluidity of the grout mix. Suitable deflocculating agents (D.A.) may therefore be used to remedy the above. The following chemicals are generally used:

Sodium carbonate	Na2 Co3
Sodium Hexameta Phosphate	
Sodium hydroxide	NaOH
Sodium silicate	Na2Sio3
Sodium Polyphosphate	$NaP4^{O}3$

These are added on percentage basis, reckoned with respect to the combined weight of clay and bentonite and are added either in the form of solution of known concentration or in the form of powder. Test results at Ukai indicate that addition of solution of known concentration was found suitable.

3.5.68. Precipitating agents: When an electrolyte is added to colloidal solution containing charged particles, the colloid is coagulated or precipitate. Colloidal solutions are often protected from precipitation by electrolytes by adding small amount of lyophibic colloids such as gelatin which are hence called protective colloids. The precipitating power of an electrolyte depends on the ion of charge opposite to that on the colloid particles. The precipitating power increases rapidly with the electric charge of the ion.

Precipitating agents are used with sodium silicate grouts to cause gel formation. The number of reactive chemicals are very many and the results can be very different with different precipitants.

Barbedette and Sabarly[23] (1958) have carried out systematic study in laboratories on treatment with silicates and have worked out that the following chemicals can be utilized for gel formation:

(1) Acids - common acids such as hydrochloric acid (HCL) sulphuric acid (H2SO4) Phosphoric acid etc.

(2) Acid salts, such as monosodium phosphates, bicarbonates of soda etc.

(3) Salts of acidic reaction such as, sulphate of alluminum.

(4) Neutral salts such as, sodium chloride, all the salts of ammonia the mtallic salts etc.

(5) Alkaline salts such as sodium aluminate.

(6) Certain bases such as, ammonia.

The number of chemicals causing gel formation is practically unlimited, but different considerations limit the choice for their practical use. These considerations are:

(1) Quality of gel

(2) Facility of use, and

(3) Economy.

The precipitates commonly used are:

Sodium aluminate	Na ALo2
Ammonium sulphate (fertilizer variety)	$(NH4)_2So4$
Sulphuric acid	H2 So4
Sodium bicarbonate	NaHCo3
Monosodic phosphate	Na H2 Po3

With sulphuric acid gel has higher strength than those with ammonium sulphate but the mixture becomes more viscous.

Mono-sodic phosphate acts as a gelling agent for sodium silicate and deflocculating agent for clay.

3.5.7. Water: The quality of water also plays its role, impurities in water may interfere with the setting of cement or form a chemical action when chemicals are added in grout mixes. In many specifications a clause is covered that water would be po~~r~~table such water may rarely contain dissolved solids in excess of 2000 ppm (parts per million) as a rule less than 1000 ppm. Brackish water contains chlorides and sulphates when chloride does not exceed 500 ppm. and So3 does not exceed 1000 ppm. the water is harmless. Sea water has a total salinity of about 3.5% (78% of dissolved being NaCI2 and 15% Mg CL2 and Mg So4) and produce slightly higher early strength but a lower long term strength. The loss of strength is usually 15% but not more, but it slightly accelerates the setting time.

CHAPTER - 4

TYPES OF GROUTS

4.1. GENERAL

Grout suspensions are mixtures of water and one or more grout material at a time depending upon the nature of the formation to be treated. Grouts can conveniently be divided into two main classes such as:

(1) Newtonian, and

(2) Non-newtonian.

Grouts which follow according to the newtonian law of motion i.e. rate of shear is proportional to the tangential stress applied, are called newtonian liquids. The chemical grouts and organic resin grouts fall in this class and are termed as fine grouts. The grouts which do not follow the newtonian law of motion are called non-newtonian liquids. Grouts formed of solid materials such as cement, clay, sand etc. are non-newtonian liquids and are termed as coarse grouts.

This chapter deals with the important properties of the grout suspension and various types of grouts which can be successfully used for the treatment of pervious soils.

4.2. PROPERTIES OF GROUTS:

4.2.1. The most important properties of the fresh grouts are:

(1) Viscosity

(2) Consistency

(3) Fluidity

(4) Stability

(5) Gelling

(6) Permanence.

4.2.1.1. <u>Viscosity:</u> Viscosity of a substance is the tangential force per unit area of either of two horizontal planes at unit distance apart, one of which is fixed, while the other moves with unit velocity; the space being filled with the substance. Viscosity is expressed in dyne per sq. centimeter dynes/cm^2 or poises. The centipoise (cp) is equal to 0.01 poise. 1 lbs/sq.in = 69,000 dynes/cm^2 and 1 gr/cm^2 = 981 poise. When a solid is subjected to a shearing force, the solid deforms, the internal stresses develop until a condition of static equilibrium is reached. Within the elastic limit of a substance, these internal stresses are proportional to the induced shearing strains (deformation), the ability of a material to reach a static equilibrium rather than deform continuously, is due to a property called shear strength. Fluids do not possess shear strength. Fluids do offer resistance to deformation, due to internal molecular friction. However, under the influence of a shearing force, deformation will continue indefinitely. The property called viscosity is actually a measure of the internal friction mobilized against shearing forces.

Viscosities of fluids are generally not measured directly, but a parameter depending upon viscosity is measured, and a predetermined relationship used to arrive at on actual value. The viscosity of a fluid suspension depends upon the solid concentration. The viscosity of water is near to 1 centipoise. Viscosity of some of the grout suspension obtained by Stormer rotational viscometer is shown in ~~the~~ Figure No.4.1. The ease with which a formation can be grouted is directly dependent on solution viscosity. Maag was the first who in 1938 worked out the relationship between grouting pressure, viscosity and the rate of penetration. The phenomena of flow of fluid through alluvial deposits may be compared with the flow of fluid through a long tube of small diameter which is governed by Poiseuille's law and is expressed as:

$$V = \frac{\Pi P r^4 t}{8 L n}$$

where,

V is the volume of liquid escaping in time t

t is the time for V

P is the pressure differential between tube ends

r is the tube diameter

L is the length of tube

n is the viscosity.

Karol[24](1963) has given the laboratory results of flow of a fluid through a long pipe one meter long and $1\frac{1}{4}$ mm. internal diameter as shown in Figure No.4.2. Extending these data to field use, the following conclusions can be drawn:

(1) For any given pressure, the rate at which a formation will accept a grout will vary inversely with the grout viscosity.

(2) Acceptance rate and pumping pressures are directly proportional.

(3) Acceptance rate is directly proportional to the fourth power of the average void size(and therefore for the average grain size, for granular deposits).

None of these conditions may exist in an actual grouting operation. The conclusion, therefore, should be used quantitatively and not qualitatively. Viscosity of a grout suspension also increases with time except for resin grouts where there is an instantaneous gellification at the end of induction time, this is shown in Figure No.4.3.

4.2.1.2. <u>Consistency:</u> The consistency of a grout mixture is related to the viscosity coefficient. The consistency of a grout suspension is influenced by the specific gravity of the grout mix and seggregation.

In a true newtonian liquid having a laminar flow, the velocity gradient is proportional to the effective pressure. The flow property of such a liquid is expressed by a single constant, the coefficient of viscosity. The consistency curve for newtonian liquid is a straight line passing through the origin and is shown in Fig. No.4.4. However, grouts are not ideal fluids and Bingham has shown that some liquids, and particularly suspensions, behave differently and do not start flowing if the effective pressure do not exceed a certain value defined as the yield value. The liquids that exhibit this property are infact plastic substances, and are known as Bingham bodies. The consistency curve for Bingham bodies is shown in Fig.No.4.5. It is evident that the flow curve for a newtonian liquid can be defined by a single measurements of viscosity, but that for a Bingham body a number of measurements must be made for various rates of shear. This is conveniently done by means of measurements by viscometers. A typical consistency curve of a plastic material or Bingham body plotted by Leonard and Dempsy[28] (1963) is shown in Fig.No.4.6. The main features of this curve is the rapid rise of stress with rate of shear until a yield point is reached after which the relation is more or less lineal. The characteristics of the consistency curve given in Fig.No.4.6 are explained below:

(1) Bingham Yield value:- The Bingham yield point is defined as the minimum stress that must be applied to a plastic material to start laminar flow. It is derived from the consistency curve by producing back to the axis the tangent to the plastic range of the curve. Ischy and Glossop[7] (1962) have shown that upon the yield value depends the minimum pumping pressure necessary to inject a grout into a soil for a particular pattern of pore size and radius of penetration.

(2) Apparent Viscosity:- Apparent viscosity is defined as the viscosity a Bingham body would have at a given rate of shear if it was a newtonian fluid i.e. exhibiting the characteristics of a pure liquid. For practical purposes it is obtained by dividing the shear stress by the unit rate of shear.

(3) Plastic viscosity: This is defined as the shearing stress in excess of the yield value that will induce a unit rate of shear.

(4) Thixotropy: A property of some but not all, Bingham bodies is that of thixotropy. Such a liquid if left undisturbed will show a marked increase in shear strength, which, will at once be lost if the liquid is stirred or agitated. But is regained if it is again allowed to stand. Thixotropic properties are possessed by many cohesive soils, and is more pronounced with the presence of fine grains. Thixotropy is of prime importance, since it enables the grout upon reaching

its destination in the ground, to stabilize itself rapidly while the permanent effect of rigidification due to other ingredients become effective, if present. The existance of this property in a grout as revealed from the observations of consistency curve plotted by the number of tests by viscometer is shown in Fig.No.4.6(a).

(5) Rheopexy: It is the reverse property of the thixotropy in which shearing resistance of the liquid increases on agitation, but decreases when left undisturbed. This is termed rheopexy and it can cause serious trouble in the mixing and injection process. Adequate use of dispersing agents is a remedy for mild occurrences, while material or combinations of materials exhibiting rheopexy to a high degree could be a abandoned at the outset. Consistency curve as obtained by viscometer is shown in Fig.No.4.6(b).

4.2.1.3. Fluidity: Fluidity is the reciprocal of viscosity. It is the property of grout suspension which penetrates the formation with ease. The fluidity as measured by the marsh cone indicates an average apparent viscosity, whereas it is the yield value of the shear resistance which is important. It is possible to determine the conditions in which grouts are able to penetrate a permeable formation and seal its voids. They are dependent partly upon the relative geometrical dimensions of the voids and of the particles used to fill them, and partly upon the surface action between the injected material and the pervious formation. Mayor[26] (1963) is of the opinion that the knowledge of the grain size curve of a suspension is not sufficient to define groutability. He has explained that when an attempt is made to grout the formation with particles having a diameter slightly less than that of pore sizes an automatically filtering plug forms inside the voids, which will be blocked gradually until it is completely sealed. According to the shape and smoothness of the particles they will more or less easily be caught up to form arches upon which the smaller particles will be wedged. In the case of cement grouting when high speed mixing takes place, as in the 'Colcrete' and high 'turbulence' mixer, the particles of cement are rounded off and the proportion of fine material is increased. This also increases the stability of corresponding suspension and facilitates its penetration of small pores.

Thus it appears that groutability is not a property that can be measured or established by a number or a formula. The same cement, put into suspension in different mixers to which different additives are added, will penetrate the fissures to a greater or less extent. The influence of pressure is equally important, for even in rock the opening of a crack can be changed. A liquid under pressure acts like a flat-jack which will temporarily separate the two faces and permit the penetration of grout which would otherwise be retained. This also applies to grouting in alluvial deposits, but the effect of pressure can be less beneficial. If the grouting pressure is excessive the grout penetrating in a more permeable zone may lift the overlying strata and create a passage in which the cement will be deposited without penetrating the stratum which it is required to seal. Mayor[26] (1963) have observed that the excessive pressures have in all cases produced a heave in the alluvial deposits and cement had collected in the voids without spreading into the mass of the stratum.

4.2.1.4. Stability: A grout must remain stable during the process of mixing and pumping, that is to say, if it is a suspension there must be no premature sedimentation, and if a liquid, no premature set. It is the property of suspensions, which enables them to preserve their original rheological characteristics. A lack of stability is revealed by the separation of the constituents, the solid particles falling to the bottom and leaving a variable depth of clear water above them. Stability is measured by allowing a suspension to remain in a container of given dimensions and determining the height of clean water above the particles after a certain time. This is very important property of suspension, and depends upon the grain size of the material, the specific surface of the particles and the intensity of surface action between the particles and the liquid. The addition of a very small %age of bentonite improves the stability of cement suspension considerably. The rise of water at the surface is also termed as bleeding. Usually the bleeding of the grout is not desirable. Mistry and Purohit[20] (1964) and Datye and Vinayaka[27] (1965) have recommended the bleeding of grouts less than 5%.

4.2.1.5. Water retentivity: Water retentivity of a grout is indicated by its ability to retain water against vacuum filtration. This property is determined by observing the time required to extract a predetermined amount of water from a fixed volume of grout subjected to vacuum. This water retentivity is considered to be a measure of the cohesiveness of the grout and to be indicated of its resistance to dilution in underwater works and to screening effects of small openings. It has been found that water retentivity, as measured by this method is closely related to the bleeding characteristics of the grout.

4.2.1.6. <u>Gelling:</u> A fundamental of a grout is that it shall develop a gel after a controllable interval of time and also shall develop a strength in keeping with the purpose for which it is designed. This is in addition to the requirements whilst flowing. Even a bentonite clay will not develop a stable gel without the assistance of additives, such as cement, silicate or other suitable chemicals. Usually referred to as rigidifying agents, their range for selection is wide. The rigidifying agents usually have an effect on the rheological properties of the grout base. In designing a grout mix, therefore, it is necessary to prepare trial mixes both in the laboratory and under field conditions in order that the required stability and viscosity during injection and gel strength in place can be achieved. To some extent this is a process of trial and error but on this basis the qualities required for the work and the limits for field controls are assessed. Gel time of grout can be controlled by addition of additives in required quantity and may range between a few minutes to 10 hours but in normal case a gel time of 30 minutes is considered. The gel time required may be determined mathematically knowing the type of grout to be used, permeability of soil to be treated and zone of influence assumed. The gel time of organic resin grout may even be a few seconds when the solution is required to seal the water flowing strata.

4.2.1.7. Permanence: A grout when set must resist chemical attack and errosion by ground water. A grout has to be permanent, only as long as it is needed. For instance, in a grout curtain under a dam the useful life of a grout should be counted in decades while in a grout curtain under an upstream coffer dam its useful life can be counted in years, or, if the grout is used to stop quick sand conditions during the driving of a tunnel, its useful life need be only a matter of weeks. Also, a grout with a shorter life than the structure can be preferred to a more permanent and more expensive grout if maintenance or corrective grouting is found to be economically feasible. Furthermore the useful life of a grout may vary with the conditions under which it is used. For instance, the use of a water soluble chemicals grout can be objectionable if ground circulation is relatively large, but will be quite acceptable if the circulation is insignificant. Clay grouts, although they never reach the consistency of a stiff clay, have nevertheless adequate rigidity of withstanding an appreciable amount of water circulation without any significant washing or leeching. A condition may exist when clay particles start migration due to improper selection of clay grout, low pressure being applied during pumping and when several types of grouts are injected without following proper sequence.

Pure silicate grouts are susceptible to disintegration and should be observed carefully under controlled condition over a prolonged period of time before use on a given project

There is no long term data available which shows the permanence and stability of silicate grouts. Even the long term stability of silicate grouts by joosten process is doubtful in absence of the data available. Even for synthetic resin grouts which are recent development there is no long term data which indicates the permanence and stability of the grout curtain. Certain resins could be attacked by aggressive water and bacteria. Tests have been made by the laboratoire control des pondres in Paris, and a small amount of Bactericide is added to the resin, and observations taken after a period of four years indicated no attack by bacteria on resin grouts treated with bactericide.

4.2.2. Rheological characteristics of typical grout:

The movement of fluid through the pore space of a soil is resisted by drag at the interface between the grains and the fluid. For true fluids the drag is proportion to viscosity and shear rate. The shear rate being determined by mean flow velocity and the geometry of the pore space. True fluids with no shear strength, are described as newtonian. Among the various common grouts, those which are simple solution of chemical compounds are ordinarily newtonian. Table No.4.1 shows the values of viscosity of grout of this type worked out by Raffle and Greenwood[28] (1961). Cement grouts have well defined shear strengths that develop immediately after mixing. Measurements made by Raffle and Greenwood(1961) on cement pastes with water cement ratio from 0.4 to 0.9 are shown in Fig.No.4.7. If a grout which has shear strength is pumped under constant pressure into the soil, the opposing drag due to the corresponding shear stress acting at the growing area of the surface wetted by grout ultimately becomes equal to the whole of the applied pressure so that none is available to maintain the viscous flow.

Table No. 4.1

Rheological constants of typical grouts

Type of Grout.		Viscosity CP	Shear strength dynes/cm^2 On mixing.	After standing
Newtonian grouts				
10% AM-9		1.2		
Sodium silicate M75		45		
Non-newtonian grouts				
Cement grouts: W/C	0.4	116	67.6	
	0.5	37	25.6	
	0.66	14	6.6	
	0.9	8.4	3.1	
5% Wyoming bentonite		10	$\angle 2$	50
5% Fulbernt clay		4.5	$\angle 2$	38

4.3. TYPES OF GROUTS

4.3.1. The grouts used at present for the treatment of pervious soils are classified in two categories according to their components:

(1) The solid phase grouts or suspension grouts such as cement and clay mixed with water.

(2) The grouts with only liquid phases such as silicates, or organic resins, also termed as colloidal grouts.

Solid phase grouts are non-newtonian and possess shear strength and are Bingham bodies. The flow of grout of this type in the early stages of injection is controlled by viscosity but in later stages may be controlled by shear strength. As permeation proceeds an increasing proportion of the injection head is absorbed in overcoming the drag due to shear strength until finally the flow ceases.

4.3.2. Cement based grouts:

4.3.21. These are classified as:

(1) Neat cement grouts

(2) Cement-clay grouts

(3) Cement-bentonite grouts

(4) Sand-cement grouts

(5) Clay-sand-cement grouts

(6) Cement mixed with admixtures grouts.

4.3.22. Neat Cement Grouts: Neat cement grout is a mixture of cement and water in different proportion varying from water-cement ratio of 20 to 0.4. This is an unstable grout as the grout suspension on sedimentation bleeds. It is used specially where strength is required. Mostly it is used for rock grouting and for coarse sand and gravel having size 1 mm. and over. Sometimes bentonite is added in proportion of 2 to 14% of the weight of the cement for good adherence to the fissures in the rock, but in soil problem of adherence is different and due to deformation of soil mass upon loading it is not the permanent hardened grout but the plastic or semi-plastic grout which adheres to the soil mass better. The studies carried out by the U.S. Water-ways experiment Station, Vicksberg reveal that neat cement grout can penetrate the cracks of size 0.5 mm, but if finer grout is used it can penetrate even cracks of sizes 0.25 mm. Grundy[2] (1955) has given that with present technique, cement grouts with admixtures can be used to seal fissures of fineness below 0.1 mm. when used after chemical injection (for lubrication) and can provide a high degree of impermeability to porous sand stone. Finely ground or air separated cement is advantageous for very fine fissures but usually ordinary portland cement is quite satisfactory. Clark[29] (1959) has suggested that neat cement grout should be passed through the finest practicable sieve to remove the oversized materials. Also, the use of a fine sieve provides the visual evidence of hhe thoroughness of the mixing. The use of the finest cement possible and wet sieving should be standard practice when grouting fine cracks. The wet screening, however, requires constant cleaning of screens, life of the equipment is also short, and requires considerable maintenance. The use of wet screening depends upon the volume of job and the degree of the results required. Properties of neat cement grout suspension are:

(i) Viscosity: Neat cement grout with water cement ratio of higher than 0.5 have low viscosity and rigidity. For lower values of water cement ratio the viscosity increases sharply and the grout requires some rigidity.

(ii) Fluidity: Fluidity of cement grouts can be improved by mechanical or physico-chemical action. High speed mixing activates the hydration of cement particles, gives a better dispersion and brings a rapid formation of small crystallized elements of hydrates of different types. Grouts obtained through energetic mixing are sometimes called activated or colloidal. Basically these grouts are more stable and fluid than ordinary grouts but only for very low water content. Wetting or dispersing agents also improve the dispersion and hydration of the cement particles. Resulting grouts are found to be more stable and more fluid than ordinary grouts but only for very low water content.

(iii) Setting: The initial setting time is normally 30 minutes and final setting time 10 hours. The viscosity increases with time and it is desired that atleast 50% viscosity of the original value should be maintained at 20 minutes of setting time. Setting time can be altered depending upon the job conditions. Setting ~~can~~ can be accelerated by addition of calcium chloride or sodium silicate or can be retarded by addition of sugar or carbohydrates but uncontrolled delays may take place. Addition of clay and bentonite in small quantities also prolong the setting time. Setting time also increases when water cement ratio increases. Somo cement never sets with water cement ratio greater than 10. All standard grouts set with a water gain.

(iv) Strength: Cements when set forms a hardened mass with compressive strength at 7 days nearly equal to 175 kg/cm^2. Density after setting is fairly constant and is independent of the water-cement ratio used. The density varies with the type of cement used, the volume of hardened cement grouts and may vary as much as 50%.

(v) <u>Bleeding:</u> The bleeding watêr accumulates under individual particles and will destroy the bond on the under side of the particles and reduce both the strength and lateral impermeability of the grouted mass. Because grouting often has the duel purpose of consolidation for strength and for the checking of the water movement, the two characteristics of low bleeding and active expansion while setting are most important. Small extra cost of the material required to produce an expanding non-bleeding grout can be saved many times over in not having to go back and redrill and regrout the hole.

4.3.23. <u>Cement-clay and Cement-bentonite grout:</u>

Kravetz[30] (1958) has given that addition of clay or bentonite in cement in very small proportions say 1 to 3% by weight of cement does not alter the basic qualities of neat cement grouts but improves the stability of the mixture without affecting appreciably its compressive strength. Clays are economical when available locally and their ability to yield a thixotropic suspension is of value in the injection of large fissures.

Addition of clay or bentonite to water cement slurries greatly extends the range over which suspension free from seggregation by settlement may be obtained. The dry clay or bentonite and cement powders may be mixed together and subsequently added to water, or slurries of clay or bentonite may be made and added to the cement slurry. Conversion of the bentonite to its calcium exchanged form undoubtedly accurs through reaction with free time roleased from the cement. The calcium bentonite is then flocculated by the excess of cations (mainly calcium) present in solution. The flocs so formed are still gelatinous and prevent sedimentation of the relatively coarser cement particles. The primary role of the clay and bentonite thus appears to be that of a suspending agent. The limitation on the quantity of clay or bentonite which can be incorporated in a mix are imposed by the following factors:

(1) Workability of the mix - increasing the concentration of clay or bentonite leads to an increasing stiffness in the slurry, eventually making it unpumpable.

(2) Final compressive strength - substantial addition of clay or bentonite to cement slurry decreases the compressive strength of the set cement.

(3) Specific gravity - Low specific gravity.

(4) Stability - The concentration of clay or bentonite in the mix must be increased to obtain low bleeding.

Clay upto 1/3rd by weight of cement could be added to ensure setting, but tests must be made to ensure that the mixture does not shrink too much on setting and that there is adequate resistance to erosion. Claims are sometimes made that clay injection is preferable to cement injection for the filling of fissures containing clay. Apart from the low cost of the clay, the resultant product with clay injection is of low density on account of the swelling action of the clay. Greater volumes of voids are filled than with the same weight of materials in cement grouts, but the economy is offset by the lesser resistance to erosion by water action so that a greater thickness of barrier is required.

The composition of a mix for a given function can only be decided by taking these factors into account. Kravetz[31] (1959) also states that addition of bentonite from 2 to 14% as an additive to cement improves its adherence to rock fissures. There is a belief that addition of clay or bentonite in cement increases its flow properties. But Raffle[32] (1963) believes that suspension formed by clay or bentonite in cement grout is very much less fluid and viscosity of combined suspension is greater than that for neat cement grout suspension. The increase in the injected quantities is not due to the fluidity due to addition of clay or bentonite in the mix but it is due to suspending action of the clay or bentonite which prevents setting near the injection point and make it possible to transfer cement particles further from the source than if the clay or bentonite was not present. He also states that decrease in strength on setting is the primary function of water cement ratio. He suggests that only one or two percent of bentonite prevents the bleeding characteristics of water cement suspension of water cement ratio greater than about 0.6 by offseting

105188

the gravity force acting on the cement particles and thus cement particles are prevented from setting.

At Ukai Project (Gujarat) high grade bentonite was added in the neat cement grout to the extent of 2% by weight of cement to improve the workability and reduce the bleeding.

4.3.24. Sand-cement grouts: Inert admixtures to cement are used for economy. Sand is excellent when large fissures or cavities exist, upto 3 parts sand to one part cement by weight is a reasonable limit. The maximum particle size of the sand can be upto 1/8th of the size of the fissures to be injected. In all cases, a final injection should be made with neat cement alone to seal the fine fissures. Sand cement grouts tend to settle if mixed in conventional mixers, and high speed colloidal mixers may be necessary.

4.3.25. Clay-sand-cement grouts: Clay is added in sand cement grouts if it is available at reasonable cost to improve the stability of the mixture. Grouts of low water-cement ratio have a tooth paste like flow which reduces wear of pump and permits coarse sand to be used.

4.3.26. Cement-admixture grouts: When the fissures are not large enough for the use of sand, useful admixtures such as rock flour, lime stone dust or fly-ash, if available at economic cost in relation to cement are added in the cement grouts. Mixes of about 1:1 proportion will ensure the setting characteristics of the mixture, but the limiting proportions are easily obtained by a site tests of mortar made from the materials. Thorough mixing of these fine materials with the cement is essential. Admixtures may be employed in grouts of various compositions to improve selected properties. The class of admixture to be used will depend upon the desired benefit ~~bentonite~~. Various types of admixtures commonly used in cement grouts and their effects are explained in Chapter 3.5.5.

4.3.3. <u>Clay based grouts:</u>

4.3.31. Clay based grouts are classified as below:

(1) Stabilized clay grouts

(2) Clay-cement grouts

(3) Clay-cement-bentonite grouts.

4.3.32. <u>Stabilized clay grouts:</u> Clay based grouts, with or without cement, have two characteristics which oppose penetration into fine passage or voids in the soil. The first is that they have discreet particles which quickly block off the available pore spaces and secondly they have initial shear strength. This initial shear strength has nothing to do with the setting of the grout and is present during and after mixing. It is represented by the Bingham yield value and a proportion of the available injection pressure is required to overcome it before any penetration can take place. The smaller, the void size of the soil the less shear strength is required to resist a given pressure. For normally constituted clay-cement grouts injected into a soil of 10^{-1} cm/sec. permeability, shear strength might be unimportant in limiting penetration. Such a grout might have shear strength of 70 dyn/cm^2 and would react a radius of about 10 ft. from the injection point under a pressure of only 12 psi. before its progress can be arrested by shear strength, but at about this value of permeability the particle size will inhibit penetration. Clay grouts on the other hand while they have a lower shear strength than clay-cement are used in rather finer soils and consequently their initial shear strength may be of more importance. The use of conditioning chemicals therefore are essential to reduce the shear strength of these grouts during mixing and injection. With the reduction of shear strength of clay grouts by conditioning chemicals, the main barrier of penetration becomes that of particle size.

In preparing these grouts, treated clay is mixed with water to form a slurry. Additives are used to check flocculation of particles and to promote gel strength. All clays can be improved by such treatment and their capacity for penetrating soil is always greater than that of clay-cement mixture. High grade bentonite may be used in grouting coarse and medium sand. But as their gel strength is not sufficient to give an appreciable increase in strength to the soil, they are generally used only for sealing, or reducing the permeability. Bentonite on addition of water swells and this swelling is reversible and its final extent is governed by the nature of the exchangeable ions. The strength of thixotrpic gel formed by bentonite suspension is strongly dependent on the setting time, the effect of concentration of bentonite in suspension and the chemical composition of the suspending fluid is shown in Fig.No.4.8. Jones[33] (1963) has given that irreversible gels can be formed by addition of sodium silicate to bentonite suspensions which leads to:

(1) A decrease in yield point and thixotropy, when the bentonite and silicate concentrations in the suspension are low, and

(2) At higher silicate concentrations the suspension gels progressively are standing.

Vora[1](1951) gives the account of clay grouting on a large scale at Madden Dam, Panama Canal, where problem was to reduce dangerous leakage in rock containing large voids, especially cavernous lime stones filled with loamy sand which will not bond with cement. Cement-clay mixtures were not found suitable and pure clay was injected through 1" diameter pipe and thrown out by air as a fine spray. The clay used was of low colloidal, plasticity of 35% and specific gravity of 2.7. He has summarized the advantages and disadvantages of clay grouting as listed below:

Advantages:

1. Low cost
2. Large amount of grouting from a single hole.
3. Time of setting can be controlled by pressure variation, and
4. Practicability of grouting in highly weathered rock and large caverns.

Disadvantages:

1. Its low resistance to erosion or disintegration (not at low pressure gradient)
2. Difficulty in obtaining pressure resistance, because of surface blow out.
3. Impossibility of grouting an opening filled with running water.

4.3.33. Clay-cement grouts: The clay-cement grouts is probably the best known and the most commonly employed. It is a slurry of clay-cement in water in different proportions. This is a stable grout in the sense that it does not bleed on set and the ability of clay to form gels is used to stabilize the cement. For soil foundation, the cement is usdd as an additive in proportions usually enough to give the grout a cement like set. This type of clay-cement grout has therefore all the basic properties of a clay grout. The cement particles in a clay and cement system are dispersed and held in place by the clay gel structure. The clay performs no important function in the final chemical set resulting from hydration of the cement. Development of strength is slow and there is no well-defined setting time. Set grouts have low crushing strength relative to neat cement grouts. Actual strength ranges from less than 0.1 kg/cm^2 to 60 kg/cm^2 depending upon cement content. Ischy and Glossop[7] (1962) have shown that set strength of 25 kg/cm^2 is obtained with clay-cement grouts with cement/water ratio of about 0.4. These strengths are vastly greater than those required to resist normal working pressures gradients resulting from water retaining structures and are usually adequate for consolidation grouting and the treatment of coarse alluvial deposits of permeability more than 10^{-1} cm/sec. In rock grouting and consolidation of open soil, high set strengths may be required. For such works the clay content is kept a minimum only a few percent of cement weight by the use of highly active clay such as bentonite and grout mix is termed as cement-bentonite grout.

Grouts containing relatively high proportion of clay, on the other hand, are referred to here as clay-cement grouts. Because of the size of the cement particles the grout mix is used for soils having permeability greater than 10^{-1} cm/sec. The clay-cement grout is superior to cement grout in respect of stability. Grouts with no water gain are desirable as the voids caused on setting are thus minimum and regrouting is not essential, delayed setting makes it possible to do intermittent grouting without loss of hole. The clay is present in the role of a filler, it increases the volume yield per unit weight of material and at the same time produces a stable grout. These grouts, therefore are often formulated not with highly active clays, but with locally occuring mixed clays, modified where necessary by additives, such as bentonite or sodium silicate. The amount of clay to be added will depend upon its quality, that is to say upon its particle size, clay content and liquid limit. The clay cement grouts are not true fluids and the gel structure which is present immediately upon mixing gives the grout sufficiently high shear strength to resist gravitational sinking and local gradients after injection but before final set. Raffle and Greenwood[28] (1961) have shown that this gel strength, although advantageous in presenting displacement is accompanied by a relatively high yield value which, in turn, requires a correspondingly high injection pressure. The demand for extra pressure is greatest in the finer soil and grouts with very high gel strength may not penetrate when pressure is limited to low values because of shallow depth.

Clay evidently be finest clay as can be economically procured and processed. Medium and coarse sand can easily be eliminated by the treatment in hydrocyclone. The fineness of natural clay should, therefore, be judged by the proportion of material below 30 microns after excluding medium and coarse sand. It is sometimes possible that clay may contain excess of silt or fine sand hence it may apparently be finer than an other clay containing more of coarse and medium sand. Obviously the coarser clay with little of silt or fine sand can be processed into better grout material as coarse sand can be easily removed by the hydrocyclone.

Datye[19] (1963) has given that grouts with cement content less than 20% of the clay have rarely been used. Cement content below this limit should, therefore, be avoided unless confirmed by laboratory tests. The fundamental qualities of clay cement grouts are:

(i) Stability: Stability is directly proportional to the quality of the clay and its proportion in the grout. A small amount of clay is usually required to obtain grouts which settle with little or no water gain. Grouts with no water gain are always preferred whenever possible since with them there is no need for redrilling and regrouting to complete the filling of voids.

(ii) Rigidity & Thixotropy: Grouts which are made to settle with no water gain can be thixotropic sometimes, that is until the cement hardens. Clay cement grouts always have some rigidity.

(iii) <u>Adherence</u>: This is a very interesting property of the clay cement grouts which are found to have a much better ádherence to clay coated walls of cracks and faults in rocks, than cement grouts.

(iv) <u>Economy</u>: This is the most important of the qualities of clay-cement grouts. For filling of same quantity of voids the quantity of clay cement mixture is required less than that for neat cement grout because of swelling properties of clay.

4.3.34. <u>Clay-cement-bentonite grouts</u>: Sometimes bentonite in small proportion by weight of clay is added in the clay-cement grouts. Resulting grout mixture is finer than clay-cement grouts. Due to colloidal particle size of the bentonite deflocculating agents are added in small proportions. The grout mixture have the same properties of clay-cement grouts.

3.4.4. <u>Silicate based grouts</u>:

4.3.41. There are 3 types of grout mixes based on sodium silicate:

1. Clay-silicate grouts
2. Bentonite silicate grouts
3. Pure silicate grouts

4.3.42. <u>Clay-sodium silicate grouts</u>: In this, clay is essentially an additive to the chemicals, to reduce cost, while the mixture retains all the properties of the chemical mix. The principles of the clay-chemical grouting and chemical grouting are, therefore, identical. The chemical is usually sodium silicate(Na_2SiO_3)mixed with a soluble reagent in proportion required to attain the specified setting time. Whereas clay chemical is suitable for medium sand, chemical alone are used for fine sand and silt. In preparing the clay chemical grout, it should be remembered that:

1) Extremely short setting time can be established during laboratory study, however, on the job the grout must be mixed pumped and made to penetrate the soil before it can set, therefore, the minimum setting time is dictated by the field conditions (usually half an hour).
2) There is a minimum amount of sodium silicate below which the setting time of the clay-chemical grout cannot be controlled although the gel will form eventually, this amount varying with the clay proportions, the reagent used, the water ratio etc.

Clay chemical grouts have a wide range of usefulness in medium sands for reducing of seepage when properly used. They are quite economical too, because the clay content reduces the requirement for active chemicals for a given volume of grout and in practice have been found to give a better water cutoff than clay-cement grouts in the same materials. The reason for this is that formations are stratified or non-homogeneous, some layers or zones being more permeable than others. A clay chemical grout flowing through and filling the open layers will also penetrate the fine grained layers much better than cement or cement-clay grouts. Furthermore, even in the more permeable areas, the gelled chemical will

completely fill the space even though the suspended mate-
tials settle.

4.3.43. Bentonite silicate grout: It is a mixture of bentonite and sodium silicate with some suitable gelling agent. It has the advantage of low bleeding percentage, high shearing resistance and ultimate consistency as compared to bentonite grout. It is more economical as compared to pure silicate grouts of equal ultimate consistency and shear resistance. The permeability is however not as good as silicate grouts. It is possible to design bentonite silicate grout of zero bleeding percentage and a shear resistance adequate to prevent washout against any imposed pressure gradient with large factor of safety by judicious selection of reagents. Laboratory tests carried out by Datye[19] (1963) reveals that the viscosity and gelling characteristics are influenced by the variations in the quality of the bentonite. Relatively small variation in the liquid limit of the bentonite was found to give rise to major variations in the characteristics of the grout. The reagent successfully used is menosodic phosphate (NaH_2po_3). This reagent acts as a gelling agent for the silicate and deflocculating agent for the clay. Silicate-sodium monophosphate grout have been used for the permanent grout curtain at Notre-Dame-De commeirs Dam in France where the height of dam is about 50 meters and depth of alluvium of about 50 meters. At Ukai Dam(Gujarat) clay was replaced by low grade bentonite of liquid limit less than 300 and bentonite-silicate-sodium monophosphate grout was used for grouting crushed rock zone. This was found to be cheaper than grout with clay-silicate

4.3.44. Pure Sodium silicate grouts: In silicate grouts, the principal constituent is sodium silicate. The reagents commonly used are:

1) Acids - the common acids such as hydrochloric, sulphuric, phosphoric etc.
2) Acid salts, such as, sodium monophosphate and sodium bicarbonate etc.
3) Salts of acidic reaction, such as, alluminum suphphate etc.
4) Neutral salts such as, sodium chloride, all the salts of ammonia, the metalic salts. etc.
5) Alkaline salts such as, sodium alluminate.
6) Bases such as, ammonia.

The acids are not very suitable because they are corrosive and have rather sharp gelling time; however, use of phosphoric acid has been reported by Bachy.

Calcium chloride has a very rapid gelling time hence it is not convenient for field work. The most common reagents are therefore:

Sodium alluminate
Ammonium sulphate
Sodium bicarbonate
Sodium monophosphate

Sodium alluminate is generally reported to be most economical but it has a tendency to gelling too fast at temperatures greater than 40°C. Ammonium sulphate has similar tendency. The choice of reagent is based on practical considerations which are:

(i) Quality of gel: Gels have different qualities such as:

(1) The time of gel formation can be regulated with precision and accuracy.

(2) Sudden gel formation without preliminary formation of gel.

(3) Certain amount of mechanical resistance.

(4) Impermeability, and

(5) Stability.

(ii) Facility for use: Some reactive agents are toxic and have corrosive reactions and are eliminated for site use.

(iii) Economy: The price of gel put in place is a function of the degree of the dilution of the silicate, the price of the precipitant and the facility of the injection at the work site.

A general study of the behaviour of different reagents with silicates has been reported by Barbedette and Sabarly[23] (1958). The study gives behaviour of different types of reagents depending on the relative proportions and the dilution with water. The study is very comprehensive and systematic. However, its value is somewhat reduced because of arbitrary criteria used for defining the time of gellification and strength of the grout. According to them the composition of gel is defined by:

$$\alpha = \frac{\text{concentration of reagent}}{\text{concentration of silicate.}}$$

$$\beta = \frac{\text{Volume of water}}{\text{Volume of concentrated silicate.}}$$

All the reagents have characteristics as above with gelling time increasing very rapidly with small changes in α.

(iv) <u>Influence of temperature on gel formation</u>: The increase of temperature decreases the time of gel formation. But the influence of temperature is more or less important depending upon the precipitants used, for example if the temperature in the field varies from 5^o to 30^oC the time of gel formation of a gel with $\beta = 5$ (with alluminate) will vary from 2 hours to 20 minutes.

(v) <u>Strength of gel</u>: Strength of gel is directly proportional to quantity of gellified silica in a given solution. For constant gelling time and for given quantity of silica in a given solution the strength is inversely proportional to gelling time. The strength of gels of long setting time is very small. The strength is therefore a function of the gelling time and concentration of gelled silica irrespective of the reagent used. However, the percentage of gelled silica for given β and gelling time is dependent on the reagent.

(vi) <u>Permeability of the gel</u>: The following factors have important bearings on permeability of gels:

(1) The dilution factor β

(2) The precipitants used

(3) The value of hydraulic gradient.

Bandedette and Sabarly have found out that the permeability of the gel is near about 1 to 3 x 10^{-7} cm/sec.

Infact the permeability achieved by fine soils due to stabilization with the treatment of silicate is very much more than that offered simply by the cohesion of the gel itself. Further, the permeability of diluted gel is of the same order as that of the concentrated gel. The dilute gel has viscosity very close to that of pure water for penetrating the fine material, thus ensuring more uniform penetration.

The silicate aluminate grout used at Ukai was of the proportion:

Silicate	300 c.c
Aluminate	13 to 16 gram.
Water	500 c.c.

The gel formed was quite firm and strong and had a good washout resistance. Syneresis and long term brittleness are of course observed for this grout. Laboratory test results are given in table No.4.2.

Recently silicate grouts have been developed which increase the strength of the treated material, as for example, a grout based on sodium silicate mixed with a system of organic eater, which gives a controlled evolution of reaction ions. This in due course results in the precipitation of silica gel. The strength of the ground treated in this way depends upon the concentration of sodium silicate in the grout. Caron[34] (1968) has given that samples of injected sand reveal strengths of 20-30 kg/cm^2. Some other hard gels have also been developed in recent years and they also involve the use of organic compounds.

Successive application in low permeability soil can also be done by the use of electro-osmosis method. The procedure is described by Bally and Antonescu[35] (1961) which shows that when an electric current in the reverse direction is passed from the perforated electrod normally used as a well, is filled with a liquid silicate grout, the rate at which the grout went into the soil is very much increased.

4.3.5. Lignochrome:

4.3.51. A new injection product has begun to appear in the United States and in Russia. Lignochrome (or ligno-sulphonate) is a residual product formed during the production of cellolose from pulp by the bisulphite process. The residue is concentrated and is produced in the form of a syrupy liquid or powder. It is known that soluble lgnosulphites mixed with a bichromate become firm gelatinous masses. The bichromate oxidizes the lignosulphite and precipitates it in the form of the salt of a heavy metal. The process of gellification is the subject of English, Russian and Swedish studies viz Smith(1952), Anger (1955), Pearl and Blyer(1956), Aaltis and Roschier(1957). Their studies have revealed the following characteristics:

(i) Setting time: The setting time varies inversely with the concentration of bichromate and may be controlled through a range from 10 min. to 10 hours. As for most chemical mixtures, the setting time is shortened by raising the temperature.

(ii) Gel strength: Gel strength by a vane test of pure gel is of the order of 500 gr/cm^2. The principal factors which have a bearing on the strength are the nature of the lignosulphites, the concentration of lignosulphite and chrome, and the P.H.

(iii) Viscosity: A freshly prepared lignochrome has a viscosity which is in the range of that of a silicate gel.

This viscosity increases with time and it is therefore comparable with that of a silicate but not to that of an equious resin, Fig.No.4.9. shows the variation in viscosity with time after mixing for two types of lignochromes and a resin grout for comparison purposes.

(iv) Toxicity: It is toxic and precautions must be taken when mixing lignochrome. After the product has gelled, it is not toxic.

(v) Stability: Pure gel or mortar samples of lignochrome show no deterioration in water. Their loss of weight is negligible and there is no reduction in strength with time.

4.3.52. Comparison between silicate gels and lignochrome:

The study has shown that lignochromes have a field of application parallel to that of silicate gels. Both are newtonian fluids with a viscosity which increases with time. The strength of the lignochrome is higher than that of the silicate gel. The advantages and disadvantages of lgnochromes are complete stability with time and easily regulated range of setting times. On the other hand they are slightly toxic and more expensive.

4.3.6. Synthetic resins (high Polymers):

4.3.61. These include a group of organic substances of which a single molecules generally under the action of a catalyst unite to form larger molecules, which may have the effect of converting an originally liquid substance into a solid. This property would suggest that polymers may be used as grouts and infact several are now so used, and a number of others are under investigation. The most commonly used products of Am-9 and phenoplast resins are only described here. Unfortunately, the polymerization which produces commercial plastics e.g. Nylon, Orlan, Dacron, Plexiglass, etc. takes place at temperatures and pressures which it would be impossible to attain on grouting sites. The following are the properties of an organic grouts:

(i) Solubility: The monomers in use are completely soluble in water. Concentration of 10-30 percent are commonly employed, and the strength achieved is proportional to the concentration used. These solutions are not colloidal, and have a viscosity very clost to that of water about 2 cp.

(ii) Polymerization: Polymerization takes place abruptly after a fixed period of time has elapsed, which is regulated by the use of one or more catalysts. Laboratory tests have led to the selection of suitable catalysts for each range of setting time, by regulating the quantity of catalysts. Polymerisation is an exothermico reaction and takes place more rapidly in an insulated medium than in the ground and this must be taken into account when injecting. It may sometimes be useful to mix the monomer solution and the catalyst at the top of the hole immediately prior to injection in order that adequate controlled setting time may be obtained.

