31^{st} IGS Annual Lecture

Development in Design and Execution in Grouting Practice

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Prof. A.V. Shroff rendered yeoman services in the development and implementation of fundamental concepts of Environmental Geotechnology and Geotechnical research-based water management. His fields of specialization are Foundation Engineering, Grouting Technology and Geosynthetics for Water Resources applications. His consistent efforts resulted in the establishment and functioning of Applied Research and Development Centre of Geotechnical Engineering in M.S. University.

Prof. Shroff was born on 19^{th} March 1942 and graduated in Civil Engineering from M.S. University, Baroda in the year 1964. After post graduation in 1966 from the same university, he obtained his Doctoral degree in the field of Civil Engineering in 1972. He had his post-doctoral training at London, U.K. under YSE Research award of the British Council during the year 1976–77.

An acclaimed teacher and researcher, Prof. Shroff has been serving the society by associating in the field of Water Resources and Geotechnical Engineering for the last four decades. He served in foremost position of a Senior Professor in the Faculty of Technology and Engineering till June 2004. Prof. Shroff held the prestigious positions of Syndicate and Senate member of M.S. University of Baroda during the year 2002–2004. He was honoured by the All India Council of Technical Education, New Delhi, which offered him Professor-Emeritus Fellowship. During 2002–04, he was the President of the Indian Geotechnical Society, New Delhi.Prof. Shroff guided five Doctoral Dissertations and 85 Dissertations of Post Graduate Students and is author of more than 150 technical papers in the National & International Journals and conferences. His books “Grouting Technology in Tunneling and Dam Construction” and “Soil Mechanics and Geotechnical Engineering” are widely read. He carried out various research and consultancy projects sponsored by CSIR, UGC, HRD, AICTE, MOWR and Ministry of Energy of Iran. He is onthe panel of experts of ropeway committee and Sardar Sarowar, Narmada Nigam, Government of Gujarat.

The Indian Science Congress honoured him by inviting him to deliver the prestigious Platinum Jubilee Lecture in 1979. In recognition of his outstanding research and development contributions in the field of Water Resource and Energy, the Central Board of Irrigation and Power, New Delhi, presented him the CBIP Pandit Jawaharlal Nehru Birth Centenary National Award in the year 1997. He received the IGS Kukelman National Award for outstanding life-time achievement in the field of geotechnical engineering. He was recently honoured as an “Honorary Fellow” of the Indian Geotechnical Society, New Delhi. He is also one of the recipients of the Diamond Jubilee award of Indian Geotechnical Society. Prof. A.V. Shroff is, at present, “Adjunct Professor” at autonomous DD University, Nadiad, Gujarat.

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Developments in Design and Execution in Grouting Practice[1]

A.V. Shroff[2]

ABSTRACT: Permeation and Jet grouting can control ground water seepage through and below the structure and can increase the stability of soils/ rocks against any structural load during and after construction, while compaction grouting can densify surrounding weak mass and/or uplifting the settled structure. The paper focuses attention on the process of development and understanding that has been generated in India and elsewhere over the last one and half decade.

Paper includes design and characterization of suspension-cement based conventional grouts along with newly developed micro fine grouts & grouted mass with reference to rheology and strength aspects under static and dynamic loadings. Long term compressive and tensile strength with creep measurements and Cyclic Behaviour of MC grouted sand under cyclic undrained load for liquefaction, fatigue life and damping ratio measurements have been dealt with including apparatus developed & typical results and their performance in the field. Use of newly developed triangular chart for design of chemical and cement based grout mixes & their field performance are worth pursuit. Some of the new patterns of grout holes with reference to site geology, volume of mass to be treated and kinematics of rig are designed and implemented for different problems. Newly developed TPC (Time-Pressure-Consumption) concept synonymous with GIN (Grout Intensity Number) has helped in the assessment of the behaviour of grout holes during grouting operation.

Computerization of grouting plant with soft wares and pick ups along with readout station can perform effective monitoring of permeation/ compaction/Jet grouting. Execution problems and methods of remediation along with control of production parameters at various grouting sites in India and elsewhere of Permeation/Compaction and Jet grouting are described. Conformation of proposed method of remediation at some project by Centrifuge Modeling and numerical methods using 3-D model are also presented.

The paper concludes with suggestions that India must adopt innovative techniques to help Grouting to emerge out as an industry of its own right.

KEYWORDS: Cement, Chemicals, Micro Fine Grouts, Liquefaction, Grout Curtain, Consolidation / Blanket Grouting, Computerized Monitoring, Execution Problems, Remediation, Permeation, Compaction, Jet Grouting.

Introduction

In the context of ground engineering, the term grouting is used for the process of pressure injection of setting fluids into pores and cavities of soil or rock. The process is widely used in the construction of tunnels, shafts and dams for the purpose of either reducing water percolation or increasing the mechanical stability. If a useful engineering purpose is to be achieved, the sealing and strengthening actions must extend a considerable distance into the formation, and it is, therefore, general practice to inject the grout into a special array of boreholes drilled into the rock or the soil.

Current grouting methods are effective in sealing cavities in both coarse and fine fissures in rock, and in sealing pores in granular materials typical of all soils short of clays and very silty sands. With the development of high-pressure pumps, cement grouts came to predominate, and they were frequently associated with the sinking of mine shafts. Local clays modified by peptizers were being used in earth and rock filled dams constructed in Europe in the 1950s and 1960s. Bentonites, either natural content or treated, had come into use, and the property of such materials to form stable and impermeable gels with the ability to penetrate medium-to-fine sands had been utilized for seepage control. In certain ground conditions such as boulder clays and in earthquake-prone areas, grouted cut-offs had been effective.

Grouts based on sodium silicate in combination with other reacting chemicals as preliminary injection to improve the penetration of subsequently injected cement gouts were in vogue during the first decade of 19^{th} century. However, the development of a particular silicate grouting technique by Joosten in 1925 greatly advanced the application of grouting, particularly for the strengthening of gravel formations. The practical difficulties of controlling the rate of gelling of sodium silicate solutions led to two distinct methods of injection, known as single-shot and two-shot processes.

In the late forties, Robert E. Lenihan, a mud jacking contactor in Long Beach, California (USA), reported using zero slump soil-cement grout pumped through pipes driven to various depths for lifting structures. Working empirically for the basic technique termed *compaction grouting*, the spacing of grout bulb injections to effectively compact a low density soil mass was developed. Graf, 1969 and Mitchell, 1970 discussed compaction and penetration grouting techniques. The compaction grouting consists of intruding a mass of very thick consistency grout in to the soil, thus both displacing and compacting it. Thus, its use is nearly always limited to soils and usually for low compaction. Brown and Warner,1973 used this technique to stabilize the soil under residences and light commercial building. However the penetration grouting has been extensively used to stabilize foundations of large structures including foundations of bridges and culverts and the ground under the tips of piles.

1 31^{st} IGS Annual lecture delivered at IGC 2009, Guntur. Remaining part of this lecture will be included in the next issue.

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The in situ deep mixing of stabilizers with soft soils to form columns, wall grids or blocks in the foundation has been developed and applied extensively in civil engineering practice since 1970s. The two type of mixing methods, namely *deep mechanical mixing* (DMM) and high-pressured *Jet-grout mixing* have been used under deep ground conditions. The application range is shown in Figure1.

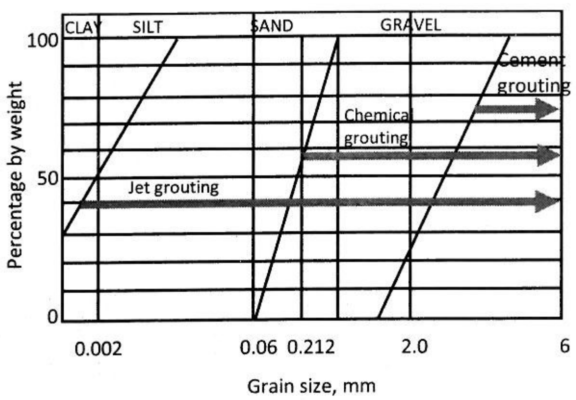


Fig. 1 Application Range-Grouting Methods

The Deep Mixing Method, DMM, uses slurry state or dry powder state stabilizer. Originally, the process ejected the grouting material itself at a pressure of 200 bars to cut soil and fill, but it was capable of obtaining an improved body only 0.5 meters in diameter. Both the methods rapidly spread nationwide in the 1970s. By 1990, creating a much larger improved body became a primary concern and a variety of approaches were tested to achieve this.

Application and Functions

Grouting can be used to control ground water flow and then to increase the stability of granular soils beneath existing structures, thus reducing lateral support requirement. After construction, grouting can be used to reduce machine foundation vibration and to eliminate seepage through crack / joints and pores. Many underpinning and anchoring processes have also employed grouting successfully. The basic functions of grouting are shown in Figure2.

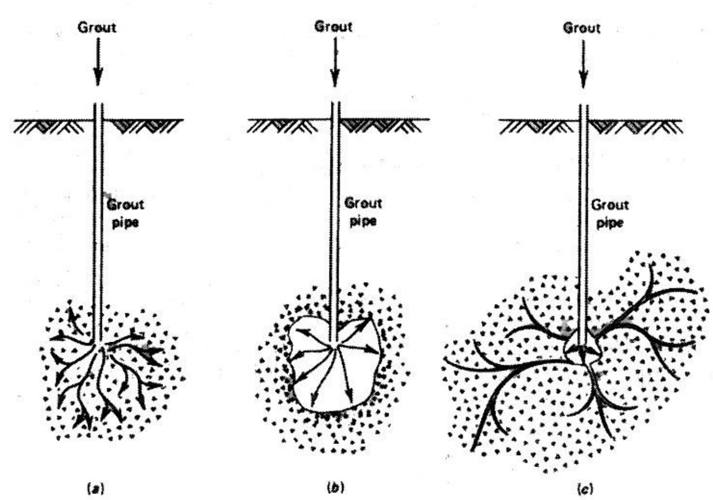


Fig. 2 Functions of Grouting

(a) Permeation or penetration: Here the grout flows into the soil voids or rocks seams with only minimal effect on the original structural arrangement.

(b) Compaction or controlled displacement: In this situation the grout remains more or less intact as a mass and exerts pressure on the soil or rock to compact and thereby densify the weaker zones of the nearby areas and/or uplifting the structures to minimize the settlement.

(c) Hydro fracturing or uncontrolled displacement: When the grouting pressures are greater than the tensile strength of the soil or rock being grouted, the latter material fails, and the grout rapidly penetrates into the fractured zone, thus, lenses of nearly solid grout are project into densified regions of the in situ soil or displaced rock masses.

Types of Grouts

Two classes of grouting materials are generally recognized: (i) suspension-type grouts and (ii) solution-type grouts. They are used for both impermeation and strength. The suspension grouts develop the strength and sealing ability when the cement hydrates and cures into a system of interlocking crystals. Water cement ratios are in the range of 0.5:1 to 5:1. The lower the water cement ratio, the greater the strength of the stabilized mass. Many types of chemicals (solution) grouts have been developed for injecting fine sands & coarse silts. The most common ones are silica gel, aminioplast, phenoplast, acrylamide, chrome lignin, vinyl polymers, epoxy and polyurethane etc.

In order that grouting results be good, one has to choose the grout based on its viscosity, setting time, strength, etc. most suitable to the problem. It is also necessary to know how to distribute the grout in the soil, for which it is necessary to make the correct choice of (i) grout hole equipment, (ii) distance between grout holes, (iii) length of injection passes, (iv) number of grouting phases, and (v) grouting pressure and pumping rate. Time-Pressure-consumption data during grouting is very useful for assessing the efficiency. Computerized electronic monitoring systems for colleting, processing, and storing data compiled during grouting work are now available helping useful analysis during and after grouting. The efficacy of a grouted mass in terms of degree of impermanence achieved and the strength of the grouted soil or rock is of paramount importance.

Classification

Broadly, grouts are classified as particulate course grouts and watery low viscous fine grouts. The major functional difference between them is that the penetrability of the former is a function of particle size relative to the size of pores or cracks to be grouted in addition to its initial viscosity, while for the latter, it is a function of initial viscosity and gel time.

Engineering Classification

The engineering classification is based on grout characteristics and its engineering performance. It includes not only the flow and strength characteristics of grouts, but also the strength and permeability of the grouting mass due to its interaction with the grout. Engineering classification and its subdivision based on grout characteristics are given by strip chart (Bell, 1957)

Rheological Classification

Rheologically, particulate and fine chemical grouts are classified as granular Binghamian and non-granular Newtonian grouts respectively, based on their initial viscosity. The flow curve for a Newtonian grout is a straight line that passes through the origin, while for a Binghamin gout it makes an intercept with the shear stress axis, which is commonly referred to as the yield value1-20N/mm^2. It is required to overcome this yield value for maintaining a uniform flow (Figure3).

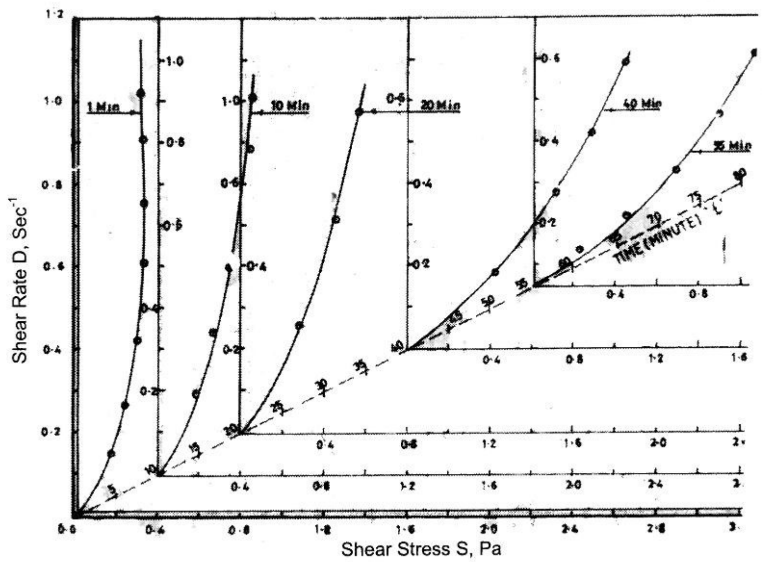


Fig. 3 Yield Value of the Grout-DST Plot

Coarse Grouts

The groutability ratio reveals that the size of the grout particle and the size of opening or pore spaces of the mass to be grouted should be in such a proportion that blockage and filtering of the grout can be minimized. It can also occur at low viscosity of these grouts which tends to gravitate towards the bottom space of passage. Roughness and adhesion between passage and particles will decide the "blockage criteria' (Figure4).

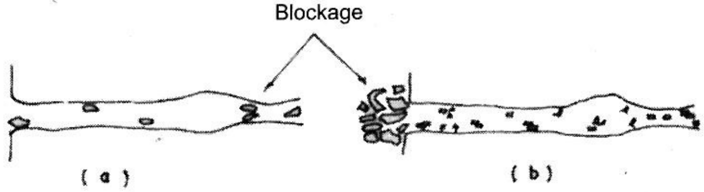


Fig. 4 Effect of Dispersion/ Flocculation - Blockage

Various researchers have suggested particle size criteria based on D_{15}, D_{10} sizes of foundation material and d_{85}, d_{95} sizes of grout material. The following criteria exist for limits of particle size for effective grouting:

(i) D_{15}/d_{85} should be between 5-24 (Kravetz, 1958) or should be at least 16 (King and Bush; 1961) or should be greater than 25 (Johnson, 1958; Karol, 1960; Mitchell, 1970)

(ii) The diameter of the grout particle must not be more than 1/10th of the D_{10} size of the soil (Bell, 1982).

(iii) The soil pore should be three time the diameter of the grout particle to avoid blockage by bridging (Raffle and Greenwood, 1961)

(iv) True Binghamian grouts such as clay, cement, bentonite or bentonite-cement, can be successfully grouted in a formation having a permeability of more than 10^{-3} cm/sec and a specific surface area of particle greater than 10^{-1} cm and its effective size (D_{10}) greater than 0.5 mm (Terzaghi, 1936; Cambefort, 1957; Glossop, 1962; Skempton, 1963).

(v) In the case of rock grouting, the size of fissure should be three time greater than the diameter of grout particle for an appropriate groutability ratio (Mitchell, 1970).

(vi) The usual cement particle size limits the penetration of cement grout to fissures in rocks of less than 160 micron width unless fracturing pressures are used (Littlejohn, 1982).

(vii) To avoid filtering of grout material from the void space, the ratio between D_{10} and d_{95} should be more than 6 (Kravetz, 1958).

From the experimental study of (Shroff, Joshi, 2005) penetrability of *newly developed micro fine (MC)* cement-grouts in different sand gradations, it is found that D_{15} / d_{85} should be more than 40 (D for sand and d for MC).

Criteria presented above provide only guide lines because of heterogeneity in pore, crack & fracture sizes of alluvium / rock & swelling characteristic of some grout materials (Shroff, Joshi,1994). Nevertheless it controls upper & lower limits of size of grout particles & the size of discontinuity of formation to be grouted. At the same time it guards against washing out of injected grout from the grouted formation.

Fine Grouts

Grout viscosity is one of the important factors in regulating the movement of grout injected into a porous formation. The penetrability of various chemical grouts in relation to soil grain size is shown in Figure 5. Pure chemical solution such as acrylamides, aminoplast or phenoplast & MC grouts are recommended for very fine soil characterized by d_{10} less then 0.02 mm and a surface area of less than 1000 cm^{-1} (Shroff, 1994). Caron (1982) ascertained that rock cracks less than 0.1 mm wide can be grouted with chemical grouts in cases where cement cannot be injected.

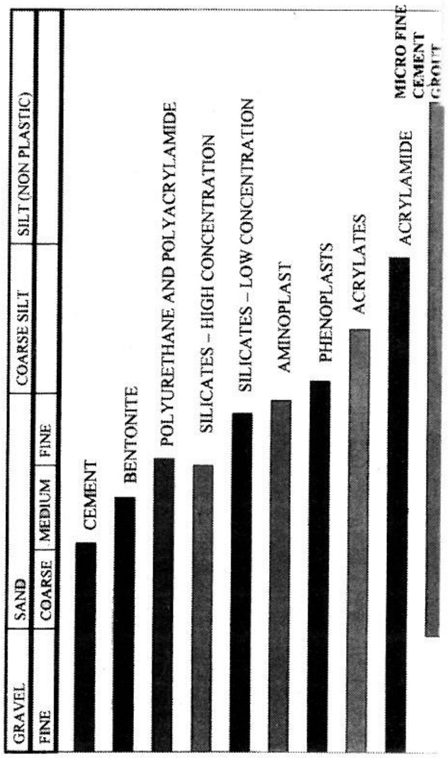


Fig. 5 Penetrability of Various Grouts

Development of Grout Mix

Grout Mix Design

The success of the grouting is dependent on selection and type of grout materials, grout mix design and suitable grouting techniques.

Ideal Frame Work Approach

Grout systems are usually developed within the framework of ideal properties, wherein the designer should employ the ingredients of the grout in such a way that fluidity and strength are balanced along with various physical properties. These ingredients should consist of proper specifications to identify quality.

The Conceptual Frame Work Approach

Shroff (1990) proposed this approach for chemical grout system based on proportion of molecules of ingredients in a unit cell of resultant ideal gel. The First step is to determine the desired general properties of the resultant gel (e.g., rigid, semi-rigid, flexible, linear, highly cross-linked etc.) Next, a conceptual framework is developed. This is an idealization of actual molecule formed in a given chemical grout system. It shall be the most representative molecule for the system that can be visualized. From this a unit cell, the smallest repeatable unit in the polymer structure is defined. This allows the determination of relative per cent of each chemical to be added. Finally, appropriate chemical species are substituted in a unit cell, again based on the desired general properties of the resultant gel. In spite of the apparent disparity between the different unit cells of all the grout systems described above, they all have something in common. Typical gel structures for different grouts are presented in Figure 6.

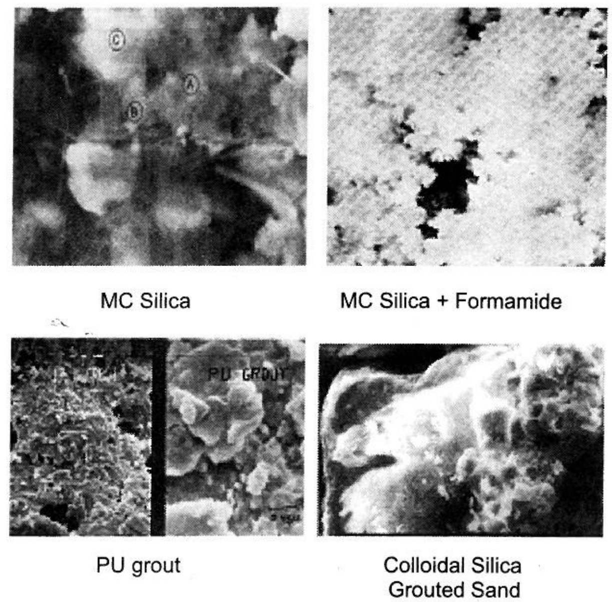


Fig. 6 Gel Structures for Different Grouts

Equivalent Weight Approach

The equivalent weight approach (Shroff et al, 1990) involves the balancing of equivalents of reactant chemical species in the system. The equivalent of their respective reactive radicals is then listed. Knowing the molecular weight of a chemical, its equivalent weight is determined Adding the equivalents of chemical of components of one group gives the equivalents of chemical components of another group, necessary for a balance reaction to take place. Multiplying the equivalents of one basic chemical by its equivalent weight gives the actual weight of it that should be used.

Testing Set-ups

In designing a coarse grouse grout mix, the conventional framework approach is utilized by balancing time viscosity by Brookfield viscometer and time strength as measured by vane shear, Triaxial compression test or servo loop material testing system (MTS) along with physical measurements such as specific gravity, water retentivity, bleeding potential, gel time, fluidity by Marsh cone, penetrability& washout test along with turbidity of leached water from grouted mass to access interaction of set grout with discontinuity of formation.

For chemical grouts, the additional tests such as PH value, syneresis, toxicity, measurements by fish mortality test are required to be performed.

Since the grouting is carried out under pressure particularly for jet grouting, the grout is subjected to hydrostatic pressures; this test has been developed to study the bleeding of grout under pressure (Shroff, 2005). In compaction grouting, the consistency is measured by slump test and fluidity by horizontal flow meter and filter press.

Interaction of Grouts with Void Space or Discontinuity

The possible microstructures resulting from the interaction between the set grout and ground are illustrated in Figures 7. (A) Set grout fills the pores and adheres to the surface of the void space, having completely displaced the ground-water. (B) Set grout only coats the surface of the space, leaving water to occupy most of the pore volume. Alluvium or rock, set grout and water are each continuous in three dimensions. A substantial part of the original permeability may be retained after treatment. Accumulation of grout at points of contact and interfaces under the influence of surface tension may give strength (C) Pores are filled by grout with poor wetting ability for the pore surface, or by one which has formed only a weak bond and has undergone subsequent "syneresis". The water film thickness remains low around the grain contacts. (D) Grout nodules loosely held within the pores may reduce permeability, so long as they are not themselves displaced by ground-water flow or by washout gradient of the reservoir water level of the dam (Figure 8). This type of interaction is always avoided.