(iii) Limit of Injectability: Using very slow pumping speeds, fine sands and silts may be injected with resin grouts. In practice, soils with permeability down to 10^{-5} cm/sec. may be effectively injected.

(iv) Stability: Organic polymers are the most stable products with respect to time. They are totally insoluble and no soluble salts are formed after setting.

(v) Toxicity: As monomers, these organic derivates are more or less toxic, but after polymerization there is no danger whatsoever.

4.3.62. AM-9: This is a product of the American Cyanamid Company's. It is a mixture of acrylamide and one of its methyl derivates in proportion which produces very stiff gels from dilute, aqueous solutions when properly catalysed. The process by which gellation occurs is a polymerization cross linking reaction. AM-9 chemical is available in powder form white in colour of friable lumps, very soluble in water, ethonal, methonal and insoluble in gasoline, kerosene and oils. The gel is formed by the following two steps.

Step I: An aqueous solution of AM-9, containing additives for controlling the gel time and one component of catalyst system (the activator), is prepared.

Step 2: The remaining component of the catalyst (the initiator) system (usually in water) is added to the solution of AM-9 prepared in Step I.

Timing of the induction period (gel time) is started. Two reactions occur in sequence. The first reaction starts almost immediately after the second component of the catalyst system is added to the AM-9 solution. The rate of formati-on of free radicals and their rate of decomposition is strongly influenced by a number of factors. Control of these by proper selection of the catalyst system allows a predetermined amount of time to elapse before polymerization of AM-9 occurs. This is known as the induction period, or gel time, during which the viscosity of the solution remains almost constant. At the end of the induction period, heat is evolved, and long flexible, polymer chains are formed. As these chains form, they simultaneously cross linke to form a stiff complex matrix which binds the water into a gel. The gel reaches its maximum strength in a matter of minutes.

There are many catalysts and mixtures of catalysts which may be used to gel AM-9. For normal use, the catalyst system is composed of catalyst DMAPN, AP(Ammonium persulphate) and KFe (Potassium ferricyanide). The following factors effect the gel time:

(i) <u>AM-9 Concentration:</u> Reducing the AM-9 concentration causes a slight increase in gel time.

(ii) <u>Catalyst concentration:</u> Changes in the concentration of one or all components of the catalyst mixture have a very marked effect on the gel time. Too much KFe or too little AP will produce week gels or none at all. Excess AP may reduce the PH to the point where the gel will not form. The lower limit is 0.2% catalyst DMAPN, 0.2% for AP and the upper limit for KFe is 0.035%.

(iii) <u>Temperature:</u> The gel time for any catalyst system increases markedly with decreasing temperature and decreases with increasing temperature. A rough rule of thumb is that the gel time is out in half if the temperature goes up $10^{\circ}F$.

(iv) <u>PH:</u> The PH of the solution, after catalyst addition, has a very marked effect on the gel time. For best control upto about ~~a~~ one hour gel time, the PH should be in the range of 7 - 11.

(v) Air: Air slows down the polymerization of AM-9 due to the effect of oxygen.

(vi) Salts: The presence of soluble salts, such as sodium chloride, calcium chloride, sulphates and phosphates, has an accelerating effect on the rate of gelation. The amount of acceleration depends on the salt concentration and should be determined by a test in the field. Salts also have the effect of increasing the gel strength, it also decreases with dilution.

The use of very short gel time helps to insure that the grout remains near the spot where it entered the formation.

The grain size and grading curve of sand influence the strength of the injected material. The finer the sand and the better graded it is, the stronger will be the treated soil. The grouted sand always has a much higher strength than the pure gel, and it is therefore impossible to judge the consolidation effect from a sample of pure resin. It is substantially impermeable to water 10^{-10} cm/sec. No syneresis have ever been observed. Under moist conditions, the gels appear unchanged for atleast 10 years. The gels are resistant to attack by fungi, dilute acids, alkalies and the ordinary salts and gases normally found in the ground.

Polymerized AM-9 is mainly elastic, deformation being proportional to the load whether the test is made on a gel or on a sample of injected sand. The modulus of elasticity of a pure resin, 10% concentration varies from 0.1 to 0.5 kg/cm^2 while when injected in sand, it can be as high as 100 kg/cm^2. This has a very interesting application for filling fissures or joints which may open or close with variation in temperature. American cyanamid have given that properties of AM-9 can be varied considerably with additives. These may be used to increase the gel strength, to increase the solution density, to increase the solution viscosity, to act as an inert filler for large voids or nearly to colour the solution to aid in subsequent loca tion of the gel.

Almost any inert solid added to an AM-9 solution will result in a modest increase in gel strength, a modest decrease in gel shrinkage, and may also reduce the cost for some application. Fern[36] (1963) has given the merits of AM-9 solution as below:

(1) The ease with which solution can be prepared.
(2) The simplicity of the equipment involved.
(3) The exact control of gel times, which is possible.
(4) The fact that the viscosity of the grout solution remains low (1.6 cp) and constant until a very short time before the onset of gellation.
(5) Although AM-9 is more expensive than any other grouting material but AM-9 solutions fill more voids space than equal volumes of most cement slurries, because each gallon of AM-9 produces at least one full gallon of impermeable gel. There is no bleeding, seggregation or setting problems. Penetration of any zone where water can be flown may be expected.

(6) Based on field experience, contractors estimate~~s~~ that grouting with AM-9 can be completed in 10-20 percent of the time required for grouting with portland cement.

(7) Due to the low viscosity of AM-9 solution and the extent control of gel time, waste of grouting fluid can be minimized and in some cases essentially eliminated. Naturally, in order to accomplish this, suitable pumping equipment and a good analysis of the problem are prerequisites.

4.3.63. Resorcinol - Formaldehyde resin: Phenoplasts are composed of phenol and formal-dehyde which is soletan-che patent process. Bakelit, which is well known is obtained by combining an ordinary phenol with formal-dehyde, but it requires temperatures and pressures which would be impossible to obtain on a grouting site. However, 1:3 dehydroxy-bensene (resorcinol) polymerizes with formal-dehyde at ambient temperatures in an aqueous solution when the PH of the solution is changed. The acid catalysts are not affected by air, indeed when using certain mild catalysts in dilute solution, it has a better set in the absence of air and this is desirable for injection purposes. The strengths obtained in the form of a pure resin, and in a mortar, are far higher than those given by AM-9. AM-9 is purely an elastic material, while resorcinol formal-dehyde is partly elastic and partly plastic. Under constant load an immediate elastic deformation is observed. This is followed by a delayed elastic deformation and then by an plastic flow. The elastic deformation is proportional to the load for the pure resin and for the mortar. The elastic moduli are much higher than those obtained for AM-9.

4.3.64. <u>Comparison of two types of resin grouts:</u> The use of AM-9 or phenoplasts depends on the mechanical properties that it is desired to give to the ground. If elasticity is desired, AM-9 is the right choice, but if strength is required, the phenoplasts are preferred.

4.3.65. <u>Pre-Polymerized resins</u>: The products which have been discussed above permit the injection of fine sands and sandy silts, but not the transformation of sand into sand stones, or the regeneration of concrete, since a polymerized resin does not have the strength of an ordinary concrete. However, if a partly polymerized plastic is used, from which the water of reaction has been eliminated, it is possible to attain strength far higher than concrete. There are large number of plastics which, partly polymerized, give unconfined compressive strengths in excess of 300 kg/cm^2. The commercial products have the drawback of being very viscous, and not useful for the injection of fine sands and fissures.

4.3.7. <u>Grouts based on bitumen:</u>- Bituminous grouts are of two types. The first consists of bitumen heated between 100-150°C. The use of such grout is exceedingly difficult and may involve preheating of the soil to be treated. The second method is to use a grout of bitumen emmulsion. The break down of emmulsion in the soil can be obtained either by the addition of an organic ester or by means of a synthetic resin.

4.3.8. Gas as a grout reagent: For the gases as grouting reagents, there are several patents from time to time which cover the use of gases in grouting, the gas generally used in conjunction with a liquid component. In theory the injection of gas is feasible but it gives rise to technical difficulties and is not of much interest from a practical point of view. One of the principal objects of grouting is to check the flow of water in the ground and it is just as easy to pump a liquid grout (of about the same viscosity as water) into the ground as to drain water from it. For this reason the injection of a gas offers no advantage over the injection of an aqueous organic monomers. In the past, before the process of polymerization was understood, the idea of a gas injection was alternative, but it is no longer so.

4.3.9. Ideal grouts:- An ideal grout must have properties which could simply be varied by altering the proportion of the constituents to cope with the various requirements demanding of it. For example, it must, in the higher concentrations provide a strengthening of the ground, and in the lower concentrations act as an impermeable barrier to water or air. Most important of all, it must at all times and in all concentrations be sufficiently cheap to be attractive to the engineers. Idealy, it must have a very low viscosity in low concentration and must have an extended gelling time for dealing with fine sands and coarse silts, or, more correctly, an extendable gelling time. For more permeable strata it is desirable that at some stage during or prior to setting time it should acquire something in the nature of a yield stress. Such a grout, of course, does not exist and it is extremely doubtful that it would ever be found. Some of the nearest approximations are perhaps the phenolic formal-dehydes, which would be widely varied in their concentration, still maintaining stable gels of the strength of which is proportional to the concentrations of the constituents.

Table No.4.2

UKAI DAM PROJECT

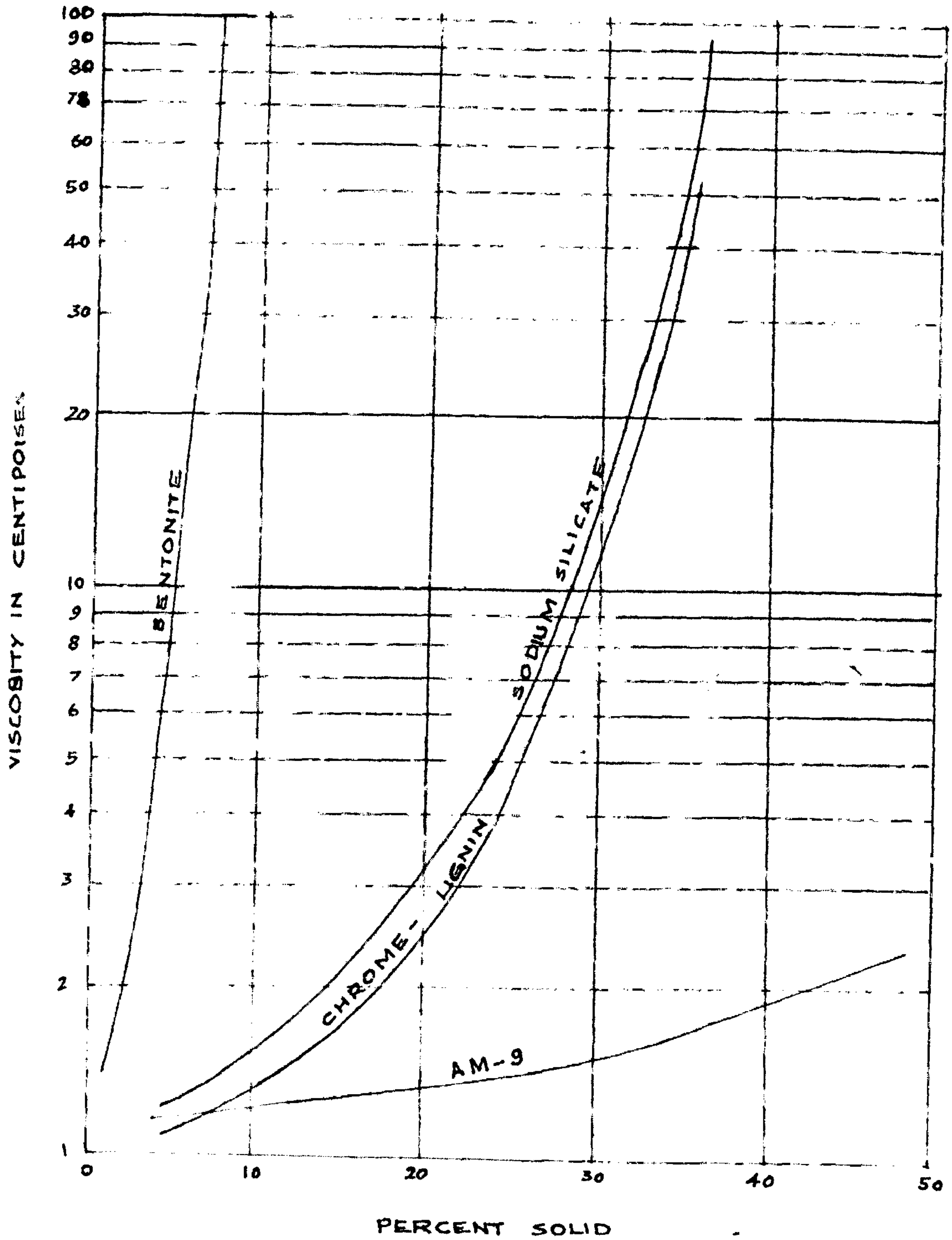
Statement showing the properties of pure chemical grout mix

Sl. No.	Quantity for 25 litres batch of grout mix. SP:gr.of silicate solution.	Vol.of silicate solution in liters	SP:gr.of aluminate solution.	Vol.of aluminate solution in liters.	Proportion of grout mix Silicate at sp:gr 1.35 in c.c.	Aluminate in grams.	Water in c.c.	Jellifying time in Hours.	Mints.	Temp. in Co.	Needle resistance test in gms/Cm² after 1 day	2 days	3 days	7 days.	Vane shear test T in lbs/ft² after (rounded figure 1 day	2 days	3 days	Re
PART I (Laboratory trials)																		
1.	1.202	16	1.030	9	300	12.69	500	0	50	35.5^0	717	887	-	-	49	52	54	
2.	1.202	16	1.030	9	300	12.69	500	0	52	35.0^0	719	875	-	-	51	55	58	
3.	1.202	16	1.03	9	300	12.69	500	1	14	27.0^0	-	-	-	-	Study of jelling time only with TGMR			
4.	1.202	16	1.03	9	300	12.69	500	1.	08	28.0^0	-	-	-	-				
5.	1.202	16	1.028	9	300	11.92	500	1	00	34.0^0	691	807	659	-	45	49	48	
6.	1.202	16	1.028	9	300	11.92	500	1	02	34.0^0	712	737	572	-	46	50	48	
PART II (FIELD TRIALS)																		
1.	1.202	16	1.030	9	300	12.69	500	0	25	35.0^0					51	-	61	
2.	1.202	16	1.030	9	300	12.69	500	0	28	36.0^0					51	62	58	
3.	1.202	16	1.030	9	300	12.69	500	0	50	25.0^0					56	52	60	
4.	1.202	16	1.030	9	300	12.69	500	0	42	25.0^0					58	62	63	
5.	1.202	16	1.028	9	300	11.92	500	0	40	34.0^0					44	46	N.T.	
6.	1.202	16	1.028	9	300	11.92	500	0	38	33.0^0					45	46	N.T.	

Note: Jelling times and vane shear strengths depend upon temperature at the time of preparation of mix. Variation can therefore be expected under field conditions, from mix and strength development with time also.

FIG: 4·1

VISCOSITIES OF VARIOUS GROUTS BY STORMER VISCOMETER (DATA AT 68° F)



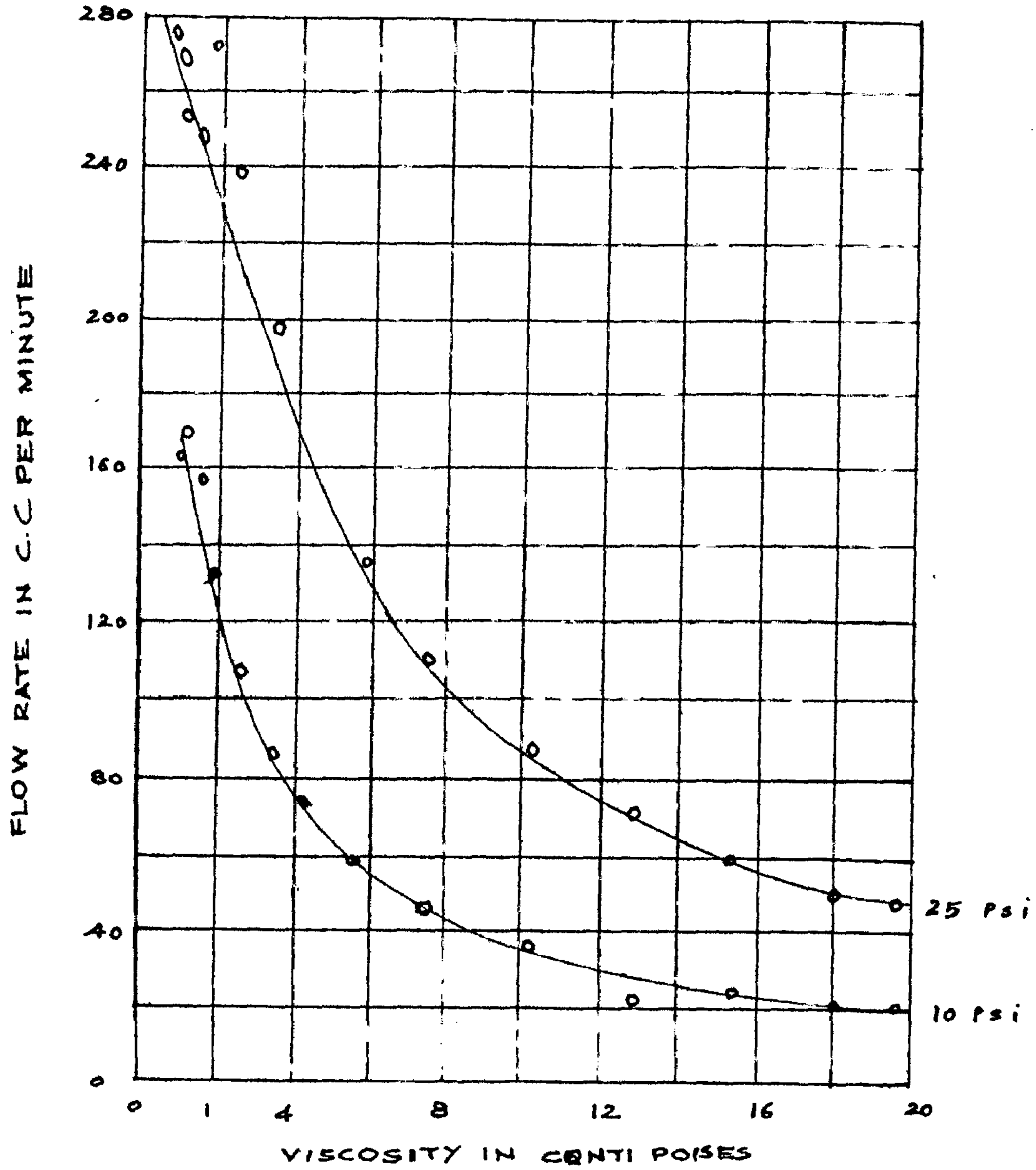


FIG:- 4.2

FLOW THROUGH PIPE BASED ON

POISEUILLE'S LAW

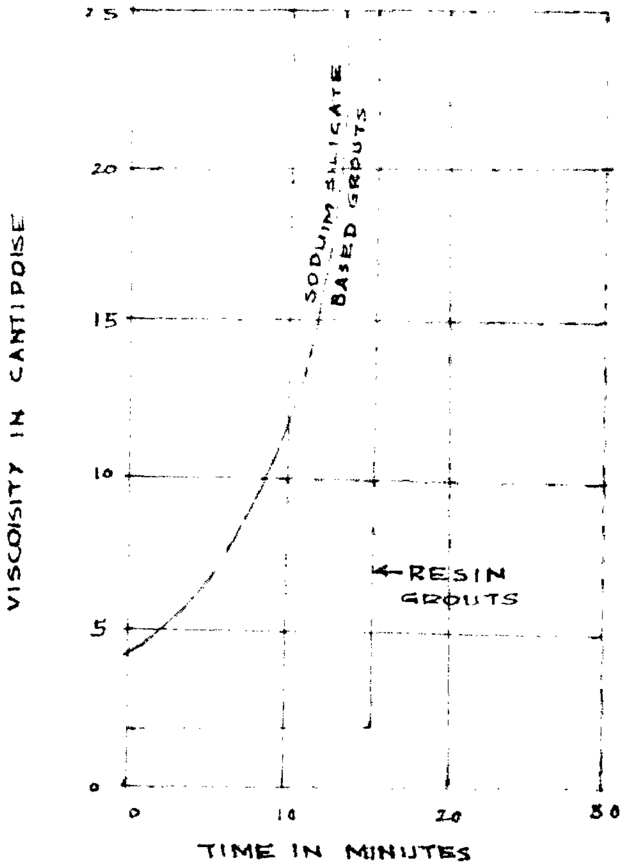


FIG: 4·3

COMPARISON OF SETTING OF RESIN GROUTS WITH THAT OF SILICATE BASED GROUTS.

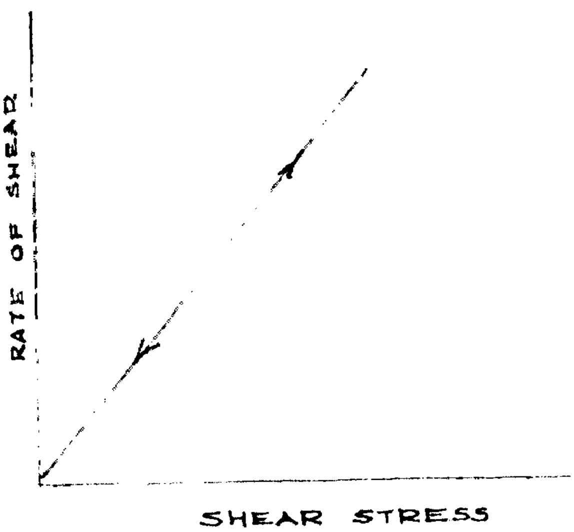


FIG: 4·4

CONSISTENCY CURVE OF A NEWTONIAN BODY

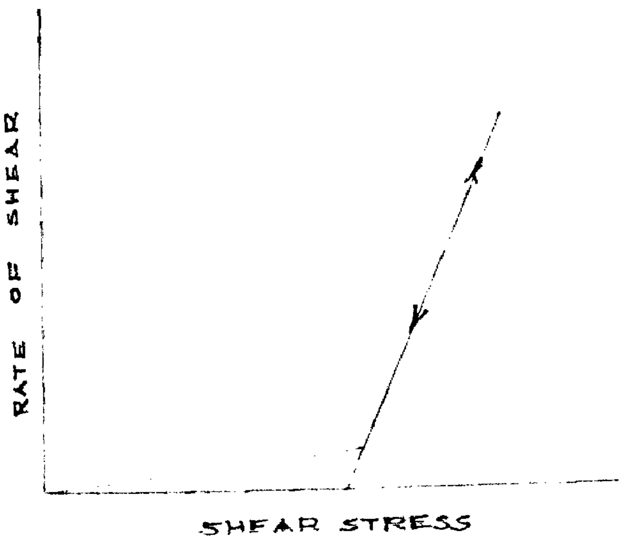


FIG: 4·5

CONSISTENCY CURVE OF A BINGHAM BODY.

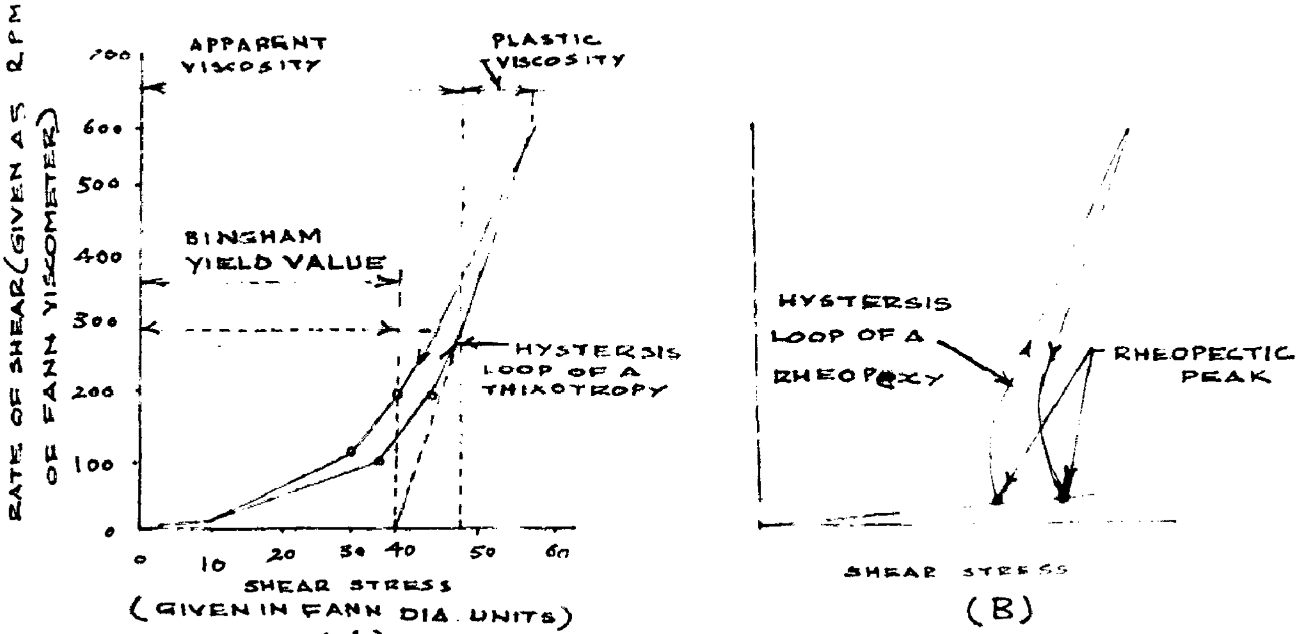


FIG: 4.6
TYPICAL CONSISTENCY CURVE

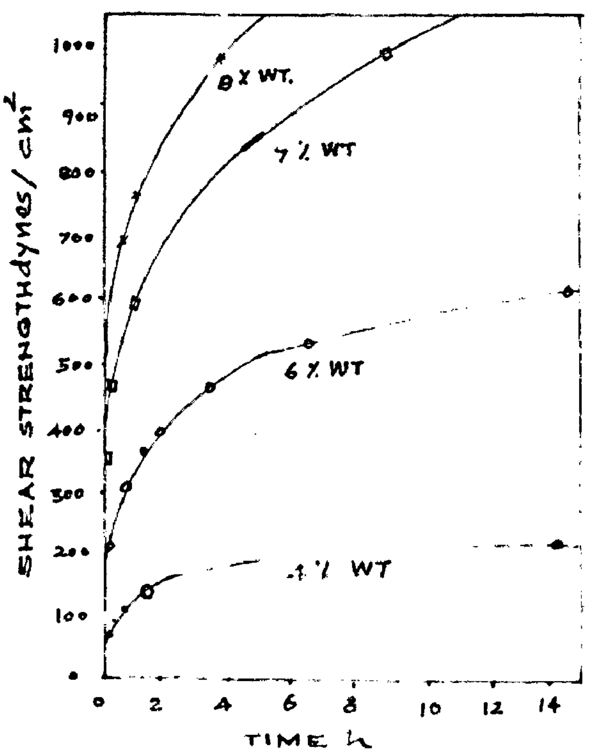


FIG: 4.8
THIXOTROPIC GELATION OF BENTONITE SUSPENSION IN WATER

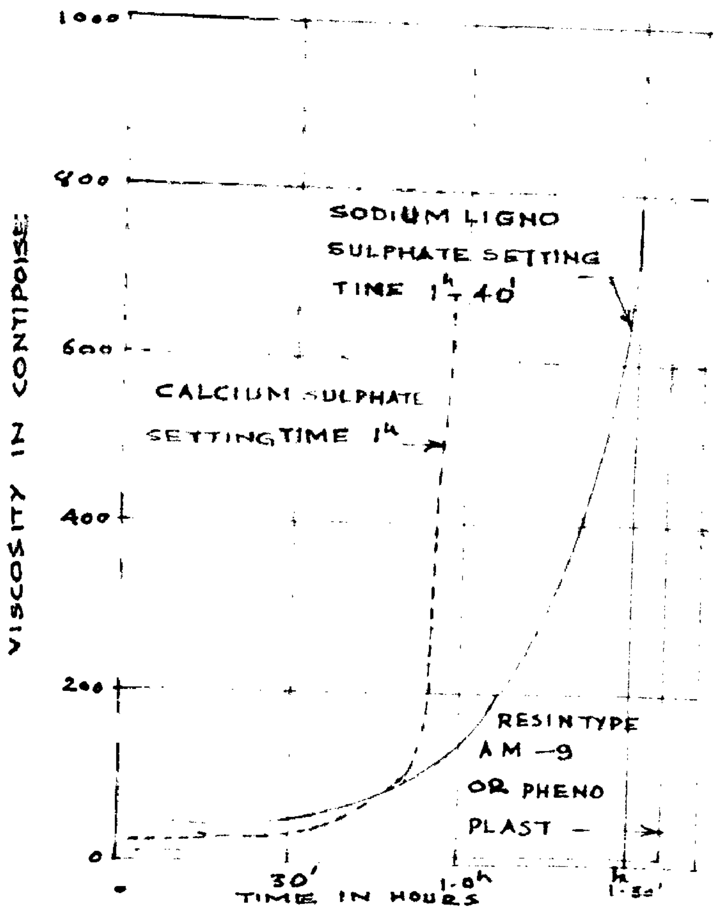


FIG: 4.9
THE VISCOSITY TIME REALATIONSHIP OF A LIGNO CHROME GROUT

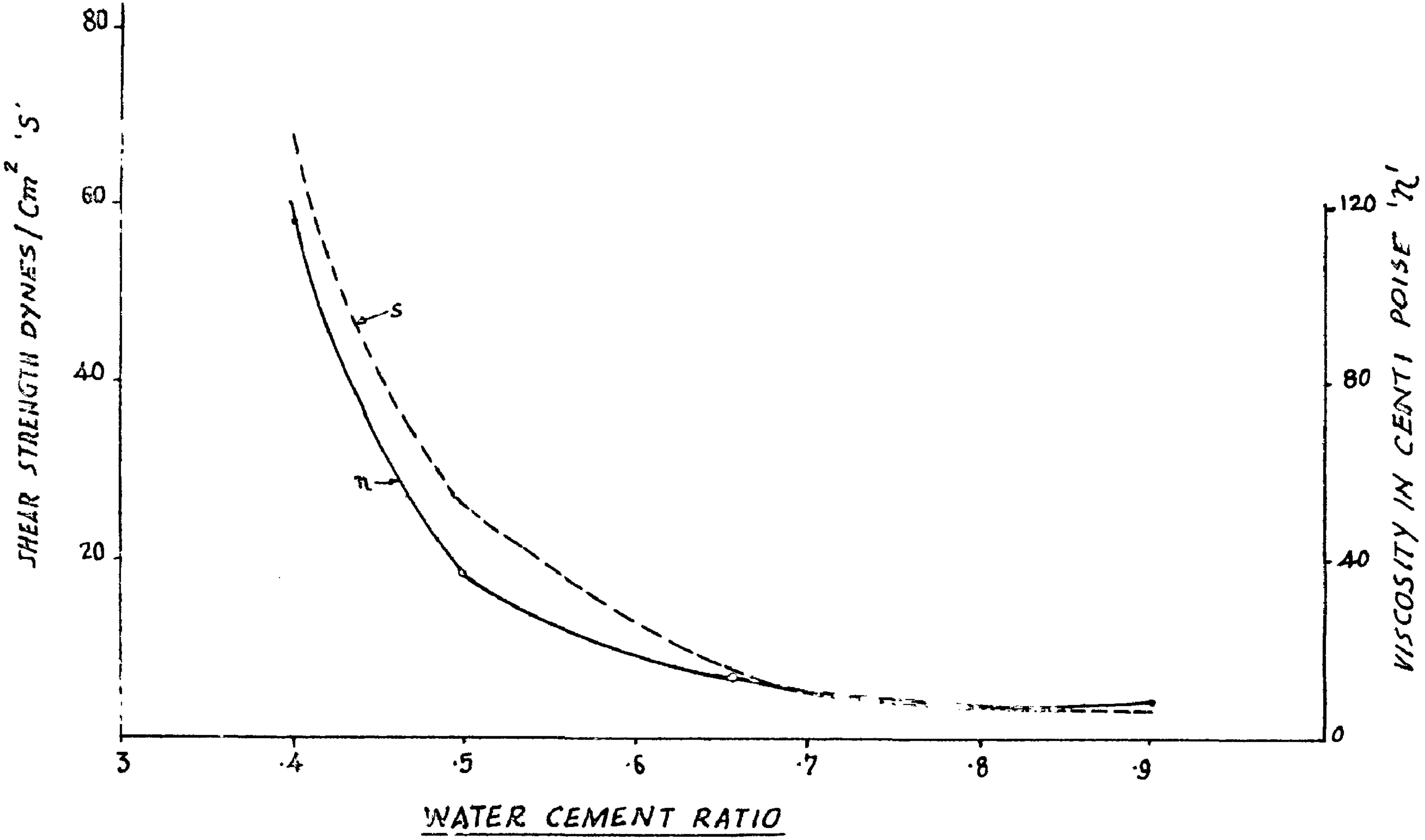


FIG. 4·7

EFFECT OF WATER CEMENT RATIO ON REHEOLOGICAL CHARACTERISTIC OF CEMENT GROUTS

CHAPTER - 5

MECHANICS OF GROUT PENETRATION

5.1. GENERAL

There are two basic factors which govern the penetrability of grout:

(1) Viscosity of the grout, and

(2) Granulometry of the grout and the soil.

It is useful to develop quantitative measurement for ascertaining the penetrability of material. However, it must not be forgotten that a structure of soil skeleton is very erratic. The shape of pores is irregular and the penetration is tremendously influenced by the presence of few open pockets. Hence the general rule stated here should not be taken as absolute criteria and field trial is always necessary before planning injection programme.

5.2. EFFECT OF VISCOSITY ON PENETRATION:

5.2.1. The seepage of water through soil formation under dams or into wells is usually described and evaluated by flownet theory and follows Darcy's law:

$$Q = KAi \quad \text{..................(1)}$$

Where Q is rate of flow

K is permeability of soil

A is area through which flow takes place

i is hydraulic gradient.

This equation may not be applicable to the grout as viscosity of grout is greater than that of water and the movement of grout through the pore space of a soil filled with water is resisted by drag at the interface between the inner shell of grout and the outer shell of the surrounding water.

Poiseuille has given an equation of viscous flow through a long tube of small diameter:

$$V = \frac{\Pi P r^4 t}{8Ln} \quad \dots\dots\dots\dots\dots\dots \quad (2)$$

where V is the volume of liquid escaping in time.

t is the time for V

P is the pressure differential between tube ends

r is the tube diameter.

L is the length of tube.

n is the viscosity.

Fundamental of this equation is that the liquid flows slowly through a long tube of small diameter. In actual field conditions the soil formation cannot be considered as a bundle of small tubes of long length. Soil formations are generally hetrogeneous and anisotropic and also viscosity does not remain constant.

Maag was the first who in 1938 published his theory of relationship between pressure of pumping, density of viscosity of the grout, the rate of injection, the permeability of the soil and geometry of the flow. His equation is:

$$t = \frac{\alpha n}{3 Khr}(R^3 - r^3) \quad \dots\dots\dots\dots\dots \quad (3)$$

Where t is the time of injection

α is the ratio of viscosity of the grout to that of water

His assumptions were that the soil is homogeneous and isotropic, that the grout is newtonian fluid, that a steady state of flow has been established and that the injection is made through an open ended pipe of small radius compared to the length of the injection pipe below ground water level and that the spherical flow takes place.

In fact these simple conditions are never realised in for the geometry of flow can be complex and the viscosity and the rheological consistency of grout may alter with time. Also this formula is limited to a situation when the influence zone of grout is far from the injection point. The Maag's formula is worthwhile as it gives a clear picture of the progress of an injection.

Raffle and Greenwood[28] (1961) have extended the treatment to cover non-newtonian grouts, both with fixed shear strength (ideal Bingham bodies) and with shear strength growing with time.

5.2.2. <u>Newtonian Grouts</u>: For newtonian liquids the flow of grout is a fully developed flow and as per Darcy's law the flow is

$$\frac{Q}{H} = 4 \Pi a K \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

Where Q is the rate of flow

h is the hydraulic head

K is the permeability of the soil

a is the effective spherical radius.

For the purpose of calculations, cylindrical injection source may be replaced by spherical source of equal area. In most cases, this simplification will lead to an exaggeration of the flow rate because the divergence of the flow near the source is greater in the spherical than in the cylinderical case. This lower divergence leads to a lower hydraulic gradient at the surface of the source and hence to a smaller flow of grout into the soil. This effect is often further exaggerated because of the confining action of nearby tight layers of ground or of ground already treated with grout.

For the flow of grout into water-laden soil, there is a resistence of the inner shell of grout and the outer shell of the surrounding water, the equation (4) requires some modification. Raffle and Greenwood have given an expression for spherically divergent, two phase flow as

$$h = \frac{Q}{4 \Pi K} \left[n\,(1/a - 1/R) + 1/R) \right] \qquad \ldots\ldots(5)$$

Where n is the ratio of viscosity of grout to that of water

R is the radius of grout spread.

and the rate of grout spread is

$$\frac{dR}{dt} = \frac{Q}{4\ \Pi\ R2e} = \frac{Kh}{e} \left[nR^2(1/a - 1/R)+R \right] \qquad \ldots\ldots(6)$$

where e is void ratio of the soil

t is time taken to travel a distance R.

From the above it follows that the time for the interface t reach a radius R is given by

$$t = \frac{e\ a^2}{Kh} \left[n/3 \left(\frac{R^3}{a^3} - 1\right) - \frac{n-1}{2} \left(\frac{R^2}{a^2} - 1\right) \right] \qquad \ldots\ldots(7)$$

The specific value of t can be found from Fig.No.5.1. plotted by Raffle and Greenwood in which $\frac{Kht}{ea2}$ is plotted against R/a for three values of viscosity ratio n. From a knowledge of the time t to penetrate to radius R the value of Q at that time can be calculated from equation (5). Scott[37] (1963) has found out that at early stages of the injection, the grout extends only a little way into the water laden soil. The flow rate is therefore higher, initially, than is predicted by equation(5), but this early phase occupies a relatively small proportion of the total injection time and the mean injection rate does not differ greatly from the value given by the simpler expression.

5.2.3. Non-newtonian grouts: Fig. No.5.1 cannot be used without reservation for such fluids as clay or cement grouts, since these do not obey Darcy's law. For these fluids the shear rate is not related to the shear stress by a viscosity coefficient. The simple newtonian relation fails because of fluids have shear strength, and if it is pumped under constant pressure into the soil, the opposing drag due to the corresponding shear stress acting at the growing area of the surface wetted by grout-ultimately becomes equal to the whole of the applied pressure so that none is available to maintain the viscous flow. Raffle and Greenwood deduce that an extra pressure gradient must be applied at all parts of the advancing grout to overcome the shear strength during injection.

$$i = \frac{2\,Tf}{\alpha} \quad \text{.......................(8)}$$

Where i is the pressure gradient.

Tf is the Bingham yield stress of the grout

α is the effective radius of an average pore passag

The value of α can be calculated from the Kozeny's equation for the relationship in terms of the soil permeability and the viscosity of water as given by the equation

$$\frac{dp}{dx} = \frac{8\,n\,v}{\alpha^2 e} = \frac{Y_w g v}{K} \qquad \ldots\ldots\ldots\ldots(9)$$

Where n is the viscosity of water

Y_w is the density of water

v is the mean flow velocity for unit area in the soil.

Using relation (9) we can tabulate value of α for ground of different permeabilities values and hence estimate the limiting value of pressure gradient for known values of α. Raffle and Greenwood have calculated the value of α for different value of K of the ground and are given in table No.5.1.

Table No. 5.1

Value of α for different values of K

K cm/sec.	α (mm) $e = 0.3$
1	0.19
10^{-1}	0.059
10^{-2}	0.019
10^{-3}	0.0059

Scott[37] (1963) has worked out the component of hydraulic gradient needed to maintain flow of grouts with various yield values in soils of given permeability as given in Table No.5.2

Table No. 5.2

Hydraulic gradient to maintain flow in non-newtonian grouts

Soil permeability cm/sec.	Yield value Tf dynes/cm^2	Minimum hydraulic gradient.
10^0	10	1.2
	100	12
	1000	120
	10000	1200
10^{-1}	10	4
	100	40
	1000	400
	10000	-
10^{-2}	10	12
	100	120
	1000	1200
	10000	-
10^{-3}	10	40
	100	400
	1000	4000
	10000	-
10^{-4}	10	120
	100	1200
	1000	-
	10000	-

5.2.4. <u>Limit of Penetration:</u> Maag's simple equation(3) gives an approximate radius of zone of influence and the ratio between the radius of injection and the time passed and the relationship is shown in Fig.No.5.2. The grout increases in viscosity for sometime before gelling, but it can be maintained within 50% of its initial value for a period of about 20 minutes. The radius of penetration after 20 minutes for various soils based on Fig.No.5.2 is tabulated below:

K cm/sec.	10^{-1}	10^{-2}	10^{-3}	10^{-4}
R cm	4000	400	40	4

Raffle and Greenwood (1961) have shown that in the case of permeation from a spherical source of radius 'a' the grout cannot reach further than a radius RL given by the equation

$$RL - a \quad = \frac{Y_w \, gh\alpha}{2\,S} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

For grouts with constant shear strength and viscosity the initial rate of penetration is determined by the viscosity. When some penetration has taken place, a proportion only of the applied pressure is available for maintaining viscous flow and the rate of penetration correspondingly falls. Expression for the time taken for the grout to reach a radius R short of RL is difficult to determine.

5.2.5. Rate of permeation determined by viscosity: By applying the data of the rheological measurements direct estimates can be made of the rate of progress of injections for the real grouts in practical cases. For the sake of illustration, injection is done from a 3 ft. long open section of bore hole of $1\frac{1}{2}$" dia. The periphical area of the section is $\Pi \times D \times 3$ = 1 sft. approximately, and the radius of a sphere of equivalent area will be 4" $= \left(\frac{1}{4\,\Pi}\right)^{\frac{1}{2}}$. Using the equation (7), the time of penetration of newtonian grouts is worked out and is given in table No.5.3 for the different radius of penetration for grouts for different permeability constants.

Table No.5.3

Time of permeation for different radius of penetration.

	Time in minutes for permeation to specified radius at 100 ft. injection pressure.								
	K = 10-1 cm/sec.			K = 10-2 cm/sec.			K=10-3 cm/sec		
Grout	1ft.	2 ft.	4 ft.	1 ft.	2 ft.	4 ft.	1 ft.	2 ft.	4 ft
AM - 9	0.015	0.12	1.0	0.15	1.2	10	1.5	12	-
Sodium silicate M-75	.35	4.1	38	3.5	41	-	35	-	-

5.2.6. Penetration limited by shear strength: For grouts possessing shear strength the limiting penetration for the different values of K is given in the Table No.5.4 calculated by expression (10):

Table No. 5.4

Limiting penetration of Non-newtonian liquids

Shear strength dynes/cm^2	Limiting penetration for 100 ft. of injection head (ft)		
	K = 1 cm/sec.	K = 10^{-1} cm/sec.	K = 10^{-2} cm/sec.
67.6	14.1	4.68	1.7
25.6		11.73	3.9
6.6			14.3

For ordinary portland cement the calculations cannot profitably be pursued for less than K = 1 cm/sec, that is very open ground, because permeation is thereafter limited by the direct blockage of voids by the larger particles of cement. For such coarse soil, the shear strength has no important effect in limiting penetration. Fine grained cement, can be produced with few particles larger than 20 microns and can therefore penetrate into fine textured soil. However, such cement forms paste which possesses even higher shear strength of 70 dynes/cm^2 and has a limiting penetration of less than 2 ft. in soil of permeability 10^{-3} cm/sec and reaches 0.7 of this distance in 180 minutes.

5.3. GRANULOMETRY OF THE GROUT AND THE SOIL

Data on sieve analysis, blow counts, water table and boulders or rock levels are most often available from soil sample tests during the planning stage of a job. It is the job of the engineer to interpret the data and translate the sieve analysis and blow counts into meaningfull terms of pore size and groutability ratio. The ability of a grout to penetrate the voids of a formation is of paramount importance in considering grouting of pervious foundations to decrease seepage beneath structures. While the penetration of suspension grouts depends on the nature of the contact surface of the solid particles and their concentration and other similar features, the primary factor involved is the size of the voids being grouted as compared to the size of the solid$ particles in the grout. Tests show that it is the smallest voids in the formation being grouted and likewise that the largest sizes of the solid particles in the grout prevents the finer size from entering the formation being grouted. These factors suggested that a "groutability ratio" might be developed relating the properties of the formation to those of the solid particles in the grout.

It can be demonstrated that there is a definite relationship between the size of the grains in a system of packed rock particles and the effective pore diameter or "critical ratio of entrance". King and Bush[18] (1961) have demonstrated from series of tests on single size aggregates made up of crushed aggregates and pea gravel that the critical ratio of entrance or pore diameter may vary from 15% of the diameter of sphere at 26% void ratio to 41% at 48% void ratio, as shown in Fig.No.6.3, by assuming sand grains to be spherical. Most naturally occuring sands or gravels in old river beds are sufficiently abraded to be considered as spheroids for this purpose and will be found to have an outside range of void ratio in place from 30% to 40% with a fair larger number in the range of 35% to 38% due to tight packing resulting from amixture of sizes. From figure No.5.3 it is seen that this range of porosity corresponds to a net pore diameter of 19.5% to 21.5% of the diameter of the large sphere. Different types of aggregates and mixed grading will give widely varying pore ratios. For design purposes we may assume pore diameter of 20% of the grain size. King and Bush have demonstrated that the pore size should be atleast three times the effective maximum grain size of the grout material. This stems from the assumption that two grains of cement or sand wedged across the opening of a pore will form a stable bridge, but that an arch formed by three grains will not be stable. In other words, if the diameter of grouting particles is not over $1/5 \times 1/3 = 1/15$ of the particle diameter of the material to be grouted, the grout will be filtered out. The theory of the penetration of grout is just reverse of the theory of filter by Terzaghi where ratio of D15 filter and d85 of base material should be less than 5 to prevent choking of filter material. The corps of Engineers Waterways Experiment Station, Vicksberg, Mississipi[38] (1955) have proposed on the basis of experimental data that grouting could be successfully accomplished if the groutability ratio were greater than $N = 15 = \frac{D15}{d85}$ where 'D' represents for alluvium particles and 'd' the size for the grout material. Suffixes such as 15 and 85 indicates the percentage of material finer than the sizes indicated.

Some believe that validity of this criteria is open to question for a grout with swelling tendency of particles of clay and bentonite which absorb lot of water and behave during penetration like a particle of much larger size and that a grout with d85 size smaller than 50 microns, satisfactory penetration may not be possible even for the groutability ratio of D15/d85 = 30. This requirement illustrates why suspension type grouts are normally limited to grouting medium sands and coarse grained soils. It is not enough to rely solely on a single point of granulometry curves for the alluvium and for the grout. It is therefore necessary to check the ratio D10/d95. King and Bush[18] (1961) have worked out this ratio as equal to 8. They have plotted the values of D10/d95 of different grouts as shown in Fig.No.5.4. These values are lower than D15/d85 but never greater than 8. Some believe that successful injections have been reported for values of 6 to 8. But the feasibility of grouting with lower values of 6 is open to question because when N approaches 6, it is known that filtering starts to take place and it will give guarantee that the soil cannot be grouted. Sometimes, the alluvium consists of a mixture of two materials resulting in a gap in a certain particle size range. It would be desirable to divide the alluvium into components when a gap in the grouting is noticed. When the voids in the coarse fraction are adequately filled by the fine fraction, the granulometry of the fine fraction will govern the groutability. On the other hand when number of voids in the coarse components are left unfilled, the groutability will be decided by the granulometry of the coarse component.

Grout containing even a very small proportion of coarse particles can form a tight filter cake in the soil face near the injection source, so that the flow rate drops to a negligible value. Scott has given that for steeply graded soil, particles larger than about 1/10th of the soil particle dimensions are trapped, if a few percent of the solid material of the grout is of this size. He has also calculated the largest size of the grout for given formation from equation (9) as Dmax of grout less than $\left(\frac{32\,nk}{e\,\gamma_w g}\right)^{\frac{1}{2}}$.

5.4. INJECTION PRESSURES

5.4.1. The determination of the allowable pressures is an essential factor in the design of a grouting programme. The use of excess pressures may weaken the strata by fissuring the rock or by opening fissures in closely jointed rock. If excess pressures are used in grouting below a foundation or a concrete cutoff in a dam, the concrete may be lifted from its seat. This would involve an uneconomical consumption of grout, induced by the grouting process itself. The damage may, of course, be much worse and result in a permanent weakening or increase in permeability. However, it must also be recognized that the pressure should be kept sufficiently high to minimize the time required to perform the grouting operations. Spacing of holes can be increased which reduce the cost of drilling. There are various criterian in common use to estimate the allowable injection pressures. The common practice is to adopt, initially, an allowable pressure of 1 lbs/sq.in. per foot of overburden cover or 0.2 kg/cm^2 per meter of overburden cover and adjust the pressure on the site as performance records become available. Grundy[2] (1955) suggests that an injection pressure corresponding to approximately double the weight of the overburden above the section being grouted may be used. Lippold[39] (1958) has stated that the safe allowable pressure will vary from 3/4 lbs/sq.in.

per foot of cover to 2½ lb/sq.in. and will depend upon the characteristics of the overlying materials. Burwell[40] (1958) suggests that the guide given by Creager,Justin and Hinds(1945) given in Fig.No.5.5 may be used to obtain an approximate estimate of the allowable pressures for various rock foundations. Zoruba (1962) has recently reviewed some other pressure specifications and shows them to be at great variance with each other. The Russian code is mentioned by Zoruba and in this code it is stated that pressures are always at least equal to the weight of the overburden above the bottom of the bore hole and higher pressures are allowed with more massive rock, with more viscous grouts and upon regrouting a hole. Zoruba also states that in the construction of some Italian dams injection pressures equal to four times the overburden pressures have been used. Very widely varying pressures have been used successfully in very different geological circumstances.

The permissible pressures for grouting the alluvium need not be governed by the overburden weight alone. After a small embankment takes place in the vicinity of the point of injection considerable shearing resistance is mobilised by arching. The heave required in such resistance to be mobilised is often permissible. Thus pressures upto 3 to 5 times the overburden weight can be used. The pumping pressures for AM-9 solution should always exceed the static water head at the point of placing, as it is seen that in any formation the gel can not be extended at pressures less than the pumping pressures required to place the solution. For most conditions specially fine sand and silt the gel can resist much higher pressures than that at which it was placed. Continental practice is to save on drilling and obtain acceptable foundation by high grouting pressures.

It is evident that it would not be possible to have a single set of specifications that could encompass the different geological and ground water conditions that are met in civil engineering works. Nevertheless, it would be useful to establish the theoretical mechanism by which fracture is induced in the rock under high injection pressures. These considerations will also suggest procedures for determining, in the field, the allowable grouting pressure for any particular project.

5.4.2. <u>Isotropic conditions:</u> The fracture of the material is assumed to follow the Coulomb Mohr failure, i.e.

$$\frac{\sigma_1' + \sigma_3'}{2} \sin \phi' = \frac{\sigma_1' - \sigma_3'}{2} - c' \cos \phi' \quad \ldots\ldots(1)$$

Where σ_1' and σ_3 are the major and minor principal stresses respectively.

ϕ' is the angle of shearing resistance

c' is the cohesion.

Vertical stress may be taken as a major principal stress and only penetrating fluids may be assumed. The effective stress before injection will be

$$\sigma_1' = rh - Ywhw \quad \ldots\ldots\ldots\ldots(2)$$

$$\text{and } \sigma_3' = K \sigma_1' = K\,(yh - ywhw) \quad \ldots.(3)$$

where Y is bulk density of the overburden

h is the height of the material above the point

hw is the height of water level above the point.

K is the principal stress ratio taken equal to or less than 1.

rw is the bulk density of water.

Increasing the pressure in the bore hole by Fe, the excess injection pressure, reduces the effective stresses by an equivalent amount so that at fracture the principal effective stresses become

$$\sigma_1' = (rh - rwhw) - Pe \quad \ldots\ldots\ldots\ldots\ldots (4)$$

$$\sigma_3' = K(rh-rwhw) - Pe \quad \ldots\ldots\ldots\ldots\ldots (5)$$

By substituting equation (4) and (5) into equation(1) it can be shown that

$$Pe = \frac{(rh-rwhw)(1+K)}{2} - \frac{(rh-rwhw)(1-K)}{2 \sin \phi'} + c' \cot \phi' \ldots\ldots(6)$$

If the major principal stress is horizontal the principal stresses at fracture are

$$\sigma_1' = K(rh - rwhw) - Pe \quad \ldots\ldots\ldots\ldots\ldots\ldots(7)$$

and $$\sigma_3' = (rh-rwhw) - Pe \quad \ldots\ldots\ldots\ldots\ldots\ldots(8)$$

Where K is greater than or equal to 1.

The excess injection pressure is given by

$$Pe = \frac{(rw-rwhw)(1+K)}{2} - \frac{(rh-rwhw)\ (K-1)}{2 \sin \phi'} + c' \cot \phi' \ldots\ldots(9)$$

If the shear stress parameters, in situ stresses and water presures are known, equation (6) and (9) could be used to predict the excess injection pressure. Neglecting friction loss in the pipe the critical injection pressure would be

$$P = Pe + rwhw \quad \ldots\ldots\ldots\ldots\ldots(10)$$

The equations (6) and (9) indicates that the grouting pressure increases with depth provided that the shear strength parameters remain constant. It can also be determined that the common specifications of 1 lbs/sq.in. per foot depth applies to a material with no tensile strength and a principal stress ratio of unity. Furthermore, the allowable injection pressure as measured at the pump cannot exceed the overburden pressure unless the material has some cohesion.

5.4.3. Anisotropic cases: When the formations consists of bedding planes, it is possible that fracture would be induced along such planes if the shear strength is sufficiently lower than that of the bulk material. Morgenstern and Vaughan[41] (1963) have found out that when the weak planes are horizontal or when the minor principal stress is equal to the overburden pressure then

$$-\sigma_1^{\prime} = C' \cot \phi' \quad \ldots\ldots\ldots\ldots\ldots \quad (11)$$

and at fracture

$$\sigma_1^{\prime} = rh - rwhw - Pe \quad \ldots\ldots\ldots\ldots \quad (12)$$

and P = Pe + rwhw

$$= rh + C' \operatorname{Cot} \phi' \quad \ldots\ldots\ldots\ldots \quad (13)$$

$$\text{Therefore} \quad \frac{P}{rh} = 1 + \frac{C' \operatorname{Cot} \phi'}{rh} \quad \ldots\ldots\ldots \quad (14)$$

In actual practice in situ stresses and strength parameters are difficult to determine and problem is further complicated by the presence of stratifications, jointing and changes in tensile strength with depth. Therefore, the most reliable method for finding the allowable injection pressure will be based upon field tests.

5.5. SYSTEMS OF GROUTING

5.5.1. Soil formations may be homogeneous deposits with relatively constant permeability but are more often stratified and lensed. It is well known that permeability is often markedly greater in the horizontal than in the vertical direction and the mean permeability frequently changes by several orders of magnitude between strata, particularly in alluvial deposits such as river gravels. The spread of grout naturally follows the easiest paths. Early phases of injection fill only the more open strata, later phases treat progressively finer and finer soils. There are two distinct methods of procedure, each with its own field of application, these are permeation and fracturing.

5.5.2. Permeation: In permeation treatment the aim is to displace the void water uniformly by the steady outward progression of the grout. If treatment is to be effective, injection pressures must not be large enough to displace the soil particles. Hole positions and stage depths are chosen so that grout from one stage complements that from adjoining stages to form an integrated mass of grouted soil. The sequence and volume of the individual injections are designed to fill open soil effectively before treatment of the finer soil. The grout must clearly be fluid enough to penetrate fine soil at speed and yet not move from position too quickly in coarse soil. In permeation treatment a clear distinction exists between one shot and two shot systems. One shot grouts penetrates as fluids and set in place when the gellation period has elapsed. Two shot grouts depend on the meeting and intermixing of separate fluids pumped into the soil, with an immediate setting action. Certain special conditions must apply if a one shot grout is to get with a well defined, simple boundary. Other quite opposed conditions apply if the fluids of a two shot grouts are to interpenetrate sufficiently for reactive products to be distributed in depth. The criterion for intermixing in depth is that the advancing interface must be intrinsically unstable.

5.5.3. <u>Fracturing</u>: In fracturing treatment the grout cuts fissures and channels in the soil and runs until it finds voids which can be filled by permeation. Fracturing is sometimes employed in the early phases of the grouting treatment of heterogeneous formations, so as to fill isolated lenses and layers of open soil from a widely spaced injection holes (Cambefort, 1961). It is also used for the treatment of formations that are barely permeable by the grout. The injection pressure required for fracturing is usually well above the overburden pressure, except when open lenses are overlain by shallow, less permeable material or where, for example, a compressible silt lies below a formation having arching strength. Compressible silts, which can scarcely be grouted by the most fluid grouts at an economic rate, can be consolidated by fracturing injection, and this practice is frequently adopted to give strength to water-laden beds encountered at depth during shaft sinking operations. Permeation is the primary objective in grouting operations, but fracturing consolidates compressible soils and both strengthens and reduces seepage in heterogeneous soils by filling inter-seated open layers and zones with grout. This method is also known as 'Claquage' used in the treatment of fine grained soils, in which the grout penetrates as a tounge through planes of weakness in the soil and thereby changes the overall conditions as the result of the effect of compaction, and, at the same time, seals off volume of soil by surrounding them with comparatively impermeable membrane.

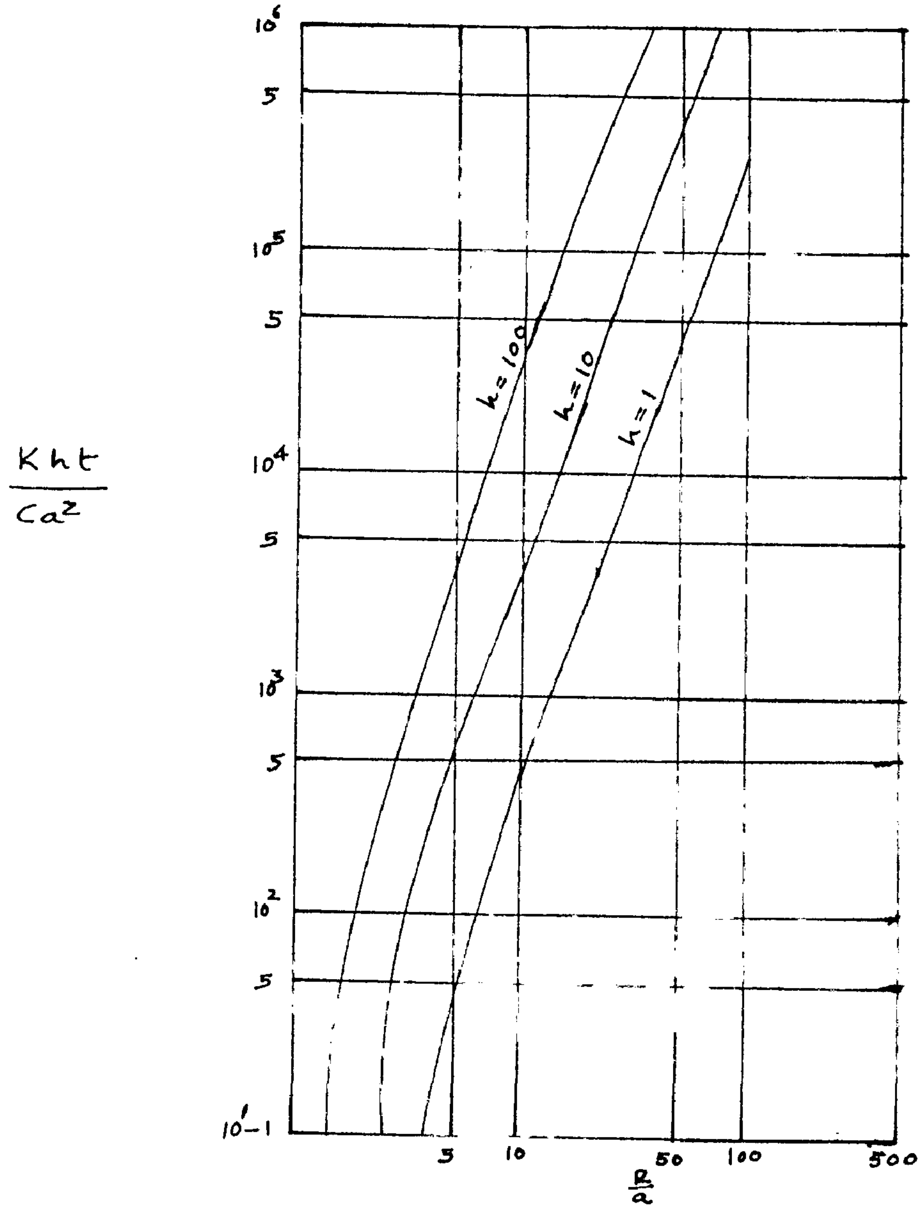
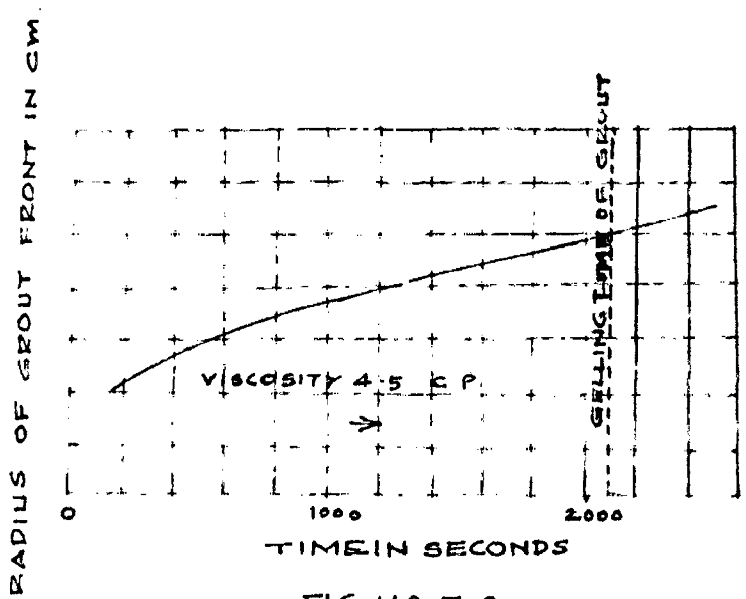


FIG: 5·1

PLOT OF THE DIMENSION-LESS PARAMETER $\frac{Kht}{ea^2}$ AGAINST R/a FOR THE CASES $n=1$, $n=10$, $n=100$



DATA

VISCOSITY 3.0 C.P.

PRESSURE 100 P.S.I

SPHERICAL FLOW ASSUMED

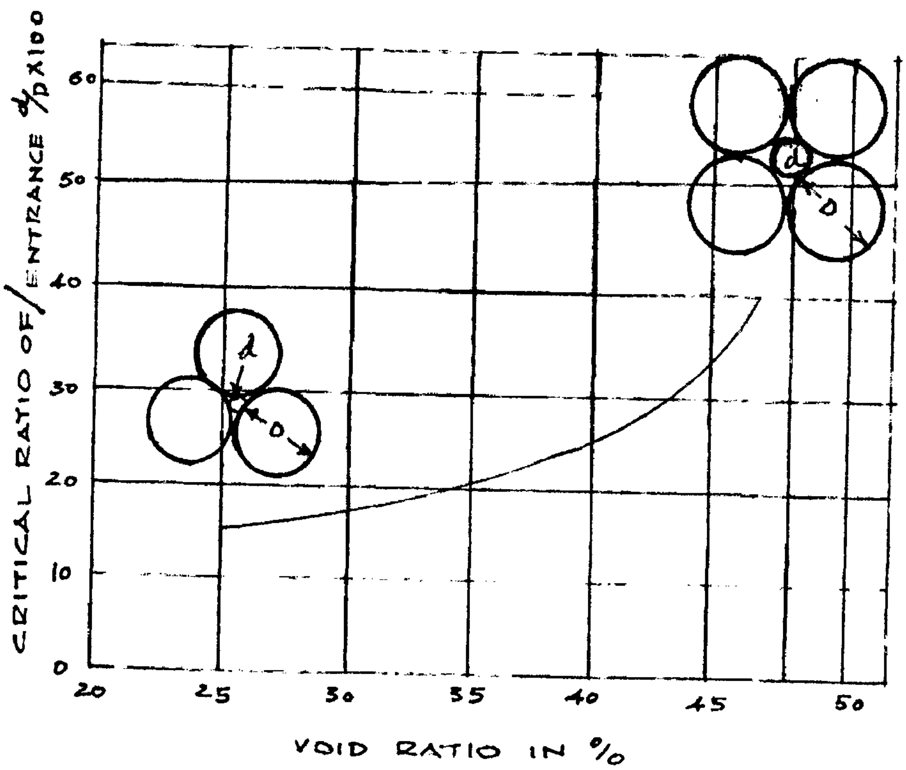
FROM ORIGIN RADIUS 2.5 CM

$K = 10^{-3}$ cm/sc.

$n = 0.3$

FIG NO 5.2

PENETRATION OF GROUT



PROTTED BY KING, J.C., FROM LABORATARY TESTS ON CYLINDERS 6" X 12" FILLED WITH COARSE SAND & GRAVEL. PROC ASCE VOL. 87. NO SM 2. APRIL 1961

FIG. 5.3.

SYSTEMATIC PACKING OF SPHERES.

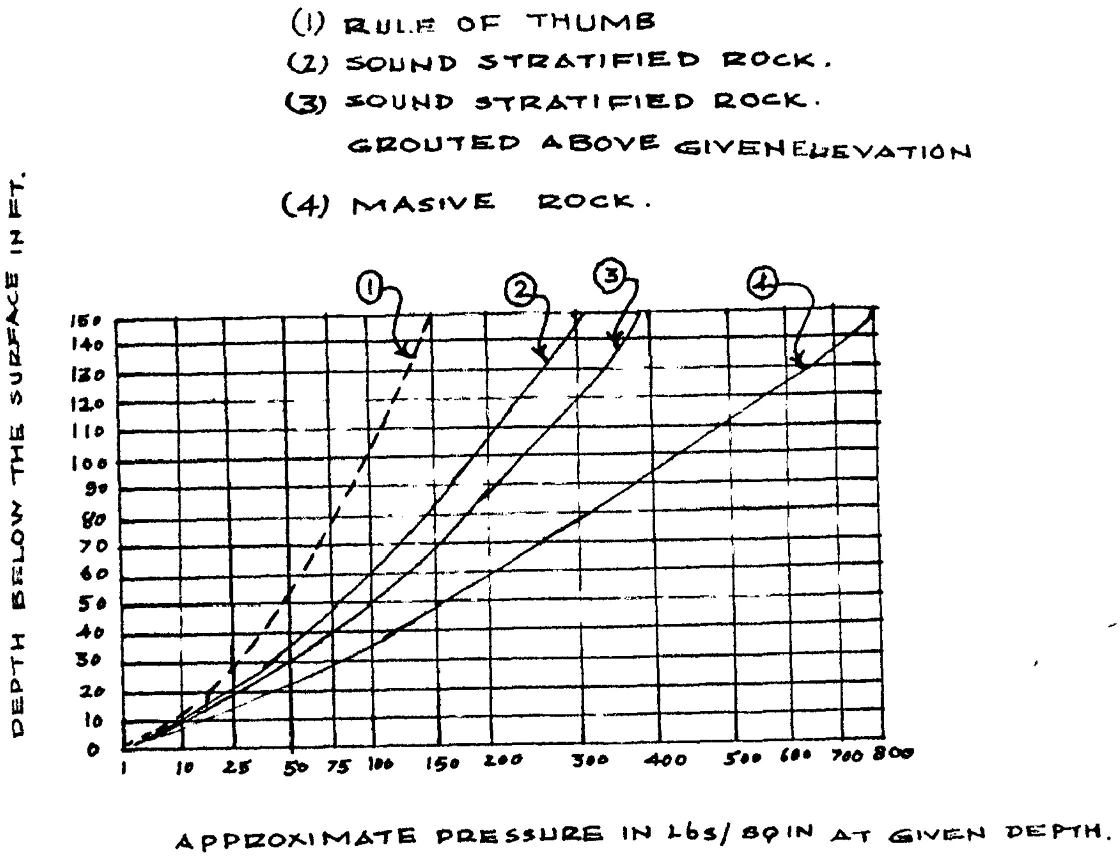
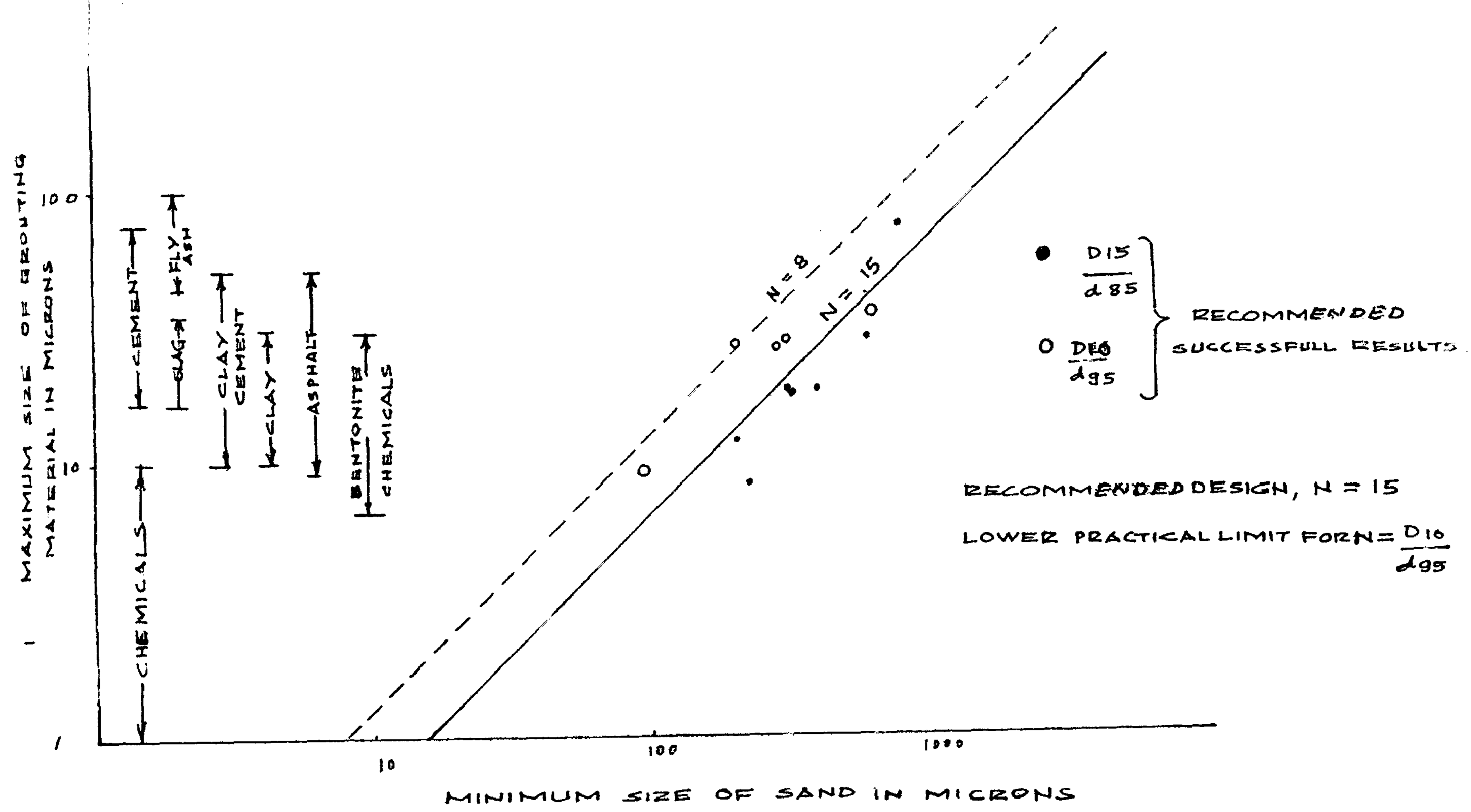


FIG: 5.5

ROUGH GUIDE FOR INJECTION PRESSURES



MAXIMUM SIZE OF GROUTING MATERIAL IN MICRONS
100
10
1
CEMENT
SLAG
FLY ASH
CLAY CEMENT
CLAY
ASPHALT
BENTONITE CHEMICALS
CHEMICALS
N = 8
N = 15
10
100
1000
MINIMUM SIZE OF SAND IN MICRONS
D15 / d85
DEO / d95
RECOMMENDED SUCCESSFULL RESULTS
RECOMMENDED DESIGN, N = 15
LOWER PRACTICAL LIMIT FOR N = D10 / d95

FIG:— 5.4

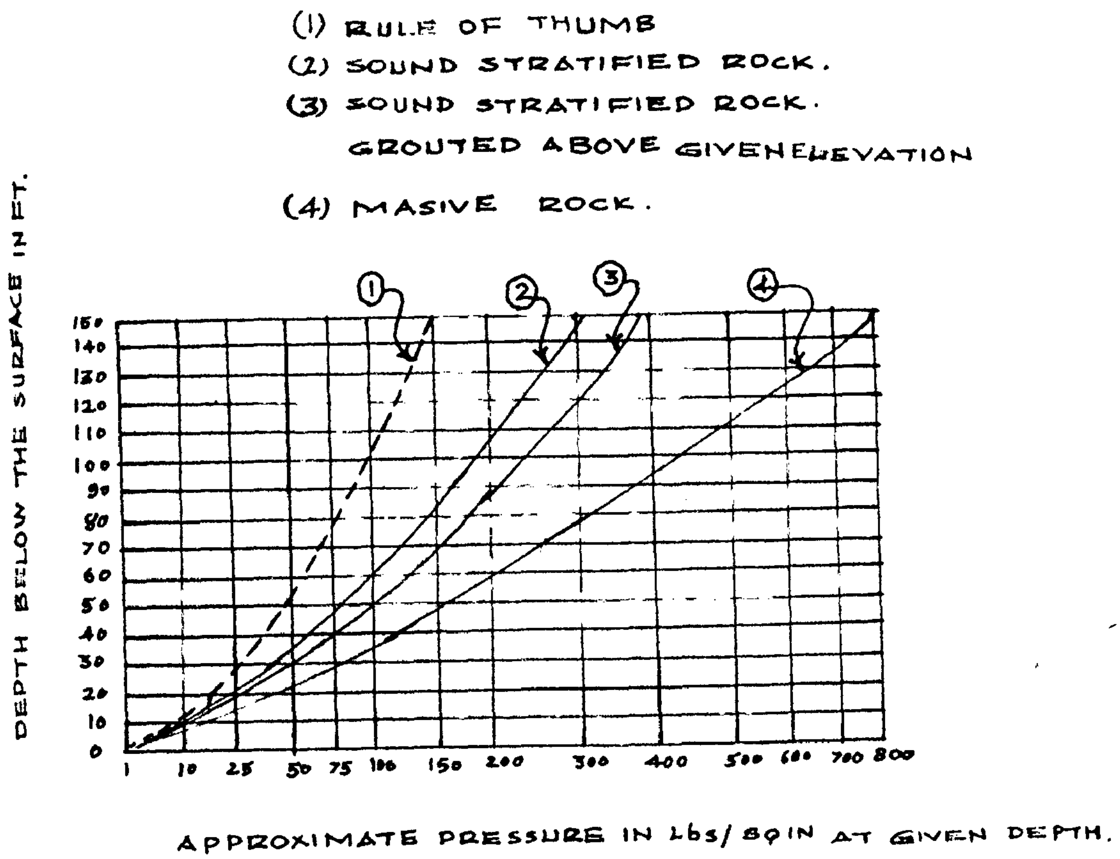


FIG: 5.5

ROUGH GUIDE FOR INJECTION PRESSURES

CHAPTER - 6

GROUTING PROGRAMME

6.1. GENERAL

The success of grouting programme of a given pressure grouting work depends upon

(1) The selection of suitable grout compatible with the site conditions coupled with economy.

(2) Design of grout mix in the laboratory for a given site condition.

(3) Field grouting test to determine suitable grout mix and allowable pressures.

(4) Efficiency of the grout curtain.

No single set of physical properties can characterize the ideal grout. Fluid grouts speed up work in fine ground, but may fall too fast in open ground. For consolidation of fine soils, choice depends on the strength required because high strength usually comes from undiluted grouts which are viscous and slow to penetrate. The most wide-spread field for chemical grouts is medium and fine sand and sandy gravels(permeability 10^{-1} to 10^{-4} cm/sec.) either to reduce permeability or to give modest consolidation strength sufficient to carry loads coming upon them. In more open soils grouts may have viscosity of 10 cp or more without disadvantage, except when close to the surface, where pressures must be kept low. Chemical grouts for this region must be cheap if they are to compete with clay grouts and clay-cement grouts, however, in finer soils (K less than 10^{-2} cm/sec.), pumping time becomes increasingly long and grouts of water-like viscosity are in demand. The gel time of grouts injected into soil should be controlable to match a variety of pumping rates and penetration distance. Injected grout does not then lie ungelled and mobile for unnecessarily long periods, in the ground. Grouts for impermeabilization do not need high strength. A coarse soil with a permeability of 1 cm/sec. filled with a grout with a shear strength of 981 poise will withstand hydraulic gradients of 100, and even less strength is needed in finer soils, and weak gels will therefore serve. Many of the chemical gels formed from relatively dilute solutions will give to a soil the strength of a weakly cemented sand. Very much higher strength can be achieved by using highly concentrated polymer grouts. In consolidation grouting the function of the grout is to add cohesion. For certain hydrogel grouts (e.g. AM-9 and TDM chrome lignin) the strength of well compacted soil behaves approximately according to Coulomb's law.

6.2. SELECTION OF A GROUT

6.2.1. The selection of a grout involves a balance between the various desired characteristics such as particle size, viscosity, strength, stability and permanence coupled with economy. The selection is also dependent on the spacing of grout holes and injection pressures. The most penetrable grouts may not often be economical and durable. For instance, the finer and more fluid grouts such as bentonite, silicates or chemical grouts may penetrate well but the weak gels filled in the large pore spaces may be vulnerable to erosion. The coarsest zones of the foundation, therefore, require the strongest gel. In the multiple line system of grout holes, generally adopted, the coarsest grout is first injected in the outer rows in order to ensure that the larger pore spaces are filled with stronger gels.

When selecting the types of grout to be used for a given job, the following basic points should be considered:

1. The nature of the soil or rock foundation to be treated.
2. The purpose of the grouting.
3. Alternative and supplementry methods.
4. The plan of injection including gel times, quantities and consumption and placement of injection points.
5. The application and equipments.
6. Economics.

6.2.2. Nature of the formation: The preliminary study which should be undertaken in the field prior to the commencement of the main alluvial grouting works falls broadly into two main parts. The first is the preliminary site investigation which establishes the basic need or otherwise, for a solution through the medium of alluvium grouting and the second is to demonstrate the application of this established method of grouting to the particular problem after consideration by carrying out a field trial to determine certain characteristics essential to the satisfactory execution of the work. The extent of this part of the study depends on the available data and the necessity to establish an appropriate specifications for the site. For the preliminary site investigation, it is important to establish, as closely as possible, the amount of voids and the nature, size and distribution of aggregates by analysing the samples in the laboratory. Such samples can also be used to get a general understanding of the permeability and hence the range of pumping rates and pressures which may be required. Naturally, the formation beds, the location of the water table and the rate at which water may be flowing are all important in establishing the nature of the formation. The rate of flowing water may be established by dye tests or actual pumping tests, in the field.

6.2.3. Purpose of the grouting: Any grouting programme must be planned with full understanding of what is to be accomplished. For example, if the purpose of the grouting is consolidation, then depending on the strength, permanency and the other qualities required from the grout, inert materials such as sand and clay can be readily discarded or used only in small proportions. Similarly, if the material to be grouted is a granular deposits requiring impermeabilization, then the use of several grouts such as chemicals, clay, cement, clay-sand cement must be anticipated from the start.

6.2.4. Alternative and supplementry methods: In a rock foundation, both wide open faults and hair cracks may be found, in a river bed deposits, coarse gravel with free flowing underground water may be interbedded with lenses of fine sand. In a grouting programme, therefore, the engineer should foresee the use of several types of grouts in order to meet any changes, sometimes very erratic, in the foundation. The range of grouts to be considered for a grouting programme are given by Kravetz[30] (1958) as shown in Fig.No.6.1. Scott[42] (1958) has refined the chart prepared by Kravetz and the same is shown in Fig.No.6.2 incorporating an additional data. According to Scott the minimum grain size or crack width that can be grouted with a given material, depends on material and field technique.

Caron[34] (1963) has summarized the work of professor B.P.Askalohoff, of the University of Moscow as follows:

Class A: Coarse sand (greater than 0.8 mm) injectable by suspension containing particles of the order of 50 microns.

Class B: Medium sand (from 0.1 mm to 0.8 mm) injectable by colloidal solution.

Class C: Extremely fine sands and silts, injectable by true newtonian solutions of low viscosity.

His studies show that:

(1) Coarse suspensions are, in general, Bingham bodies i.e. they possess rigidity.

(2) Grouts formed by colloidal solutions are newtonian and their viscosity increases with time as shown in Fig.No.4.3. and

(3) Mixtures based on organic monomers, also newtonian, have a viscosity which does not vary with time.

It is observed that the compressive strength of soil treated with cement or clay-cement suspension (Class A) is independent of the grain size of the soil, while that of soil treated by products in Class B and C is greater as the soil becomes finer. These latter products behave somewhat as adhesives. Study of the grouts in Class B and C must include not only the characteristics of the pure product, but also the behaviour of the product in the soil.

Datye[19] (1963) has given the range of application of different types of grouts as given in Table No.6.1.

(i) Analysis of grouting diagrams

(ii) Study of the evolution of permeability.

(iii) Watching of the possible uplift of the ground.

(iv) Observing the characteristics of the treated soil.

(v) In-situ observations from open pit excavation or from inspection shaft driven.

(vi) Geotechnical tests.

The quality control staff guides the supervising staff regarding the quality of material and mix to be injected in the grout hole. Quality control staff performs tests in two places:

(i) In the field, and

(ii) In the laboratory.

Tests performed for various types of grout are as below. Allowance may be made between the field testing and laboratory testing due to variation in temperature. In some of the chemical grouts the effect of temperature variation is significant.

<u>Cement Based grouts:</u>

(i) Field tests:

Bleeding at 24 hours.

(ii) Laboratory tests:

Quality: To conform to I.S.-269 1951.

Strength:

<u>Clay-cement-bentonite grout:</u>

(i) Field tests:

Clay slurry SP:gr.

Combined slurry: SP:gr, Marsh cone value.

Bleeding at 24 hours. Not more than 5%

Gellification time.

(ii) Laboratory tests:

Cement: Quality to conform to IS-269-1951

Bentonite: Atturberg's limits, clay content, swelling index.

Clay: Mechanical analysis, Atturberg's limits.

D.A. Quality, dispersing effect.

Strength: of grout. Needle penetration resistance test at
1 day
3 days
7 days

Washout tests: For pressure heads 5 to 10 times of normal pressure head to come over grout curtain.

<u>Chemical grouts</u>

(i) Field tests:

Combined suspension SP:gr, Marsh cone value.

Gellification time:

(ii) Laboratory tests:

Sodium silicate: Chemical analysis, SP:Gr.
Alkalinity ratio
$\frac{Nao}{Sio3}$
PH Value

Strength: N.P.R.T. at
1 day
3 days
7 days

Vane shear strength at 7 days.

Washout tests: For pressure heads of 5 to 10 times.

Gelling agents. Chemical analysis.

Resin grouts: Not yet perfected in India.

The results of the various tests performed by the quality control staff are communicated in a tabular form as given in table No.7.4 to the supervising staff for timely action if required.

7.5.2. <u>Frequency of testing:</u> In the field thetests viz; SP:gr of slurry, bleeding and wash out tests shall be carried out for each batch of grout injected in a stage. The needle penetration resistance tests on the samples prepared for bleeding shall be taken in the laboratory for 1 day, 3 days and 7 days and if the results show the strength of grout less than specified, the stage in which grout was injected shall be regrouted to correct it for strength.

In the laboratory the tests on the quality of materials to be used in a grouting programme shall be carried out on each new batch of material arrived and shall be carried out before the injection of such materials in a grout hole. If the tests do not conform to the specifications laid down, the whole batch may be rejected or a grout mix may be redesigned. In case of doubt about the quality of the materials in use such tests shall also be repeated in the laboratory to confirm the quality of material according to specifications.

In using the cement care shall be taken to use fresh cement as old cement loses its strength as much as 50% with age.

7.6. <u>GROUTING DATA</u>

During the progress of drilling and grouting the following records shall be kept upto-date:

The day to day record of drilling and grouting hole wise shall be kept as given in table No.7.5 and 7.6. The table No.7.5 gives the actual working hours, idle hours, reasons for idleness, core recovery, classification of soil of the holes drilled. Table No.7.6 gives the injection pressure, intake, time and the mix proportion used. On the completion of hole these data are plotted graphically showing the water loss (preferably in lugeons) for different stages, core recovery, grout intake in liters or in kg. of solid for each stage and over-all consumption of grout in liters or in kg. of solids. All such holes are preferably plotted with same datum to distinguish between shallow and deep holes.

When the grouting is completed, a plant shall be prepared showing the regular holes as per grout pattern adopted, check holes if taken shall be shown in different colours to distinguish from the regular pattern, deep holes, holes in which water tests are performed, permeability holes, if any, inspection shaft if driven, open pit excavation, if any and the elevation at which grouting is carried out. A longitudinal section of the grouted portion shall also be prepared showing the depth of overburden and rock levels as observed in drill holes and the elevation at which grout curtain is taken down. A consolidated statement shall also be prepared from the register holewise and in the serial order of holes to see that no hole is left out to be drilled and all the stages of the holes are treated properly. If there is any missing of hole or stage the same shall be carried out in the field till then the installation of grouting equipment shall not be allowed to be shifted from the site.

In addition to above day to day record, the following further records shall also be maintained for efficient grouting performance:

(i) Pressure intake curve for each hole stage wise shall be plotted from the registers to know the behaviour of formation to the pressures applied and intake. From these plots a further relationship of core recovery and pressures at which crack occurs and plugging pressure at which hole is completed can also be plotted. This plot will give some indication for further grouting works to determine the grout pressure to be applied without causing any crack after knowing the core recovery or type of formation.

(ii) Records of measurements of heave at the surface as the soil becomes saturated with grout.

(iii) Record of any check hole or test pit which might have been put down to expose the grouted soil for inspection and the results of any laboratory test such as permeability, crushing strength etc. made on samples.

(iv) The results of plate bearing tests on the grouted soil if taken any.

(v) The record of piezometric levels when installed upstream and downstream of the cutoff shall also be kept after the dam is put in service to measure the efficiency of the grout curtain.