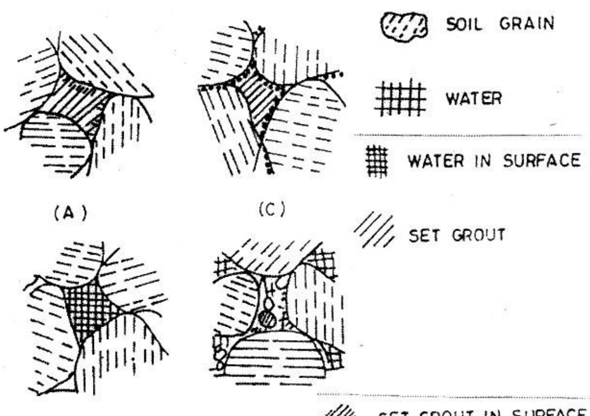


Fig. 7 Interaction of Set Grout with Void/Discontinuity of Alluvium or Rock

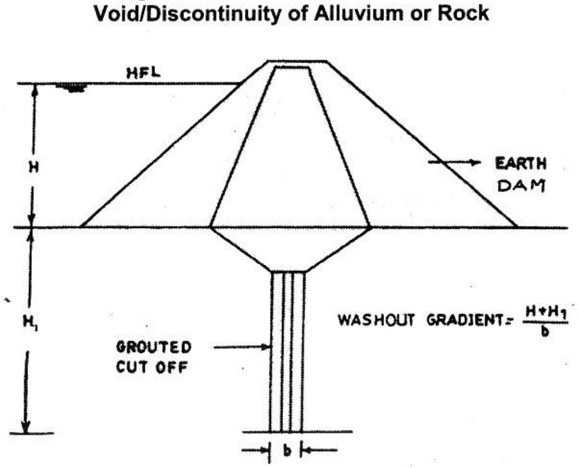


Fig. 8(b) Washout Gradient against Grouted Curtain

Cement Based Grouts

Also known as suspension grouts, cement grouting is the most important and the most widely used method in the construction industries for reducing the mass permeability and / or increasing the strength of natural formations.

Neat Cement Grouts

The production of a durable, satisfactory cement-based grout is dependent on the water: cement ratio for fluidity control, the rate of bleeding (less than 5%) for making stable grout, and subsequent ultimate strength of the grout. (Mistry and Shroff, 1980)

The stages of gelation of cement grout after mixing is dormant, setting and hardening. Neat cement grout in the early stage of injection is controlled by viscosity, but in the later stage may be controlled by shear strength. Hydro gel is formed by hydration of the calcium silicate present in cement. Tricalcium-aluminate is found to increase the strength or consistency of the paste. Water cement ratio between 0.6 and 5 gives a grout with sufficient fluidity (Marsh cone viscosity is 28 to 42) to be pumped and after injection cement content to act as a strengthening medium.

Stress strain relation of neat cement grout is shown in Figure 9. One day and seven day stress-strain curves for thick and thin mixes show distinct characteristics. More per cent strain is required to reach a peak value as w:c increases. The one-day compressive strength: 10.5 kg/sq. cm at 0.6 w:c drops to 1 kg/sq. cm at 2 w:c, thereafter, a marginal drop is observed. The seven-day compressive strength increases from 9.8 kg/sq. cm at 2 to 18.9 kg/sq. cm at 0.6 W. In brief, a grout mix having a water : cement ratio of 2 W bears distinctive characteristics, below which it exhibits higher viscosity, more yield value, lower flow value and more short-term strength up to seven days, and above which, i.e., in all thinner mixes, the reverse is true.

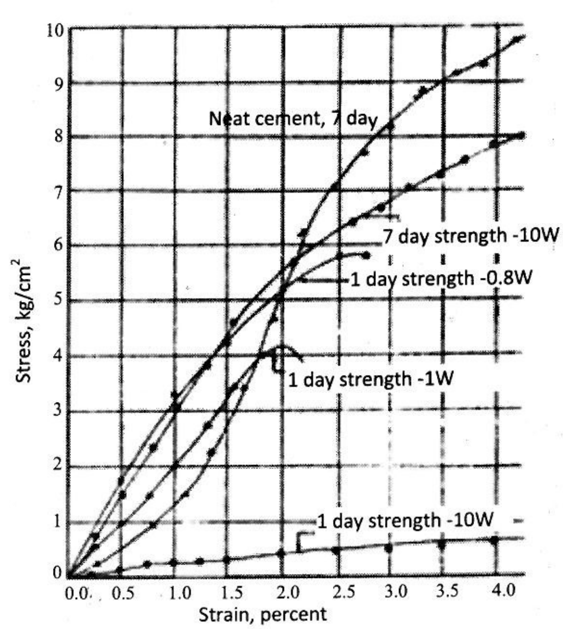


Fig. 9 Stress-Strain Relation-Cement Grout

Cement with Bentonite as Admixture

Two per cent bentonite by weight of cement markedly decreases the amount of solids settle in cement grout due to the thickening caused by complete swelling of the bentonite that absorbs lime and alkalis leached from the cement. Bentonite helps in cracking flocs of cement, penetrating small voids increasing grouted area and preventing settlement of particles, making the cement dense and impermeable. Marsh funnel viscosity of cement grout with bentonite appreciably increases the funnel viscosity of the slurry. It lowers the strength by 50 to 70 per cent in thinner grout mixes while in thicker ones the strength loss is only 25 per cent.

Cement with Fly Ash as an Admixture

The fly ash markedly decreased the consistency at a constant water :

Cement –fly ash ratio. Fly ash with OPC provided a grout of lower viscosity than with special cement. Neyveli fly ash reduces gelation time, afflux time, bleeding and seven day's unconfined compression strength. Figure 10 presents the stress – strain relationship for cement –fly ash grout mixes.

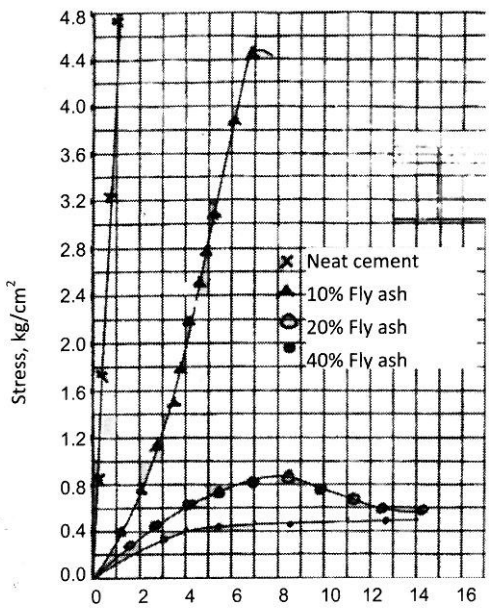


Fig. 10 Stress-Strain Relation-Cement Fly Ash

Cement with Chemical Admixtures

Calcium lignosulphonate as a water-reducing admixture on cement suspensions separates particles, promoting penetration in a given crack with grout of lower water cement ratio. Bleeding can be reduced somewhat and setting time increased. Ten per cent volume of micro air bubbles produced by an air-entraining agent can reduce strength up to 50 per cent. Accelerators do not significantly alter the rheology of a cement paste at an early age. The quicker stiffening of an accelerated paste may result in higher viscosity at a later stage. Figure 11. shows time viscosity relation of cement with various admixtures Shroff and Shah (1992).

Sodium silicate as an admixture slows long term strength while calcium chloride accelerates the short gelation. Accelerators display time-viscosity curves which exhibit high viscosity at any time for a given water: cement ratio. The water reducer shows linear Binghamian in the low stress range compared to neat cement while the accelerators remain non-linear Binghamian in the same low stress range. Accelerators show early strength build up and visoelasticity compared to neat cement while water reducer gives consistent delay in stress build up. The grout with accelerators remains elastic for higher peak stress compared to neat cement (Figure 12). Shroff et al, 1992. Sodium Silicate exhibits high compressive strength compared to neat cement. It is higher by 21, 49 and 25 percent over that of neat cement at 7, 28 and 90 days respectively.

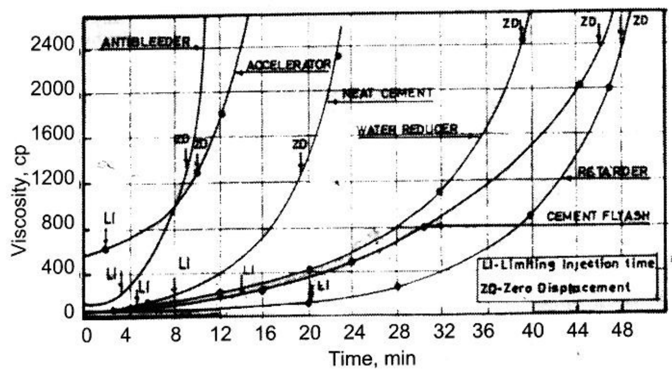


Fig. 11 t-η Relation of Cement with Admixtures

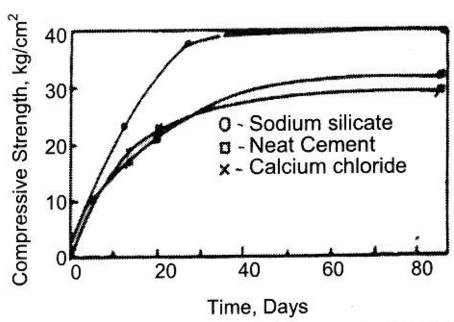


Fig. 12 q_c time Relation of Cement +SS $CaCl_2$

Addition of Methyl Methacrylate in neat cement grout increases its initial fluidity reducing bleeding potential and reducing gel time. The general nature of time viscosity relationship indicates gradual increase of viscosity initially, and rapid increase after zero displacement (ZD) time Figure 13.(Shroff, 1992) Optimum content of Methyl Methlacrylate 2% show compressive strength =162 Kg/cm^2, while E=395 Kg/cm^2 of neat cement increases to 2600 Kg/cm^2 with MMA (Figure 14) and tensile strength =18.4 Kg/ cm^2 of cement grout (w : c =1). Table1 shows optimum per cent of admixtures based on conceptual frame work approach (Shroff, 1992).

Use of Azo-bis- isobutylnitrite and Benzoyal Peroxide to the neat cement grout containing optimum amount of MMA further accelerate the reaction, as well as increase long term strength.(Shroff et al, 1992.)