Table No.7.4

DAILY PROGRESS REPORT OF TESTS FOR GROUTING BETWEEN

Date	Chainage.	No.of holes grou - ted.	Proportion					Results of Tests							Needle resistance (Gms.per sq.c.m.)				Vane shear tests g/cm² at 7 days.	Remarks (order of adding the ingradients etc.)
			Clay	Bento- nite.	Cement	Water	D.A. %	Marsh cone (second)			Jellifying time		% Blee- ding	Speci- fic gravi- ty.	1 Day	2 Days	3 Days	7 Days.		
								Initial	Atte- raddi- ng D.A.	After 20 Mt. 8b.	Hrs.	Mts.								
1.	2	3	4a	4b	4c	4d	4e	5a	5b	5c	6a	6b	7a	7b	8a	8b	8c	8d	9	10

Table No. 7.5

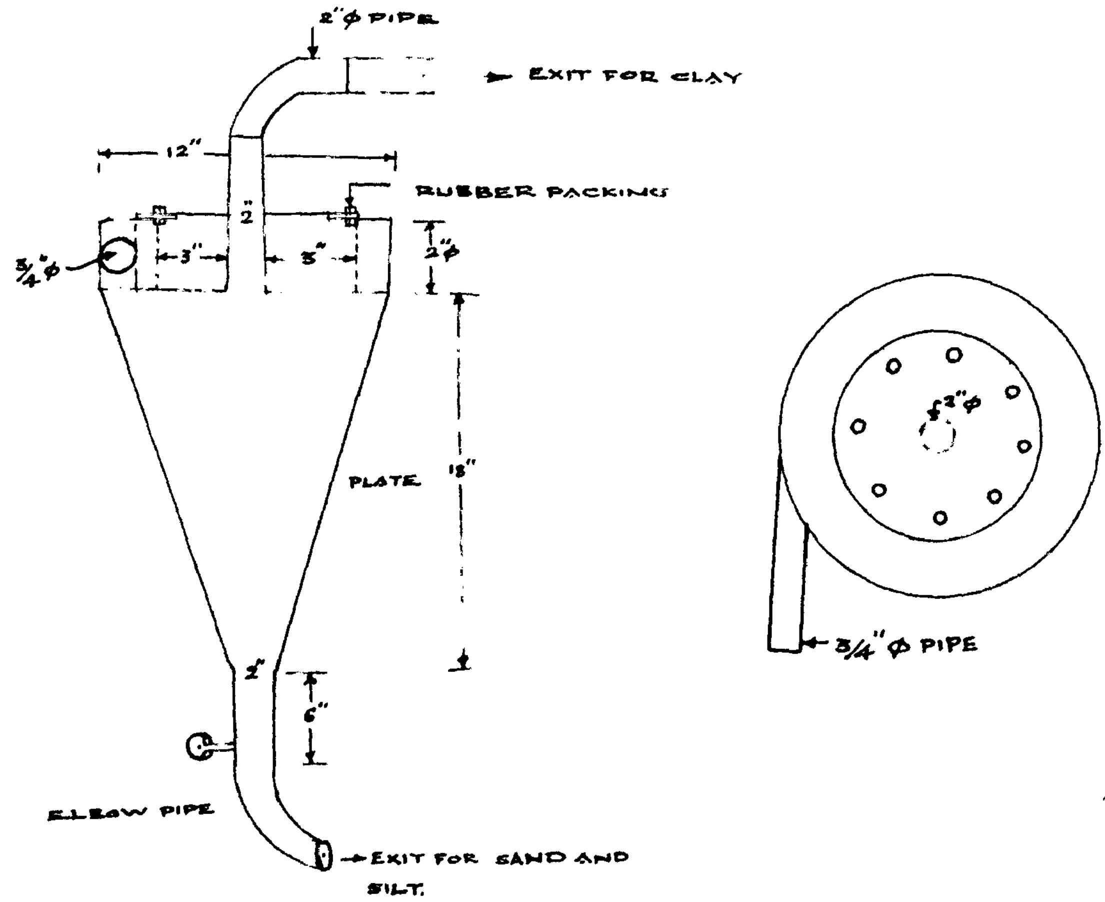
SHOWING THE DRILLING DATA

Date	Shift.	Name of machine.	Working hours			Depth of drill			Recovery.	Classification.	Idle Hours			Reason for idling.	Signatures.	Remarks.
			FM	To	Net	FM	To	Net			FM	To	Net			

Table No. 7.6

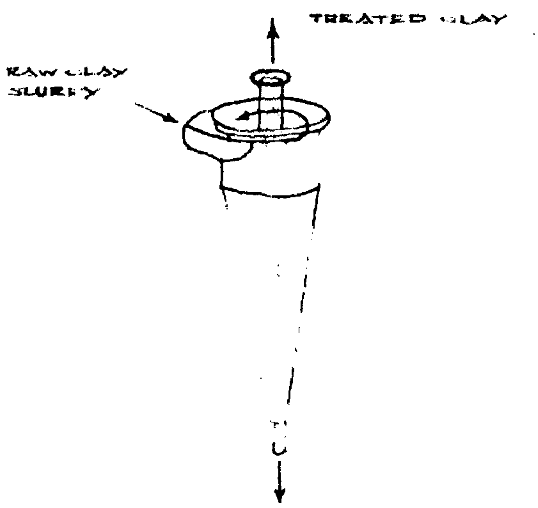
SHOWING GROUTING DATA

Date & Shift.	Location.	Time From	Time To	Total	Pressure in Kg or lbs.	Consumed in gallons.	Proportion used.	Remarks.



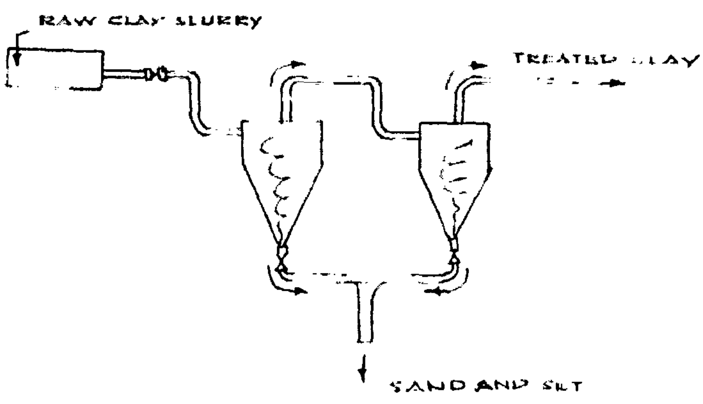
(a)

FIG. 7·1 DETAILS OF HYDROCYCLONE



(b)

FIG. 7·1 SECTIONAL VIEW OF HYDRO CYCLONE



(c)

FIG. 7·1 HYDROCYLONE IN SERIES

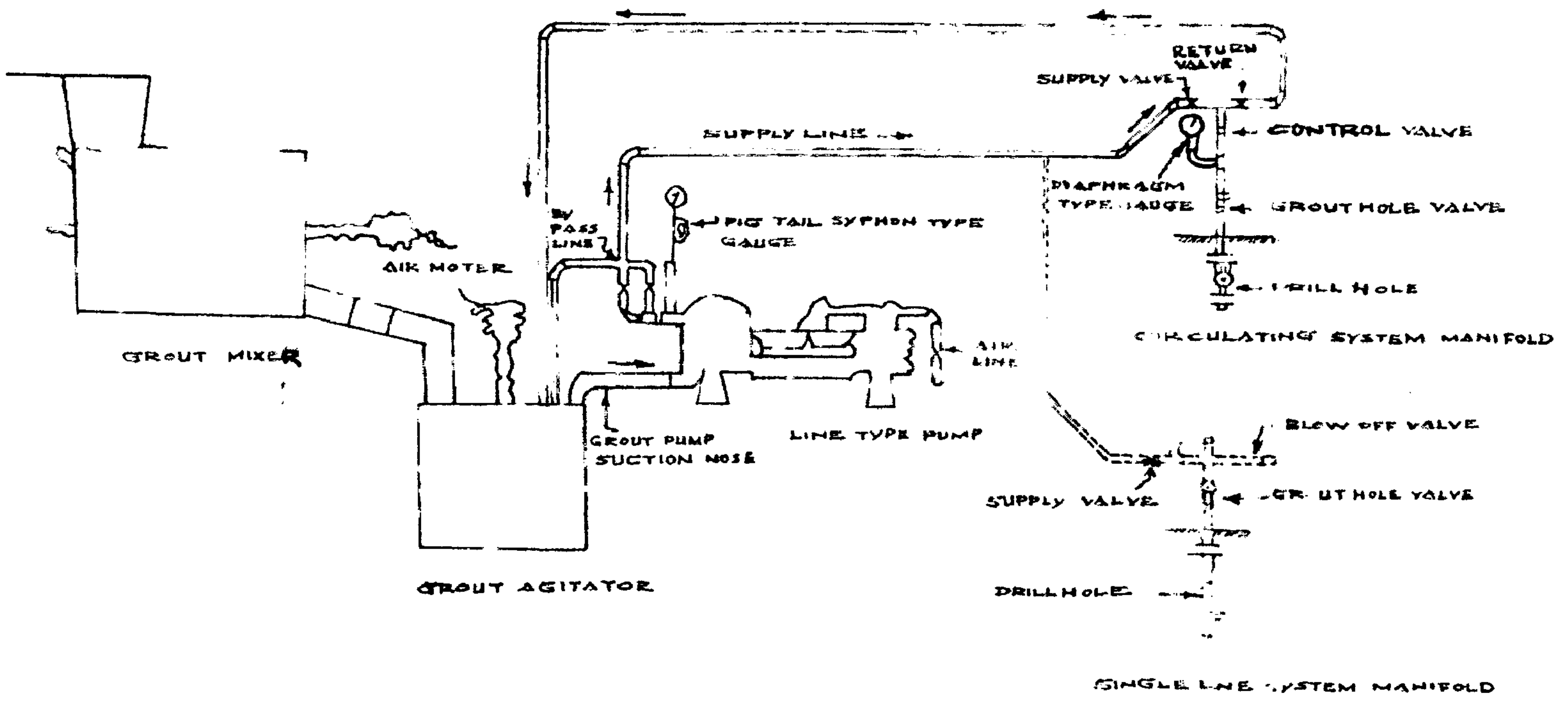


FIG. 7·2 TYPICAL GROUTING PLANT

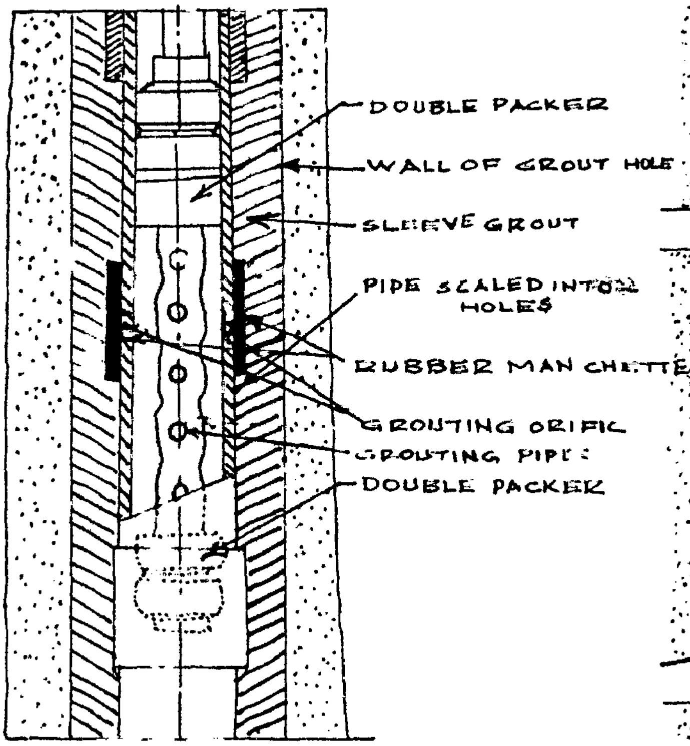


FIG 7.3(a)
TUBE-A-MAN CHETTE

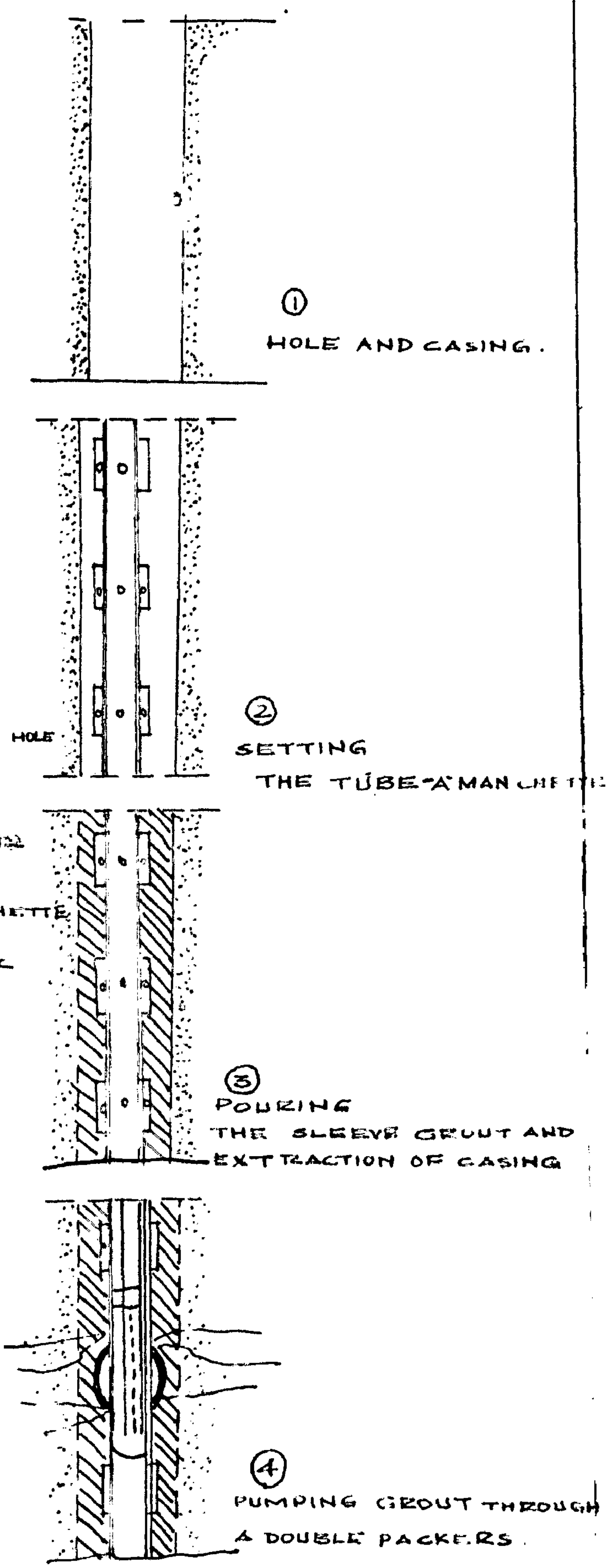


FIG: 7.3 (b)
PRINCIPLE OF THE TUBE-A-MAN CHETTE
GROUTING METHOD

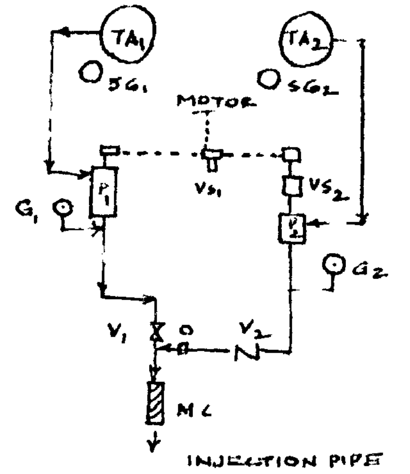


FIG: 7.4
PROPORTIONING SYSTEM

TA1 -- MIXING TANK FOR AM 9
TA2 – MIXING TANK FOR AP
SG – SIGHT GAUGE
P1-P2 – PUMPS
VS – VARIABLE SPEED DEVICE
G1-G2 – GAUGES.
V1-V2 – VALVES.
O – ORFIC
M.C – MIXING CHAMBER.

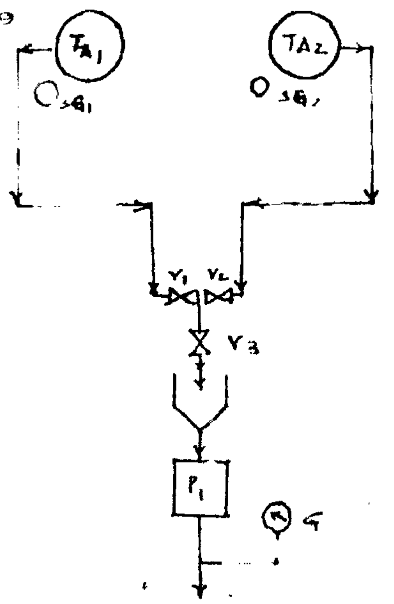


FIG 7.5
TWO SOLUTION SY.

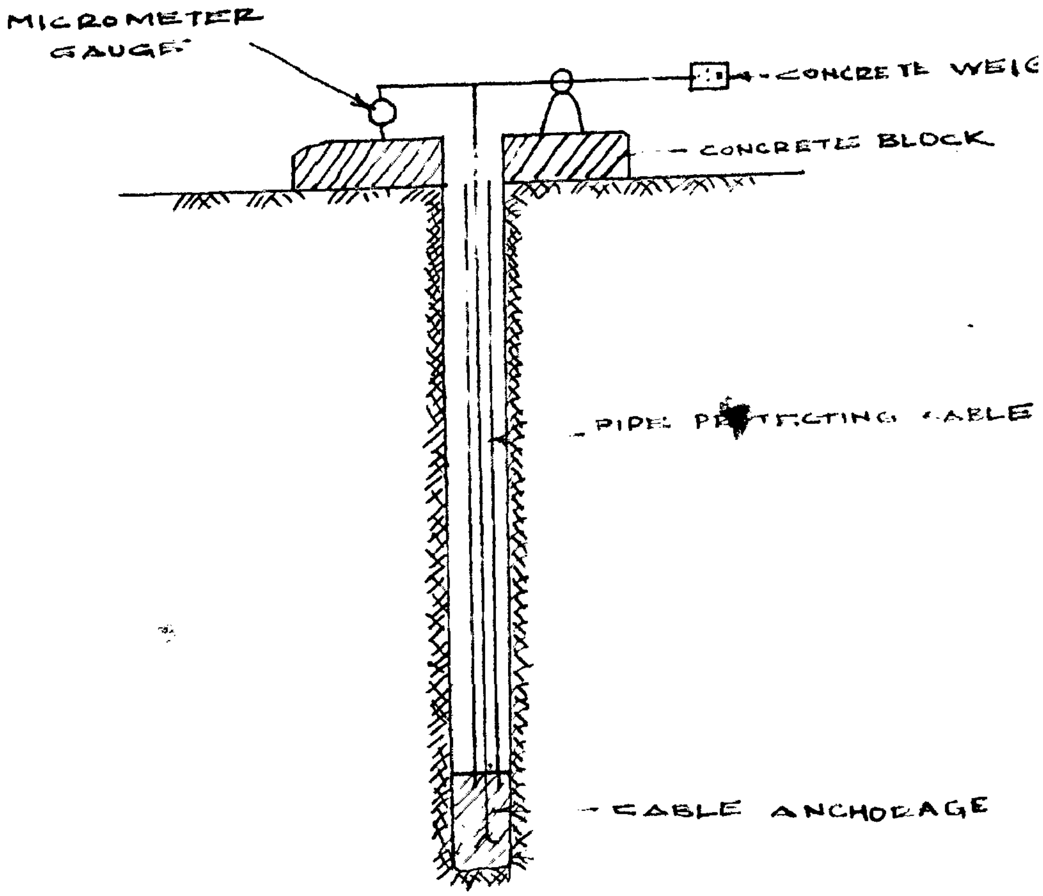


FIG: 7.6
DIAGRAM OF UPLIFT GAUGE

plotted. The sudden increase in intake shows the fissure formation in the strata. Such pressure intake curves with respect to time may be plotted for all stages in each hole to determine the behaviour of grout travel with respect to pressure and the stratification of formation. In practice various types of intake pressure curves are possible as shown in Fig.No.6.10. These should be studied closely before deciding upon the consistency of the grout and pressure to be applied.

After the grout is finished in a test plot permeability test may be conducted to determine the permeability after grouting to know the efficiency of the grout. A large diameter hole say 1 meter diameter may be drilled for visual inspection of the sub-surface to see the filling of voids with grout. Samples taken from the large diameter hole may be tested in laboratory for compressive strength. If possible an open pit may also be excavated to inspect the seepage of water through the grouted zone.

At Girna dam field trials indicated that for border cases of formation say for sand fraction of size 0.8 mm. clay-cement grout under overburden pressure failed to penetrate and satisfactory penetration was obtained under high pressures.

The success of the grouting operation, therefore, depends upon the inference of the results obtained from the field trials.

6.5. EFFICIENCY OF GROUT CURTAIN

6.5.1. The efficiency of the grout curtain is related to the criteria, targets and the results achieved.

6.5.2. <u>Criteria:</u> There is no specific rule which fixes the limit of the permeability of the foundation materials to be achieved after grouting. For rocky foundation Lugeon in 1932 has fixed a criteria of water loss of 1 lugeon (ref. para 6.2.56) which generally corresponds to a permeability of 10^{-5} cm/sec. This criteria so fixed was only for sound rock foundations as at that time all the dam foundations which he had analysed were resting on sound rock. This limit can be relaxed in soft rock formations. At Mangla Dam, even a water loss of 2 lugeons was considered acceptable. For alluvial deposits this criteria can also be applied. In some of the projects permeability not exceeding 10^{-4} cm/sec. after grouting was accepted. A reduction in permeability of 1000 to 10,000 times of the initial permeability is most warranted depending upon the type of material to be grouted.

6.5.3. <u>Targets:</u> How targets of reduction in permeability are fixed? If a complete cutoff of seepage past the dam foundation is required as in case of irrigation and power projects when every drop of water is valuable, a high target may be fixed, when a complete cutoff is not required as in case of coffer dam where some seepage can be permitted or where the leakage past the dam is not so much of significance, so long as the dam foundation is safe as in case of flood control schemes, a low target may be economical.

6.5.4. <u>Results achieved:</u> The results achieved in the filed seldom agree with the targets fixed, due to hetrogeneous and anisotropic condition of the foundations. For high targets, rigid control may be exercised to achieve the results in the field about 99% of the targets fixed. For low targets, results achieved shall not be less than 90% of the target.

6.5.5. Measurement of Efficiency: In grouting alluvial deposits the efficiency of the grout curtain increases in direct proportion to the width and number of grout rows. In coffer dams, the efficiency of grouting could easily be observed by open excavation but under dams, the effectiveness in arresting seepage through the dam foundation may be observed by measuring the head drop by means of carefully located piezometers on both the sides of the cutoff. This will be effective if the head drop across the grout curtain is not less than about 90%. Records had recently been published for the Rocky Reach dam on the Columbia river, U.S.A. where clay cement and chemical grouting were used to form a cutoff in gravel deposits with permeability as high as 5 cms/sec. Fig.No.6.11 shows the piezometric levels at the highest river level before completion of the grouting and after the first filling of the reservoir. This shows a reduction in pressure upto 90%.

Hydraulic, economic and safety requirements may impose a maximum value of seepage of water through a dam foundation. In order to provide some standard of comparison the following standards are suggestive:

(i) Target T $= K_t - K_p$

(ii) Reduction R $= K_t - K_f$

(iii) Grout Efficiency E $= \dfrac{K_t - K_f}{K_t - K_p}$

$$= \frac{R}{T}$$

Where,

K_t is coefficient of permeability before grouting

K_p is coefficient of optimum grouting

K_f is coefficient after grouting.

From the above the maximum improvement that could be obtained will be Kt/Kp and minimum Kt/Kf. However, the designer will be more interested in the amount of seepage which will take place even after grouting. Although, the grouting efficiency gives a measure of the reduction, very high initial permeabilities may mask the fact that even with highly efficient grouting, the final results may still permit rather more seepage than is desirable. It must be borne in mind that laboratory grouting results are usually on the safe side and that success of a grouting programme depends on the pattern of grout holes, the grout sequence and methods and combination of grout used.

<table>
<tr><th rowspan="2">PURPOSE OF GROUT</th><th colspan="5">TYPE OF FOUNDATION</th></tr>
<tr><th>ROCK LARGE FAULTS & CAVITIES</th><th>ROCK BLAST CRACKS SMALL VOIDS</th><th>COARSE GRAVEL COARSE SAND $D_{10} > 1$ mm</th><th>COARSE TO MEDIUM SAND $D_{10} > 0.2$ mm</th><th>MEDIUM TO FINE SAND $D_{10} > 0.1$ mm</th></tr>
<tr><td rowspan="7">IMPERMEABI - LIZATION</td><td></td><td></td><td></td><td colspan="2">ASPHALT EMULSOIN</td></tr>
<tr><td></td><td></td><td></td><td colspan="2">CHEMICALS</td></tr>
<tr><td></td><td></td><td colspan="2">CLAY - CHEMICALS</td><td></td></tr>
<tr><td></td><td></td><td colspan="2">CLAY</td><td></td></tr>
<tr><td></td><td colspan="2">CLAY - CEMENT</td><td></td><td></td></tr>
<tr><td colspan="3">CEMENT</td><td></td><td></td></tr>
<tr><td rowspan="2">COARSE * GROUT</td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="6">CONSOLIDATION</td><td></td><td></td><td></td><td></td></tr>
<tr><td colspan="3">CEMENT</td><td></td><td></td></tr>
<tr><td></td><td colspan="2">CLAY - CEMENT</td><td></td><td></td></tr>
<tr><td></td><td></td><td colspan="2">CLAY - CHEMICALS</td><td></td></tr>
<tr><td></td><td></td><td colspan="3">CHEMICALS</td></tr>
<tr><td></td><td></td><td colspan="3">ASPHALT EMULSION</td></tr>
</table>

* SAW DUST, SAND - CEMENT - CLAY, ETC.

FIG. 6·1

EXTENT OF PRESSURE GROUTUSE

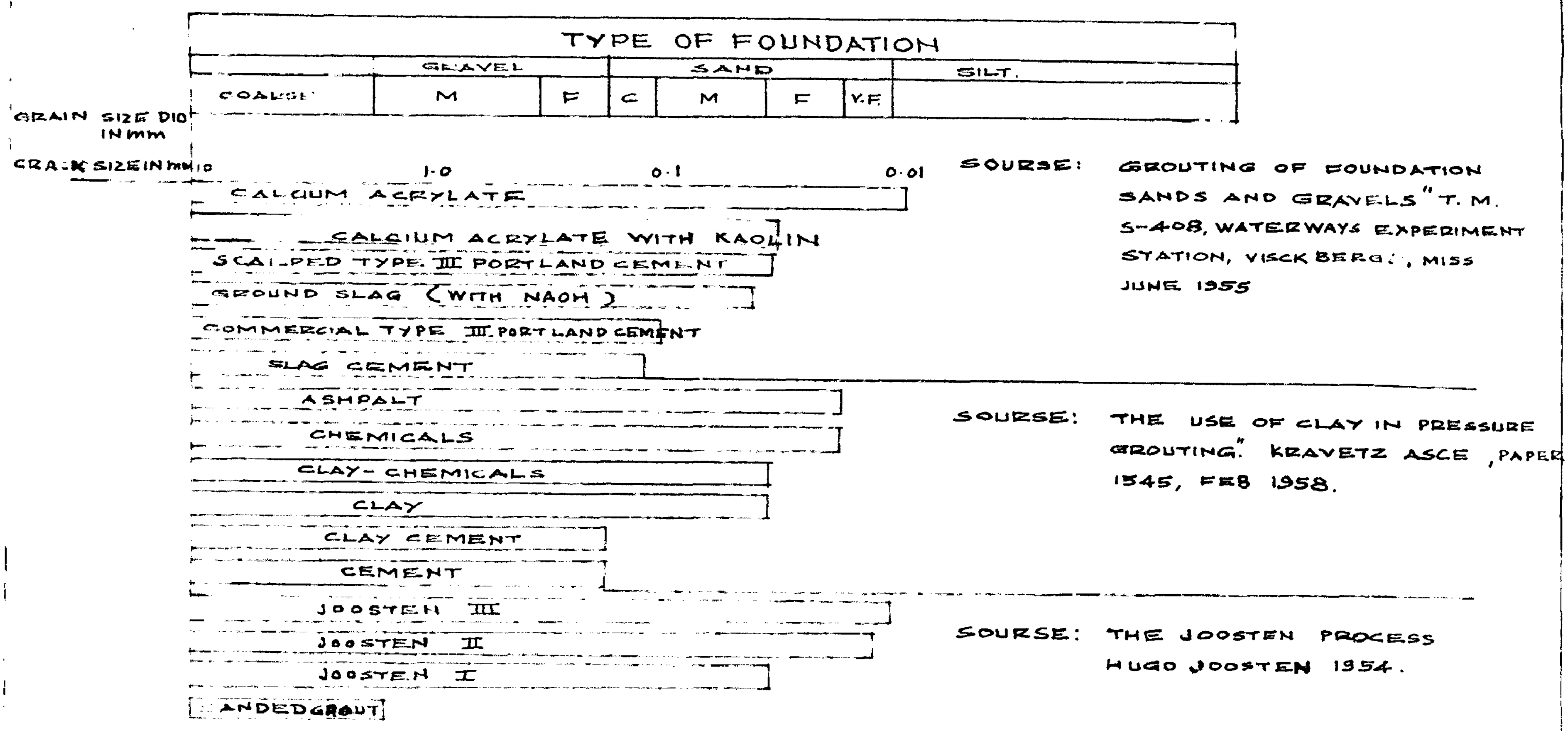


FIG: 6·2

EXTENT OF PRESSURE GROUT USE

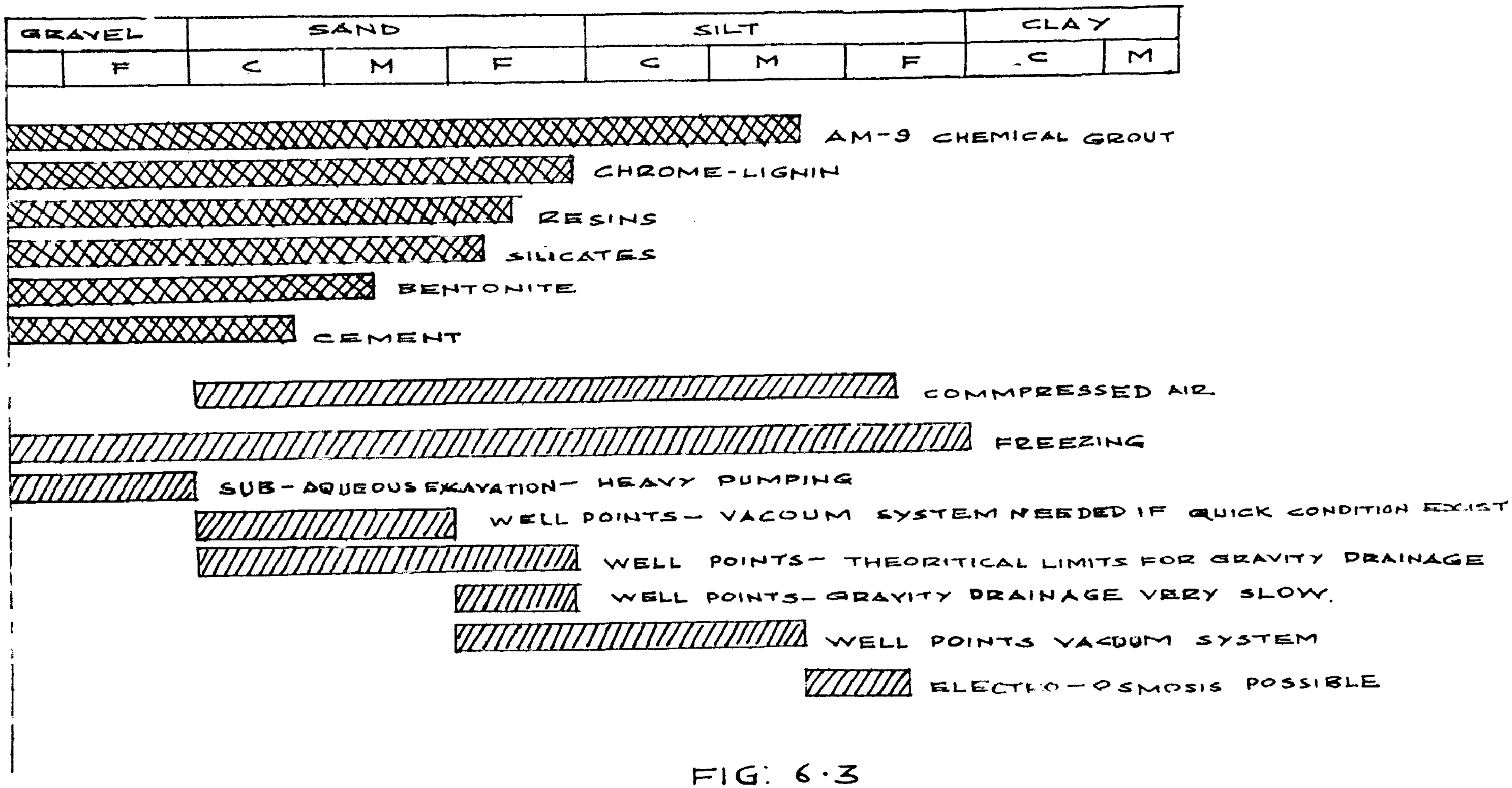


FIG: 6·3

COMPARISON OF METHODS FOR STABILIZING AND DEWATERING SOILS

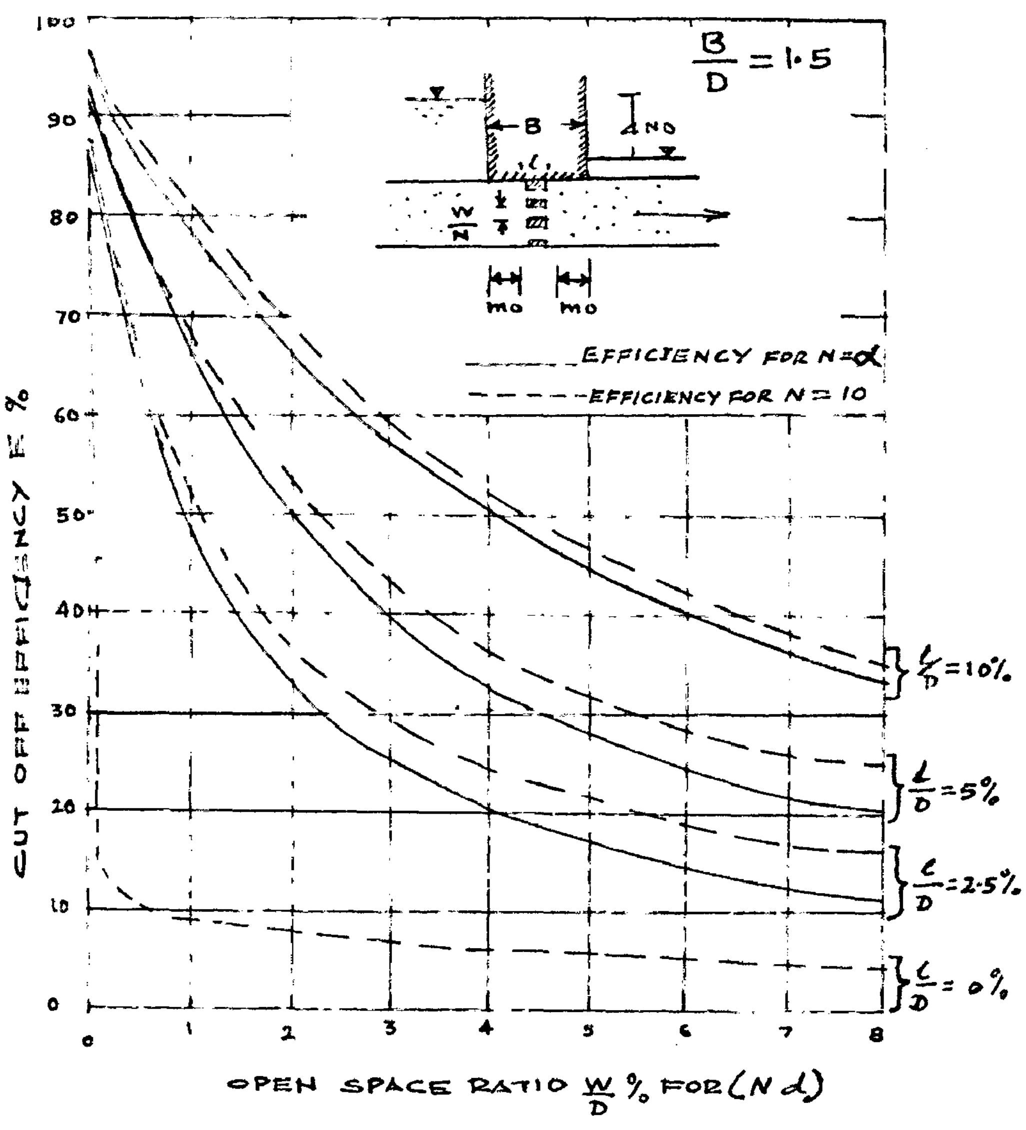


FIG: 6.4 (a)

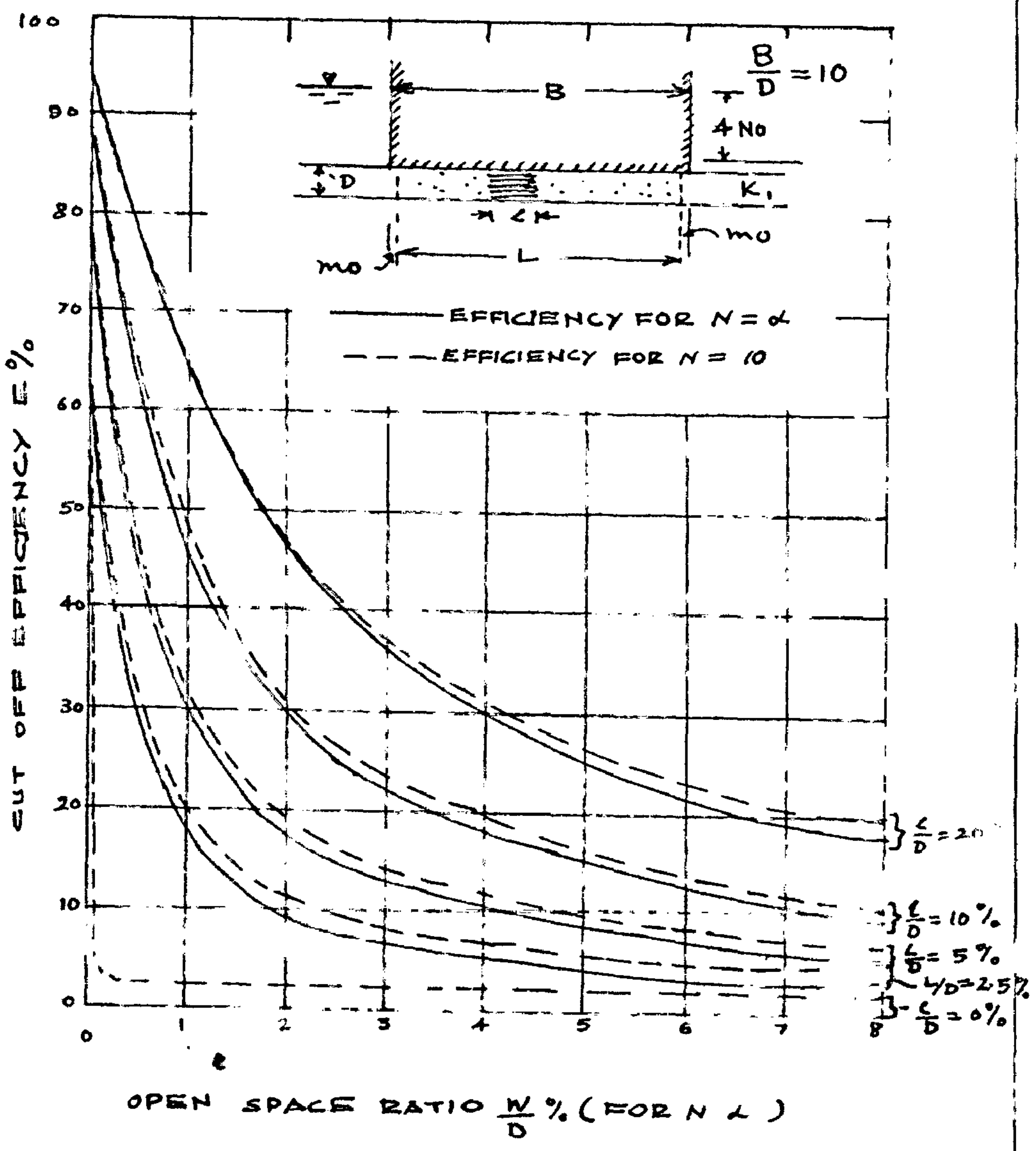


FIG:- 6.4 (b)

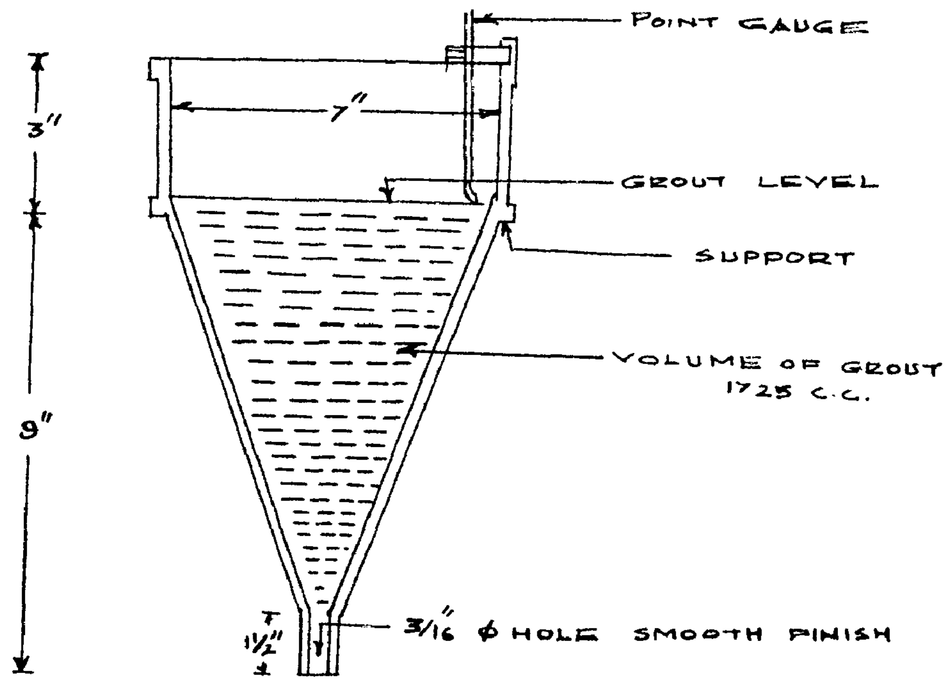


FIG 6·5

FLOW CONE.