Table 1 Optimum Admixture to Cement Grout (Shroff *et al.*, 1992)

Admixture	*Chemical*	*Optimum dosage, % cement weight*	*Remarks*
Accelerator	Calcium chloride	0.5–2	Accelerates set and hardening
	Methyl Methal Acrylate	1% + 1.2% Benzoyal Peroxide or AIBN*(1.2)	
	Sodium silicate	0.5–3	Accelerates set
	Sodium aluminate	0.5–3	Accelerates set
Retader	Calcium lignosulphonate	0.2–0.5	Also increases fiuidity
	Trataric acid	0.1–0.5	
	Sugar	0.1–0.5	
Fluidiser	Calcium lignosulphonate	0.2–0.4	Two times UCS, Effective water reducer
Expander	Aluminum Powder	0.005-0.02	UP to 15% expansion
Filler	Fly ash	0.8c + 0.2 Fly ash	Reduce afflux time and bleeding with same long tern strength
Antibleeder	Bentonite	2.0	For making mix dense and stable
	Aluminium sulphate	5.0	

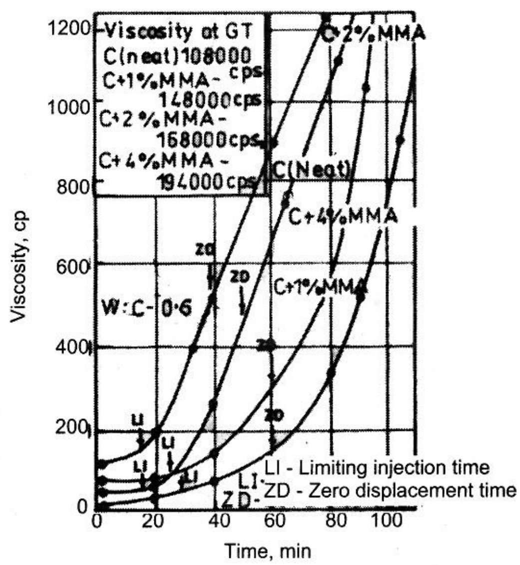


Fig. 13 t-η Relation-Neat Cement with MMA

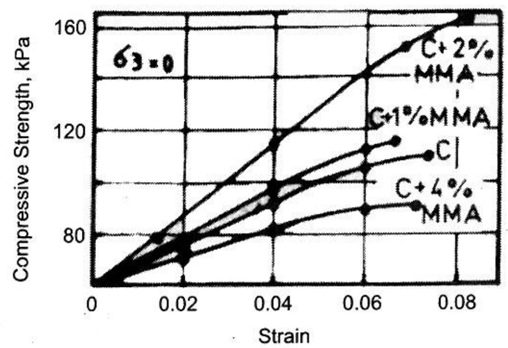


Fig. 14 (a) σ–ε Relation: Cement + MMA

Fig. 14 (b) Sand Particle Interaction with Chemical Grout Magnification: 230X

Cement Sand Grouts

Various sand cement ratio (2.5:1, 2:1, 1.5:1) and water cement ratio (0.9, 0.8, 0.7) are tried to get the desired flow value and strength. Initial viscosity of sand cement grout is sixty times the initial viscosity of neat cement grout for the same water cement ratio. Addition of small percentage of triethanolamine (TEA) reduces the set time and bleeding potential as well as it improves flow and strength properties. Grout mix consisting of cement sand ratio 1:1 shows three days compressive strength of 127.59 Kg/cm^2, 198.97 Kg/cm^2 and 183.15 Kg/cm^2 on addition of 0.5 per cent, 1 per cent and 1.5 percent TEA respectively (Shroff & Patel,1987), indicating 1% as optimum amount. Water-reducing admixtures are more effective in increasing pumpability and water retentivity than fly ash. The use of a water reducing agent somewhat improves the water retentivity of grout.

In Diversion tunnel project, Bias dam, India (1982) used a sand-cement grout for sealing voids between the tunnel lining and rocks with sand cement ratios varying from 0.5 : 1 to 2:1 with w:c = 1. A mix containing sand cement ratio (s: c) 1.25 with w: c of 0.7 is used at Sardar Sarovar project for filling interface contact between crown of concrete key plug and base rock, while grout with s: c ratio of 2 and 0.78 w: c ratio is used at Kalindri dam project, Saurashtra, Gujarat project for filling solution cavities of cavernous limestone.

Also, during perennial canal lining by ULO mat technique used at Ukai branch canal, Gujarat, cement-sand grout along with 2% Bentonite and flyash (0.8 c + 0.2 flyash) is used.

Cement-Bentonite-Clay

Among the various grout mixes, proportions of bentonite : cement : water (1 : 0.5 : 8) have given good strengths of about 75 gm/sq. cm. to 300 gm/sq. cm. at different time intervals of 1 to 7 days. Sodium silicate and monosodium phosphate in 1:1 proportions improved workability increased strength from 20 gm/sq. cm. to 900 gm/sq. cm. and reduced gelation time from 5 hours to 48 minutes.

A hydro cyclone Figure 15, which works on the centrifuge principle, was employed to separate clay from the local Tadkeshwar soil at Ukai for designing an economical stable cement-clay grout mix as shown in Table 2. Figure 16 shows viscosity-time curve of cement-bentonite grouts. Thixotropic bentonite gel can be made irreversible by addition of sodium silicate. Also, rate of gelation and final shear strength were influenced by several concentrations. Sodium and Potassium chloride induce volumetric expansion in Bentonite that can seal and reduce permeability.

Table 2 Various Grout Mixes used at Ukai and Daman Ganga Projects (Gujarat) Shroff and Mistry (1980)

Clay	*H.G. Bentonite*	*Cement*	*Water*	*+Peptizing agent*	*NPR gm/cm²*	*Washout Pressure (Kg/cm²) (hydraulic gradient)*	*Average reduction in permeability, %*
–	1.0	0.5	8	-	1628	2.45 (81)	
0.8	0.2	0.6	6	0.1 to 2 %	1804	7.14 (63)	
–	0.4	1	5	2 %	2475	2.8 (92)	
–	0.7	1	5	2 %	2060	2.45 (81)	99.03
0.4	0.6	0.6	6	2 %	1775	2.75 (85)	
–	1	0	8	Sodium silicate = 1 Monosodium phosphate	1250	6.42 (90)	
–	1	0.5	8	S / P = 1	2350	2.9 (92)	

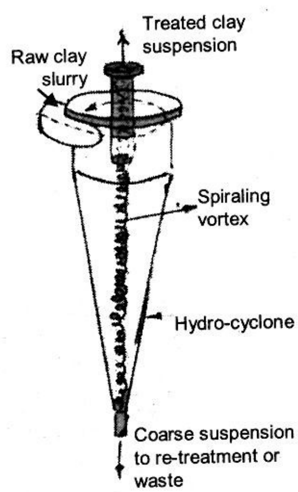


Fig. 15 Hydro Cyclone

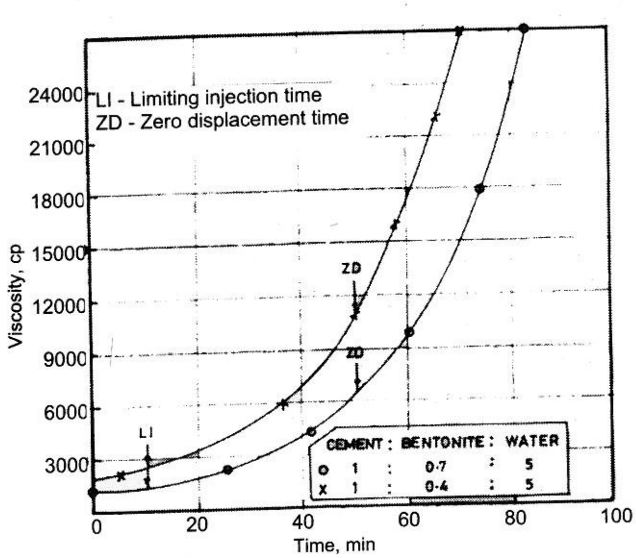


Fig. 16 t-η Relation of Cement-Bentonite Grouts

Chemical-Based Fine Grouts (Solution Grouts)

Shroff and Joshi (2003, 2006) introduced the use of sodium silicate silicate in conjunction with other reactive chemicals along with micro fine cement based grouts for strengthening sandy foundations and jointed & fractured rook formations. Shroff, Amin and Shah (1988) and Shroff and Shah (1987, 1988, 1989) have done fundamental research on various chemical grouts employing various precipitants with sodium silicate, chrome lignin, aninoplast, phenoplast, acryl amide, epoxy resin, polyester resin and polyurethane grouts. Shroff and Shah (1984) have studied the physico-chemical relation of new precipitants along with basic chemicals. Sensitivity & toxicity of chemical grouts are tested by lethal Dose (LD50) or fish mortality tests. Strength aspects have been discussed by shroff and Amin (1982) and Shroff and Shah (1984, 1985) and Shroff Amin (2005). Guide specifications were laid down for proper testing procedure of chemical grouts by Mistry, (1964) Shroff and Shah (1983, 1987, 1988, 1992, 1995).

Sodium Silicate

In silicate grouts, initial minimum viscosity of grout that can produce gel is by SiO_2: Na_2O ratio 3.9, having pH value 8.5 to 9.2 for a given dilution within ideal frame work of gel time. Rate of reaction (gel time) and gel strength are directly proportional to concentration of acidic or basic catalysts and silicates in grout at constant temperature respectively. Various reactants (a) inorganic i.e. acids and acidic salts (mainly sodium aluminate and sodium tetraborate (b) organic (mainly formamide, ethyl acetate, silica fume (c) colloidal silicate (mainly with potassium chloride) (Shroff *et al.*, 2004) and acid silica sol (Yonekura, 1997) are used for producing medium stiff, hard and very stiff silica gel. Table 3 illustrates various components.

Table 3 Sodium Silicate Grouts with Organic and Inorganic Precipitants (Shroff *et al.* 1999)

Sodium Silicate	*Water*	*Precipitants*
Inorganic		
300 cc	500 cc	Sodium aluminate =13 gm
260 cc	900 cc	Sodium aluminate =17 gm
100 cc	1750 cc	Phosphoric Acid = 20 cc
Organic		
70 ml	30 ml	9 cc (ethyl acetate)
30 ml	60 ml	10 cc (5 ethyl acetate + 5 $Cacl_2$)
30 %	54%	12.5% formamide + 1.5% $Cacl_2$)
50 %		Silica fume (in slurry form with 50% solids)
30 %	60 ml	5 ml ethyl acetate + 5 ml formamide.

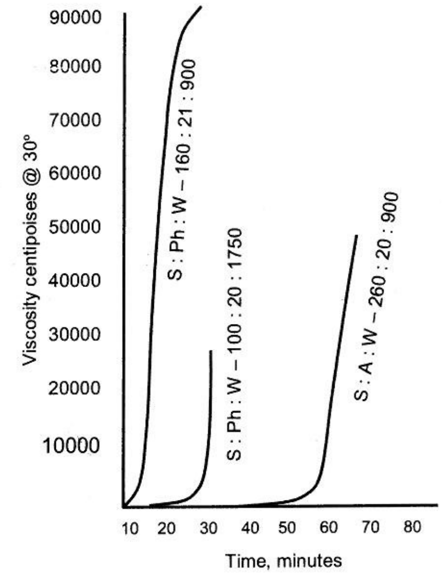


Fig. 17 t-η Relation SS Grout

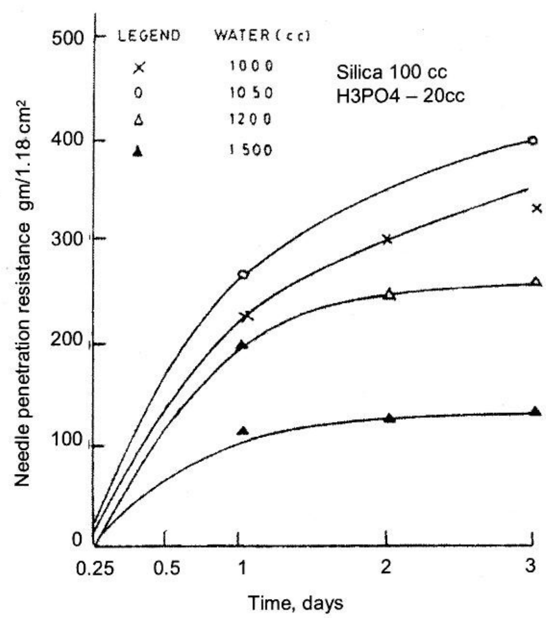


Fig. 18 Time-Strength Relation-Silica with H_3PO_4

A soft to medium stiff gel can be formed by neutralizing with weak acidic salts, sulphate salts etc, by forming small chains of dense spherical aggregation of polymerized silicic acid, while formamide, ethyl acetate, ethyl alcohol & acetic acid form a hard gel that neutralizes sodium silicate by the saponification process. Silica gel is resultant of the reaction obtained by the addition of bivalent or trivalent cat ions, causing polycondensation that leads to a three-dimensional network structure of highly coiled & interlinked.

Sodium silicate is generally considered non-toxic and non-corrosive, and thus free from health hazards and environmental effects (Caron, 1963). Bicarbonate and phosphate gels have an almost insoluble framework that prevents the passage of water, while in other gels the framework diminishes its importance more or less rapidly. At time of equal gelations, it seems that sodium silicate has a linear susceptibility that is independent of the type of reactant used. Lower the gel time higher the gel strength. The gel strength of silica is almost independent of the nature of the reagent added to promote gelation, but the stability of the gel with the passage of time depends on the gelling agent. The short-term strength of pure silica gel is inversely proportional to setting time, since setting time is related to catalyst concentration, strength increases with increasing sodium silicate. Factors affecting the syneresis of hardened gels are density of the grout mix and environmental changes.

Shroff and Moghe (1980) studied the time viscosity and time-strength relationship of silica gel developed by using phosphoric acid and sodium aluminate (Figs. 17 and 18). These grouts are used in curtain grouting at Girna dam, Maharashtra, India.

It is found that 10.29 meq of HCl produces gel strength of 0.35kg/cm^2 with 15meq sodium silicate. The gel time is influenced considerably by the amount of sodium aluminate and to a lesser extent by temperature. Formamide helps increasing rigidity compared to other precipitants.

Lower increase of viscosity of silicate formamide during limiting injection and zero displacement compared to aluminate and phosphoric acid but higher viscosity at gel time. Limiting injection, zero displacement and gel time of aluminate are longer than phosphoric acid but shorter than formamide. It seems that formamide remains watery for longer time for any stress range compared to other precipitants (Figure 19a). Grout with formamide precipitant produces highest stress build up at gel time and also higher order transformation from semi viscous-elastic compared to other precipitants. Shroff and Shah (1988) produced a hard silica gel employing formamide and presented its interaction mechanism with sand and used during box jacking operation for underpass below railway track alignment to prevent raveling of sand towards portal at railway yard, Baroda, India.

Typical stress-strain curves of SF-grouted samples indicate that increase in peak deviator stress with increase of time interval at any confining pressure reverting time-strength relation of silicate grouted sand as strongly time dependent linear elastic and time

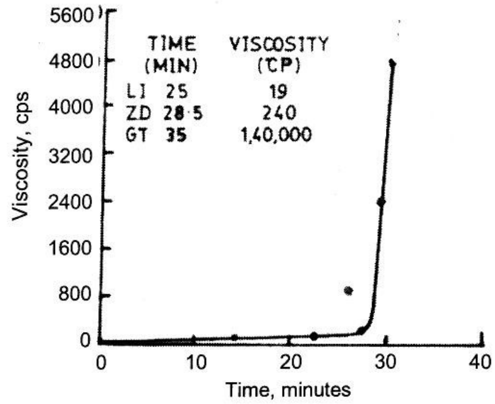


Fig. 19(a) t-η Relation-Silica Formamide Grout

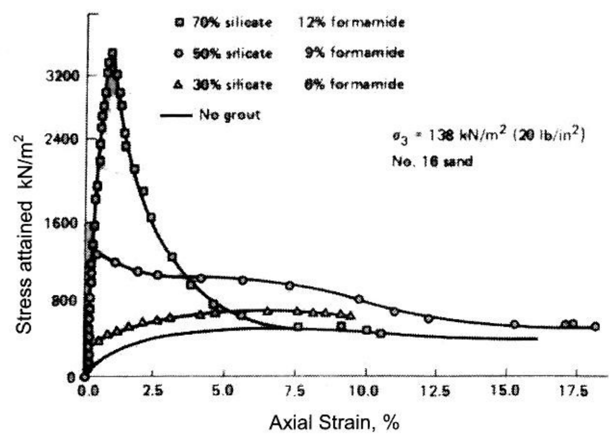


Fig. 20(a) Stress Difference (kN/m^2) vs. Strain (%) Relation-Silica-Formamide Grouted Sand

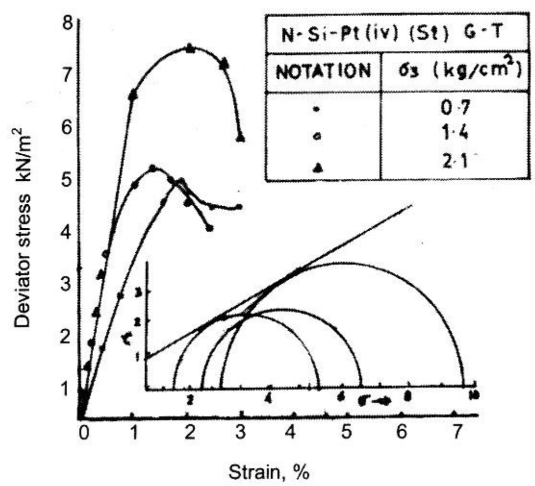


Fig. 19(b) σ_d-ε Relation Silica Formamide Grout

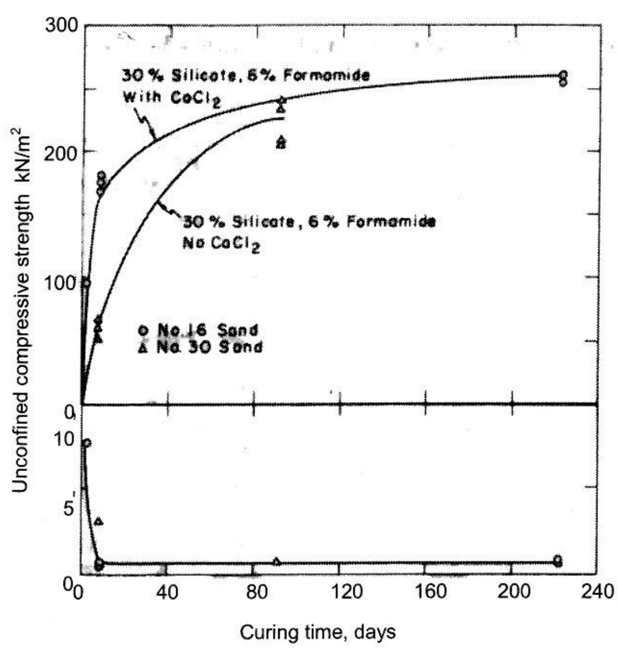


Fig. 20(b) q_c-t Relation of Silica Formamide Grout

dependent plastic leading to rupture of solid material. Over the range of pressure studied, the strength of each grouted soil giving c = 1.4Kg/cm^2, characterized by linear Mohr envelop with no significant change in frictional angle (Figure 19b.) Under Ground heavy water flow, flash-setting grout is developed employing sodium silicate and a hardener containing a combination of bisulphate, sulphate and bicarbonate.

Silica gel by formamide in dune sand gives unconfined compressive strength q_c = 38 Kg/cm^2 for moist cured sample. Clough *et al.* (1979) mentioned that as silicate content and confining pressure increases peak strength & stiffness increase as well as failure becomes more brittle and strains at failure get smaller (Figure 20a). However, rate of increase is strongly dependent on the amount of formamide. For long term loading conditions, the grouting component of strength is subject to creep rupture at load levels that are 50% of those defined as failure in rapidly loaded UCS test (Figure 20) Effect of curing time on unconfined strength is shown in Figure 20(b).

Mixture of ethyl acetate and formamide in silica grout reduces setting time. The adhesive shear and tensile strength of 50 kPa and 40 kPa are measured after seven days and 28 days respectively by special apparatus fabricated for rock grout interface testing. Stress-strain curve of compressive strength of grouted sand (35kPa at 7days) is linearly elastic during initial stage transforms to elasto-plastic leading to failure (Shroff, Joshi, 2004). Set- grouted sand mass imparts cohesion of 96 kPa with no change in angel of internal friction and reduces permeability 145 times giving adherent washout strength (AWS) of 246 kPa (82 hydraulic gradient) against washout forces. (AWS=14.67,q_c=0.6) (Joshi, Shroff, 2004).

Hoshiya *et al.* (1982) developed a silica gel using non-alkaline sodium silicate, which is capable of solidifying in several seconds to several hours (Figure 21). This gel possesses a high strength because no water glass was left unreacted and all the silicon dioxide was embraced in gelation to control the injected area, therefore, the combination of both slow-set grout and flash-set grout is the most effective.

The gelling of the silicate solution imparts an artificial strength and stiffness to the soil that provides it

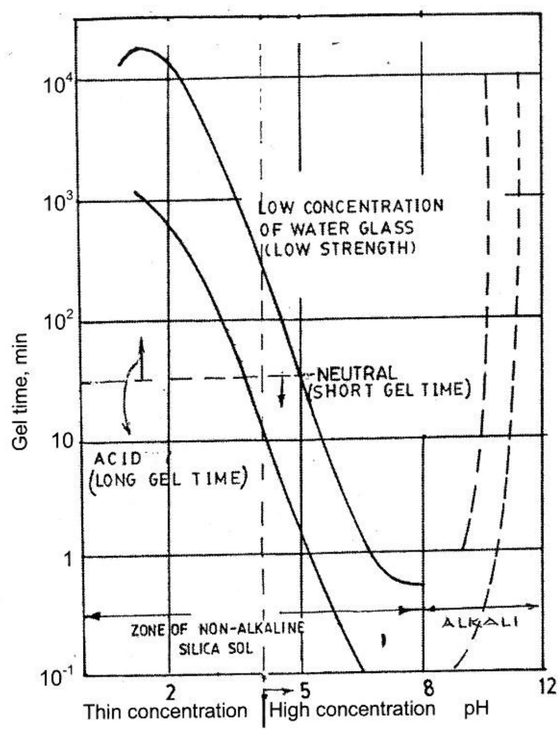


Fig. 21 pH-Gel Time Colloidal Silica Gel

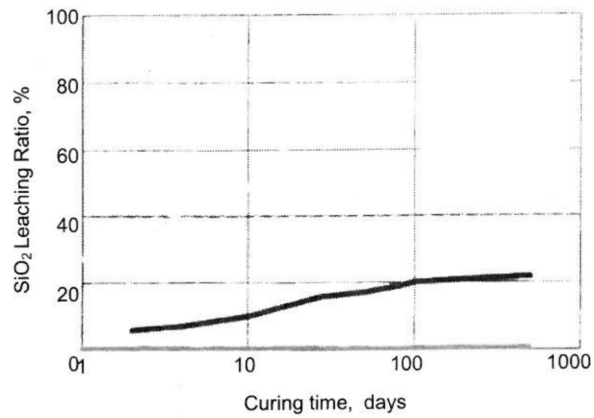


Fig. 22(a) SiO2 with Organic Hardener Leaching vs. Days

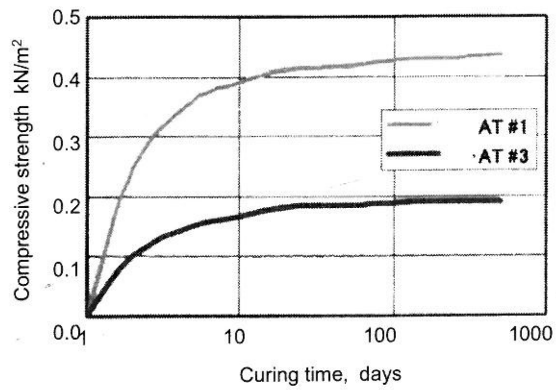


Fig. 22(b) q_c vs. Time of Colloidal Silica

with additional resistance to deformation under loading such that produced by shallow tunneling in an urban area.