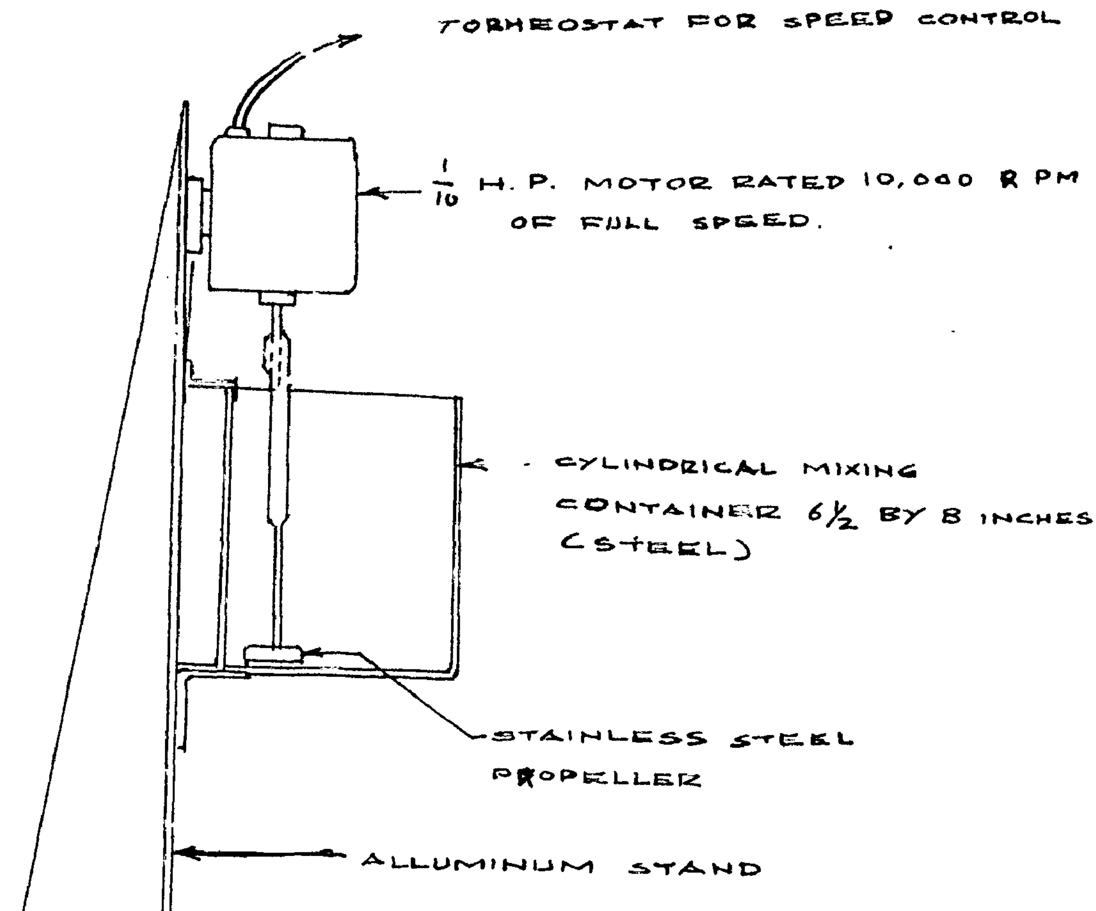


FIG: 6·6.

LABORATORY GROUT MIXER

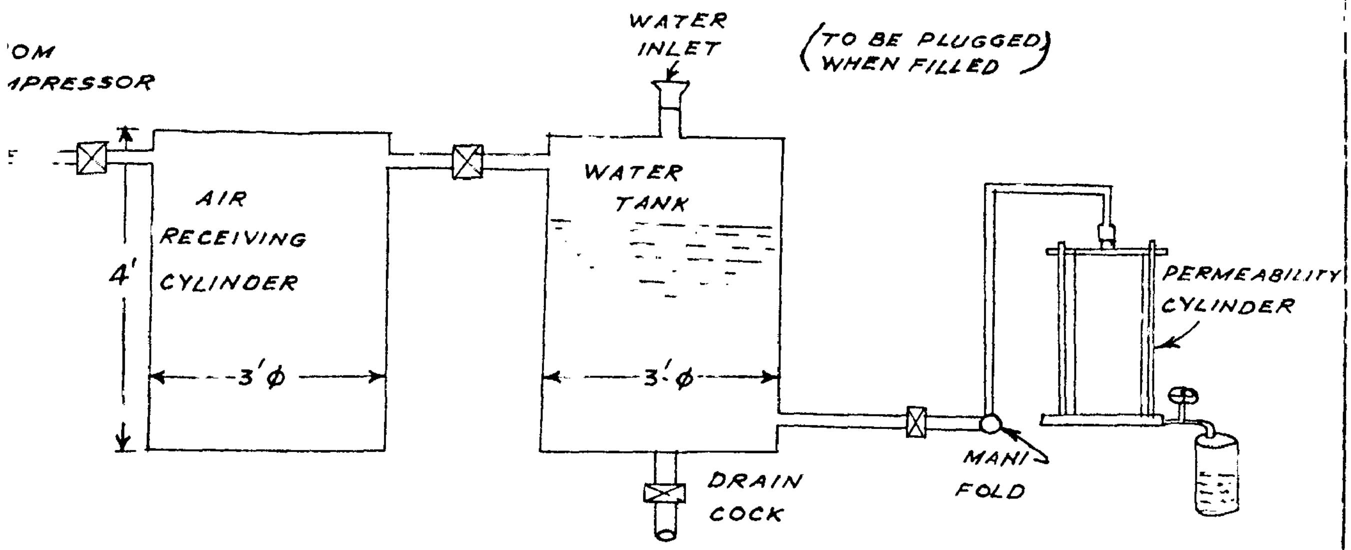


FIG. 6·7. SHOWING WASH OUT SET UP.

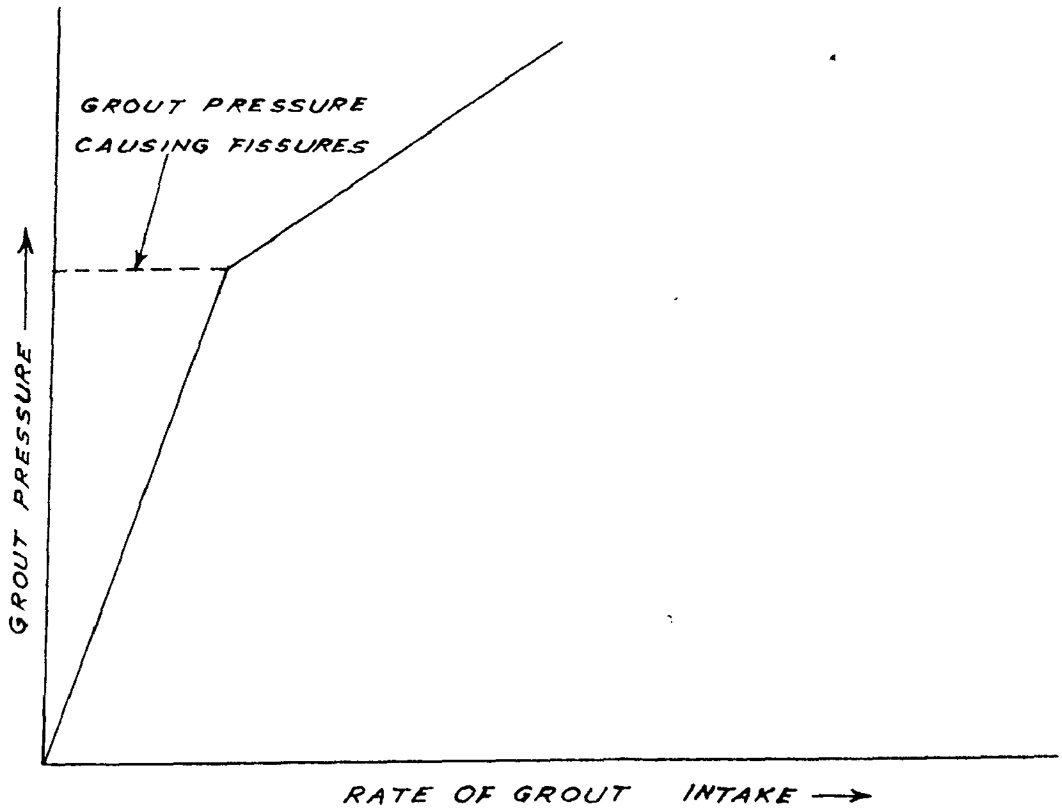


FIG. 6·8

PRESSURE INTAKE CURVE

REF:— O = POINTS FOR INCREASING PRESSURES.

● = POINTS FOR DECREASING PRESSURES.

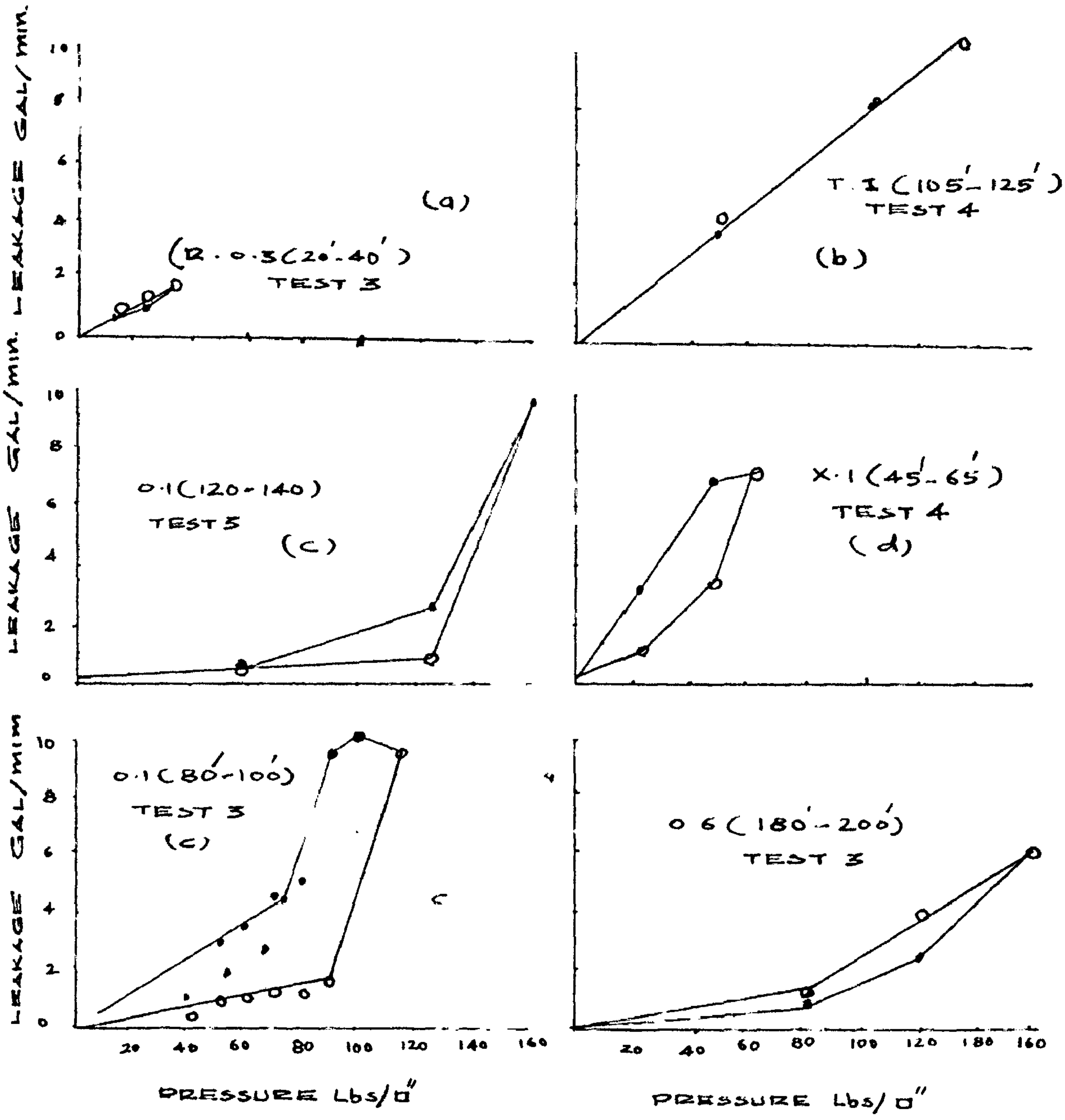


FIG: 6.9

TYPICAL WATER PRESSURE TEST RESULTS.

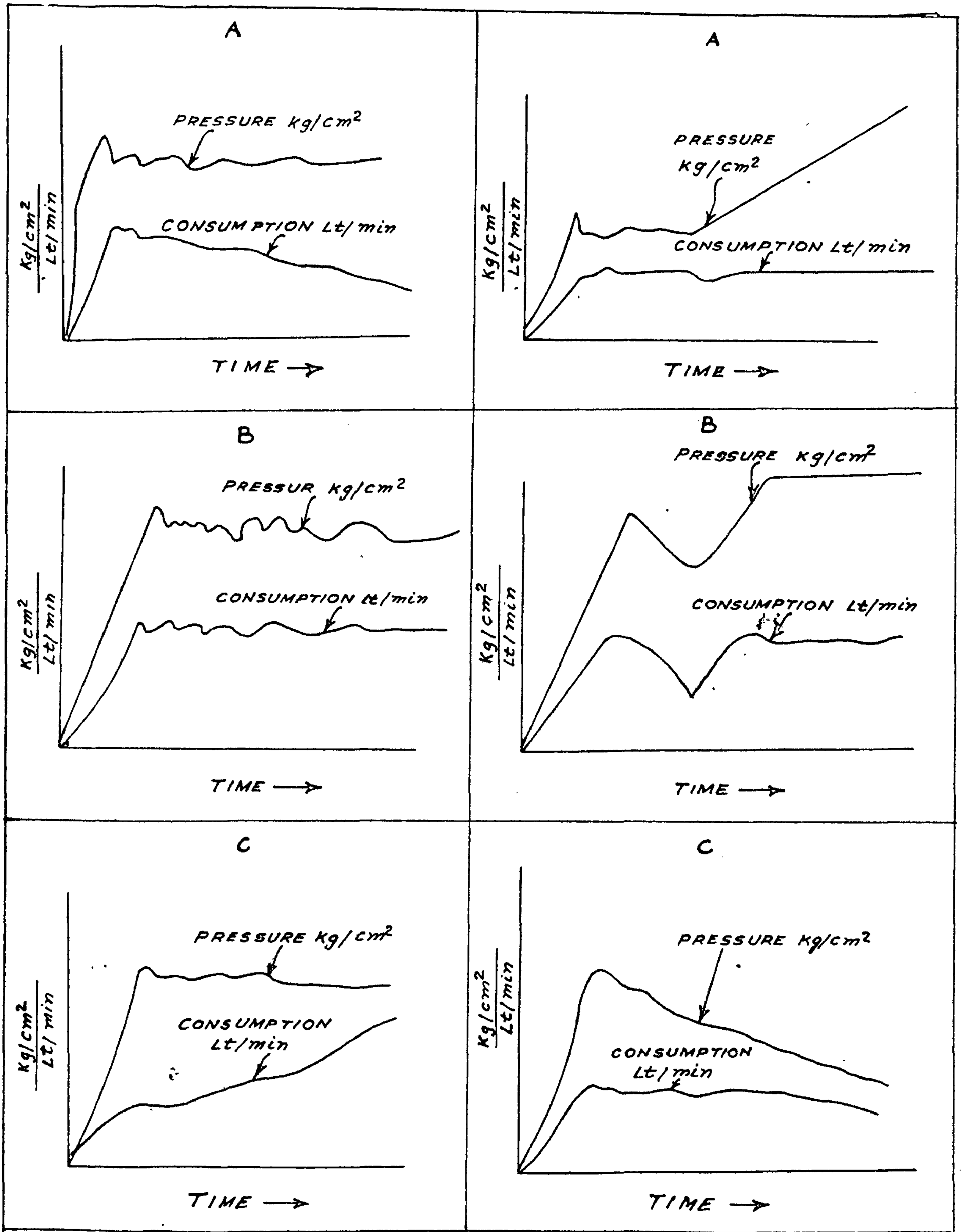


FIG. 6·10

TRENDS OF PRESSURE AND
RATE OF CONSUMPTION OF GROUT

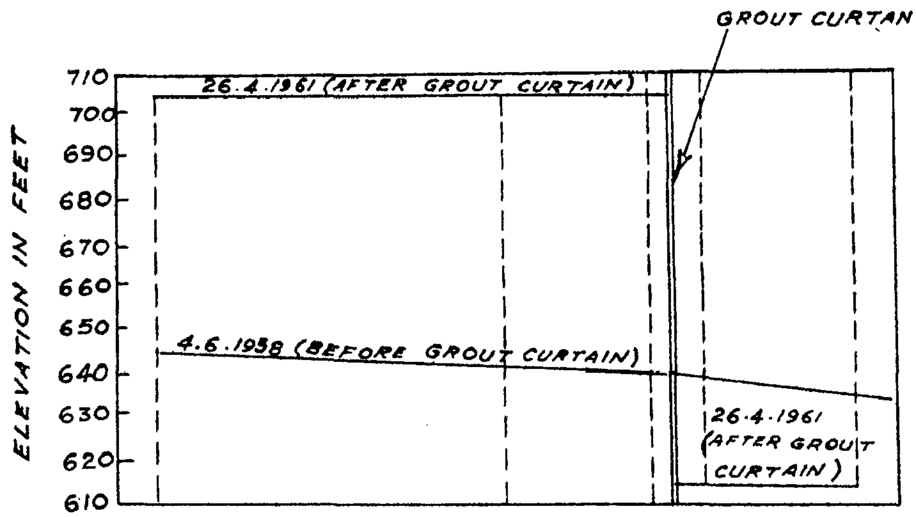


FIG. 6.11. ROCKEY REACH DAM. PIEZOMETRIC LEVELS ACROSS GROUT CURTAIN BEFORE COMPLETION OF THE GROUTING AND AFTER THE FIRST FILLING OF THE RESERVOIR

CHAPTER - 7

TECHNIQUE OF GROUTING

7.1. GENERAL

After the selection of a grout mix for a given job condition by laboratory tests and field trials, the success of the grouting depends upon the following operations in the field:

(1) Grouting process

(2) Field control

(3) Field problems

(4) Quality control

(5) Grouting Data

7.2. GROUTING PROCESS

7.2.1. The grouting process can be divided into the following component operations:

(1) Drilling

(2) Processing of raw materials

(3) Mixing of grout materials

(4) Injection of grouts.

7.2.2. Drilling: This consists of drilling holes and installation of grouting pipes. There is no standard drilling rig that can be used for all jobs. Therefore, the considerations governing the choice of drilling equipment are economy and speed. Drilling of foundation holes of 1½" to 2" dia. size is carried out by rotary drills, percussion drills, wagon drills and jack hammers. Augers are also used for drilling holes of 4" to 6" size and depth upto 20 ft. For larger holes 5" to 48" dia. calyx shot core drills are used. Drilling carried out by rotary method uses drilling mud for stabilizing the sides of bore holes which avoids the use of casing pipe and facilitates driving in alternatively hard and soft strata. Different types of rotary bits are used such as steel cutters and fish tail bits for soft cohesive silty strata, rotary cutting bits with tangestan carbide inserts for partially cemented strata, Tricone bits are used for sand and fine gravels, for larger coarser gravel and bouldery strata Tricone bits with carbide inserts are used. It might appear that mud drilling would be objectionable for grouting work, if the bentonite suspension or chemicals are to be injected subsequently. However, the drilled mud is eventually replaced by sleeve grout which cracks under pressure. The disadvantage of mud drilling is therefore obviated in all methods involving the use of sleeve grout and specially in tube-a-machnette method. Wagon drills are most versatile and can be used for drilling in rock as well as soil with suitable bits. The drilling performance at Ukai project by above drilling methods shows that wagon drill is most economical. The comparison of progress per hour and rate of drilling per meter for different drilling equipments as observed at Ukai project is given in Table No.7.1.

Table No.7.1

Comparison of output and rate of drilling with various equipments

Sl No.	Name of Equipment.	Output per hour in meters		Rate per meter	
		Soil	Rock.	Soil	Rock
1.	Special jack hammers ($1\frac{1}{8}$" dia.)	Not suitable.	3	-	6.0
2.	National calyx.	0.9 to 1.0	0.25	33	105
3.	Diamond drill.	1	0.5	27	60
4.	Wagon drill.	4	2.5	7.5	12.0

7.2.3. Processing of raw materials:

7.2.31. Processing of clay: Naturally occuring clay contains silt and sand in varying proportions. Clay is generally used to grout sand of size 0.8 mm. and above. Therefore, for satisfactory penetration it is desirable to remove fractions of 30 microns and above. Clay in natural deposits is available either in dry lumps or in moist conditions. Dry lumps can be broken by hand or in mechanical crushers and formed in powder which is stored in shed for use. In the moist clay water is added and slurry is formed and stored in tanks. From dry powder sizes over 10 mm. can be removed by screening whereas by wet screening it is not possible to remove coarser fraction from clay slurry as the sieves get choked up. The studies carried out at the C.W.& P.R.Station,Poona in 1959 revealed that it is not easy to remove silt and sand from clay slurries by simple sedimentation in tanks because silt and sand can settle only in a diluted suspension of clay. The concentrated clay suspension forms a thick fluid of high viscosity which does not allow the sand to settle under gravity. Thin suspensions are not desirable because afterwards it would be very difficult to remove the excess water to achieve a consistency required for the grout. Insuch cases, the separation of the sand would be achieved by the use of "Lavadune", and "Hydrocyclone". The principle of Lavadune is very simple, and consists of a sloping tube through which the clay slurry travels up and the particles that settle down roll along the bottom of the tube and are finally collected through a slot discharging to waste.

Particles greater than 1 mm. sizes can be separated out by this apparatus. The further separation of coarser materials above 30 microns can only be achieved by an apparatus working on centrifugal principle called the "hydro-cyclone", a patent of Neyrpic, France. Field application of this apparatus has confirmed that penetration could be achieved effectively in a soil of permeability 1½ times lower than those treated with non-hydrocyclone clay grouts. Fig.No.7.1 shows the principle of the hydrocyclone. It can be operated by a pump or simply by gravity head. The clay slurry enters at the top of the cyclone tangentially with a velocity sufficient to cause the heavier particles to be thrown to the sides and drop at the bottom, where there is a throttle which prevents the loss of too much of the usable clay slurry. The process is admittedly an imperfect one and consequently the hydrocyclones are used in multiple units to give successive treatments, and so ensure an efficient separation of sand and silt from the grout. Its successful performance depends on three factors:

(1) Diameter at the top

(2) Velocity of injection of clay slurry, and

(3) Side slopes of the cone.

The smaller the scale of conical chamber and the greater the pressure the finer the size of separation. The treated clay should contain about 98 to 99% of clay plus silt. Results obtained at Ukai are given in Table No.7.2.

Table No.7.2

Results of clay treated in Hydrocyclone

Experiment No.	%age of clay & silt before treatment.	%age of clay & silt after treatment.	Remarks.
1	90	94 96 99	Ist treatment repeat.
2	74	86 96 99	Ist treatment repeat.
3	83	94 98	Ist treatment.
4	88	95 98	Ist treatment.
5	90	93 95 98	Ist treatment repeat.

7.2.32. Processing of bentonite: The commercial products of bentonite contain a little less than 10% of fine sand and silt. For efficient penetration of fine sand it is desirable to remove particles of size above 5 microns. This can be achieved by smaller size of hydrocyclone.

The bentonite used at Blackwall tunnel,London contained 10-15% by weight of particles in the silt and sand range and were removed by hydrocyclone. The result showed that untreated suspension passed freely through soil of permeability 4×10^{-1} cm/sec but flow was quickly blocked in a sand of 9×10^{-2} cm/sec. permeability. After treatment in hydrocyclone the suspension passed freely through fine sands of 2×10^{-2} cm/sec. permeability and only blocked in soil of permeability 3×10^{-3} cm/sec.

7.2.33. Processing of chemicals: Chemicals are mostly available in dry lumps. Their solution of required specific gravity are prepared and stored in tanks and subsequently diluted as desired.

7.2.4. Mixing of grout materials:

7.2.41. There are two types of grout mixers viz;

(1) Paddle type, and

(2) Colloidal type.

The paddle type or low speed mixers are commonly used with propeller blades instead of straight paddles. The speed of paddle type mixers are given in Table No.7.3.

Table No.7.3

Speed of paddle type mixers

Material.	Type of mixer.	R.P.M.
Cement.	Double drum.	Ist drum 1500 2nd drum 800
Chemicals.	Single drum.	800
Final mixing of grout slurries already prepared.		800

The size of the mixers is generally about 2 to 3 ft. diameter and 3 to 4 ft. height. The colloidal type mixer has a high speed disc similar in design to the colcrete mixer. Messers Bachy in France have developed a twin cylinder mixer in which two cylinders rotating at the speed of 1500 r.p.m are placed in a drum which is given a special shape.

Moor and White[50] (1959) have suggested that grouts with better penetrating qualities will be obtained by using colloidal mixers. This premise appears to be based on the assumption that clumping and flocculation occur in neat cement and cannot be overcome by the paddle mixer. Tests made at the U.S.Army Engineers Waterways experiment

Station[51] (1958) indicated that the colloidal mixer mixed to about the same homogeniety in about half the time required by the paddle mixer. However, prolonged mixing with the colloidal mixer heats the grout and causes rapid increase in viscosity. The value of colloidal mixing has been generally appreciated and it is now almost universally used. The only point for alternative study is whether the colloidal mixer should be used for the final mixing after addition of cement or before the addition of cement or chemicals. It seems necessary to use the colloidal mixer for cement only when high proportion of cement is used. However, when the proportion of cement is small or when chemicals are used the final mixing can be done with an ordinary paddle mixer. It should be normally satisfactory to mix the clay and water in a colloidal mixer to form the slurry which can be stored before final mixing. Addition of dispersing agents to the clay during the colloidal mixing process is desirable.

Recent development in mixing clay is by jet mixer in which dry clay powder falls through opposing high pressure water jets which create sufficient turbulence to mix the highly active clay slurry. This was tried at Blackwall tunnel, London.

7.2.42. Grouting Stations: On work site processing of materials and mixing is done under shed called the grouting station. A grouting station is a kind of small modern factory, schematically, it consists of a stocking area, a clay processing plant, mixers, storing tanks, measuring tanks and pumps where numerous and constant controls for quality and quantity can be carried out.

7.2.5. Injection of grouts

7.2.51. The injection of grout in the ground involves two stage process viz;

(1) Pumping of grout at the head of the grout hole

(2) To carry out the grout into the ground by grout pipes embedded in the ground.

7.2.52. Grouting Plant: The pumping of grout at the head of the grout hole is done by setting up a grouting plant at the site of the work. For suspension type of grout, the grouting plant consists of a grout mixer, a grout agitator, a grout pump and piping with connections to the grout hole. Two types of piping system are used for the grout as shown in Fig.No.7.2. The circulating system has a return line to the grout agitator tank through which grout which does not go into the hole is returned. With the single line system all the grout pumped goes into the hole or is wasted through the blow off valve as shown in Fig.No.7.2. The circulating system is used almost universally at the present time. The primary advantage of the circulating system is that the grout has no opportunity to set in the supply line. Engineers who prefer the single line system believe that the high stage of pulsating pressures in the grout supply line coming directly from the piston stroke of the pump are important in the process of grouting the maximum amount of grout in the soil. These peaks of pulsating pressure generated in the fluid grout in the hole are largely dissipated through the return line when the circulating system is used.

The agitator in the storage tanks is used only for keeping the grout in uniform suspension. Various types of pumps have been used satisfactorily. Most satisfactory equipment for injecting grout is a pump of double reciprocating duplex type. This gives asteady flow which is more effective, specially in fine seams than the intermittant flow produced by compressed air equipment and the operator has a better control of pressure. Unless extreme care is used the latter may force air into seams if grout does not flow faster. There are other types of positive displacement pumps which may prove to be more suitable. One type of pump which may even replace the positive displacement pump is the centrifugal pump specially equipped for grout injection. The pump should be specially fitted with renewable lines rubber pistons etc. for grouting purposes.

Grout mixers are usually simple tanks with slowly rotating mechanically operated paddles, or high speed impeller type. A water meter is provided in the system to measure the water entering into the grout mixer. Pressure gauges must be protected from direct contact with the grout. This is done by connecting the gauge and the grout line with an oil or grease filled pipe. Grout lines exposed to the sun should be covered with wet cloth to prevent them from getting too hot, which sometimes causes early setting of the cement and increase in viscosity, thus poor grouting results. In cold weather, grout must be kept from freezing before it is pumped into the rock.

For silicate grouts the grouting equipment is the same as for suspension grouts except that agitating tanks are not necessary.

The grouting plant for organic resin grouts consists of separate tanks for grout components drawn to a mixing chamber by pumps and forced in the ground. The pumps have variable pumping ratio to draw the required quantity of solution. Sometimes the different components of the grout mixture are pre-mixed in advance depending upon the job requirements and ease of working.

7.2.53. <u>System of injection:</u> There are two systems of injecting grout fluids into the soil formations:

(1) Single fluid system, and

(2) Two fluid system.

(1) <u>Single fluid system:</u> Most of the single fluid injection processes require the mixing of two or more components such as cement, clay, bentonite and a wide selection of chemicals, before the resulting single fluid is injected into the ground. Chemical reaction commences at the moment of mixing and will produce a gel or set grout within a definite period of time known as gel time. It is a practice to premix the components of a batch in fixed quantities, to consume within the gel time. In some cases, where the injection proved to be slower than had been expected, the grout would have inferior flow characteristics towards the end of the batch and the gel might have completely formed before the batch had been injected and thus involves considerable waste in discarding the material. Increasing the gel time to allow a factor of safety on the expected time of injection for the batch is not a satisfactory solution, as increased gel times usually meant decreased gel strength and in addition it is not desirable to have ungelled grout in the ground for extended periods. The solution of this problem has been found in a proportioning pump. This consists of two pump units interconnected in such a way that two solutions prepared from the grout components are drawn from two separate tanks in the required propor-

tions, forced through a mixing chamber and thence to the ground. The pumping ratios of the two solutions can be easily altered on the pumps and thus the gel times varied as necessary according to the ground conditions. By this means the full gel time of the grout is available for it to permeate the soil, there is no danger of partially gelled grout being pumped and shorter gel times generally can be worked to.
The single fluid grouts are generally of a complex character and require careful premixing before injection. The single fluid grouts used for treatment of pervious soils include:

(1) All suspension grouts, cement based and clay based grouts.

(2) Sodium silicate with suitable salts reagents such as sodium aluminate, bicarbonate, phosphate etc. These have weak gel strength.

(3) Sodium silicate with a system of organic esters, which have a controlled evolution of reactive ions leading to the precipitation of silica gel. Soil crushing strength of 20 kg/cm^2 can be obtained.

(4) Chrome-lignin grouts. This grout has good penetrating ability and can produce an elastic and insoluble gel.

(5) Organic polymer grouts. These grouts have low viscosity.

(2) Two fluid system: Two fluid system grout is only used with silicate grouts. In this system a concentrated solution of sodium silicate is injected into the ground followed by a stronger electrolytic saline solution of calcium chloride. These two solutions react instantaneously in the ground and form a soft gel which bind the sand grains together at their

point of contact, giving the whole mass a crushing strength of 30 to 50 kg/cm^2. This method was first used in 1925 by Joosten and is still favoured by some engineers because of its reliability and high strength. Other advantages are:

(1) Ground heave does not occur since the fluids, which are injected in small dozes, react almost instantaneously. For this reason, the grout is confined within an established radius of the injection position, and prevent the grout escaping into the more permeable zones.

(2) The reactants form a reliable gel not noticeably affected by temperature variations or by the presence of brine or brackish water. Furthermore, the gel is chemically inert and immune to sulphate attacks, and the chemicals employed may be used safely in confined spaces since they are non-toxic and non-inflamable.

Guttman has modified the Joosten's process by adding sodium carbonate to the silicate solution to reduce its viscosity and thus to permit the treatment of rather fine grained soils.

7.2.54. <u>Injection methods</u>: There is no standard method of injection of grout. In each case, methods must be adopted to suit the nature of the soil, the results to be achieved, and the conditions under which the work must be executed. The suspension grouts are usually injected by the:

(1) Rising tube method

(2) Desceding stage method, and

(3) Tube-a-manchette.

Silicate grouts are either injected by

(4) Single shot process, or

(5) Two shot process.

Organic polymer grouts are injected by

(6) Proportioning system

(7) Two-solution system, or

(8) Batch system.

These procedures are described below:

(1) Rising tube method: In this method the hole is drilled to the full depth and the tube is installed down to the bottom of the hole. The grout is injected at the bottom of the hole as the casing is withdrawn in short stages of 1 to 2 ft. Shorter stages are recommended by Ischy & Glossop[7] (1961) for alluvium grouting. For rock grouting the stages are generally much larger than that for alluvium grouting. The pipe is lifted after each pass. A flush coupled pipe is necessary so that it can be pulled out conveniently. This method is suitable for sands, but is rarely used in deep holes as it is apt to give trouble by premature setting of the grout in the hole, or because the grout may travel up the outer surface of the casing and seal it into the ground.

(2) Descending stage method: In this method a hole is drilled upto a certain depth and grouting is done for zone drilled. For this operation the tube may be gradually pulled or a perforated section of length equal to the stage required to be grouted may be used. After one stage is grouted drilling resumed and the grouting pipe advanced for a further length corresponding to the depth of the next stage. In this way, the entire zone requiring grouting is injected in a descen-

ding stages. This procedure (piccollis method) is used for the preliminary treatment of grouts for very permeable strata. Experience has shown that a crack develops in each stage, which takes most of the grout and communication develops along the grouting in the entire length of stage. It cannot therefore, be assumed that satisfactory treatment of layers smaller than the height of stage can be achieved.

(3) Tube-a-manchette: The tube-a-manchette invented by Ischy in 1933 is the most satisfactory method in use at present time. It is described in details by Ischy and Glossop[7] (1961) and is shown in Fig.No.7.3. It consists of a steel tube of 1½" to 2½" diameter. At interval of about 1 ft. along the length of the tube rings of 1/4" dia holes are bored through it and each group of holes is covered by a tightly fitting rubber sleeve. In operation bore hole of 3" to 4" dia. is sunk to the full depth by normal methods of drilling and the manchette tube is placed in it. After lowering the manchette tube the casing is withdrawn and the annular space is filled by sleeve grout of clay and cement mix which is plastic.

Grouting is then carried out, usually beginning at the bottom of the hole and working upwards, through a 1" dia.pipe perforated for a short distance at its lower end and fitted with two opposed U.Packers which can be centred over any one of the rings of injection holes. When pumping starts the pressure on the grout pipe builds up and expands the rubber sleeve which in turn cracks the plastic grout filling and allows the injection fluid to pass out into the alluvium. The break of sleeve grout takes place generally in three directions at 120° near the sleeve and sleeve expands 10 to 15 mm. When pumping is stopped the passive pressure of the grout closes the cracks in the sleeve grout. The rubber also acts as a oneway valve. The rubber valve prevents any short circuiting of the grout back into the tube, and the sleeve grout prevents its return to the surface.

The great advantage of this method is that the tube once in place can be used at any interval of time with a whole range of different grouts and in this way the coarser beds can first be treated, and the finer ones later from the same hole. Moreover, if necessary, grouting can be resumed in any hole after lapse of any length of time. Also the operation of boring and of injecting any hole are entirely distinct and can be carried on separately and at any time. This method has been patented and is generally considered to be a very satisfactory method. Finally and most important, the method is extremely flexible as the manchette tube can be left in the ground and same hole can be grouted more than once and if necessary with grouts of different composition. Care shall be taken to wash the manchette after injection for reuse.

Army Corps of Engineers have used a combination of rising tube and descending stage methods. By this method first zone is drilled and grouted by conventional stage grouting method, then second stage zone is drilled and grouted by using expandable packers.

The rising tube method was used at the Kota barrage (Rajasthan) and for trial injection at Girna and Mula dams, Maharashtra State. It is observed that a major defect of this method is the leakage along the grout tubes and consequently, building up of pressure at the required position is not possible, resulting unsatisfactory penetration of grout. An improvement over the rising tube method used at Kota can be achieved by injecting a thixotropic sleeve grout into the space between the grouting tube and the alluvium. A short pass of 1 ft. should be used and the sleeve grout at the bottom of each pass cleaned with air and water. Datye has carried out experiments at Mula dam to perfect this method. The leakage along grout pipe was entirely prevented by use of a sleeve grout. But the grout pipes tended to adhere to the holes and broke while lifting especially for holes deeper than 40 ft. However, it is not possible to eliminate the major defect of this method and redrilling is necessary, if we have to return to the same elevation with a different types of grout. When the grout mix is decided in advance and changes in mix are not necessary over the whole length of the hole, this method could be satisfactory and economical, otherwise the ultimate cost of re-drilling would be higher than the tube-a-manchette.

Descending stage method has the obvious defect that the drill rig is kept idle for a long time between the stages. To be really effective, the stages should be short but the drilling progresses very slowly. This method would therefore, be considered only as a last resort to be used when all alternatives have failed.

To summarize, a satisfactory grouting system must satisfy the following requirements:

1. It should permit individual treatment of lenses and layers with short passes.
2. It should permit, repetitive grouting with different types of grouts and under various selected pressures.
3. The leakage along the grout pipe must be prevented.

It is obvious that the tube-a-manchette method is a highly satisfactory procedure. The cost of drilling per meter for the tube-a-manchette method, is, however, greater than that for the rising tube due to larger diameter of hole. The cost of redrilling in rising tube method are partly offset by the initial cost of the grout pipe and sleeves in case of tube-a-manchette method. However, heavy grouting pipes are necessary in rising tube-a-method to withstand the driving force and breakage of pipes is to be allowed for. Hence, the choice between the two method is largely a matter of availability of equipment.

(4) Single shot process: Single fluid grout mixtures are injected into the grout either of the methods used for suspension grouting.

(5) Two shot process: Joosten was the first to inject silicate grouts by the method known as two shot process. This consists of injection of concentrated solution of sodium silicate through pipes fitted with a point and perforated with small holes for a length of about 2 ft. at its lower end. The pipe is driven into the ground in stages, and silicate solution injected. This passes into the sand, driving the ground water ahead of it and adhering to the grains. Having reached the limit of the zone to be treated the pipe is withdrawn in a similar stage and an injection of calcium

chloride is made at each stage. This solution passes through pores already occupied by sodium silicate solution partly displacing it and partly reacting with it to form a soft gel which bind the sand grains together at their point of contact. This reaction is almost instantaneous. This method of injection ensures that the pipe will not stick fast in the treated ground and ensures complete treatment of the soil and to prevent the grout escaping into the more permeable zones. Joosten placed the pipes at close intervals of less than 1 meter. This process is still favoured by some engineers even though it was introduced in 1925.

(6) Proportioning system: A suitable proportioning system is shown in Fig.No.7.4, considerable flexibility has been designed into the system to vary gel times, pumping rates and pressures over a moderate range during any application. This system allows one man to control and eliminate adjustments of chemical concentrations and mixing procedures during an application.

AM-9, catalyst DMAPN and KFe are mixed in tank TAI, AP is made up in tank TA2. The pump P2 is selected to handle a small volume of AP solution (5-20% AP in solution) relative to the volume of AM-9 solution handled by pump P1. Thus the flow through P2 can be varied on the job to produce large changes in gel time without materially changing the concentration of AM-9 in the final mix or the total volume of solution entering the ground. The capacity of P1 is usually about five to fifteen times that of P2. The starting and stopping of the pumps can be controlled by a switch and the injection is controlled by opening or closing the valve VI. The orifice O should be sized so that AP solution will spray into the mixing chamber for good mixing. This makes P2 run at a higher pressure than P1.

The sizes of all pumps lines and valves must be chosen according to the pressure and flow rates anticipated. For all applications a separate water source should be available so that water may be pumped through P1 or another pump to clear the injection points or the formation adjacent to them. The American cyanamid company has designed a light portable proportioning pump with a mechanical control over a wide range of gel times. The pumping unit, with a volume range of 2.5 to 14 gel/min. and a pressure range of 0 - 150 lbs/in^2, weighs approximately 375 lbs.

(7) Two solution system: Two solution system has been used on some types of applications. It permits working conveniently with only one predetermined gel time. If the gel times are to be changed, careful weighed amount of AP, catalyst DMAPN or KFe can be added from time to time to the respective tanks. The process is shown in Fig.No.7.5. AM-9, catalyst DMAPN and KFe are mixed in tank TA1 and the AP in tank TA2. Tanks TA1 and TA2 should be of identical size and volume, and surface of the solutions should be at the same elevation. Equal volumes of solutions are drawn from tanks TA1 and TA2 into blending tank TA3, continuously blended and fed to pump P1. The concentrations of chemicals in each tank must therefore be twice that desired in the final grout solution . This system may be modified by using separate pumps of equal capacity for the AM-9 and AP solutions. Such a modification, of course, eliminates tank TA3 and pump P1 and permits the field use of very short gel times.

As in the case of proportioning system, all the components of the equipment must have the capacities and working pressures required by the anticipated grouting operations. A separate water supply is needed.

(8) Batch System:- In this method the AM-9 and catalysts are mixed in one tank and the batch is then pumped into the ground using almost any type of pump. This system has threee major limitations:

(1) The entire batch must be placed during the established induction period. This is not always possible since pumping rates often decrease as pumping continues, thus the danger of gellation in the equipment is always present.

(2) It is difficult to vary the induction period during pumping of a batch. For some applications, economy and varying water conditions necessitate changing the induction period.

(3) The most severe limiation is that, by its nature, it is not possible to use a batch system with very short induction period. Very short gel times are essential at some times during most applications.

7.3. FIELD CONTROL

7.3.1. Field control of grouting process is indeed a formidable problem especially when the work is carried out on the basis of limited exploratory data. The extent of pockets and lenses often may not be known from the exploratory holes (especially in alluvial deposits) and it is generally impracticable to obtain reliable permeability or grain size data during drilling of grout holes. In practice, solution of this complex problem is achieved by adopting a rigid field control over the injection of different types of grout. Field control is exercised on the following operations:

(1) Sequence of drilling holes

(2) Sequence of grouting holes

(3) Injection pressures

(4) Intake

(5) Upheaval

(6) Success of grout operation

(7) Guide to grout supervisors.

7.3.2. <u>Sequence of drilling holes:</u> There is no objection to completion of drilling in advance of grouting except when the stage grouting method is used. Clogging of grout pipes and its jamming into the holes by the hard setting grouts injected from neighbouring holes does not arise in the case of the tube-a-manchette method. The sleeve grout protects the grouting tube. Similarly if sleeve grout is used for the rising tube method, the objection to drilling in advance can be avoided. However, it is not possible to carry out drilling work while grouting is in progress within the influence zone of the hole which is being grouted, because the grout finds its way into the mud circulation or water circulation of the drill and makes the drilling operation difficult. This can be avoided by doing the drilling outside the influence of grouting, generally 30 meters.

7.3.3. <u>Sequence of grouting:</u> It is always desirable to create confinement while grouting alluvium. It is therefore desirable to grout the outer lines of the grout hole first and then take the interior lines proceeding always from outside towards the centre. The injection of the outer lines requires heavy consumption which should be reduced by using closer spacing and using a grout with more cement. The choice of spacing is evidently on relative costs of grouting and drilling. Lateral confinement is achieved by grouting the outer lines with material that sets early. For open and very coarse alluvium such setting type of grout is desirable in order to develop adequate resistance to wash out. Thus the right method would be to start with the treatment of outer lines with grouts that set well and for the central zone, the more fluid grouts which would penetrate the finer alluvium. In any line the general practice is first grout holes at interval of 4 times the normal spacing of holes. This is called the first stage and the method is known as split spacing method. Intermediate holes are grouted subsequently, reducing the spacing progressively.

For vertical confinement grouting is started with coarse sand zone followed by grouting of medium sands and finally fine sands are grouted. When any particularl zone or stratum is taken for grouting, the work is generally done from bottom upwards. However, a few manchettes at top of each hole are grouted initially to check the escape of grout.

7.3.4. Injection pressures: The control of pressures during grouting can be achieved by the observations of pressure intake trend with regards to time. Theoretically the resistance to flow of grout would increase as the grout travels away from point of injection and as the grout thickens. Hence the grout intake should progressively be reduced at a constant pressure or increasing pressure will be necessary to maintain a constant intake. With high pressures the increase in resistance is compensated by increase in zone of influence due to formation of the cracks. Hence it is reasonable to presume that the intake may remain constant at constant pressure. On the other hand, if the cracks tend to enlarge by upheaval the intake rate may increase at constant pressure. Such a trend is therefore definitely undesirable.

Control of pressure during grouting can, therefore, be exercised in the following manners:

(1) No restriction should be placed on the pressure for opening sleeve.

(2) After the sleeve is opened and intake starts, the pressure should be brought down to the overburden pressure and increased gradually.

(3) A trend of pressure and intake with regards to time should be observed continuously so that the pressure is immediately brought down if any tendency is noticed for the intake to increase with constant or decreasing pressures.

(4) Grouting should be stopped if upheaval is observed on the surface or high intake persists even after increasing the cement content and lowering the pressure.