Colloidal Silica

It is produced by extracting alkali using ion-exchange resin. SiO2 leaching ratio and syneresis or volume change is highest in inorganic reactants than organic reactants and nil to minimum in case of colloidal silica (Figure 22) Average diameter of the particle is 10-20 nm. Time viscosity and strength study along with microscopic gelation is shown in Figure 23 (Shroff, Joshi 2004). Jellification occurs at pH vale above 9.5 at optimum concentration 20 to 30 g/l potassium chloride within 15 to 25 minutes, while with 35 g/l potassium chloride flash set is at w : c = 2. Increase content of KCl has prominent effect on yield value and viscosity. Ten days vane shear strength of inorganic & UCS of organic reactants give 67 kPa & 162 kPa while colloidal silica gives 165–180 kPa for w: cs = 2. For grout w : cs = 1, after 45 days the UCS = 340 kPa with KCl = 30 g/l, while peak deviator stress after 45 days reaches to 700 kPa with w : cs = 0.5 at confining stress 196.2 kPa and Kcl = 30 g/l. Cohesion ranged from 59 to 150 kPa with no much variation in ϕ (Figure24b).

UCS of grouted sand exhibits increase of 60 to 65 per cent with decrease of w: c ratio: 2 to 0.5, Kcl concentration and curing time. Peak deviator stress of 711 kPa at 45 days with confining stress 196.2 kPa is observed in grouted sand compared to 415 kPa of organic reactant. Grouted and raw set-grout samples follow Mohr-Column failure criteria. Cohesion range from 60 kPa to 130 kPa with little change in φ for grouted sand sample. Adherent wash out strength 430 kPa of colloidal silica (Threshold hydraulic gradient: 198) which is 1.75 time with organic reactant while 1.8 times with inorganic reactant.

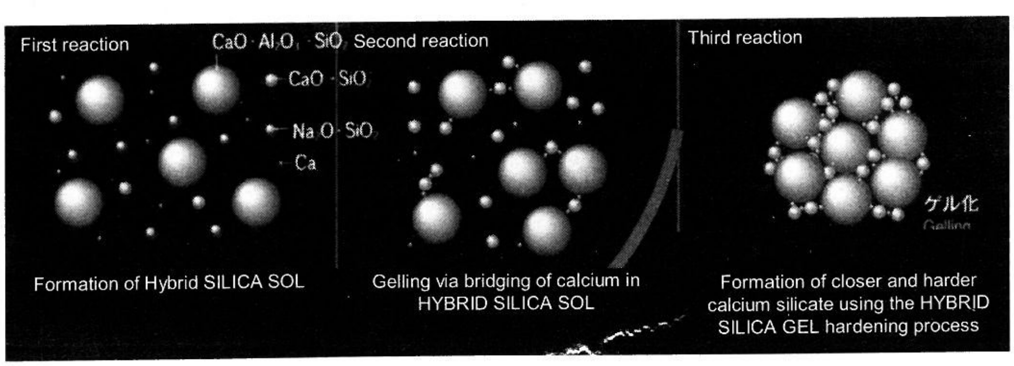


Fig. 23 Time-Viscosity Relation of Colloidal Silica Grout along with Progress in Gel Development

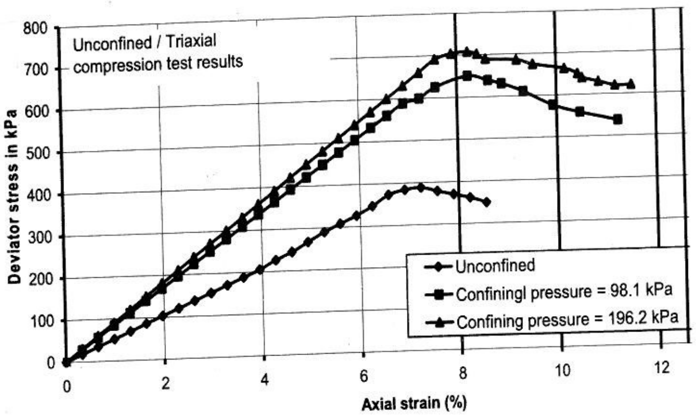


Fig. 24 (a) Stress-Strain Characteristics of Raw Colloidal Silica Grout

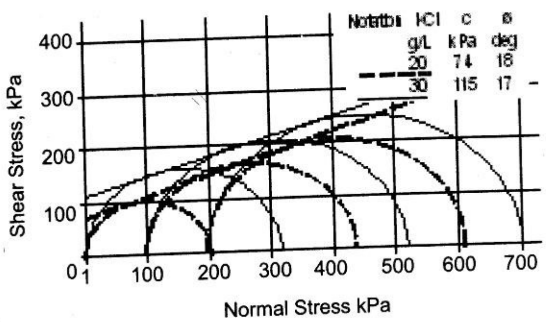


Fig. 24 (b) Mohr's Stress Circle of Raw Colloidal Silica Grouts (W:CS=0.5, 45 days curing)

The gel net work of colloidal silica grout is formed by polymerization and condensation of silanol radicals on the surface of colloids. During initial time after mixing hybrid silica sole is formed while nearer to LI time gelling by bridging K + occurs in hybrid silica. During final jellification, formation of closer and harder potassium silicate with hybrid silica gel forms. Pile of spherical colloids will be connected close and harder potassium silicate in hybrid silica gel increasing strength of grouted mass (Figures 23, 24).

Long Term Behaviour (Durability)

The acid silica sol grout had lower initial strength which increased with time until about 200 days. The strength then decreased slightly with time but nevertheless maintained about 75% of peak strength after 1200 days. In colloidal silica grout, strength constantly increased with time and after 1000 days it is four times the initial strength proving to be strongest amongst all solution grouts (Figure 25 a). The structures of silica grouts are illustrated in Figure 25(b).

Long term permeability test on stabilized sand under the hydraulic gradient of 50 ($5N/cm^2$) confirmed that the grout remain stable and impermeable state upto 3000 days in organic reactants. In case of colloidal silica they remained impermeable state for 10 years or more.

Lignosulphonate

It is known that soluble lignousulphite mixed with dichromate becomes a firm gelatinous mass. The increase or decrease in setting time is controlled by a higher of lower pH value, adjusted by acid or alkaline salts (Moghe, Shroff, 1980). A lignin gel is produced by reacting it with a catalyst and accelerators. Figure 26 portrays the time viscosity relationship of grout mix 1 (lignin + dicromates): 4 to 5 parts water. Caron (1963) reported curves of equal strength and time- viscosity relationship for this grout.

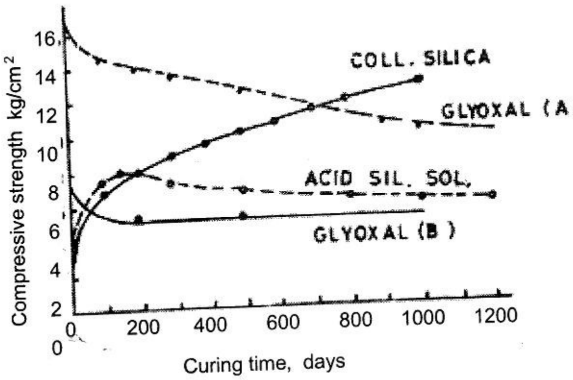


Fig. 25 (a) Long Term Behaviour of Stabilized Sand with Silica Grout

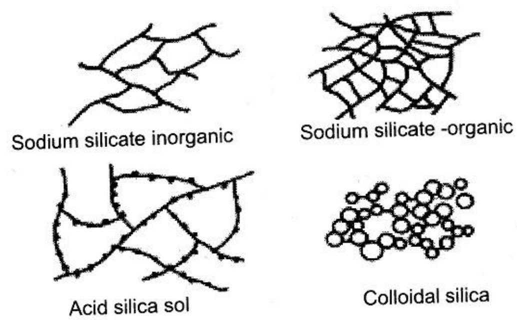


Fig. 25 (b) Structure of Stabilized Sand with Silica Grouts

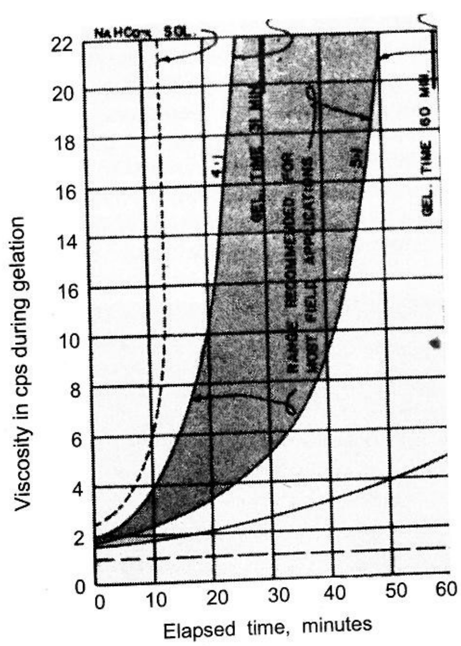


Fig. 26 t-η Relation Lignin Dichromate Grout

Urea Formaldehyde-UF (Aminoplast)

When it interact in aqueous media in the presence of acid, condensation polymerization proceeds by a steady chemical combination of small molecules into macromolecules, developing a chain network; as the reaction proceeds the viscosity of the solution steadily rises and ultimately a set mass is formed (Figure 27a). MERI (1979) produced a hard gel by condensing urea and formaldehyde in the presence of dilute acid along with acryl amide powder and a catalyst to activate the desired hetero polymerization that would evolve a gel having a compressive strength of 55.37 kg/cm^2 and a tensile strength of 5.5 kg/cm^2. Shroff, Shah and Bendale (1985) evolved an insoluble stiff gel with minimum syneresis by reacting urea- formaldehyde with a catalyst, admixtures and fillers. Shroff and Shah (1988) produced a UF gel by employing oxalic acid and proposed a new concept that of the 'chelate complex', wherein the metal ion of alluvium, or rock links up with the UF gel.

Raw gel is insoluble in water and grouted mass remains intact under water submergence. It develops high resistance to washout forces making mass less permeable to water. Grouted mass shows increase in strength with increase in confining pressure (Figure 27b). The injection of grout into sand is observed to lead to a material that adds cohesion component in addition to original friction component of the sand.

Rate of reaction and gel strength are directly proportional to concentration of oxalic acid and urea-formaldehyde respectively. Mathematical model as suggested (η = exp {0.015 (s)$^{0.4}$/D}) accounts all the rheological changes of UF grout system, which follows reasonably time viscosity curve. New interactive chemical process of double order condensation in presence of melamine ion of alluvium to be grouted and non-metallic ion of oxalic acid help increase the strength of grouted mass.

Table 4 summarises the new UF mix designs developed by Shrof and Shah (1995)

Resorcinol Formaldehyde-RF (Phenoplast)

Phenol and resorcinol with formaldehyde polymerized with acid (3 ml hydrochloric acid) or alkali (3 gm Sodium hydroxide) catalyst; give an instantaneous non-toxic gel at 33° C with pH value 1.7. Bell (1975) studied the influence of gel time of catalytic concentrations of hydro gel of polyphenolic polymer with formaldehyde in the presence of ferrous sulphate or sodium dichromate at several temperatures.

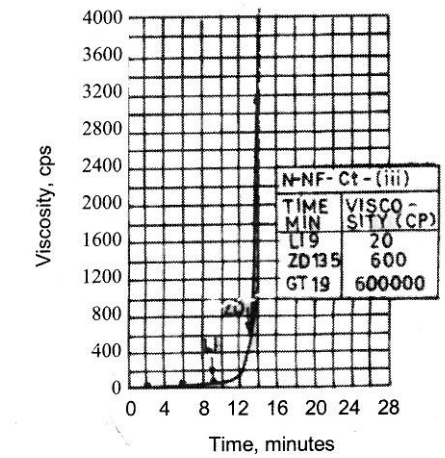


Fig. 27(a) t - η Relations UF Grout

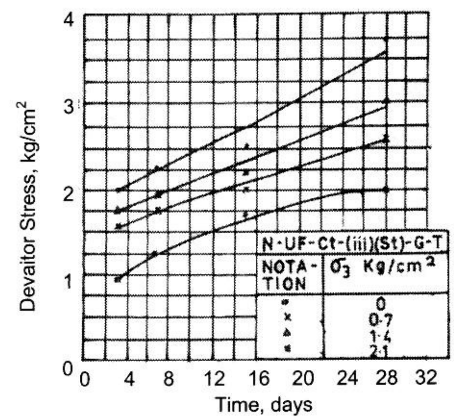


Fig. 27 (b) Time–Strength Relation UF Grout

Watery initial viscosity of 2 cp facilitates injection of grout in fine cracks and voids of sizes 0.1 mm or less, where cement or other chemicals are not workable. Time-viscosity relationship obtained from D–S-T (Shear rate-shear stress-time) plot reveal abrupt transition from sol to gel unlike silicates and aminoplast grouts. The grout has controlled jellification time under moist condition and practically non-toxic.