(5) The grouting is then resumed only after a lapse of 12 - 24 hours.

In case of low pressures the spread of grout is attained entirely by permeation. The pressures are, therefore, restricted to overburden pressure or slightly above the overburden pressure. No upheaval is permitted and control of pressure is so exercised that the intake decreases at constant pressure or the pressure has to be increased progressively to maintain the desired intake rate. It may be obligatory to use fluid grouts to achieve the desired spread.

7.3.5. Intake: No attempt need be made to grout to refusal. It is better to limit the injection to a predetermined value. If injection is not found to be adequate after tests, it is better to return to the hole rather than attempt final and complete grouting in the first instance. It is not possible to fill all the voids space and the consumption of grout is 60 to 80% of the void space depending on the type of grout and the nature of alluvium.

7.3.6. Upheaval: In rock grouting no upheaval is permitted as it develops numerous fine cracks which increase the permeability of the strata as they are difficult to be grouted. In alluvial deposits, however, some are of the opinion to permit upheaval to take the advantage of high pressure grouting. Some engineers limit the upheaval in alluvial deposits at 1 cm/meter depth. At some of the dams in Europe viz; at Serreponcon where the depth of overburden is nearly 100 meters the upheaval due to high pressure grouting was in the order of 30 cm. or nearly 1/3rd of the limit specified above. When the grouted zone is overlain by a compact and brittle soil which is not grouted, it would be desirable to minimise upheaval and it cannot be conclusively said that high pressure grouting and upheaval would not damage the foundation. Impervious backfills are sufficiently plastic and the superficial cracked zone can always be excavated and removed after completion of grouting operations. In the case of sand and gravel deposits some upheaval may not be objectionable as in these deposits open cracks would not form.

Upheaval is measured by standard gauges installed in the area to be grouted. A typical diagram of a standard uplift gauge is shown in Fig.No.7.6. Uplift can also be measured by a crude method of levelling which may be fairly reliable for alluvial deposits but for rock formations it is desirable to install standard uplift gauges.

7.3.7. Success of grouting operations: Grouting cannot be successful without individual treatment of different lenses and pockets. Due to the wide range of the size of pore space of different adjoining pockets, it is desirable to inject in short passes of 1 ft. It is also necessary to return to the same elevation with different types of grouts so as to adopt the grout best suited to the type of alluvium met with.

It is necessary to prevent leakage along the grouting pipe and create adequate confinement so that sufficient pressure can be built up. Attempt need not be made to grout to refusal. As grout takes time to set, refusal cannot be achieved during grouting. Grouting must be stopped when:

(1) The predetermined consumption limit is exceeded
(2) Pressure does not rise even after prolonged grouting.
(3) Heave exceeds the permissible value.

There is no objection to stoppage of grout operation and resumption after passage of time because there is no single path of travel for movement of grout through alluvium. New paths can be created by application of high pressure and creating artificial small cracks. Such cracks are unavoidable and are not objectionable as long as the consumption of grout is not excessive.

7.3.8. Guide to grout supervisors: The grout supervisor should keep the following points in mind for successful supervision of the work:

1. Don't judge the efficiency of grouting operators solely on the number of bags reported as being injected. Look for leaks.

2. Don't change grout supervisors unless absolutely necessary. It is hard on the supervisors, but the experienced ones are able to secure instantaneous action on almost anything required to expedite methods or obtain more effective results. This may not be true if the grout crew does not feel that supervisor knows more about the work than they do.

3. Don't put too much of emphasis on static or standing pressure alone. Grout that is slightly too thick, pumped slowly, will close almost any hole, during the injection of grout.

4. Don't be too willing to call a hole completed because it suddenly builds pressure. This may be due to filtering action of grout.

5. Don't pump water into a hole longer than necessary. There is no advantage in needlessly pumping water into a free hole.

6. Don't be too hasty about deciding that a hole is tight. Hang on to it with water so long as there is any perceptible pick up in the pump speed at a normal pumping

14. Don't antagonise the contractors personnel by untactful approaches.

15. Don't fail to report any unusual incidents to the authority.

16. Don't forget that you are expected to direct the grouting procedure.

7.4. <u>FIELD PROBLEMS</u>

7.4.1. In the field following problems are generally encountered:

(1) Treatment of contact between rock and alluvial deposits.

(2) Grouting of top zone.

(3) Variation in pressure and intake, and

(4) Consumption of grout.

7.4.2. <u>Treatment of Contact zone:</u> The most difficult zone to be treated is the contact between bed rock and alluvial deposits. Ischy and Haffin[52] (1955) have described the treatment of the contact zone with clay-cement grout to form a roof. Then treat the rock with cement grout and regrout the roof with pure clay after the rock is treated. Once the roof is formed leakage does not choke the manchette as it passes through the soil and tube is protected by surrounding sleeve grout. In some cases where it is difficult to distinguish between large blocks of rock and bed rock, grouting may be carried out by descending stage method instead of tube-a-manchette. Similarly the contact between the backfill and alluvial deposits may also be treated by descending stage method. The permeability of the contact may be determined by point permeability test.

The consumption of grout in the contact zone may be very high say 10 times of theaverage consumption.

7.4.3. <u>Grouting of top zone:</u> From the experience of field trial tests at Serreponcon, Mayer[53] (1958) has concluded that grouting of shallow layers is impossible and certain weight is necessary to counteract the effect of high grouting pressures. Trials have shown that for the top 5-7 meters depth, perfect grouting results cannot be expected. The treatment of top layer can be successfully achieved by raising the embankment 5-7 meters height, and carry out the grouting from the top of the embankments.

7.4.4. <u>Variation in pressure and intake:</u> There are three cases of pressure intake relationship as shown in Fig.6.10.

(1) Case A: when intake decreases with constant pressure or when increased pressure is required to maintain the constant intake.

(2) Case B: when intake remains constant with constant pressure or when intake follows the pressure.

(3) Case C: when intake increases with constant pressure or intake remains constant at decreasing pressure.

The case A is an ideal case as resistance to flow of grout would increase as it travels away from point of injection and pressure applied is required to overcome this resistance. Hence the grout intake should progressively be reduced at a constant pressure or increasing pressures will be necessary to maintain a constant intake. In case B, zone of influence is increased with increase in pressure due to formation of cracks and intake may follow the pressures. In case C, the cracks are enlarged by heave caused by high grouting pressures and intake rate may increase at constant pressures. Such a trend is undesirable and may be avoided. Grouting should be stopped and further grouting should commence after a lapse of some period say 12 - 24 hours to settle the grout. The clay based grouts remain soft even after a lapse of 7 to 10 days.

7.4.5. Consumption of grout: It is not practicable to grout alluvium to refusal as grout would keep on spreading even after it has reached the limit of the zone required to be treated. To prevent overtravel the consumption of grout is restricted to a limit decided from void space in the zone to be treated with suitable margin for overtravel. While the consumption in the low pressure approach will be definitely restricted to the void space, in the high pressure approach some allowance may be necessary. In open material the resistance to penetration is poor and the grout will spread rapidly resulting in a constant rate of intake at constant pressure over prolonged period. Such a tendency is controlled conveniently by thickening the grout, and by providing the confinement by cheaper grouts in the outer rows.

7.5. QUALITY CONTROL

7.5.1. Numerous and various controls are exercised during the execution of the job. These are exercised by two separate organisations viz;

(1) Supervising staff, and

(2) Quality control staff.

The supervising staff is responsible for workmanship and efficiency of the grout curtain. Following tests are generally carried out to determine the efficiency of the grout curtain:

pressure. Many apparent tight holes loosen up and take grout freely after a few minutes under pressure.

7. Don't think that there is one optimum water and solid ratio for every hole. It changes as the hole approaches completion, generally increasing.

8. Don't pump thin grout into a hole that will take thick grout readily.

9. Don't attempt to reduce the water solid ratio too rapidly; you may plug the hole.

10. Don't fail to inject the grout at all times at the maximum allowable pressure or at such pressure as can be sustained throughout the grouting. Strata which lies above the natural ground water level will absorb the moisture from the grout causing it to thicken and slowly clog the voids in the formation.

11. Don't vary the pumping speed needlessly i.e. stopping unnecessarily or permit pumping at less than normal speed if there are no leaks.

12. Don't, when a test is made to see that the line is opened during grouting operations, permit the byepass valve to be closed quickly on a fairly tight hole. You will almost surely lose it.

13. Don't stand ready to pounce on the contractor for something gone wrong. But be alert to perceive and advice or warn him before the damage is done. Look ahead and advise others of details they may not yet have decided or considered. The supervisor who thinks everything is running smoothly may be neglecting his work, the unsolved problems and conditions of improvement.

Table No.6.1

Range of application of different grouts

Type of grout.	Granulometry of soil treated D-15 of soil.	Value of permeability lower limit.	Remarks.
Cement.	1.0.mm coarse sand & gravel.	0.5×10^{-1} to 10^{-1} cm/sec.	Depends upon fineness of cement.
Clay-cement bentonite.	0.8 mm coarse sand.	0.5×10^{-1} cm/sec.	d85 of clay under 30 microns.
Bentonite-chemicals.	0.4 mm medium sand.	10^{-2} cm/sec.	d85 of bentonite under 5 microns.
Chemicals (silicates)	0.1 mm fine sand & silt.	10^{-3} cm/sec.	Depends on viscosity and shear resistance of grout.

American Cyanamid Company[43] (1965) have given the range of different grout materials for treatment of pervious soils and mechanical methods for underground water control for various range of grain size of the formation materials, as shown in Fig.No.6.3 and table No.6.2. The upper six bars in Fig.No.6.3 show the different types of grouts applicable for different range of alluvium while the lower eight bars show the well developed and widely used mechanical water control techniques. The fact that there is so much overlapping of the materials and methods that each application should be carried out only after an economic consideration of all the different possible techniques. Such factors include time for completion of the job, number of personnel required, simplicity of application, control of the results and case of fitting the grout programme into the overall operation. In many jobs, particularly in large construction jobs, more than one grout mixes are used. Suspension grouts of cement-clay-bentonites are used for filling large voids and fine strata is finished with colloidal grouts based on silicates or organic monomers. Sometimes when there is a considerable flow of water in which the suspended grout may be washed away, it is desirable to inject first with organic grout of low gelling time to arrest the flow of water and then suspension grout is injected for filling of voids for the purpose of strengthening the foundation and for impermeabilization.

Table No.6.2

Range of different grouts

Grout.	Material grouted.	Finest size that can be grouted D10 in mm.
Cement.	Fissures.	0.1
Cement.	Soil.	0.65
Cement(fine 30 mircons)	Soil.	0.29
Cement.	Large sand.	0.5
Cement.	Dense sand.	1.4
Silicates.	Soil.	0.1
Silicates.	Loose,not too fine sandy soil.	0.12
AM-9.	Soil.	0.013

6.2.5. <u>The Planning of Injection:</u>

6.2.51. For the planning of the injections of a given pervious formation and for the desirable results, it is important to determine the following:

(1) Time of injection.

(2) Width and location of a grout curtain and No. of grout rows.

(3) Spacing, size and depth of grout holes.

(4) Consumption and injection pressures, and

(5) Check holes.

6.2.52. <u>Time of Injection:</u> The time of injection of a grout mix depends on the gel time. The time of injection can be determined mathematically from equation(7) Chapter No.5, knowing the type of grout to be used, permeability of soil to be treated and zone of influence of grout. Normally gel time is taken as 30 minutes unless this is accelerated or retarded depending upon the job conditions. For cement based grouts the gel time is controlled by initial setting time of cement which for ordinary portland cement is 30 minutes. Therefore all the operations of mixing and injections should be completed within that time. Clay based grouts remain in suspension if stirred continuously. Therefore, the injection time of clay based grouts depends upon the quantity of grout injected and the economy of operation. For newtonian grouts except organic resin grouts the viscosity increases with time. The rate of penetration of this class of grout is inversely proportional to the viscosity. The strength of silicate grouts is also varies inversely with the gel time. Therefore, the gel time of this class of grout should be small enough. Normally it is taken 30 minutes. Ischy and Glossop[7] (1962) state that at least 50% viscosity of its initial value should be maintained at 20 minutes. For organic resin grouts there is an instantaneous polymerization and hence the gelling time can be from a few seconds to few hours depends upon the job conditions.

6.2.53. <u>Width and location of grout curtain and No.of grout rows:</u>

(1) <u>Width of Grout curtain</u>: The width of grout curtain is an important factor which governs the efficiency and the cost of the curtain. The width of the curtain is taken as the distance between the outer rows plus the spacing between rows (to allow for spread beyond the outer rows of holes). In a wider curtain a few pockets of high permeability may not significantly effect the efficiency of the curtain, but in a thin curtain it is necessary to block almost the entire surface of the curtain. The grout curtain may fail due to erosion under high pressure gradient and it may cause piping. Lane[44] (1935) has made a study of 280 dam foundations, which include 150 failures. From these studies he has concluded that an average hydraulic gradient for foundation of gravels and boulders may be 3 and for fine sands may be 8. This corelation between foundation soil type and piping potential has some value as a guide to judgement, but it should not be used as a design criteria. Sheraard[45] (1963) has classified the width of the clay cores which can fairly be taken as a width for a grout curtain. These are:

(i) Cores with a width of 30 to 50% of the water head have proved satisfactory on many dams under diverse conditions and is adequate for any soil type and dam height.

(ii) Corse with a width of 15% to 20% of the water head are considered thin but, if adequately designed they are satisfactory under most circumstances.

(iii) Cores with a width of much less than 10% of the water head have not been used widely.

Ambraseys[46] (1963) have co-related the width of grout curtain, with the base width of dam, depth of alluvial deposites, permeability of soil, before and after treatment and is shown in Fig.No.6.4. He has calculated that wider grout curtains are more efficient to arrest the seepage water. It is a normal practice to provide a cutoff of width of 1/3rd to 1/5th of the water head tappering at the lower depths in case when permeability of soil decreases with depth as for the same efficiency of cutoff low permeability ratio would require less width of curtain as shown in Fig.No.6.4. In case of homogeneous soil it is advisable to continue the same width of the curtain to the impervious strata.

(2) Location of grout curtain: The location of grout curtain is governed by the location of the impervious core in the body of the dam. If the impervious core is centrally located then the grout curtain is also located at the centre of the dam confined within the base width of the core. If the core is inclined towards upstream then the grout curtain is also provided towards upstream but confined within the base width of the core. When the section of the dam consists of a homogeneous material which is fairly impervious then the best location of grout curtain would be just near the upstream toe of the dam to keep the base of the dam dry and thus reduce the pore pressures. In no case the grout curtain is to be provided at the downstream toe of the dam or towards downstream toe as it will increase the pore pressures in the dam.

(3) Number of grout rows: For alluvium grouting, the grout curtain should be formed of minimum 3 rows of grout holes, in case of rock grouting, curtain may be of a single row of holes though a tow row curtain is preferable. The No.of rows of grout holes depends upon the minimum width of curtain required, and spacing of holes.

6.2.54. Spacing, size and depth of grout holes:

(1) Spacing of grout holes: The spacing of grout holes depends upon the type of grout to be used, permeability of soil, pressures to be adopted and time of injection. Knowing the time of injection we can calculate the radius of zone of influence from equation (7) and Fig.No.5.1 of Chapter No.5.

For rough estimates spacing of holes can be taken as given in Table No.6.3.

Table No.6.3.

Rough estimate of spacing of holes

K cm/sec.	Depth 30'-60'	Depth / 60
10^{-1}	9	11
5×10^{-2}	7	8.5
10^{-2}	5	6

(2) Size of Holes: The equations 2 and 3 of Chapter No.5 indicate that the size of hole should be large enough to inject a given quantity of grout in short time. In small diameter holes say ½ to 1" size the grout stiffens in the pipe and is difficult to clean them afterwards. In large size holes the grout segregates. Holes used for grouting are ½" to 5" diameter size. U.S.B.R. have specified a minimum diameter

of 1 3/8" for grout holes. Most grout holes are drilled with EX bit which gives a hole of approximately 1½" diameter. In some of the works where a size of grout hole was specified as 1½" diameter the contractors have drilled larger size holes at their own expenses in order to avoid difficulties and delays due to the hole caving in and squeezing. When grouting is done with rising tube method or tube-a-manchette process the size of hole varies from 3" to 4". It is also customary to drill about 10% of holes with NX size bits which give a hole of 3" diameter to obtain cores to explore the foundation materials, which are later grouted or left ungrouted as desired. It is also a practice to drill one or two large diameter hole of say 1 meter size in grouted zones to inspect visually the efficacy of grout penetration in voids of the soil and to conduct permeability tests. This large size hole may be plugged subsequently.

(3) Depth of holes: The depth of grout holes should be chosen primarily on the basis of the results of engineer's and geologist's study of the conditions of bed rock under-lying the alluvium deposits and on the purpose of the grout curtain. Depth of grout holes should never be chosen on the bases of some predetermined formula or on the basis of the average depth of grout curtain used under other dams. In the design stage the engineer determines the approximate depths of grout holes which are flexible and more accurately depends upon the geological formation in individual holes. The depth of hole also depends upon the type of cutoff to be provided. The site where the average permeability of the foundation soil decreases with depth below the surface or when there is a single conti-

nuous impervious layer into which the cutoff can be connected. In case when formation is homogeneous and permeability does not change with depth, even curtain of 95% of the full depth are not effective to arrest seepage. For this reason, only a complete cutoff should be provided and keyed into sound rock.

6.2.55. Consumption & Injection Pressures:

(1) Consumption: The consumption of grout in case of rock grouting is assumed as Kg of dry material per metre depth of hole whereas for alluvial grouting consumption of grout is calculated on the basis of soil mass treated. The soil contains voids ranging from 25 to 35% and can be determined by laboratory tests. For estimate purposes voids may be assumed 30% of the soil mass.

The consumption of grout is related to the injection pressures, viscosity of grout, gel time, permeability of the soil and spread of the grout. From knowledge of the above properties of grouts the quantity of grout required to spread in the zone of influence can be calculated by the equation(5) Chapter No.5. For the purpose of estimating the consumption of grout may be assumed 30% of the soil mass treated. When low pressure grouting is adopted, the allowance for overtravel is implicit in the assumption of 30% because all the voids in a soil mass can never be filled by grouting. With high pressure grouting, a limit of grout consumption of 45% of the soil volume can be realized with good control. In the absence of close control on consumption of grout, consumption of 60% of soil volume may be allowed.

First viscous grouts are injected followed by grouts of lower viscosity and greater penetrating power, to impregnate the finer grained and less permeable beds. In this way the degree of saturation of the soil can be gradually increased. Thus where it is wished to reduce the flow of water through an alluvial deposit it may be sufficient to seal the more permeable beds only, but where the grouted zones is to become part of a permanent structure, and strength is required as well as low permeability, then the ground should be saturated as completely as possible. Ischy and Glossop[7] (1963) have stated that in practice, complete saturation of the soil is never obtained, except perhaps with a sand of almost uniform grain size. Taking into account the loss of excess water from certain grouts under the pressure of injection, experience has shown that under normal conditions an alluvial materials will have been satisfactory treated when the volume of grout injected is equal to about 50% of the volume of the void. The percentages of voids that can be filled by various grouts is given in Table No.6.4.

Table No.6.4.

Consumption of various types of grouts.

Type of grouts.	Allowance for overtravel.	%age of voids filled by grout slurries.
Clay-cement.	½ of the spacing of grout holes.	60 - 70
Bentonite-Chemicals.	-do-	70 - 80
Chemicals.	-do-	80 - 90

(2) <u>Injection Pressures</u>: If the shear perameters of the soil foundation and the in-situ stresses and water pressures are known then the permissible injection pressure can be calculated

by the equations given in Chapter 5.4. In common practice permissible injection pressure is taken as equal to the weight of the overburden i.e. 1 lbs/sq.in. per foot depth of soil or 0.2 Kg/cm^2 per meter depth of soil.

The injection pressure is also governed by the system of grouting i.e. by permeation or by fracturing. If the grout travels by permeation, then pressures are to be applied roughly equal to the weight of overburden. But it is not practicable as the viscosity of grout increases with time and increase in shear strength restricts the spread of grout under permissible overburden pressures. Mathematically spread of grout can be calculated for the type of grout used and soil conditions. Minimum hydraulic gradient required to maintain the spread of grout can be found out from table No.5.2 which will give the injection pressure required to permeate. But this mathematical solution evolved for the rate of spread of grout is approximate and it is difficult to predict the actual spread of grout in the field. The limiting pressure for groutings has been a matter of great controversy and the wide difference exists between the practice followed in Europe and in U.S.A. The American Practice limits the pressure to the overburden pressure while pressures exceeding 5 times overburden pressures have been used with success in Europe.

The grouting pressures should be as high as the job conditions will permit but should not be excessive to open up the cracks and cause the grout to spread beyond the required zone of influence. On the other hand high injection pressure means a higher gradient for the flow of grout, which increases the velocity of grout to flow and permits thicker grout which increases the rate of penetration thus allows lower labour and machinery cost per unit of volume injected. Higher pressures also increase the spread of grout and thorough penetration is ensured with a wider spacing of holes apart from the saving in No. of holes to be drilled. Ischy and Glossop[7] (1962) have stated that fissure formation although undesirable is sometimes provoked deliberately by high pressures to create a network of grout filled fissures between which the silt pockets which are difficult to grout are encased and protected against internal erosion. High pressures in rock grouting may cause heave beyond the permissible limits and open up the fine cracks thus increase the permeability. In case of pervious foundations the higher pressures may open the passages either horizontally or vertically and cause the grout to escape causing negligible upheaval. When cracks are formed at the surface due to upheaval the backfill can be excavated and refilled after completion of grouting operation. Therefore, the choice between the high pressure and low pressure system of grouting depend upon the type of the structure and characteristics of the foundation soil. When the grain size of the soil is closed to the limit of penetrability of low cost grouts, restrictions on pressure might lead to an uneconomical spacing of holes. It may be dangerous to resort to a wider spacing of holes in such cases and to inject a low strength grout at low pressure in a soil with large pore space capable of absorbing the coarser high strength grouts. As the high strength chemical grouts are either not available or prohibitively expensive, the choice of grouting techniques is governed by the balance between the cost of increased grout consumption in the high pressure system except when the cracking and upheaval is likely to damage a rigid structure overlying the grouted zone. It is, therefore, desirable to undertake field trials to assess the injection pressure that can be applied to a grout selected for injection to permeate the zone of influence assumed.

6.2.56. <u>Check Holes:</u> Check holes are provided mostly on inner row of holes to determine the efficacy of grouting by pumping in or water test. For rock foundation Lugeon(1932) has given that the water intake should be less than 1 lugeon which is equal to loss of water of 1 liter per meter per minute at a pressure of 10 Kg/cm^2 tested for 10 minutes. In British units this is equal to 0.067 gallon per foot of hole per minute at a pressure of 142.2 lbs/sq.in. One lugeon is generally accepted as corresponding to a permeability of approximately 10^{-5} cm/sec. Lugeon's recommendations were based on experience of gravity dams founded on fairly good rock in Switzerland and this recommendation need not necessarily apply to soft rock foundations. At Mangla Dam water loss of 1 to 2 lugeon was considered safe. For alluvial deposits Neelands[47] (1963) has given that in practice the lower limit of permeability attained by grouting is of the order of 10^{-5} cm/sec. Relatively open grounds are generally easier to reduce to that figure than ground which started with a high degree of impermeability, but this depends on the presence or absence of finer material in the range 10^{-3} to 10^{-4} cm/sec. which may make the final limit relatively hard to reach. There are reports that fine gravel treated with CEMEX-A gave a figure of about 10^{-7} cm/sec. and treated by AM-9 solution gave a figure of about 10^{-10} cm/sec. But it is difficult to determine the permeability below 10^{-7} to 10^{-8} cm/sec.

At Aswan High Dam a criteria for reduction in permeability after grouting was fixed at 5×10^{-4} cm/sec. at Mangla dam it was considered to be 5×10^{-5} cm/sec. and at Girna Dam it was considered less than 10^{-4} cm/sec. At Aswan High Dam high permeability was considered perhaps because of provision of impervious blanket and relief wells.

6.2.6. <u>The application and equipments:</u> The selection of application of grout and grouting equipments to be used depend upon the type of grout to be used. For suspended grouts collo-idal mixers are preferable to conventional type paddle mixers. For cement grouting circulating type piping system is preferred whereas for clay grouting single line system is better. Silicate grouts are pumped in the soil by two shot process or a single shot process. Single shot process is better as gelling time is controlable whereas in two shot process gellification is instantaneous. Organic resin grouts are pumped into soil by three basic types of equipments which are:

(1) Proportioning system

(2) Two solution system, and

(3) Batch system.

Of these proportioning system is the best and easiest to use.

Grouts are injected by:

(1) Descending stage method.

(2) Rising tube method, and

(3) Tube-a-machette process.

Of these tube-a-machette process is the best as the manchette tube can be left in the ground and same hole can be grouted more than once, and if necessary with grouts of different compositions. Grout pumps mostly used are of double reciprocating duplex type. Centrifugal pumps specially equipped for grout injection are now-a-days commonly used.

6.2.7. Economics: Chemical grouts are expensive comparative to suspended grouts. The suspended grouts cost about 20% of the cost of injection process. Therefore, low cost grouts may be used for coarser strata of coarse sand and gravel and medium to fine sands may be treated with chemical grouts. Where the foundation strata consists of medium to fine sands the outer lines may be injected with low cost grouts under high pressure to form barrier and inner lines may be injected with chemical grouts to avoid excess consumption due to low viscosity of the grout. Sometimes even the upper layers of the soil which are more permeable are treated with low cost grouts to provide confinement for costly chemical grouts injected for lower stratas. Mollex[48] (1963) has worked out typical figures for the net cost of grouting materials per cft. of finished grouts as shown in Table No. 6.5. These costs are for materials only contained in 1 cft. of finished grout and do not include the labour and plant to prepare and mix the grout. Furthermore the costs can vary from place to place according to availability of materials and will also depend on mix design.

Table No. 6.5

Costs of various grout materials.

Gꝏut mix.	Cost in S-d
1:2 clay cement.	2-0
3:1 fly-ash cement.	2-0
Bentonite-silicate.	2-8
1:1 sand cement.	3-0
Silicate.	3-6
Neat cement.	6-0
Guttman mix.	6-3
Joosten mix.	7-2
Urea Formal-dehyde.	10-9
Resorcinol-formal-dehyde.	27-0
AM-9	35-0

Mistry[49] (1965) has worked out the net cost of 1 cft. of hardened grout for various grout mixes as shown in Table No. 6.6.

Table No. 6.6

Cost of various grouts

Sl. No.	Grout Mix Clay.	Bentonite	Cement.	Water	Cost in Rs./cft of grout.
1.	-	-	1.0	4.0	4.19
2.	-	.4	1.0	5.0	1.11
3.	0.8	0.2	0.5	5.0	0.74

Neat cement grouts are expensive because neat cement grouts tend to bleed heavily, as such occupies less volume when set than fresh neat cement grout. Therefore to yield 1 cft. of hardened grout more than 1 cft. of fresh neat cement grout is required. For the clay based grouts due to the swelling properties of clay, bleeding is negligible and one cft. of fresh grouts yields approximately the same volume when set.

Joosten grouts are generally more expensive than the soft gel grouts for the reasons:

(1) The silicate is pure instead of being diluted, which increases the cost of the injected material.

(2) As the joosten process consists of 2 stage injection in the ground a system of very closely spaced bore holes are required.

6.3. DESIGN OF A GROUT MIX

6.3.1. The general procedure for preparing and designing a grout comprises mixing of different materials viz; cement, clay, bentonite, chemicals etc. in varying proportions. These materials may not be used all at a time and their selection depends upon the requirement of the job conditions. Mixes should always be prepared by adding different solid materials in their order of fineness. Water is poured first, reagents for obvious reasons are mixed last. Laboratory mixing is normally done with high speed mixers whereas in the field standard grout mixers are of much slower speed, and this point may be considered in laboratory mix design. The grout mix should satisfy the following properties:

(1) Specific gravity

(2) Consistency

(3) Fluidity

(4) Stability

(5) Gellification

(6) Permanence.

(7) Rigidity.

Each of the above properties can be verified and tested in the laboratory as described below.

6.3.2. Specific Gravity test: Specific gravity is the ratio of weight of grout to the weight of equal volume of water. The specific gravity influences the viscosity and the consistency of the grout. Specific gravity of mixture should be greater than water in the range of 1.05 to 1.15 to drive it out from the pores of the soil.

Procedure: This is a standard laboratory procedure in which a standard specific gravity bottle is used for determining the specific gravity. However, for a grout different apparatus is used. In which nozzle is dipped in the grout, the ball is pressed and then grout is sucked in the cylinder slowly in sufficient quantity to keep the float inside the cylinder floating. The reading given on the stem of the float gives directly the specific gravity of grout mix.

6.3.3. Consistency: The consistency of a grout mixture is related to the viscosity coefficient. There are various viscometers available for the measurements of consistency of a grout mixture. The consistency determined by rotational viscometer is described below. In this viscometer the force developed in the grout, that will resist the motion is proportional to the area in contact and the velocity gradient:

$$F \qquad A \times \frac{dv}{dr}$$

$$F = n \times A \times \frac{dv}{dr}$$

Where n is the coefficient of viscosity.

Procedure: The rotational viscometer consists of two concentric co-axial cylinder. The outer cylinder may have a variable speed of rotation, while the inner cylinder is fixed to a torsion wire. The space between the two cylinders is filled with the suspension or liquid to be tested, and the deflection of the internal cylinder is noted for each velocity of the external cylinder.

A predetermined quantity of grout is poured into the outer cylinder, keeping the inner cylinder raised above the top of the outer cylinder. The inner cylinder is then lowered into the outer cylinder slowly and trucly vertical, so that no grout can enter the bottom cavity of this cylinder. The outer cylinder is rotated slowly and its speed is increased from about 2 r.p.m. to about 50 r.p.m. gradually by means of arrangement consisting of a motor, a variometer and a worm reducer. During this period the rotation of the inner suspended cylinder is observed and the angle of its rotation at different speeds of the outer cylinder is measured with the help of "a lamp and scale arrangement". The speed of the outer cylinder is then gradually reduced to 2 r.pm. and the angle of rotation of the inner cylinder at various different speeds are measured. Graph is plotted of the values of different speeds of the outer cylinder as ordinates and the corresponding values of angle of rotation of the inner cylinder as Abscissa.

If ϕ is the angular velocity of the outer cylinder, Rc its internal radius, the external radius of the inner cylinder, n the height of the fluid in contact with the inner cylinder and M the moment of external force applied to ensure that the internal cylinder will be stationary, it can be shown that for an ordinary newtonian fluid of viscosity n

$$\phi = \frac{M}{4 \Pi hn} \left(\frac{1}{RP^2} - \frac{1}{Rc^2} \right)$$

and that for Bingham fluid

$$\phi = \frac{M}{4 \Pi hn} \left(\frac{1}{RP^2} - \frac{1}{Rc^2} \right) - \frac{Tf}{n} \log \frac{Rc}{RP}$$

The relationship between ϕ and M is linear but the line does not pass through the origin.

When the suspension is thixotropic the characteristics vary with time and the state of previous agitation of the liquid. If, starting from a suspension in a liquid state, the angular velocity of the cylinder is increased or decreased, the plot of moment against angular velocity will be of a loop shape. The value of ϕ gives the yield value of the grout.

6.3.4. Fluidity:

Fluidity is the reciprocal of viscosity, and bears an approximate linear relationship with viscosity.

Procedure: Fluidity of a grout is measured by observing and noting the flow of grout from a standard apparatus known as a Marsh cone shown in Fig.No.6.5. It consists of a cone 9 inches high and 7 inches in diameter at the top. At the bottom of the cone, a highly polished 3/16" nozzle is provided through which suspension of 1725 c.c. is allowed to flow out and the time of the flow noted. Time of flow of equal quantity of water is also noted for comparison. In some laboratories the diameter of the nozzle is adjusted to give 27 seconds of time for flow of 1000 cc. of water and the time of efflux of equal amount of grout is noted and compared with the time of efflux of water.

6.3.5. Stability: The grout mixture should be stable when sets without separation of free water known as bleeding.

Procedure: 1000 c.c. of fresh grout mix is poured in jars and the grout is allowed to set for 48 hours in the jars. At the end of 48 hours volume of free water that may separate out at the top is measured and is expressed as a percentage of 1000 c.c. of the original grout. The solids level indicates the amount of contraction, or, if the grout contains an admixture producing expansion of the grout, the amount of expansion.

6.3.6. Gellification: The time of gellification is defined emperically as the time at which there is no appreciable deformation. The grout after injection should gellify and set. This property will permit greater speed of grouting in the field, because if the grout injected in a hole does not gellify early it would delay the grouting of adjacent holes.

Procedure: Fresh grout mix is poured in beakers of 5 cm. diameter and 100 c.c. capacity. Then at the interval of 15 minutes, these beakers are tilted and behaviour of the top surface of grout is observed. The time at which this free surface of grout does not show any deformation when the cylinder is tilted is noted and is termed as time of gellification. The time of gellification varies from a few seconds to few hours depending upon the job condition.

6.3.7. Permanence:- The grout should remain permanent in position when set and should not be washed away under high hydraulic gradients imposed upon. In the laboratory the test is carried out under higher than the hydraulic gradient imposed upon to safeguard against other disturbance in the foundation which may be caused due to collapsing of structure or earthquakes. This test measures the stability of the grout, its resistance to wash out forces and permeability of alluvium after grouting.

Procedure: Steel cylinder or perplex glass sheet cylinder 4" internal diameter and 12" long are used. These cylinders are first cleaned from inside and they are coated inside with bentonite cement mix. The cylinders are then filled with alluvial deposits and tamped slightly. The moulds are then grouted and after 24 hours the cylinders are removed from the moulds and kept in water for 6 days. After 6 days these are removed and then fixed in permeability moulds. The moulds are then connected to the manifold shown in Fig.No.6.7. The water vessel is then filled to about half of its capacity with water, keeping the cock at the bottom closed and calculated pressure is developed in this vessle by joining it to air compressor. The cock is opened and water under pressure is allowed to enter the grouted alluvium from top of the cylinder. The bottom cock of the permeability moulds are then opened and water coming out from the cock is collected. The colour of the coming water is then compared with distilled water by visual observation and wash out of the grout if any, is examined. If the water has even slight test of turbidity the mix of the grout is changed and this test is repeated. After several such trials a stable grout is determined for its final use in the field. If the water collected is clean as distilled water, the rate of flow is measured and permeability of grouted alluvium is calculated by Darcy's law.

Sometimes during the wash out tests through pipes are developed along the sides of the cylinder and give impression of washout of grout and high permeability results. This factor should be thoroughly studied and investigated before concluding the inference from the results. If such through pipes are developed they can be very well sealed by plastic clay or wax.

6.3.8. Rigidity: This test measures the rigidity or stiffness of the grout. It is, however, not possible to recommend any value of the strength in gram/cm^2, but it is better to determine this value on various types of grout mixes which prove successful in the wash out tests. After determining the different values, this turns out to be quick test to check various trial mixes of grout.

Procedure: This test measures the strength of the grout as the intensity of pressure in gr/cm^2 that is required to be applied to penetrate a wooden needle having a cross sectional area of 1.18 sq.cm. through a depth of 2 cm. in 5 minutes. In this test fresh grout is poured in plastic cylinder of 7 to 8 cm. diameters and the grout is allowed to set. After 24 hours, test is started. The mercury is allowed to fall in the beaker placed on the top of the needle. The rate of fall of mercury is varied and so adjusted that the needle penetrates 2 cm in 5 minutes.time. In practice this criteria involves observational errors due to personal judgement. The rate of penetration of needle is never uniform and 5 minutes is apparently taken as a time limit rather than actual period of penetration. This test is used only for preliminary trials.

6.3.9. Method of Laboratory mixing of grout:

6.3.9.1. Suspension grouts: The mixing vessel is shown in Fig.No. 6.6. The estimated quantity of water necessary to produce a grout of desired consistency is placed in the mixing vessel, the motor is started, and the rheostat is adjusted so that the speed of stirring is about 500 r.p.m. The cementing material is introduced gradually into the mixer over a period of 1 minute. The time of starting this operation is considered as the start of mixing. As the cementing material is fed into the mixing vessel, the speed of stirring is increased gradually until the maximum speed is attained. Three minutes of mixing is sufficient to produce a homogeneous mixture. If the grout is to contain sand, the sand is introduced into the mixture after 3 minutes of mixing, gradually pouring it into the mixing vessle over a period of about 1/2 minute, and continuing the mixing for an additional 2½ minutes. The total elapsed time in this case is 6 minutes from the time of starting.

For suspension grouts the order of adding grout material to be kept as clay, bentonite, cement and the deflocculating agent (D.A.) Bentonite is added prior to cement as ordinary portland cement contains more than 95% of complex compounds containing calcium as one of the elements and base exchange will take place. D.A. is added on percentage basis reckoned with respect to the combined weight of clay and bentonite and is added in the form of a solution. Results of laboratory tests on grout mix carried at Ukai Laboratory are described by Mistry[49] (1965). Two grouts mixes were investigated in the laboratory viz;

(1) Bentonite + Cement + Water + D.A.

0.4	1.0	5.0	variable.

(2) Clay + Bentonite + Cement + Water + D.A.

0.8	0.2	0.5	5.0	variable.

In the field it was found that the properties of grout mixes prepared in the laboratory and at site differed considerably although the proportions of the mixes were identical and the ingredients were of the same quality. The reason was that the order of mixing the ingredients was different; this is also emphasized by the King and Bush[18] (1961). Observations made during mix design at Ukai laboratory are given below:

If cement is added prior to the addition of bentonite, the marsh value is very low irrespective of the stage at which the deflocculating agent is added. The gelling time is also high, compared to the gelling time when bentonite is added prior to the cement. Similarly the specific gravity is also somewhat higher. The most remarkable difference is in bleeding and needle resistance test, the results of which are very high. The bleeding ranges from a minimum of 25 to maximum of 47%. The 7 day needle resistance values from 9400 gr./cm^2 to very high values. This, however, is not profitable in view of the very high bleeding observed. These results indirectly indicate that due to the cement added prior to bentonite, some Ca^{++} ions must have formed and replaced the Na^+ ions of the bentonite as a result of which the swelling of bentonite must have been marred. This accounts for the very low marsh cone values and very high bleeding percentage observed. If the D.A. is added first, or in between the bentonite and cement, the bleeding is high when it is added after adding bentonite and cement, the bleeding is less than 5%. Also the reduction in marsh cone value is greatest in the last mentioned order. Thus the sequence of adding as first clay, bentonite, cement and then D.A. appears to be the most suitable. Two chemicals were used as deflocculating agent viz;

(1) Sodium-hexameta-phosphate.

(2) Sodium carbonate.

The 2% addition of sodium hexameta phosphate gave better results, there was no wash out of grout and permeability was less than 10^{-4} cm/sec. but its cost is prohibitive and hence sodium carbonate which is cheaper was tried. 4% addition of sodium carbonate resulted in decrease in gelling time, increase in specific gravity, decrease in bleeding and the needle resistance values increased considerably.

The test results at Mula Dam laboratory carried by Datye and Vinayaka[27] (1965) reveals that the viscosity of bentonite-cement is much less than clay-cement grouts(Marsh cone value 30-33 versus 40-45 clay cement). Penetration resistance also related to cement content and decreases with leaner grouts. The wash out resistance results indicated that clay-cement grouts needs to be richer in cement when grouted in sands of 0.8 - 2 mm.range. It is difficult to achieve uniform penetration and good resistance to wash out with sands of lesser sizes near 0.8 mm. As revealed by the wash out resistance tests, a stronger grout is required in larger voids space to resist seepage under pressure gradients. The needle resistance of clay bentonite cement grouts has to be over 1000 grams to resist wash out in a sand skeleton of 0.8-2.0 mm. range, while a resistance of 300 gr. is sufficient in case of silicate grouts for the same purpose in sands of 0.2 to 0.4mm. It is, therefore, essential to ensure that all void spaces capable of accepting coarser and stronger grouts are filled before commencing the grouting of the finer material, the softer gels will be suitable for injecting the relatively finer materials. To prevent the over travel of grout when the foundation material is coarse and highly pervious, addition of cement is helpful in clay cement grouts.

6.3.9.2. Silicate Grouts:- The solution of sodium silicate and precipitants are prepared separately of known dilution and are mixed together in laboratory in high speed stirrer as shown in Fig.No.6.6. Laboratory tests carried out at Mula Dam laboratory by Datye and Vinayaka[27] (1965) and their results are summarized as below.

Three precipitants viz; sodium aluminate, Ammonium sulphate and sodium bicarbonate in varying proportions were tried. The effect of these on the properties of grout mix are given below:

(i) Gel: The gel formed ranged in colour from dull white to bright white in the order of the type of precipitant used aluminate, sulphate and bicarbonate. The brittleness also increased in the same order.

(ii) Gelling time:- The increase in the doze of precipitant increased gel strength and reduced gel time, but there is a limiting dozage beyond which only the gelling time is reduced and the strength actually is very poor. A reduction in strength was noticed with 20 gram doze of sulphate, with 8 to 10 gram of aluminate and 30 grams of bicarbonate in 100 cc. solution of sodium silicate.

(iii) Gel strength: As mentioned above the gel time and gel strength bear a relationship upto a given doze of precipitants, an increase in precipitants increases the gel strength. The results showed that the ammonium sulphate appeared to give a better coverage than the aluminate or bicarbonate in producing gels of equal strength or gel time.

(iv) PH: It was observed that PH of the fresh grout did not bear any relation with gel time or strength. Viscosity results also indicated that it practically remained unaltered in 20 minutes after mixing.

(v) Bleeding & shrinkage: The shrinkage of gels due to 'syneresis' is known to be of frequent recurrence in dilute gels. The shrinkage of gels increased in the following order:

Aluminate	(0-4%)
Sulphate	(0-6%)
Bicarbonate	(0-40%)

The extent to which syneresis occurs depends on the system but in general decreases with increasing concentration of gelling agent.

(vi) Wash out resistance: The data obtained showed that in 0.4 to 0.8 mm. sand grouted with different mixes with hydraulic gradient of 10 there was no wash out and permeability was in the order of 1.7×10^{-4} cm/sec. Even for 0.2 - 0.4 mm. sand specimen under hydraulic gradient of 30 - 60 the specimen have high resistance and the permeability coefficient was around 10^{-4} to 10^{-6} cm/sec.

The test results indicated that sodium aluminate and ammonium sulphate are preferable to sodium bicarbonate as precipitants but the aluminate gel is more susceptible to temperature fluctuations, this is also confirmed by laboratory tests results carried out at Ukai and shown in Table No.4.2. Though the sodium bicarbonate has been used on many jobs, the use of this precipitant is not considered advisable in view of high syneresis than at early ages and also its poor durability.

6.3.9.3. Organic resin grouts: Solutions of organic monomers and catalyst are prepared separately and added only when required as gelling starts with mixing the two solutions. Solutions are prepared in conventional mixers and sotred before use. There is no laboratory record which gives the properties of the mixture and their effect. These grouts are patent and the proportion of various mixes and properties are generally being given by the firms.

6.4. FIELD GROUT TRIALS:

The object of the field grouting trial is to prove the practicability of establishing a satisfactory penetration of grout in the alluvial deposits at a given work site. Field test determines the spacing of holes required, pressure of injection to be applied, consumption of grout per unit volume of mass treated, and the consistency of grout required and its co-relation with the mix designed in the laboratory, permeability of the alluvial deposits before and after grouting and estimating the efficiency of the grout curtain to check the seepage water. The scope of the test may depend upon the economy and time available to carry out. Test plot should be minimum 30 ft. long to have sufficient number of holes, preferably it may be 100 ft.long for better co-relation of results. It should be located outside the seat of the dam or may form part and parcel of the main grout curtain. The sequence of field grouting trials are:

(1) Drilling of holes.

(2) Water test

(3) Grouting injection

(4) Water test

(5) In-situ observation.