Base catalyst gives 2.5 times more strength of raw gel & grouted sand compared to acid based catalyst. Mass has adherent strength of 2.10 kg/cm^2 against wash out forces. Fish life study of washed out water from grouted mass has given the percentage mortality and BOD value within the range of effluent water. Grouted sand exhibited qc = 6.5 kg/cm^2 at 7 days and deviator

Table 4 New UF Mixes Designed & Developed by Shroff and Shah (1995)

Urea formaldehyde	*Catalyst*	*Water*	*Row gel UCS 3 Days*	*Grouted sand UCS 3 Days*	*Grouted sand UCS 28 Days*	*Adherent strength*
30 cc	1.5 cc oxalic acid	100 cc + 0.5 gm Urea + 1.5 gm Malamine + 0.5 gm Copper Sulphate	qc=2.9 kg/cm^2	1.1 kg/cm^2	3.4 kg/cm^2	1.95 kg/cm^2
55 cc	5 cc H_3PO_4	200 + 4.0 gm Sodium Aluminate	1kg/cm^2	–	–	0.98 kg/cm^2
50 cc	10 cc H_3PO_4	200	0.65 kg/cm^2	–	–	0.55 kg/cm^2
50 cc	5 cc HCL	200 + 3 gm Sodium Aluminate	1.75 kg/cm^2	–	–	1.65 kg/cm^2

stress, σd value = 10.25 kg/cm^2 at confining pressure of 21 kg/cm^2. Strength of each grouted fine sand characterized by linear Mohr envelope showing cohesion = 2 kg/cm^2 with least variation in φ value (Figure 28). The reaction of resorcinol with formaldehyde under alkaline conditions produces hydro gels which fill the pore space of granular soils. The resultant product may be regarded as a matrix of set grout densely filled with sand. Acidic RF grouts were used successfully for grouting the Woolwich and Reading beds at Blackwell Tunnel in UK. The alkaline RF gouts used in grouting deep rock for mine shaft sinking at North Yorkshire.

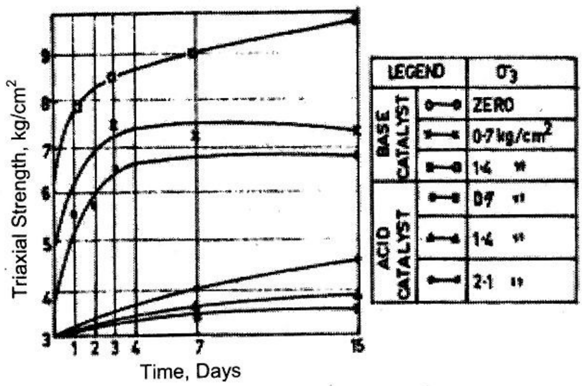


Fig. 28 (b) σ_d-t Relation RF Grouted Sand

Polyurethane

Hydroxyl group (pu1) is reacted with an isocynate to from urethane gel. Isocynate componate is qasi-prepolymerised by mixing hydroxyl compound (OH) with excess amount of isocynate (NCO) in the ratio NCO/OH = 4:1 to reduce toxicity and efficient reaction (Shroff, 1992). Increase in hydroxyl compound-pu$_1$ with constant volume of 10cc and 20 cc of prepolymer-pu$_2$ increased the strength. Blowing agent, methyl ethyl ketone and adipic acid improves the strength property of grout system.

With initial watery viscosity, polyurethane grout is injectable in fine cracks, joints and voids of sizes 0.1 mm or less (Figure 29a). Unlike the other chemical grouts, this grout possess the characteristics of expanding into the voids of a soil mass or crevices of the rock mass after jellification and producing strong grouted mass. Polyurethane grout has controlled gel time (30minutes) with high gel strength. It is observed that though 20 % of water in 1:1 ratio gives maximum strength of 1.53 kg/cm^2 and gel time within limit, but 70% of water can be considered as economical mix giving optimum strength of 1 kg/cm^2 and gel time within specified limit of 30 mnt. Raw gel and grouted mass are insoluble in water.

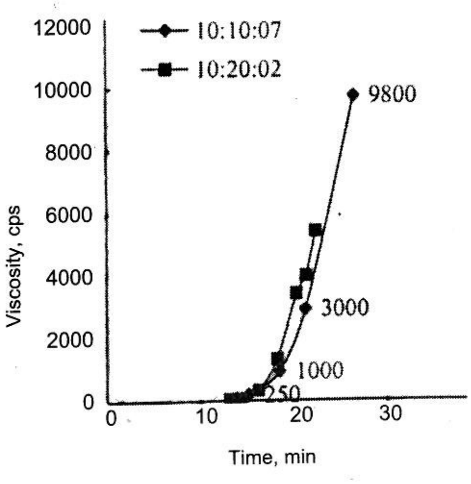


Fig. 29 (a) t-η Relation of PU Grouts

An elastomer coupling of gellified foam in void spaces of grouted mass develops consistent compressive & tensile strength and resistance against any wash out forces. It is observed that grouted sand give nearly same compressive and tensile strength. Dry cured submerged sample shows comparable higher compressive and tensile strength than moist and saturated submerged cure sample. The respective compressive strength values are respectively: 3.15kg/cm^2, 2.037kg/cm^2 and 1.18kg/cm^2. Also the study of dry cured submerged sample by extension test with confining pressure of 0.07kg/cm^2 exhibited ultimate tensile strength of 3.71kg/cm^2. (Figure 29b)

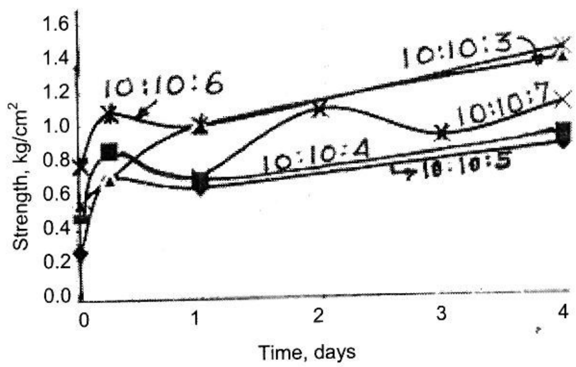


Fig. 29 (b) q_c-Time Relation of PU Grouted Sand Grout Proportion PU1:PU2:Water

Apart from the use of polyurethane as soil penetration grouting, many other possibilities exist to use polyurethane in reinforced earth construction, reduction of machine foundation vibrations, Geotextile construction, pipe jacking operation and other geotechnical engineering applications.

Epoxy

Epoxy resins have often been used for various problems in civil engineering works in India, during the last decades. The sealing of cracks (Koyna, 1968; Konar, 1971; Hirakud, 1976) in concrete structures, plugging if leaks in hydraulic structures and bonding of fresh concrete to hardened concrete are the main applications.

The epoxy gel is developed by interacting chemical compound derived from bis-phenol-A with epichlorohydrin and catalyst or hardener consisting of amines and polyamides. As per the requirement, the resultant jellified mass, either rigid, elastic or elastoplastic can be cultivated by proportioning resin, hardener and water component. The time-viscosity relationship shows that the initial viscosity drops to 50 per cent up to 12cp, thereafter increases to semi viscous limiting stage transferring suddenly to gel stage at about 35 minutes (Figure 30).

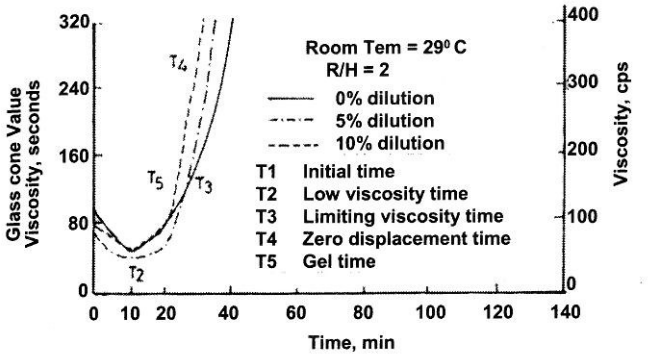


Fig. 30 (a) t-η Relation of Epoxy Grouts

The higher initial viscosity is attributed to the formation of primary micro gels of 0.5 micrometer to 1 micrometer, which grow into secondary micro gels interacting with each other before the onset of physical gelation immediately after mixing. At a critical concentration the secondary micro gels (10 micrometers) pack together abruptly to form and gel mass having a high cross-linked density that explains the typical time-viscosity curve.

A mix having resin: hardeners ratio 2, gives compressive strength of 662.8 Kg/cm^2. 775.36 Kg/cm^2 and 800.12 Kg/cm^2 at 1 hour, 14 days and 21 days respectively (Figure 30b). Stress-strain curve indicates that peak stress increases with increases of curing time reverting strength-time relationship of epoxy set mass as strongly time dependent. Two categories basically identified, mainly time dependent linear elastic and time dependent non-linear elastoplastic leading to brittle failure. High pressure Tri-axial test gives cohesion value of 450 Kg/cm^2.

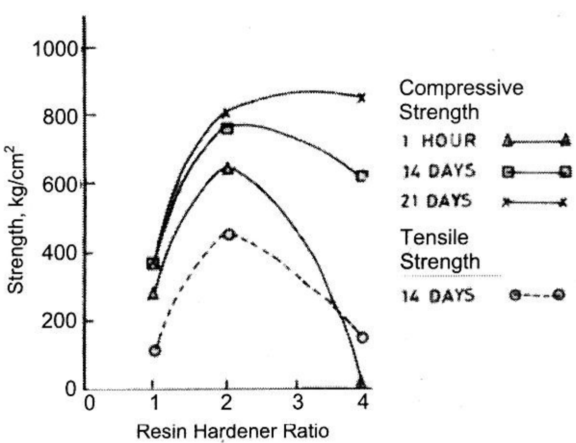


Fig. 30 (b) Compressive and Tensile Strength for Epoxy Grout

Considering the above time-viscosity and strength behavior, the grout is selected for injection purpose consists of optimum ratio of resin: hardener. Viscosity of 50 cp is desirable for cracks of the order of 0.15 mm. It is observed from the metamorphic and igneous group of rock specimens vein lets, bends & minor fractures that adhesion over discontinuation and cracks by epoxy grout always exceed the strength of the rock. Rupture always occurs through the rock and never through bonded crack.

Rock Joints of Basalt and sand stone specimens having single and multiple joint configurations are grouted. The test results on wet surfaces also showed that significant strength could be developed across joints by bonding them with these epoxy resin grout.

A suitable injection gun is developed for grouting epoxy in rock sample in metal box 10 cm × 10 cm × 10 cm (Shroff, Amin, 1990).

It infers from universal compression test on raw set mass that peak compressive strength of 1500 kg/cm^2 and residual strength of 1450 kg/cm^2 with initial tangent modulus of 50000 kg/cm^2. Tensile strength measurement gives 120 kg/cm^2, elongation 10.2% and water absorption as 0.21%.