After the hole is drilled to the required depth determined from the rock levels, water intake and slurry intake tests are carried out. These are very useful for assessing the groutability of different stratas. Samples of the soil shall be collected and tested in the laboratory for mechanical analysis. Water tests shall be conducted in the test hobes and grout injected in the hole and co-relation of water loss and grout consumption shall be determined. The water intake is generally observed at or lower than overburden pressure depending upon pore pressure in the hole. Intake rate of grout slurries can also be found out by adopting similar test procedure. The water intake tests can only give an idea of the relative permeability of the strata. It is, therefore, desirable to compare the permeability test results with the water intake test results. Poor co-relation between these would indicate defective procedure of water intake test or unreliable procedure for permeability test.

Water intake tests carried at Mangla Dam are shown in Fig. No.6.9. In Fig. a and b water absorption is directly proportional to pressure, Fig. c, d, e and f show fissure formation with increased pressure.

The grout is injected in the holes at varying pressures and pressure intake curve as shown in Fig.No.6.8 are

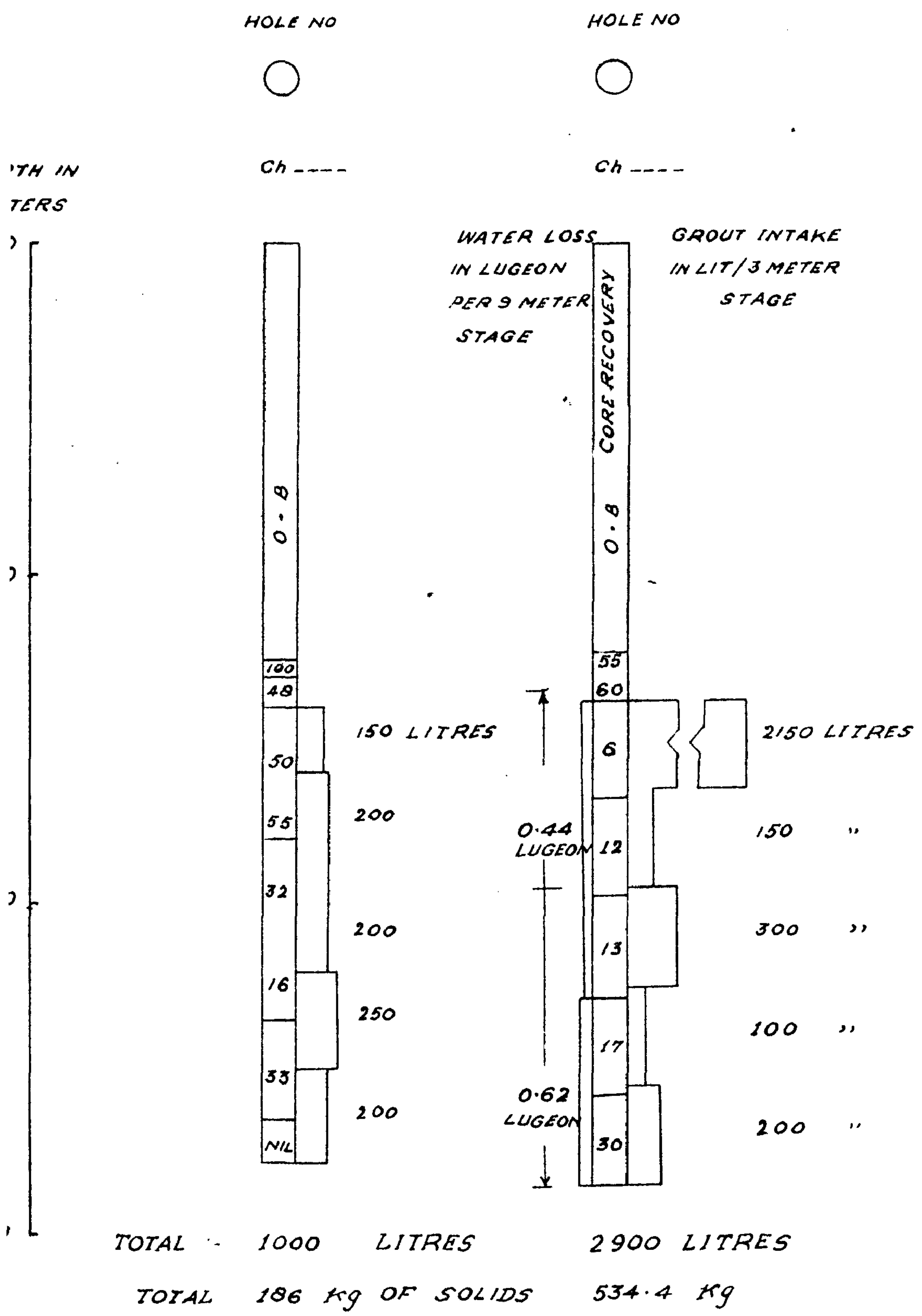


FIG NO 7.7. SHOWING THE DRILLING AND GROUTING DATA OF GROUT HOLES

CHAPTER - 8

ECONOMICS OF GROUTING

8.1. GENERAL

The economics of grouting operation depends upon

(i) Cheap cost of grouting operation, and

(ii) Should be compatible with other methods of cutoff.

8.2. COST OF GROUTING OPERATION

8.2.1. The cost of grouting operation includes the cost of the following components:

(1) Cost of exploration and field grout tests,

(2) Cost of installation of drilling and grouting equipments on site,

(3) Cost of drilling holes,

(4) Cost of preparation and injection of grout in the holes,

(5) Cost of testing the efficacy of the grout, and

(6) Overhead charges.

8.2.2. Cost of exploration and field grout tests: The cost of the preliminary investigations and foundation explorations are covered under the separate sub-head of the project estimate viz; under sub-head "A-Preliminary". On the basis of these investigations, specifications and drawings of foundation treatment by grouting are prepared. The grout pattern and mix provided in the specifications are tentative and may not be applied directly in the field before ascertaining its suitability. Therefore, it is necessary to take some exploratory holes and field grout tests to determine the grout pattern and mix to be adopted. No general rule can be framed for estimating cost of exploration and field grout tests. The following schedule may be useful for preliminary estimate and may be included into the overall cost of the grouting operation.

i) The bore holes 150-200 mm. dia. at 30 meters c/c may be drilled for sampling and permeability tests.

ii) Large diameter bore holes 450-500 mm. dia. at 150-200 meters c/c may be drilled for pumping tests to determine the average permeability.

iii) One or two exploration pits to full depth or partial depth may be taken to determine the characteristics of the foundation materials.

iv) Field grout tests in a test plot of length preferably four times the spacing of holes may be taken to determine the pattern of holes, grout mix, injection pressures and permeability to be achieved in the field. This test plot may or may not form the part of the main grout curtain.

The cost of exploration and field grout tests may range between 5 to 10% of the overall cost of the grouting operation.

8.2.3. Cost of installation: It is better to estimate this item separately as the cost is not dependent on the total volume of grouting but on the size of the installation, which includes the cost:of;

i) Electrical wiring, switch gears and installation of generators.

ii) Water supply pumps, storage tanks, and pipe lines.

iii) Compressors and air pipe lines.

iv) Clay slurry storage tanks and their processing plants.

v) Grout station installations which includes, grouts mixers, injectors, grout pipe lines etc.

The above cost may be based on the plant layout for any specific site.

8.2.3. Cost of drilling:- The unit cost of drilling holes in overburden depends on the strata met with and generalisation are difficult to make. On any major grouting job, trials with different types of equipments must be carried out at the exploration stage to choose the equipment and to estimate the probable output. Drilling in overburden is not so cheap and simple as is commonly considered. Presence of hard layers, pebbles and boulders can make the drilling operation as expensive as in soft rock. Cost of drilling often constitutes major part of the cost depending upon the type of equipment used and shall include cost of casing pipe and cost of tube-a-manchette if installed. The cost of drilling depends on:

i) Size, spacing and depth of holes,

ii) Number of grout rows, and width of grout curtains, and

iii) Equipment used.

8.2.4. Cost of grout injections: The cost of grout injections includes the cost of processing of raw materials, preparing grout slurries and injection into the holes etc. It is better to estimate and pay for grouting on the basis of the dry weight of solids injected as the consistency of the grout slurry varies from hole to hole. This method places a premium on the consumption of materials and creates a tendency to increase the tonnage of materials. The cost of injection of a grout consists of:

(i) Fixed cost, and

(ii) Variable cost.

The fixed cost includes the cost of installing the grout pipes in holes, pressure gauges, testing and removal of grout pipes and other sundry stores required for grouting. This varies from 20 to 30% of the cost of grout injection. The variable cost includes the cost of processing raw materials, cost of materials, preparing grout slurries and injection of slurry into a hole. This varies from 70 to 80% of the cost of grout injection and will depend on the quantity of solids injected and rate of injection. The cost of ~~m~~ injection also depends upon the power supply. When a bulk power supply is available at reasonable cost, the operating and capital costs of plants and equipments can be considerably reduced. The cost of injection alone can be reduced to the extent of about 20%. The cost of the dry materials may be based on the supply price at the site of work plus 15% for wastage, handling etc.

8.2.5. <u>Cost of test checking:</u> It is very important to make a provision in the estimate of test checking the efficacy of the grout, this includes:

1) Permeability holes at suitable intervals to find permeability at any elevation after grouting. These may be grouted or simply plugged as desired.
2) One or two inspection shaft of larger size may be driven to permit visual inspection of the strata grouted. These may be plugged subsequently.

8.2.6. Overhead charges: Over and above the actual cost of the grouting operation the estimate shall also include the overhead charges as below:

i) Contingencies charges ranging from 3 to 5% of the actual cost of grouting for contingent works viz; store sheds, cost of photographs, film etc. cost of transporting the staff to the site and back, cost of forms, stationery etc., and other ancillary works related to the main item.

ii) Cost of work charged supervisory staff and quality control staff. The experience at some of the projects has indicated that the cost of supervisory staff comes to about 2% of the total cost and cost of the quality control staff comes to about 1% of the total cost.

These over head charges do not include the over head charges and profits of the contractors. This may be added at about 15% in the rates for different items.

8.3. DIFFERENT METHODS OF CUTOFF

8.3.1. The alluvial deposits below the foundation of dams can be made water-tight by the means of

i) Rolled earthfill cutoff

ii) Steel sheet piling

iii) Concrete diaphragm walls

iv) Groutings

v) Slurry trench, and

vi) Combination of more than one method.

The limitations and potentialities of each method are described below.

8.3.2. Rolled earthfill cutoff:

8.3.21. This is a positive and most reliable method of providing cutoff below the dam. In this method cutoff trench of desired bed width is excavated through the alluvial deposits, down to bed rock and backfilled with rolled impervious soil with similar construction methods as used for constructing an impervious core of the dam, or with plastic concrete of clay and cement. The trench shall preferably be keyed in rock to avoid formation of leakage paths along the contact. If it is left in the weathered rock this may be grouted with suitable grout mix to make it water tight. The rock foundation below the cutoff trench may or may not be grouted depending upon the type of rock formation, content of fissures determined from core recoveries, and the degree of the seepage control desired. The excavation and backfilling of cutoff trench is to be carried out in a dry state where it can be inspected and controlled at all times. The limitations of rolled earthfill cutoff are viz;

i) Depth

ii) Dewatering, and

iii) Time.

8.3.22. Depth: The rolled earthfill cutoffs are mostly constructed to depths upto 25 meters. The cost increases rapidly below this depth, although some trenches have been excavated to more than 50 meters depth for high dams, the construction tends to be expensive and troublesome.

8.3.23. Dewatering: The principal difficulty in the construction of a rolled earthfill cutoff is frequently the dewatering of the trench and holding down the water level until the trench is excavated to the final depth and backfilled. It is difficult to make the reliable estimate of the seepage water in the trench which is required to be pumped out. There are various methods of keeping the area dry as shown in Fig.No.6.3. Well points are installed on the slopes of the trench as the excavation is being made or deep well pumps are located on Upstream and Downstream of the trench to lower the water table. In few cases sump pumps are installed at the bottom of the excavation and have proved adequate for controlling the seepage inflow. At a few dams the contractors have driven tunnels in the rock formation below the alluvium deposits and dewatered by draining the water downwards into the tunnels.

8.3.24. Time: The construction of a rolled earthfill cut off is a very slow process. This involves construction of coffer dams or ring bunds in case of flowing water, excavation in overburden and rock to the desired depth and backfilling with suitable soil and raising to the safe stage in a single working season before floods can be diverted ~~passed~~-over ~~it~~. The grouting in rock below the bed of C.O.T. if required may be done in advance from overburden though costly to gain time of completion of cutoff trench. For large depths of alluvial deposits, these operations may not be accomplished in a single working season and therefore it may not fit in the overall working schedule of the project.

8.3.25. Potentiality: The potentiality of the refilled cutoff trench is that it provides a tight and effective barrier to seepage. The reduction in permeability is to the extent of 10^{-5} cm/sec. and pressure head drop more than 90%.

8.3.3. <u>Steel sheet piling</u>:

8.3.31. The steel sheet piling was among the several important types of foundation seepage barriers used in pervious soils, 30 years ago, Because it is relatively expensive and experience has shown that the leakage through the inter locks between the individual piles is considerable, it is, therefore, chosen much less frequently today. However, in some circumstances when the alternative cutoff methods are expensive or time consuming, steel sheet piling can provide the best under seepage barrier. The limitation of steel sheet pile cutoff are viz;

i) Depth

ii) Formation, and

iii) Leakage.

8.3.32. <u>Depth</u>: The steel sheet piles can be used as a seepage barrier upto 30 to 40 ft. depth only. With a larger depth heavy section of sheet piles may be required to withstand the static pressure head of water and the dynamic pressure of driving and may also buckle while driving. Also driving of sheet piles to a greater depth would be difficult and serious drawback is the noise and vibration associated with the driving.

8.3.33. <u>Formation</u>: The effectiveness of steel sheet piling as a cutoff seems particularly dependent on such factors as:

i) Density of soil

ii) Presence of bolders in soil formation

iii) Characteristics of formation viz; homogeneous or heterogeneous, and

iv) Rock surface.

There is no reliable guide which gives the relationship between the coarseness or density of a soil and the type of the steel sheet piling which should be used in it. The presence of boulders in the formation may make the driving of sheet pile difficult and may be damaged during driving, with no indication of trouble at the surface, the interlocks tear open or the pile buckles. Therefore, driving and pulling out tests should be made at site when there is any doubt. In some cases heavy section of sheet piles have to be driven assisted by jetting through coarse gravel mixed with boulders. If the clay stratum is met with the penetration of sheet piling will be very difficult through the clay stratum as the side friction would tend to produce buckling of the piles. The sheet pile wall is more effective in homogeneous soils. In stratified soils, the permeability in the horizontal direction is many times more than that in the vertical direction, so that a large part of the total pressure head loss is due to seepage in the vertical direction as the water enters the foundation upstream from the dam and leaves it at downstream. In such cases a relatively small part of the pressure head loss occurs as the water travels horizontally under the dam, so that the influence of the sheet pile wall is relatively much lower than for a dam with homogeneous formation. For the most satisfactory use of sheet pile wall, the underlying bed rock should be sufficiently weathered, soft and with uniform surface to allow the pile to penetrate a few inches into the rock. During this distance the driving resistance increases rapidly. Very hard rock with an irregular upper surface may cause the piles to deflect and break and it may happen that only one edge of a pile may rest on rock while the other edge may be hanging in the soil formation which will increase the leakage along the contact zone at the base.

8.3.34. Leakage: The leakage and the pressure head loss of water flowing through a steel sheet pile will depend primarily on an average width of the openings in the interlocks. Both theory and laboratory tests indicate that, if no stresses exist in the interlocks the average flow channel between individual piles is approximately equal to the seepage resistance of 10-15 meters of the soil width. In actual practice it is observed from the measurements of pressure head across the sheet pile cutoff installed under dams that the resistance to seepage actually offered by the sheet pile is usually equal to that offered by the length of soil at the site varying from 100 to 500 meters. The fact that the measured resistance is higher than indicated by the theory and the laboratory tests is due to:

i) Interlocks stresses either tension or compression which closes the interlocks openings.

ii) Plugging of the interlocks openings by corrosion and/or by fine soils.

The sheet piling have been provided for earth dams, as a seepage barrier on Missouri river projects, U.S.A. and on D.V.C.dams, India. The piezometric data at Missouri river projects indicated hardly any reduction of pressure head of about 8 to 18% of the total head across the pile. With passage of time it increases significantly most probably due to corrosion and migration of adjacent soil to clog the interlocks.

8.3.35. Potentiality: The principal value of a sheet piling seems to be its ability to prevent piping progressing beyond the sheet pile wall. This was the major reasons for its installation at Missouri river projects. The driving of sheet piling is fast and when other methods of cutoff are expensive and time consuming, steel sheet piling can provide the best under seepage barrier.

A steel sheet pile wall with special design to reduce leakage in the interlocks and to prevent breaking or buckling was used in 1957 in the lower 85 ft. of the foundation material at Swift Creek dam in Washington. It consists of a double row of straight section of sheet piling welded to 18" wide flange steel beam. After the driving the space between the beams were excavated and the cells were filled with concrete.

8.3.4. Concrete diaphragm walls:

8.3.4.1. There are various methods and patents of constructing concrete diaphragm walls under earth dams as seepage barriers. Few of them are illustrated below.

8.3.4.2. Vertical walls: The excavation of a trench is made by shoring and struting and backfilling with un-reinforced concrete to form a wall usually 5 to 6' thick after necessary dewatering of foundation.

A vertical concrete walls of various types have been installed under a few earth dams as foundation seepage barriers. When founded on rock over their whole length, concrete walls are perfectly satisfactory and provide almost complete imperviousness. However, they have not been used to any large degree. This may be perhaps due to high cost of concrete which makes the rolled earthfill cutoff economical. One of the last major dams in U.S.A. at which a cutoff of this type was used is the U.S.B.R's Tieton dam completed in 1925. There, a 5 ft. wide concrete wall was carried down more than 130 ft. deep. Difficulties with dewateringand shoring during construction discouraged further use of similar design. These are still common in England and have been put down from 150 to 200 ft.deep.

8.3.4.3. Compressed air Cassion type: Other type of very deep concrete cutoff wall was installed as a compressed air cassion constructed under a few major dam in eastern U.S. in 1930. It is doubtful whether such a design would be economically feasible under any circumstances in the present day.

8.3.4.4. Intrusion Prepakt method: A new form of concrete cutoff walls which show promise in several cases (when its brittleness would not result in cracking and where a deep barrier is not needed) is a continuous row of large diameter bored piles drilled with special drilling rigs and backfilling with cement mixed with natural foundation soil or by pumping a sand-cement grout into the hole from the bottom up as the drill bit is removed. This method has been used at Shek Pik dam Hongkong where a 22" dia. holes are drilled to form a continuous cutoff wall 30 ft. deep to act as a barrier to grout, injected between the diaphragm walls. The Prepakt process has been used to build continuous underground impervious walls for many types of civil engineering structures, however upto the present it has been employed for permanent seepage barrier under only 2 small diversion dams of the U.S.B.R.

8.3.4.5. Veder Process: The Veder Process is the patent of ICOS firm of Milan, Italy. In this process holes are drilled continuously and are held open with a bentonite mud slurry and clay-cement concrete is placed with a tremie from the bottom of the hole. In India this process is used by Rodio group for many civil engineering jobs viz; To act as a seepage barrier for dry excavation of C.O.T. at Ukai project, for open excavation of foundation of Air India Building, Bombay, for open excavation of machinery foundation at Ranchi and to act as a grout barrier at Obra Dam, U.P. the width of wall is kept 2 ft. cutoffs have been formed under a considerable number of low earth dams in Europe and south America. At Manicouagan dam Quebec this process has been used (completed and tested satisfactorily in 1962) in the upstream coffer dam where a 2 ft. thick wall is put down to a maximum depth of about 250 ft. through coarse alluvium containing boulders upto 3 ft. diameter.

8.3.4.6. The following are the limitations of the concrete diaphragm walls:

1) depth
2) Stability
3) Seepage
4) Permanence, and
5) Cost.

8.3.4.61. Depth: The records available indicate that concrete diaphragm walls are taken down to a maximum depth of 250 ft. only. Beyond this depth drilling becomes difficult and costly due to weight of driving bit on the rig.

8.3.4.62. Stability: It is doubtful whether a thin concrete diaphragm walls can withstand the high hydraulic gradient. It acts as a cantilever slab fixed at base and may deflect due to high hydraulic pressure. It may even crack due to its brittleness. At the surface it may look alright but the in-situ conditions in the soil may be different.

8.3.4.63. Seepage: If the keying into rock is not perfect, it may leak at the base. In Rodio's process alternate blocks usually 2 to 4 meter long are cost and then gaps are filled in. If any opening is left in the interlocking of the blocks, this may cause considerable leakage.

8.3.4.64. Permanence:- The permanence of concrete diaphragm walls is doubtful and are not used as a permanent cutoff below the dam. Mostly these are used as a supplement to grouting or t rolled earthfill cutoff, and as a temporary seepage barriers, for open excavation of foundations.

8.3.4.65. Cost: The cost of concrete diaphragm wall is prohibitive and costs about 8 to 10 times to that of rolled earthfill cutoff. The construction also takes much time as the excavation in rock for keying is most difficult and special chisels are used to excavate. The concrete also remains plastic for longer period and on setting it becomes brittle.

8.3.4.66. Potentialities: The bore holes can be drilled in any type of soil formation containing boulders where driving of sheet piles is difficult or in any permeable zone where excavation of cutoff trench is very difficult due to high seepage. Therefore when other methods of cutoff are not feasible concrete diaphragm walls may be employed.

Best cutoff can be obtained when these walls are supplemented with grouting or rolled earthfill cutoff to form a thick mass perfectly water-tight.

8.3.5. <u>Grouting:</u>

8.3.5.1. It was in 1955 when grouting was first adopted at Serre-Poncon to form a permanent cutoff below the earth dam resting on alluvial deposits. This practice has since been used to several dam sites and the piezometric data available indicates the pressure drop across the curtain more than 90% and reduction in permeability more than 10,000 times of its original value. With modern practice of grouting and advance in soil mechanics and grout technique, it is now possible to provide a cutoff in any soil formation ranging from permeability of 10 cm/sec to 10^{-4} cm/sec. by the coarser grout of cement-clay-bentonite and fine grout of silicates and organic resins respectively. The limitations of grout process are:

1) Cost.

8.3.5.2. <u>Cost:</u> The only limitation is the cost. It is costlier than rolled earthfill cutoff for low depth of alluvial deposits. For large depths the cost of rolled earth fill cutoff increases many times and grouting would only be solution.

8.3.5.3. <u>Potentiality:</u> The grouting can be done to the depth to which holes can be drilled. Therefore, depth is not the limitation. Grouting in various forms has a great potential for effective cutoff in alluvial beds and it holds promise of replacing sheet piling and rolled earthfill cutoffs. It has also great possibility for solving dewaterring and coffer dam problems many of which defy solution to-day.

8.3.6. <u>Slurry trench method:</u>

8.3.6.1. It is the latest development in providing permanent cutoff below the dam to check seepage through the alluvial deposits. In a slurry trench method, a trench is excavated by the conventional method of excavation by dragline or back hoes and the trench is kept continuously full of bentonite mud slurry to prevent the walls from caving in so that no sheathing or shoring had to be used. After the trench is excavated to bed rock with the drag line, the bottom may be cleaned with clamshell buckets and air jets. The excavated materials (a mixture of sand and gravel with bentonite slurry) may be stock-piled adjacent to the trench to drain out water. The mixture may be blended later with 15 to 20% silt. This mixture may then be dozed in the trench from one end displacing the bentonite slurry until the backfilling is complete. The material is expected to have a permeability coefficient of less than 10^{-5} cm/sec.

This procedure had been employedto construct ground water seepage barriers for various purposes but except for some dykes below the MCNary dam, had never been used under a permanent dam structure until the Wanapum dam built in 1958. Here the foundation material at site consisted of an erratic deposits of sand, gravel and boulders extending to a depth of 80 ft. The coefficient of permeability varied from 3 to 10^{-1} cm/sec. Studies indicated the driving of sheet pile difficult and excavation of trench for rolled earthfill troublesome and would have been costly due to heavy dewatering. After successful field tests, a 10 ft.

wide foundation seepage barrier 80 ft. deep was formed of slurry trench. The limitations of this process are:

1) Depth

2) Permanence

8.3.6.2. Depth: The trenches can be excavated to the depths to which buckets of draglines and back hoes can be lowered safely. Therefore for deeper curtain this method is not suitable.

8.3.6.3. Permanence: The core walls constructed are very thin say from 3 to 10 ft. wide, and the slurry in trench remains in a very soft conditions for very long times after construction. The material has a consistency somewhat like that of stiff butter. Therefore where early loads are to be imposed upon the soft mixture may shrink. Moreover, till the mixture is hardened embankments cannot be raised over them. This may effect the overall programme of construction.

8.3.6.4. Potentialities: The compaction of slurry in the trench is by its own weight and requires no rolling. The process has many advantages, and now it has a precedent on a major project, it will undoubtedly be used more frequently in the future at sites where depth of cutoff is not great and trench can be excavated by trenching machines or by a back hoes, the trench width can be reduced 3 to 4 ft. which will reduce the cost.

8.3.7. Combination of more than one process: The combination of more than one processes are now-a-days preferred for perfect cutoff walls in alluvial deposits.

8.4. COST COMPARISON

8.4.1. The cost comparison of different cutoff processes are illustrated below.

8.4.2. Rolled earthfill trench: The cost is worked out on the basis of volume of earthfilled in the trench and includes:

The rate of excavation in O.B depending upon lift and for normal lead of 50 m.	Rs. 3 to 5 per m^3
The rate of excavation in rock depending upon lift and for normal lead of 50 m.	Rs. 14 to 20 per m^3
The rate of dewatering while excavating.	Rs. 0.5 m^3
The rate of dewatering while refilling.	Rs. 0.5 m^3
Refilling with imp. soil.	Rs. 7.0 m^3

8.4.3. Steel sheet pile: The rate is worked out on the weight basis. This can be converted to surface area basis inclusive of cost of driving.

The rate of sheet pile.	Rs. 2000 MT
Weight of sheet pile/sq.m. 120 to 180 kg. Rate per/sq.m. depending upon the section and manufactures.	Rs. 240 to 360 per sq.m

8.4.4. Concrete Diaphragm wall: The rate is worked out on the basis of surface area inclusive of cost of drilling, bentonite slurry and placing concrete.

The rate for a 0.6 meter thick un-reinforced diaphragm wall.	Rs. 380/sq.m.

8.4.5. Grouting: A detailed estimate is to be prepared from which rate per m^3 of mass treated can be worked out.

8.4.6. Slurry trench: The rate is worked out on the basis of surface area and includes most of excavation, cost of silt, blending and dozing.

8.4.7. Problem: Economics of different processes of providing cutoff at a given site are worked out as below for the problem:

Depth of alluvial deposits consisting of sand & gravel.	15 m.
Depth of water retained.	60 "
For comparison assume a unit length of	100 "

8.4.71. Rolled earthfill cutoff:

(i) Quantities:

Assume trench bed width.	15 m
Side slopes	1:1
Excavation $\frac{45+15}{2}$x15 x 100=45,000m^3	45,000 m^3

(ii) Abstract of cost:

Quantity.	Items.	Rate per.	Amount.
45,000 m^3	Excavation.	5.0 m^3	Rs. 225,000
45,000 m^3	Dewatering	1.0 "	Rs. 45,000
45,000 m^3	Refilling.	7.0 "	Rs. 315,000
			Rs. 585,000
	Contingency	3%	Rs. 17,550
			Rs. 602,550
	W/C.	2%	Rs. 12,050
		Total:	Rs. 614,600

8.4.72. Steel sheet piling:

(i) Quantities:

Surface area one row only. 15x100=1500 m^2

(ii) Abstract of cost:

Quantity.	Items.	Rate.	Amount.
1500 m^2	Sheet piles.	350 m^2	Rs. 5,25,000
	Contingency	3%	Rs. 15,750
			Rs. 5,40,750
	W/C.	2%	Rs. 10,815
		Total:	Rs. 551,565

8.4.73. Concrete diaphragm wall:

(i) Quantities:

Surface area 15 x 100 = 1500 m^2

(ii) Abstract of cost

Quantity.	Items.	Rate per	Amount.
1500 m^2	Concrete diaphragm wall.	380 m^2	Rs. 570,000
	Contingency.	3%	Rs. 17,100
			Rs. 587,100
	W/C.	2%	Rs. 11,742
		Total:	Rs. 598,842

8.4.74. Grouting:

(i) Quantities:

Width of curtain	15 m
Rows of grout holes.	5 Nos.
Spacing between rows.	3 meters.
Spacing of holes.	3 meters.
∴ No.of holes(100/3+1)x5	170 Nos.
Depth of holes 170x15	2550 M
Quantity of soil to be treated = 15x15x100	22500 m
Assume 45% filling.	x 45
	10025 m^3
Assume 80% with clay based grout:	8020 m^3
20% with clay-silicate grout	2005 m^3

Proportion of clay-cement-bentonite grout.

Clay + Bentonite + cement + Water + D.A.

0.4 + 0.2 + 0.6 + 6 + 2%

In m^3 of slurry solids will form 186 kg. of which

Cement 93 Kg.

Clay 62 Kg.

Bentonite 31 Kg.

D.A. 1.86 Kg.

Proportion of clay-chemical mix: 2+1

This will contain 114 Kg of solids in m^3 of slurry out of which

Clay 76 kg.

chemicals 38 kg.

Rates are assumed as below:

Clay (processed)	Rs. 100 MT
Bentonite	Rs. 300 "
Cement.	Rs. 200 "
D.A.	Rs. 5 Kg.
Silicate	Rs.1000 MT
Injection charges.	Rs. 100 M.T.

Requirement of dry materials, clay-based grout:

Cement $\frac{93}{1000} \times 8020 = 750$ M.T.

Clay $\frac{62}{1000} \times 8020 = 500$ "

Bentonite $\frac{31}{1000} \times 8020 = 250$ "

D.A. $\frac{1.86}{1000} \times 8020 = 15$ "

Chemical grout:

Clay $\frac{76}{1000} \times 2005 = 150$ M.T.

Chemicals $\frac{38}{1000} \times 2005 = 75$ "

(ii) Abstract of estimate:

Quantity.	Items.	Rate	Per	Amount
2550 Rm	Drilling including placing manchettes.	75	Rm	Rs. 191,250
2550 Rm	Sheath grout.	5	"	Rs. 12,750
650 M.T.	Clay	200	MT	Rs. 130,000
750 M.T.	Cement	300	"	Rs. 225,000
250 M.T.	Bentonite	400	"	Rs. 100,000
15 M.T.	D.A.	5100	"	Rs. 76,500
75 M.T.	Silicate	1100	"	Rs. 82,500
2 No.	Permeability tests.	1000	"	Rs. 2,000
				Rs. 820,000
	Contingency	3%		Rs. 24,600
				Rs. 844,600
	W/C	3%		Rs. 25,338
		Total:		Rs. 869,938

8.4.75. <u>Slurry trench</u>:

(i) Quantities:

Width of trench	3 meters
Excavation 100x3x15	4500 m^3
Assume 50% overcut.	2250 "
	6750 m^3
Refilling	6750 "
Silt for blending 20%	1350 "
Bentonite	100 kg/m^3

(ii) Abstract of cost:

Quantity.	Items.	Rate	Per	Amount.
6750 m^3	Excavation.	5	M^3	Rs. 33,750
1350 m^3	Silt.	5	"	Rs. 6,750
1350 m^3	Blending of silt.	2	"	Rs. 2,700
675 M.T.	Bentonite (LG)	200	"	Rs. 135,000
6750 m^3	Refilling	4	"	Rs. 27,000
				Rs. 205,200
	Contingency.	3%		Rs. 6,156
				Rs. 211,356
	W/C	2%		Rs. 4,227
		Total		Rs. 215,583

8.4.76. <u>Abstract:</u> The cost worked out above for different types of cutoff for a unit length of 100 m is given below in descending order:

1.	Grouting	Rs. 869,938
2.	Rolled earthfill.	Rs. 614,600
3.	Concrete diaphragm wall.	Rs. 598,842
4.	Sheet piling.	Rs. 551,565
5.	Slurry trench.	Rs. 215,583

The cost comparison shows that slurry trench is the most economical but other limitations viz; if there is a standing pool of water or flow of water in the river will require coffer dams which will add to the cost. This limitation will also apply to the rolled earthfill cutoff. The cost of coffer dams may make these processes very costly and time consuming. The cutoff provided by sheet piling is not perfect in early stages of the reservoir operation and driving may also be difficult in gravel and boulder formations which may make the cutoff ineffective. Therefore only method which can compete the grouting process for given job conditions would be concrete diaphragm. The stability and reliability of concrete diaphragm are not yet developed in this country, therefore grouting would only be feasible. When there is a flow of water in the river the grouting operation can be carried out from floating pontoon.

CHAPTER - 9

EXAMPLES OF APPLICATION OF GROUTING

9.1. Few examples of application of grouting of dam foundations resting on alluvial deposits are illustrated below:

9.2. SERRE-PONCON (1956)

Location: The first use of alluvium grouting on large scale was at the dam at Serre-Poncon on the river Durane in the French Alps. The dam is constructed of rolled earth fill 120 meters high.

Problem: The dam is founded on water bearing alluvium 100 meters deep highly permeable with permeability of 10^{-1} cm/sec. The foundation consists of 3 types of alluvial materials to be grouted viz;

(1) Coarse material with minimum grain size of 1.00 mm,

(2) Fine material with grain size of 0.1 mm, and

(3) Intermediate materials.

Treatment: The impermeable cutoff below foundation has been formed of 12 rows of grout holes of which 4 extend down to the bed rock. The thickness of the cutoff is about 35 meters at its contact with the core and progressively decreases to about 15 meters thick from adepth of 30 meters down to bed rock in which it penetrates by several meters. The upper part of the cutoff is reinforced with 7 rows of shallow holes so as to cause a satisfactory contact between the cutoff and the core of the dam, and to reduce as far as possible hydraulic gradient in the contact zone.

Coarse alluvial materials of size 1.00 mm were grouted with clay-cement grouts. The intermediate materials were grouted with colloidal clay of L.L.105% and finely ground slag a sulphate resistant material with caustic soda as a setting agent. The layers of fine materials were grouted by silicate grouts.

Results: The permeability of 10^{-1} cm/sec. measured before grouting was reduced to 10^{-5} cm/sec.

From the filling of reservoir in 1960 regular observations made in many piezometers installed both upstream and downstream of the cutoff show that the flow of water through it is exceedingly small. In fact the gradient of the ground water immediately downstream of the cutoff is lower than that which existed there before the construction of the dam.

9.3. SHEK PIK DAM

Location: The Shek Pik dam is located on the island of Lan Tao in Hongkong just 500 yards from the mouth of the river. Dam is constructed of rolled earthfill 187 ft. high and 2500 ft. long intended for the water supply of Hongkong.

Problem: The investigation of dam site foundation indicated alluvium deposits of about 30 ft. deep containing many boulders, overlying about 20 ft. of much decomposed granite with many of the physical properties of an alluvium. Beneath this, there is a zone of 20 to 30 ft. of less decomposed rock. Sound rock is available at about 100 ft.depth below the surface. The ground water was almost at surface level. Alluvium deposit was found to be very permeable and containing large boulders. Conventional methods of constructing a cutoff of sheet piles and a concrete filled trench were out of the question. Even backfilled cutoff was also found to be very difficult.

Treatment: The cutoff consists of two rows placed at 20 ft. apart of bored pile diaphragm 22" dia. extending through the alluvium into weathered rock. The grout was injected subsequently into the alluvium in between the bored pile diaphragms. The grout curtain is formed of 4 rows of holes two between the diaphragms and one on either side and out side, extending to alluvium. Upstream row within the diaphragm was extended down to weathered rock. The grouting was carried out by tube-a-manchette method. A wide range of grout mixes were used including grouts made from locally occuring clays after processing and with addition of some bentonite and cement. Four successive dozes were given in stages, idea behind the stage grouting was to allow time for water displaced by the grouting to dissipate and so reduce the heave. The choice of the grout for the various zones was determined as the work proceeded and was based on accurate records of grouting pressures, rates of acceptance etc. 15 ft. thick embankment was placed over the cutoff and grouting was carried out from the embankment to increase the grouting pressures.

Results: After grouting the permeability was reduced to about 10^{-5} cm/sec. Some heave was also noticed in the embankment which had to be removed before further raising of earth fill.

The section of earth dam showing cutoff is shown in Fig.No.9.1.

9.4. KOTAH BARRAGE (1958)

Location: Kotah Barrage is located across the river Chambal near Kotah city in Rajasthan. The barrage has a total length of 1810 ft., 1000 ft. long spillway on its left resting on rock and remaining earth dam portion on its right abutting old palace wall. The earth dam is about 120 ft. high above the river bed.

Problem: In earth dam portion in a length of about 210 ft. rock was found to be about 60 ft. below the river bed. In remaining length rock was met at higher level where masonry cutoff was provided. Sheet piles could not be driven due to presence of big size boulders in the formation. Open excavation was also not found suitable as it would have delayed one year and would have required more power for dewatering which was not available. then. Therefore it was decided to grout the alluvium.

Treatment: It was the first time in India when an alluvium grouting was to be attempted. Methods and processes were not developed then and hence reputed firms viz; Solletance of France and Cementation Co., Ltd. were consulted. The grout mix used was

Clay	Cement.	Sodium silicate
90 lbs.	4½ lbs.	160 c.c.

The consumption of grout was found to be about 46% of the total volume treated. The thickness of cutoff is about 30 ft. made up of 5 rows of grout holes spaced at 6 ft. and holes at 6 ft. c/c.

Results: The field permeability tests carried out after the grouting operations gave the average value of 8×10^{-3} cm/sec. against the original value of 1.8×10^{-1} cm/sec. before grouting.

9.5. ROCKY REACH DAM (1959)

Location: The Rocky Reach Hydro electric project is located on the Columbia river in North Central Washington.

Problem: The Columbia river at dam site is a relatively shallow fast flowing river about 1300 ft. wide. It lies on a high bed rock against the western wall of the canyon, and is bordered on the east by a high broad terrace which extends more than 3500 ft. from river edge to the canyon wall on the east. Boring and seismic exploration made by the U.S. Army Corps of Engineers indicated that extensive granular deposits existed in the terrace areas. These erratic deposits, formed by the river in its previous course were highly permeable with permeability in the order of about 5 cm/sec.

Treatment: After a careful review of the seepage problem in view of this large coefficient of permeability and from the field grouting test results, a grouted cutoff was decided. The cutoff consists of three rows of hole spaced at 10 ft. and holes along the row spaced at 12 ft. c/c. A construction trench was excavated to facilitate the drilling and grouting operations. The grout used was cement-clay followed by cement-bentonite. The field testing and the review of data indicated that sand formation of significant extent still remained ungrouted and that residual permeability was still higher than the desired value. The fine sand formation could only be grouted with chemical grouts. Sodium silicate-aluminate grout was not considered as high PH value in excess of 11.0 was noticed in ground water and it was doubtful whether gelling of grout could be assured. Chromelignin grout was rejected as long term stability was questionable. AM-9 grout was selected for further grouting because of its ability to penetrate the sand and silt zones which remained open after the cement grouting. Gel time for AM-9 grout used was usually in the range of 15-45 minutes and grouting was done in stages of 5 ft. by descending stage method.

Results: Field permeability tests and continuous record of piezometric levels before and after grouting indicate that initial permeability of 5 cm/sec. was reduced to $5x10^{-4}$ cm/sec. after the completion of cement and chemical grouting. The efficiency of thecutoff has remained at the same high level following several years of operation at full reservoir level.

Cross section of cutoff wall is shown in Fig.No.9.2 and piezometric levels after grouting are shown in Fig.No.6.11.

9.6. MISSION DAM (1959)

Location: The Mission Dam is located in the valley of the Bridge river in British Columbia. It is located at the down-stream toe of the existing diversion dam 60 ft. high cons-tructed in 1948 to divert the water to the power station. In 1955 it was decided to raise the reservoir level by 140 ft. by constructing a new dam. The dam is built of earth and rockfill. Professor Terzaghi was incharge of design and construction of Dam.

Problem: The site is underlain by two pervious acquifers, separated by a thick stratum of highly compressible clay. The permeability of the sediments beneath the clay stratum was of the order of 10^{-1} cm/sec. and the pattern of the stratification was that of a sediment laid down by a swiftly flowing river.

Treatment: The presence of two acquifers separated by the clay stratum required two separate cutoffs for the new dam. The lower cutoff consists of a grout curtain extending from the bottom of the clay stratum to bed rock and the upper

one consists of a sheet pile wall which reinforces the original clay cutoff of the diversion dam. The grout was designed to achieve the permeability of 2×10^{-4} cm/sec. after grouting. The curtain was designed with hydraulic gradient of 3 to 4 times. Thus for 180 ft. depth of water, curtain width was kept 50 ft. made up of 5 rows of grout holes spaced at 10 ft. apart and holes along the rows in row 1 and 5 were spaced at 10 ft. apart whereas for inner rows holes were spaced 15 ft. apart, staggered. The contract for grouting was awarded to Solctanche, France. The grouting was carried out from an embankment of 25 ft. height upto El.2010 down to bed rock El.1555. The principle constituent of the grout were portland cement type I and glacial lake clay with clay content of 40 to 60% with clay fraction less than 2 u, LL 45 to 55% and P.I. of 19%. Grout rows 1 & 5 were injected with relatively thick and strong grout to prevent the escape of large quantities of less viscous grout from the central portion of the curtain across rows 1 & 5. Grout was made up of cement clay ratio of 0.63 and cement/water ratio of 0.02 by weight.
The grout was injected under pressures of 200 to 500 p.s.i. Intermediate rows No.2&4 were injected with grout of cement/clay ratio of 0.22, cement/water ratio of 0.07 and injected under pressure of 200 to 400 p.s.i. sometimes reaching pressures of 800 p.s.i. Central row No.3 was injected with clay chemicals made of clay-sodium silicate and mono sodic phosphate which acted as a gelling agent for the silicate and deflocculating agent for the clay. The properties of clay chemical grout used were:

Initial viscosity	36 sec.
Marsh cone.	45 Min.
Shearing strength after 8 hours.	10 g/cm^2

The grout was injected at pressure varying from 600 to 1300 p.s.i. occasionally reaching to 1500 p.s.i. The grouting was carried out by a tube-a-manchette process. The grout pressures were used 3 to 6 times overburden pressures and exceptionally as much as 10 times. Observations of heave indicated no where more than 0.2.ft.

Results: After grouting the permeability to be achieved was 2×10^{-4} cm/sec.

The section of the cutoff wall is shown in Fig.No.9.3.

9.7. GIRNA DAM (1964)

Location: Girna dam is located on the river Girna a tributary of the river Tapi. The project was undertaken to supplement the irrigation demands of the existing canal system offtaking at 25 and 60 miles downstream and generation of power. The dam is 3160 ft.long and 183 ft.high above river bed. Masonry spillway is located from Ch. 0 to 1400 and rest is of rolled earth fill.

Problem: For masonry dam portion rock foundation was available at shallow depth. For earth dam portion rock was about 80 ft. below the surface. The overburden deposits in the river bed consist of two types viz;

(1) From Ch.1400 to 2200 the deposit is young and consists of homogeneous clean sand with grain

(2

During 1959-60 from Ch. 1400 to 1920 and from Ch. 2750 onwards the cutoff trench was excavated down to rock and backfilled from Ch. 1920 to Ch. 2750 cutoff trench could not be excavated fully down to rock because of high seepage and was stopped at 20 ft. above rock. It was, therefore, decided after consultation with A. Mayor, French Engineer, to provide a grout curtain.

Treatment: Partial cutoff excavated was backfilled and 15 ft. high embankment was raised over it. The grouting with cement-bentonite grout was under taken departmentally in May 1960 between Ch. 2450 to 2600 with three rows of grout holes spaced at 7 ft. apart and holes along the row spaced at 10 ft c/c. The grouting was carried out by rising tube method. The work was slow and hence further work was given on contract. The remaining grouting was carried out by tube-a-manchette method, four times overburden pressures were applied and grout mix used was clay-cement-bentonite. The consumption was found to be 70% off the total volume treated. The width of the grout curtain was kept 24 ft. The piezometers installed showed poor drop of head across the curtain after treatment with clay-cement-bentonite mix. After trials, silicate grout was suggested with 2 rows spaced at 6 ft. apart and holes along the row at 6 ft. c/c. The grout mix used was:

Silicate	300 c.c.
Aluminate	11 to 16 gram
Water	500 c.c.

The consumption of grout was found to be 300 lit/ sleeve at pressures of about 300 p.s.i. During grouting it was noticed that some high permeability pockets did not accept grout at 300 p.s.i. and hence higher pressures even upto 1000 p.s.i. were applied. There was increase in intake, the high pressure used may be due to non-opening of sleeves.

Results: After grouting permeability results show reduction in permeability to the desired degree of 10^{-4} cm/sec. from original permeability of 10^{-1} to 10^{-2} cm/sec. Cross section of earth dam showing the cutoff is shown in Fig. No. 9.4.

9.8. ASWAN HIGH DAM

Location: The Aswan High Dam is located on the river Nile, the life line of Egypt and Sudan. It is located 8 Km. upstream of the existing Aswan Dam built in 1902 and raised twice. The dam will be 111 meter high above the river bed and is being constructed in sand and rockfill. Sand available is desert sand uniform in particle size. At the dam site there is a back water depth of about 35 meter. The novel features of the project are that when completed it will have storage of about 55 maft. which is more than the combined storage of all major dams in India. Once filled, rain or no rain it can feed irrigation and power generation for consecutive 3 years. It will take some 10 years to fill the reservoir initially. The project is under construction and is likely to be completed very soon.

Problem: There is no hearting zone material available, hence desert sand is placed in the hearting zone and grouted with cement grout to make it impervious. At the site the rocky gorge has been filled by alluvial deposits of sand and gravel with lenses of silt from an elevation of $+85^{m}$ to -40^{m}. This is underlain by a stratum of indurated sand and silt. The deepest rock is at elevation of -135^{m}.

Thus the maximum depth of overburden is 220 meters. Sand fill of 30 meters depth is placed upto El. +115^{m}. Thus the total depth of overburden works out to 250 meters. Depth of weathered rock is about 10 meters. The overburden is lenticular, the fine sand zones are probably formed from wind blown desert sand deposited in the back water of old Aswan Dam. There are some zones of rock debris or talus mixed with gravel. The permeability characteristics of various stratas are as below:

Permeability for fine sand and silt 5 x 10^{-3} cm/sec.

Permeability for coarse sand & gravel: 10^{-1} cm/sec.

Treatment: To avoid the risk of foundation failure of the dam, the design of the seepage control is based on a combination of impervious blanket, grout curtain and relief wells. The grout curtain is the largest in the world with regards to the depth of the grouted zone 250 meters deep from El.+115^{m} to -135^{m}. The width of the curtain provided is 40 meter whereas model studies had indicated 25 meters thick grout curtain. Grout curtain is tapered down with increasing depth. The curtain consists of 8 rows of grout holes spaced at 5 meter apart and holes along the row spaced at 2.5 meter c/c. In case permeability after grouting is not reduced, additional 6 rows were also contemplated, spaced at 2.5 meters apart with holes at 2.5 meter c/c. The grout used for medium and coarse sand was cement-and clay grout in outer and barrier rows of the grout holes and silicate clay grout for the inner rows. For fine sand layers bentonite grout was used in outer and barrier rows of thegrout holes and aluminate-silicate grouts for inner rows. Clay-cement grout was also used for intermediate rows to prevent formation of large horizontal lenses of clay-silicate grout in the highly pervious pockets. Over travel of clay-silicate grout is thereby prevented and more effective control is exercised over the consumption. Clay and bentonite were used after processing in a hydro cyclone to eliminate coarser particles. The grouting was done through the tube-a-manchette and pressures were applied just sufficient to open the sleeves and thereafter the pressures were brought down to maintain the flow. Maximum pressure used was about 60 kg/cm^2 at an injection depth of about 120 meters. Normally the pressures applied were 20 kg/cm^2.

Results: A system of piezometers is installed on the upstream and downstream of the grout curtain. Results of pressure drop are still awaited. From the grouting carried out so far it has been found that permeability has been reduced to 5×10^{-4} cm/sec. from the initial value of 10^{-1} cm/sec.

9.9. OBRA DAM:

Location: Obra Dam is located on the river Rihand 30 miles downstream of Rihand Dam in U.P. to supply water to the thermal power station. It is 91 ft.high dam built in rolled earth fill with spillway in masonry.

Problem: The geological data of the dam site shows the pervious overburden extending to a depth of 80 to 100 ft. overlying lime stone rock in the deepest portion. The overburden comprises of medium sand with fine sand in top 20 ft. depth. Permeability of the foundation material is about $3x10^{-2}$ cm/sec.

Treatment: It is the recently built dam and hence the latest practice in the foundation treatment by grouting was employed for a complete cutoff in the form of a grout curtain to control the adverse effects of under seepage through the foundation. The conventional method of backfilled cutoff trench upto the rock level would not have been economical as such an alternative method of deep grout curtain was proposed. The treatment provide was similar to that provided at Shek Pik dam Hongkong. Two rows of reinforced concrete diaphragm walls, 0.6 meter thick were provided 3 meters apart and extending one meter into rock. The in-between space was groutedwith silicate grout as the grain size of the foundation material would not have accepted cement or clay based grout.

Results: The permeability after grouting is found within desirable values.

The cross section of earth dam showing the cutoff is shown in Fig.No.9.5.

9.10. OTHER EXAMPLES

In addition to the examples illustrated above the following are further examples, where attempts are made to provide grout cutoff walls.

1) Sylenstein (Germany)
2) Stramentizzo (Italy)
3) Notre-dame de Commeirs (France)
4) Mattmark (Switzerland)
5) Kvistrorsen (Sweden)
6) Asen (Sweden)

The depth of cutoffs provided are shown in Fig.9.6 and the comparison is given in Table No.9.1.

Table No.9.1

The comparison of depth of the grout curtain

Name of Dam.	Country	Depth of grout curtain in meters
1. Aswan High Dam.	Egypt	250
2. Mission Dam.	British Columbia.	150
3. Serre-Poncon	France.	115
4. Sylenstein	Germany	100
5. Mattmark.	Switzerland	100
6. Stramentizzo	Italy	85
7. Asen	Sweden	45
8. Notre-dame de Commier	France	44
9. Kvistrorsen	Sweden	40
10. Rocky Rea-ch.	British Columbia	40
11. Obra	India (U.P)	30
12. Girna	India (Maharashtra)	30
13. Shek Pik	Hongkong.	25
14. Kotah	India(Rajasthan)	20

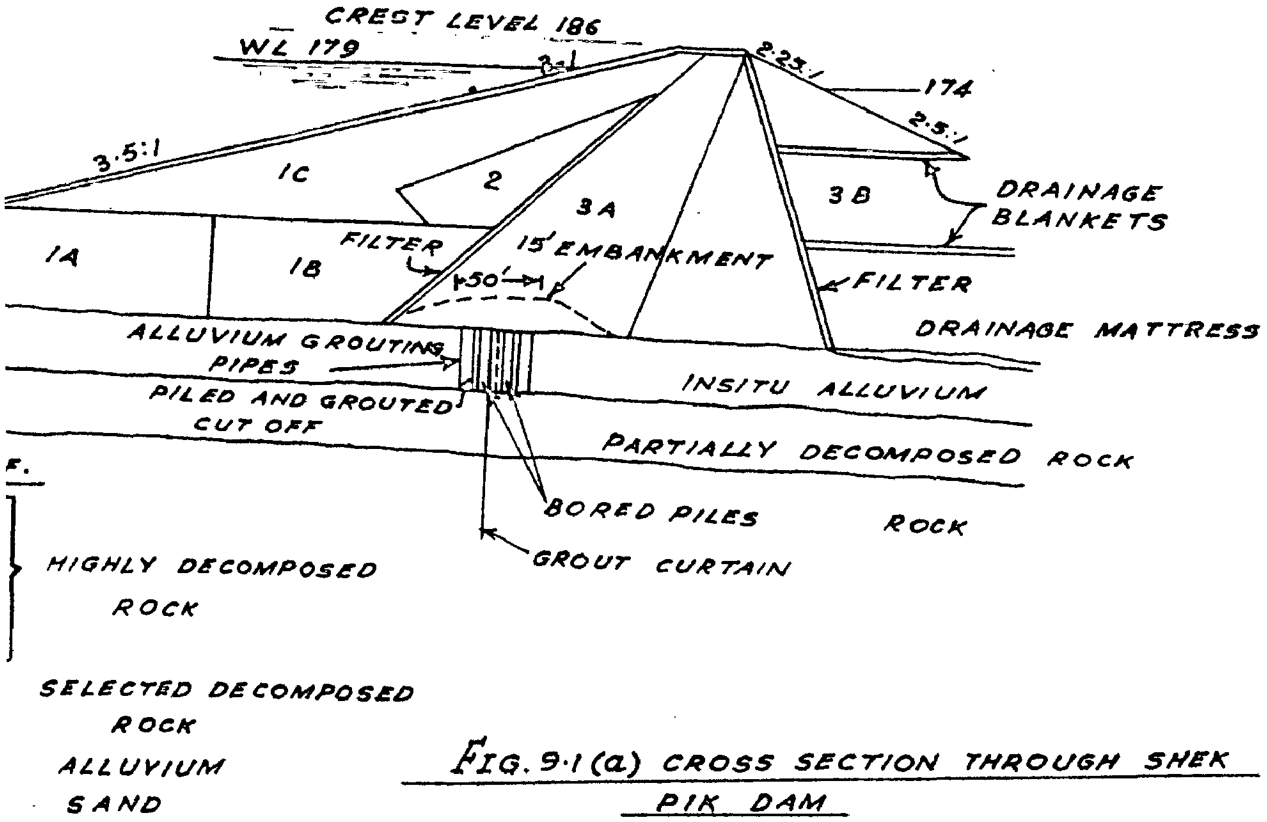


FIG. 9·1(a) CROSS SECTION THROUGH SHEK PIK DAM

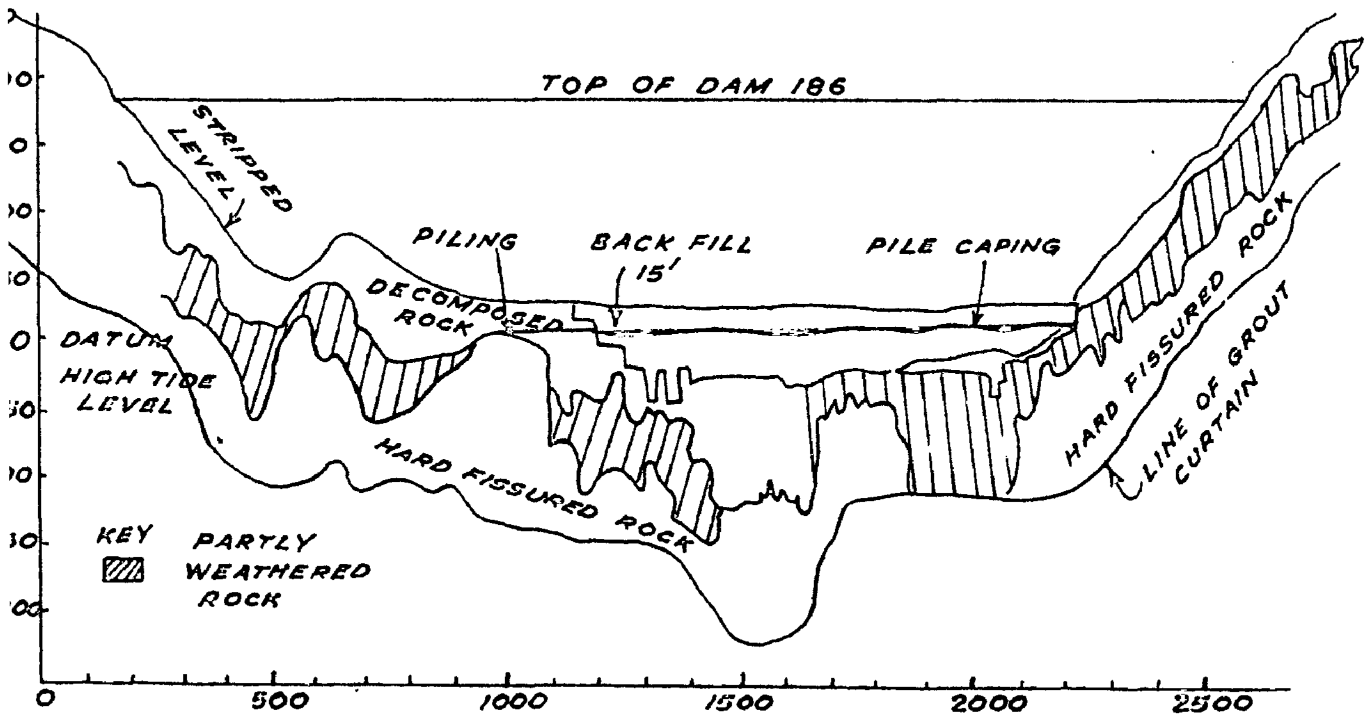


FIG. 9·1. (b) SHEK PIK DAM LONGITUDINAL SECTION ALONG THE CUT OFF

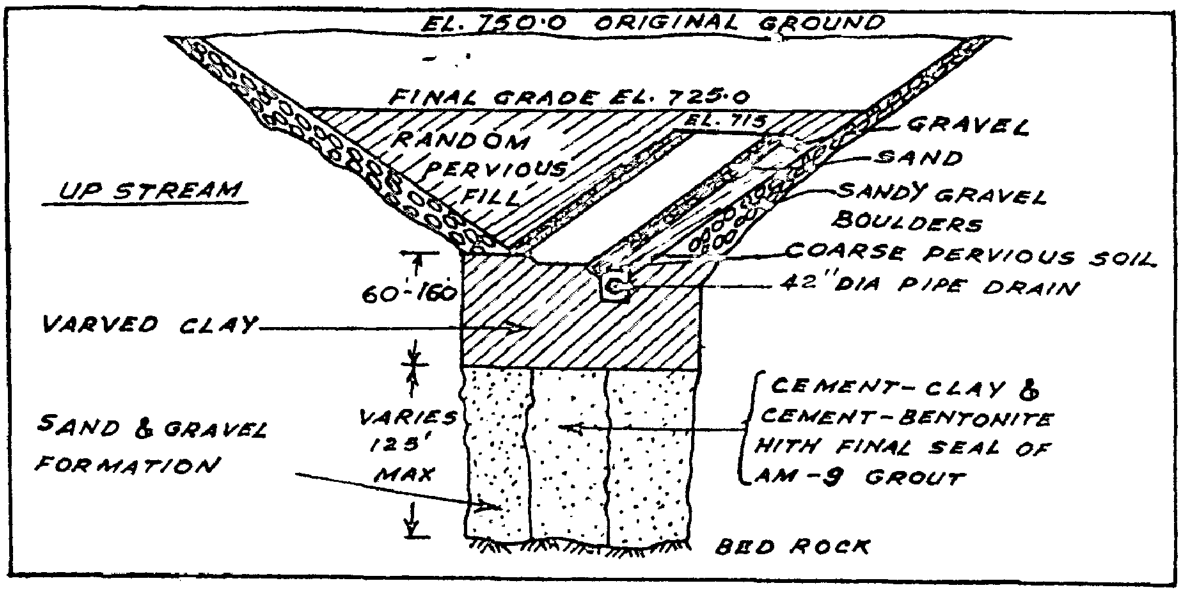


FIG. 9.2. SHOWING SECTION OF CUT OFF WALL AT ROCKY REACH DAM.

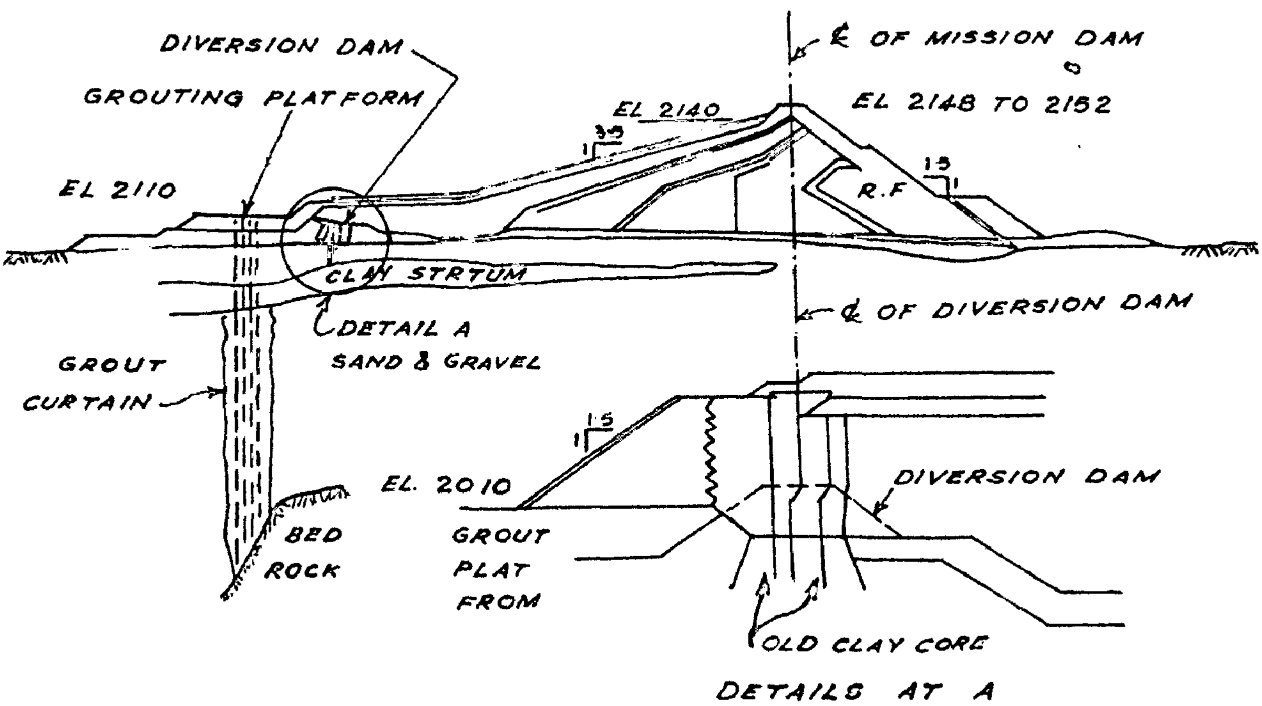
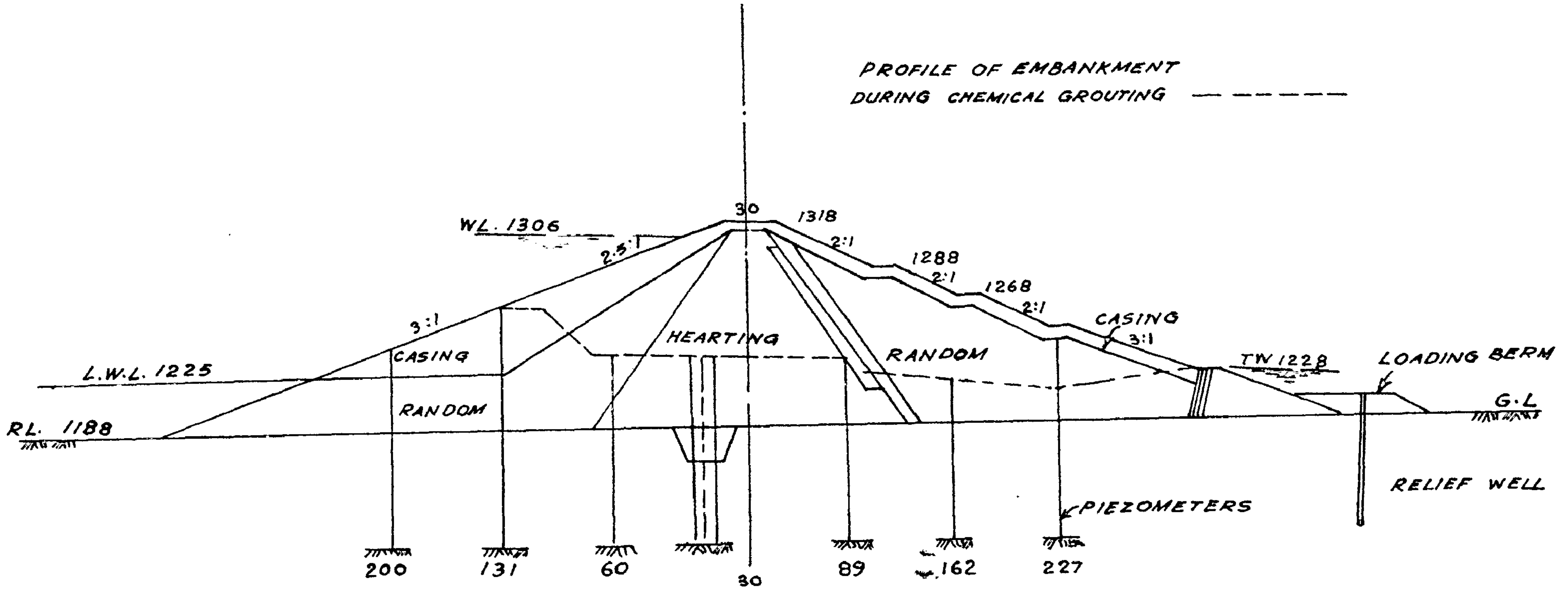
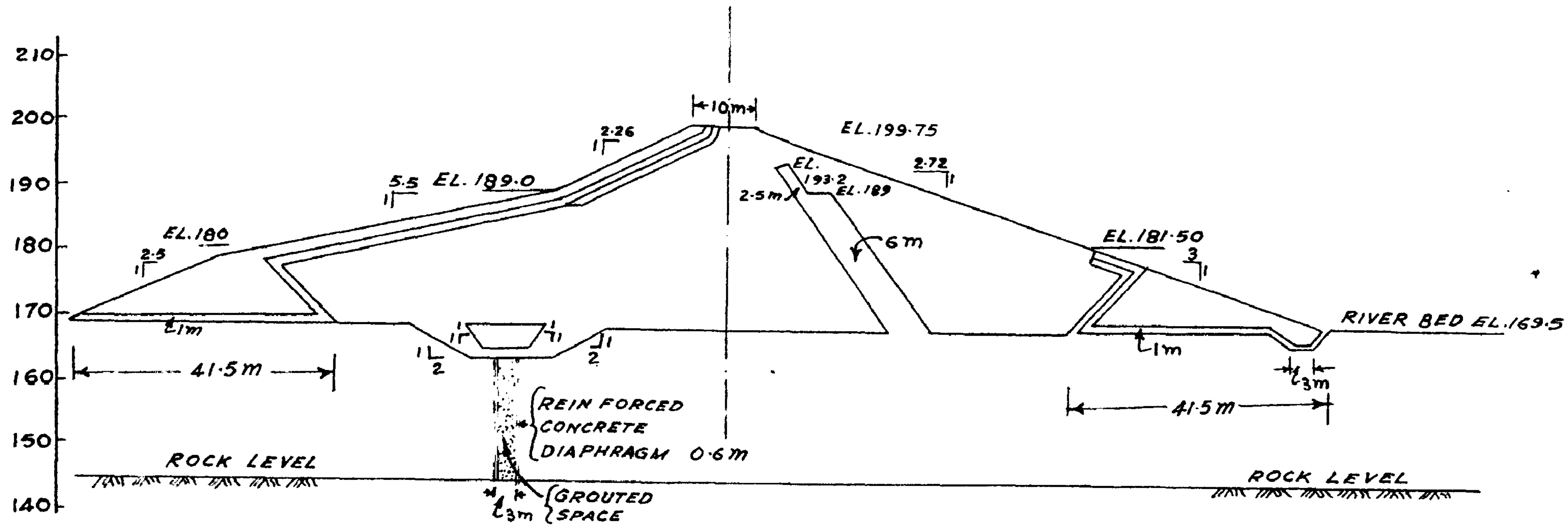


FIG. 9.3 CROSS SECTION OF MISSION DAM SHOWING CUT OFF WALL



CROSS SECTION AT CH 2150

FIG. 9.4. SHOWING THE EARTH DAM SECTION AND CUT OFF OF GIRNA DAM



SCALE = 1 cm = 10 m

FIG. 9·5. SHOWING THE EARTH DAM SECTION AND CUT OFF OF OBRA DAM.

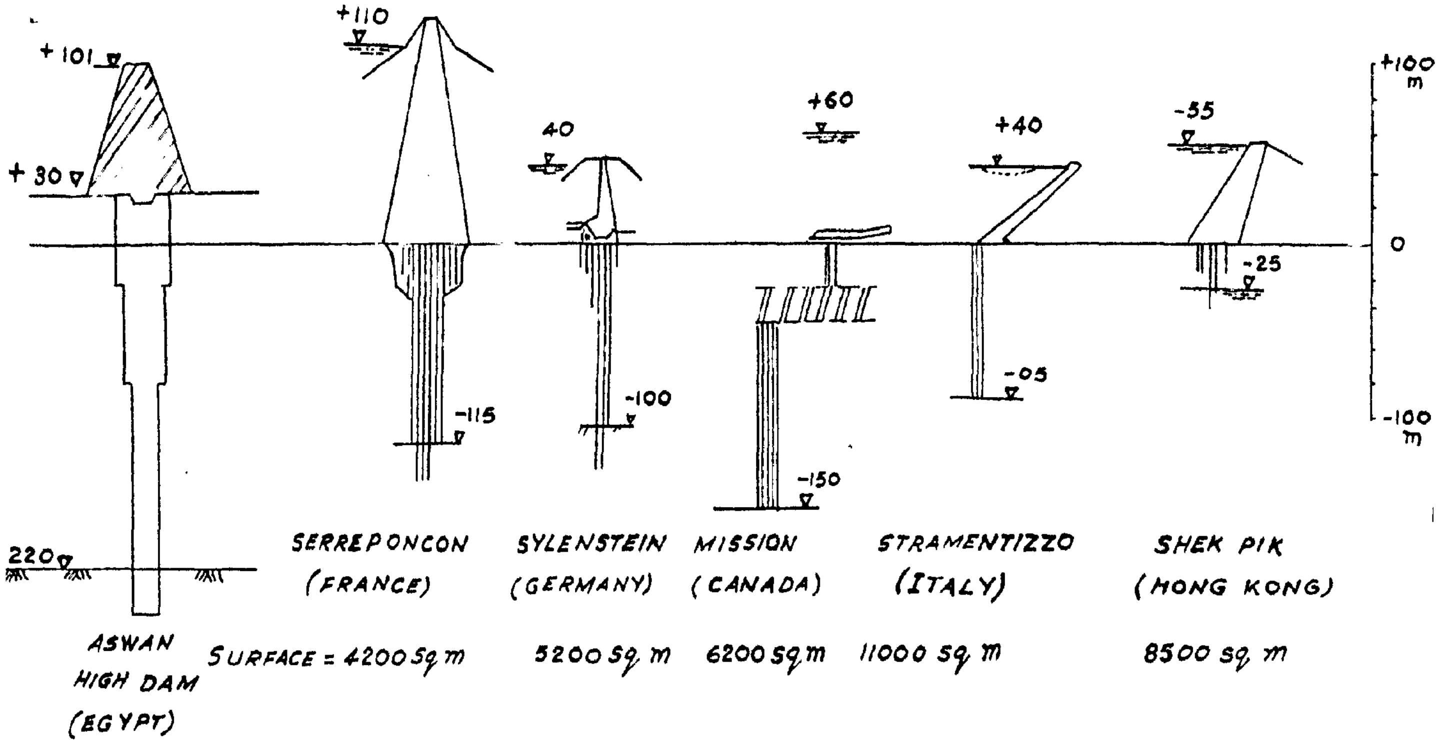


FIG. 9·6 - SHOWING GROUT CURTAIN IN ALLUVIAL SOIL FOR DAM FOUNDATIONS

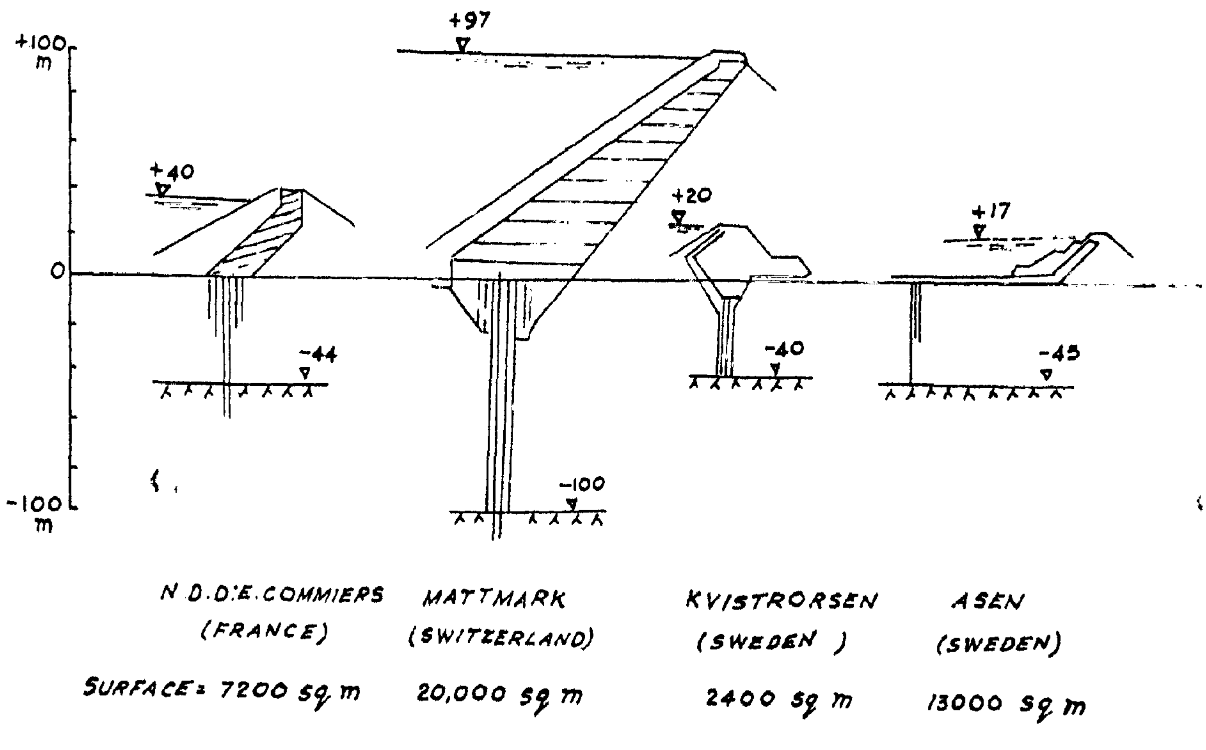


FIG. 9·6. - SHOWING GROUT CURTAIN IN ALLUVIAL SOIL FOR DAM FOUNDATIONS.

CHAPTER - 10

C O N C L U S I O N

The grouting is an art of changing the properties of the formation. The studies have shown that there is a basic difference between rock grouting and alluvium grouting. Though the grouting process was first attempted 166 years ago in 1802 it is only recently that alluvium grouting has now been established engineering practice which is most useful in the foundation of deep cutoffs for dams. In recent years this practice has been followed more scientifically at the Aswan High Dam where a thorough study of the characteristics of the alluvial deposits, permeability, zone of formation and different grout ingradients were made to give separate treatment to each zone to form a grout cut off. The subject is a complex one, and brings together the Civil Engineer, Mechanical Engineer, Geologist and a Research Officer. In this dissertation attempt has been made to demonstrate the basic principles of the subject, but it will no doubt remain one in which experience is more important than theory.

It is concluded from the studies that there is a definite relationship between the granulometry of grout material and the soil, and viscosity and injection pressures. The grouting of coarse sand and gravel of sizes greater than 0.8 mm. and permeability of greater than 10^{-1} cm/sec. can be grouted with neat cement grout having D85 size of 40 microns. But the neat cement grout is coarse and unstable and hence cheap grout of clay-cement-bentonite which is finer and stable are now-a-days preferred. The medium sands of size 0.2 to 0.8 mm. having permeability of the order of

10^{-2} cm/sec. can very well be grouted with clay-silicate or bentonite-silicates grouts depending upon the coarseness of the formation, the availability of good quality of clay and bentonite. The D85 size of clay is less than 30 microns and that of bentonite is about 5 microns. The fine sands having permeability of the order of 10^{-3} cm/sec. can only be grouted by colloidal grouts of silicate as the clay based grout will not penetrate the formation. Very fine sands and silt having permeability less than 10^{-3} cm/sec. will not even accept silicate grouts due to high viscosity and will only accept low viscosity grouts of organic resins. The study of injection pressures has indicated that high pressures are advantageous for cheap grouts of clay and cement to grout the coarser and open formations for confinement and to increase the zone of influence thus to reduce the drilling cost. The chemical grouts which are costlier may be injected at low pressures to avoid the overtravel. This is achieved by closer spacing of holes. The method of injection by tube-a-manchette is best as the same hole can be grouted any number of times with different grout mixes and at any stage.

The present study has revealed that grout cutoff walls are costlier than other methods of cutoff walls, but there are limitations where other methods may be expensive and troublesome. For example rolled earthfill cutoff will be expensive and troublesome where deep curtain are required to be provided and also in running water, which requires additional cost of constructing coffer dams. The limitations of sheet pilings are the depth and bouldery formation. The sheet piles are driven upto the depth of about 15 meters successfully and where the formation contains boulders the

the successful penetration is doubtful. The slurry trench method will be suitable for shallow depths where dragline bucket can be lowered. Stability of concrete diaphragm wall and permanence are doubtful. The efficiency of the grout cutoff walls has been observed to be more than 90% after operation and the reduction in permeability has been achieved more than 10,000 times of an original permeability. However, for high dams a provision is made in the design to provide impervious blanket and relief wells to avoid any mishap due to foundation failures. To induce the adoption of grout cutoff walls, the process should be economical and shall be able to compete with other methods of cutoff for all types of foundations. It requires a further investigation of cheap grout materials, cheap methods of drilling and grout injection processes, use of high pressures to increase the zone of influence and thus effect the saving in drilling cost etc.

At present there are no standard specifications of the grout material or process in I.S.I. or A.S.T.M. If the process is to compete with other methods of cutoff, then standard specifications shall have to be framed to standardize the process for all types of foundations. The quality of commercial bentonite and silicates varies from manufacturer to manufacturer, and requires a great deal of laboratory testing to prepare a mix. Slight variation in the quality of the material, greatly affects the properties of the mix. If the commercial products are manufactured according to the standard specifications for grouting, then much further laboratory testing can be reduced which will reduce the ultimate cost and time. Use of resin grouts is not yet developed in India and if we are going to use these products in India we have to investigate the source and manufacture the product indigenously to save foreign exchange and also make the process cheaper.

Systematic recording of grouting data shall also be explored to avoid wasting of much time in collecting unnecessary data and omitting useful information. The success of grouting operation depends upon the interpretation of the results achieved.

There is also a shortage of trained personnel in grouting operations. Often the persons trained in grouting operations are transferred to other jobs. It will be hardship for them to remain only in one branch of civil engineering but the success of the operation depends upon the mature knowledge and judgement of the grouting inspector. The civil engineering institutes and the Government departments should also train their personnel in this field if the process is to compete with other methods of cutoff.

R E F E R E N C E S

1. Vora, A.V., Modern Practice in Grouting Dam Foundations Annual Session, Bombay Centre of the Institution of Engineers(India), Paper No.3, pp-91-110, 1951.

2. Grundy, C.F., the treatment of grouting of permeable Foundations of Dams, Trans.5th International Congress on Large Dams, Paris, Vol.I, R-66, May 1955, pp-647.

3. Glossop, R., The Invention and Development of Injection Process, Part-I, Geotechnique, 10,3, 91-100,1960 Part II Geotechnique, 11,4, 255-79, 1961.

4. Hays, J.D.Gunite Blanket Impervious foundation Grouting for Earth dam, ASCE, Civil Engineering, Vol.23,No.4 April 1953, PP 58-59.

5. Dams in India, C.B.I.(India) Publication No.48,1950.

6. Khosla, A.N., Bose, N.K., and Taylor, E.M.,Design of Weirs on permeable foundations, C.B.I.(India) Publication No.12, 1954 (Reprint).

7. Ischy, E., and Glossop, R., An Introduction to Alluvial Grouting, Proc. Institute of Civil Engineers,London, Vol.21, March 1962, pp 449-74.

8. Terzaghi, K., and Peck, R.B., Soil Mechanics in Engineering Practice (1948), John Willey and Sons,London.

9. Jumikies, A.B., Soil Mechanics, D.Van Nostrond Co.Inc. Prince Town, New Jersey, New York.

10. Karol, R.H., Engineering Properties of Soil, Prentice Hall.

11. Clark, J.F.F., Discussion Proceedings of Conference on Grouts and drilling muds in engineering practice, May 1963, Butterworths, London, pp-114.

12. Krynine and Judd, Principles of Engineering Geology and Geotechnique, McGraw Hill Book Co.,1957.

13. Skemption, A.W. and Cattin, P., A full Scale Alluvial Grouting Test at the site of Mangala Dam, Proceedings of the conference on grouts and drilling muds in engineering practice, May 1963, Butterworths,London pp-131-135.

14. Desai, M.D., Field permeability test at Ukai, Journal of Indian National Society of Soil Mechanics and Foundation Engineering, Vol.4, No.2,April 1965, pp-237.

15. Casagrande, A., Control of seepage through foundatio and abutments of dams, Geotechnique 11,3 - 159-81,1

16. Mayer, A., Quelques reflexions Sur L' Utilisation Des Injections Dams les barrages, Geotechnique 11, 4-328-32, 1961.

17. Troxell, G.E., and Davis, H.E., Composition and Properties of Concrete, McGraw Hill, New York, 1956.

18. King, J.C. and Bush, E.G.W., Grouting of Granular Material, Proc.ASCE, Vol.87, No.SM-2, April 1961, Paper No.2791, pp.1-32.

19. Datye, K.R., Notes on grouting technique. Summer School of Soil Mechanics and Foundation Engineering, University of Baroda, Baroda, May 1963 q(Unplublished).

20. Mistry, J.F., and Purohit, M.U., Tests for a Stable grout mix in curtain grouting for the earth dam at Ukai, Journal of the Institution of Engineers(India) July 1964.

21. Grim, R.E., Clay Minerology, McGraw Hill, New York, 1962.

22. Mukherjee, M.M., Utilization of Bihar Bentonite in Irrigation Projects, Journal of Indian National Society of Soil Mechanics and Foundation Engineering, Vol.4, No.3, July 1965, pp-331-44.

23. Barbedette, R., and Sabarly, F., Contribution towards the technical Improvements and Economy of Sodium Silicate Treatment, Proc. 6th Congress on Large Dams, New York, C.7, VI, 1958.

24. Karol, R.H., Grout Viscosities, American Cyanamid Company, March 1963.

25. Leonard, M.W. and Dempsey, J.A., Clay for Clay grouting, Proceedings of Conference on Grouts and Drilling muds in engineering practice, May 1963, Butterworths, London, pp-116-26.

26. Mayer, A., Modern Grouting technique, Proceedings of Conference on grouting and drilling muds in engineering practice, May 1963, Butterworths, Londong, pp.7-9.

27. Datye, K.R. and Vinayaka, M.R., Studies on grouts for treatment of pervious soil, Journal of Indian National Society of Soil Mechanics and Foundation Engineering, Vol.4, No.2 April 1965, pp.149-76.

28. Raffle, J.F., and Greenwood, D.A., Relation between the rheological characteristics of grout and their capacity to penetrate soil, Proc.5th International Conference on Soil Mechanics and Foundation Engineering, Paris 1961, Vol.2, pp.789.

29. Clark, B.E., Discussion pressure grouting fine fissures, Proc.ASCE, Vol.85,No.SM-3, June 1959, pp.65.

30. Kravetz, G.A., Cement and Clay grouting of foundations, the use of clay in pressure grouting, Proc.ASCE, Vol.84 No.SM-1, Feb.1958, Paper No.1546, pp.1-30.

31. Kravetz, G.A., Discussion, the use of clay in pressure grouting, Proc.ASCE, Vol.85, No.SM-2, April 1959,pp.1-9

32. Raffle, J.F., Discussions, Proceedings of Conference on Grouts and Drilling muds in engineering practice, May 1963, Butterworths, London, pp.165.

33. Jones, G.K., Chemistry and flow properties of bentonite grouts, Proceedings of Conference on grouts and drilling muds in engineering practice, May 1963, Butterworths, London, pp-22-28.

34. Caron, C., The development of grouts for the injection of fine sands, Proceedings of Conference on grouts and drilling muds in engineering practice, May 1963, Butterworths, London, pp.136-41.

35. Bally, R.J.L., and Antonesev, I.P., Application de L'lectre Osmosc a L'elendu des sols ct a leur Consolidation, Proceedings of the 5th International Conference on Soil Mechanics and Foundation Engineering, Paris 1961, Vol.1, pp.7.

36. Fern, K.A., Application of Polymerization techniques to solution of grouting problems, Proceedings of Conference on grouts and drilling muds in engineering practice, May 1963, Butterworths, London, pp.146-49.

37. Scott, R.A., Fundamental Considerations governing the permeability of grouts and their ultimate resistance to development, Proceedings of Conference on grouts and drilling muds, in engineering practice, May 1963, Butterworths,London pp-10-14.

38. Technical Memorandum No.C-419, Tests of Standards Grouts U.S.Army Corps of Engineers Waterways Experiment Station, Vicksberg, October 1955.

39. Lippold, F.H., Pressure grouting with Packers, Proc.ASCE. Vol.84, No.SM-1, Feb.1958, Paper No.1549.

40. Burwell, E.B., Practice of the Corps of Engineers, Proc.ASCE,Vol.84,SM-1, Feb.1958,Paper No.1551.

41. Morgenstern, N.R., and Vaughan, P.R., Some Observations on allowable grouting Pressures, Proceedings of Conference on grouts and drilling muds in engineering practice, May 1963, Butterworths, London pp.36-42.

42. Scott, B.E., Discussion, the use of clay in pressure grouting, Proc.ASCE, Vol.84,No.SM-4,October 1958, Paper No.1828.

43. A.M-9 Chemical Groutings, American Cyanamid Company, 1965, Princetone, New Jersey.

44. Lane, E.W., Security from underseepage, masonry dam on earth foundations, Trans.ASCE, Vol.100, 1935,pp.1257.

45. Sheraard, J.L., and others, earth and earth rock dams, 1963, John Willey and Sons, New York.

46. Ambrasey, N.N., Cutoff efficiency of grout curtain and slurry trenches, Proceedings of Conference on Grouts and drilling muds in engineering practice, May 1963, Butterworths, London, pp.43.46.

47. Neelands, R.J., Discussion on Proceedings of Conference on grouts and drilling muds in engineering practice, May 1963, Butterworths, London, pp:174.

48. Moller,K., Discussion on Proceedings of Conference on Grouts and drilling muds in engineering pnactice,May 1963, Butterworths,London, pp.174.

49. Mistry, J.F., Clay grouting works at Ukai Dam(Gujarat State), Journal of Indian National Society of Soil Mechanics and Foundation Engineering, Vol.4,No.3,July 1965, pp.313-22.

50. Moor and White, Discussion, Pressure grouting fine fissures, Proc.ASCE, Vol.85, No.SM-2, June 1959,pp.65.

51. T.M.No.C-419, Report No.4, U.S.Army Engineers Waterways Experiment Station, Vicksberg, October 1958.

52. Ischy, E., and Haffén, M., Barrage De Serre Procon Compagnes De Reconnissances - 5th International Congress on Large Dams, Paris,1955, R-80, pp.803.

53. Mayer, A., Grouting with clay cement grouts, French Practice Proc. ASCE, Vol.84, SM-1, Feb.1958,Paper No.1550.