Special Laboratory Tests for Epoxy Grout

Mix Viscosity and Pot Life: The flow cup test is relatively simple in which viscosity is determined from the time of efflux of fluid at room temperature.

Penetration Test: It involves estimation of pressure and the time required for an epoxy mix of known viscosity to travel through the gap of known width and length.

Bond Strength Under Tension: Once the sealing system is set, the grout system is injected through the inlet nipple using a small syringe. The specimens are then left undisturbed for curing for 7 days. Tensile testing machine is used to evaluate bond strength of epoxy grouted with concrete or rock. For underwater applications the gap between the prisms is grouted underwater so that the water in the gap is replaced by grout system.

Flexure Test: These cracks are then grouted by epoxy system under study. The grouted rock cracks are then tested under flexure mode.

Pressure Bearing Capacity of the Sealing System: After a curing period of 7 days, water under pressure is injected in the gap which is grouted through the hole provided in one of the cubes. Water pressure is then gradually increased till failure of the sealing.

A trial field injection was made with the above resin grout mix (Shroff, Amin, 1990). It is found that, it provides better bonds to moist rock and has more flexibility than other chemical grouts to accommodate movement before bond or shear failure occurred and has shown lower volumetric shrinkage during curing. The stratascopy inspection at the site has indicated that larger cracks and smaller cracks are appeared to be filled up and interacted properly with epoxy grout.

Acrylamide

The acrylamide grout is developed by polymerizing the monomers acrylamide and methylene-bis-acrylamide in about 90:10 proportion with the requisite amount of the catalyst, ammonium persulphate (0.5%) and the accelerator, triethanolamine (25% in 75% water diliution) and inhibitor potassium ferricyanide (KFe = 0.05%) (Shroff, Shah, 1995). Although the grout exhibits good penetrability, with a constant low viscosity during the induction period and better gel control with adequate strength, it can be dangerous and toxic if certain basic safety and handling precautions are not observed.

A new low toxic Acrylate based grout is developed (Shroff,1999) having long term stability, employing the non-toxic catalyst calcium hypochlorite, exhibiting higher initial viscosity, requiring a one-hour gel time and possessing a lower strength than the above grouts.

Crack Grouting

Shallow cracks (extending up to about 60cm depth) can be grouted by grouting through surface nipples/surface entry ports (SEP) and only in the remaining depth of crack the grouting through inclined drilled hole (IDH) method is employed. Figure 31 shows two types of nipples and their fixing details. After sealing the cracks, the surface is sealed by cutting 'v' notch. This method was successfully used fro u/s top corner of drainage gallery of Koyna dam and Hirakund dam.

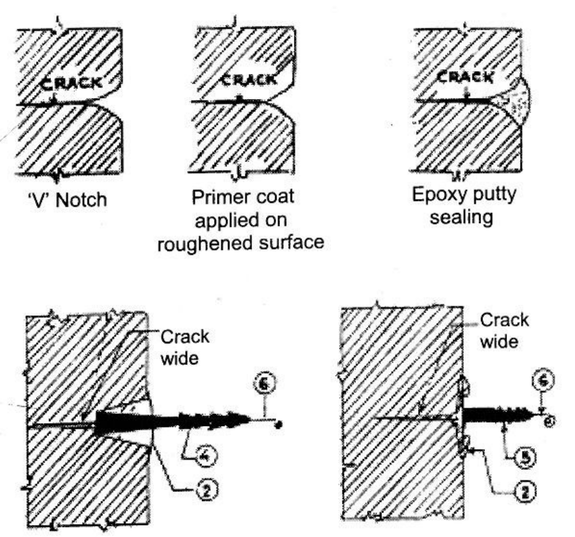


Fig. 31 Nipples and 'v' Notch Details

Micro Fine Cement and Slag Based Grouts

These grouts can be used for the following applications: Dam curtain grouting, Foundation consolidation, nuclear waste secondary containment, Hazardous waste, plume stabilization, preventing heave and boiling condition in cofferdams, Water sealing during tunnel construction, pipe insertion work, dam bottom ducts, reservoir bottoms, preventing seepage in river embankments in fine sands to coarse silt where cement grouts are impenetrable.

These grouts are prepared indigenously by grinding OPC and for blast furnace Slag in ball mill to a required specific surface area of particle (5000 to 8000 cm^2/gm). The special measurements are particle size analysis by blain air permeability, Laser particle size analyzer and true permeation under gravity.

Micro Fine Cement Dust Grouts (MCD)

Superfine cement is used at various Dam sites of Gujarat particularly in curtain grouting at Sardar sarovar project. MCD grouts are developed having fines: 8115 cm^2/gm with particle 50% grain size 13 μm. Gel time with 1% dispersant (Naphthalene sulphonate) varies 60 to 120 min. at 0.6 to 4w:c ratio can be further reduced to 30 to 100minutes with 1% sodium silicate along with decrease in bleeding potential. It is observed that MCD grouts possess consistently lower initial viscosity compared OPC grouts (Figure 32a).

Figure 32b shows UCS & initial E value of MCD gel which is higher at all w:c ratio when compared to OPC. The cohesion imparted to grouted sand with OPC, & MCD are 70, 125 kPa respectively. It is resisting threshold hydraulic gradient of 260 against wash out forces subjected to grouted sand. The unconfined compressive strength of MCD grout is about 1.33 times the unconfined compressive strength of OPC.

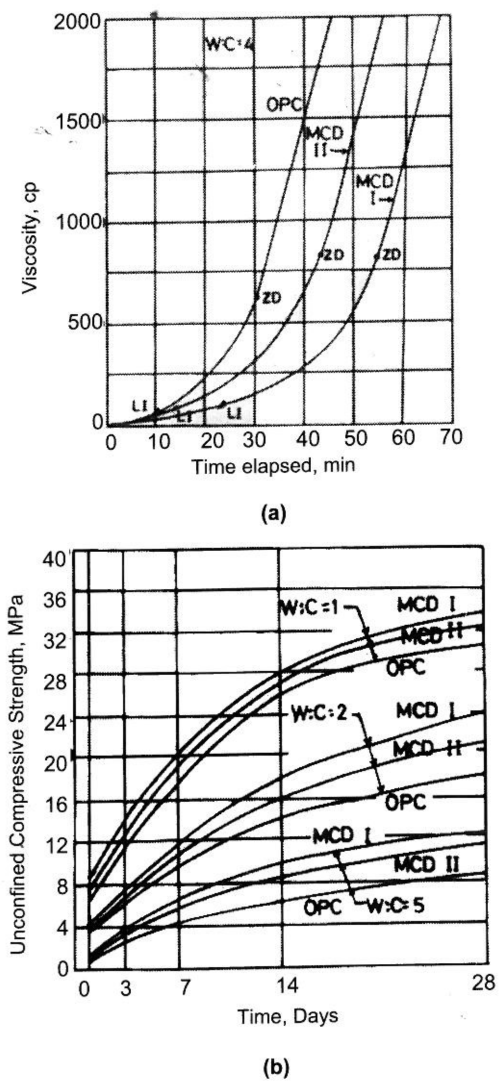


Fig. 32 (a) t-η Relation MCD, OPC Grout, (b) q_c-Time Relation OPC, MCD Grout

Slag Based Micro Fine Cement Grouts

Group-1 Grouts (MC-I, MC-II & MC-III) are developed by adding 25%, 50% & 75% Slag to 75% 50% & 25% OPC respectively in various proportions of solid to water in presence of 1% Naphthalene sulphonate as a dispersants. *Group-2* is only Slag MC grouts along with chemical activators 0.1% to 2% in various proportions of water to solid ratio in presence of 1% Sulphonated melamine formaldehyde dispersant are designed. Chemical activators employed are carbonate & floride of sodium & potassium, and Sodium sulphide (NAF). *Group-3* grout consisting of 25% OPC & 75% slag (MC-III) is activated by above chemicals in the

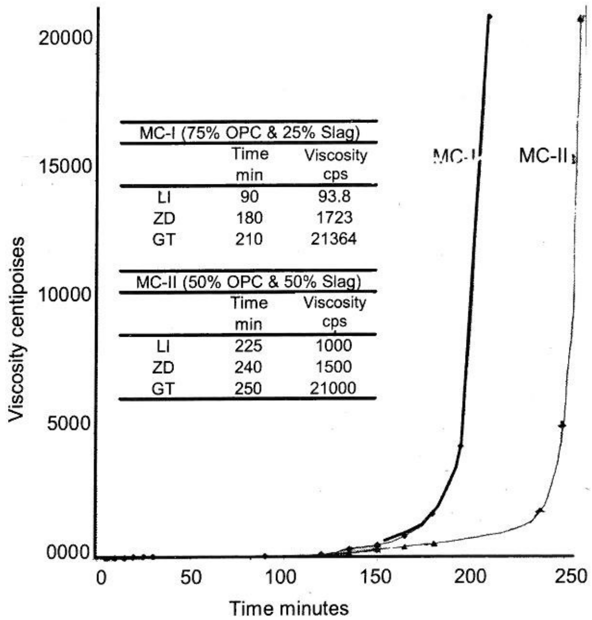


Fig. 33 t-η Relation MC-I, MC-II

concentration of 0.02% to 0.10% to improve its performance in presence of above dispersant. In Group-4 MC-III grouts, Sodium silicate are mixed as component 1:1 proportion- MC:SS & also MC_III is mixed with 1, 2 & 3% of bentonite (MC-bentonite) and Silica fume 4, 12 & 20% (MC-SF) as admixtures to improve its grouting performance.

Group-1

Time-viscosity and Time-Strength: MC-II remain watery- Newtonian longer time (100 min) compared to 60 min in case of MC-I at same w:s ratio, & grout MC- III solidify slowly at 200 min compared 150 min in MC-I. MC-I grout (75%OPC + 25% Slag) seems to be ideal with respect to initial & gel time (Figure 33).

The stress-strain curves for all neat grouts indicates elastic-plastic behavior with maximum failure strain of 4.66%.For grouts prepared from 75% OPC +25% slag showed maximum UCS of 6315kPa after 90 days of curing at w:c = 0.8 which decreased to 2587 kPa at w:c = 5. For grouted sand these strength are 3768 kPa at w:c = 0.8 and 96 KPa at w:c = 5. For slag + OPC grouts higher per cent of slag gives better strength of grouted sand for low w:c and higher per cent OPC gives better strength for high w:c ratio (Table 5).

Triaxial compressive strength of slag+OPC grouts are tested with confining pressure of 100 kP and 200 kPa. 50%OPC +50% slag gives highest deviator stress (5860kPa) with cohesion 1050Kpa amongst various

Table 5 Effects of per cent OPC on Ground Granulated (microfine) Blast Furnace Slag Grout (Shroff, Joshi, Ghrisma, 2005), Group-1

Proportion slag+OPC	*Unconfined Compressive Strength, kPa*												*Indirect Tensile Strength, kPa*			*Flexural strength kPa*	*Adherent washout strength, kPa*
	Raw gel						*Grouted sand (RD=30%)*						*Grouted sand*			*Grouted sand*	*Grouted sand*
	0.8W		2W		5W		0.8W		2W		5W		0.8W	2W	5W	2W	5W
	3*	90*	3	90	3	90	3	90	3	90	3	90	90	90	90	90	7
25%+75%	3685	8156	1022	4665	615	2611	516	3661	308	1196	296	986	3275	1594	-		
50%+50%							1050	5036	883	3978	650	2527	2955	1380	-		
75%+25%	1060	15935	626	8124	418	4800	1060	4810	575	2100	108	361	450	260	125		810

water cement ratio (0.8 to 5) for any OPC and slag variation. In MC-I & MC-III of grouted sand it is observed that there is no much variation in ϕ value (40° to 45°).

Figure 34 shows indirect tensile strength vs. time relationship. MC-I grout gives higher indirect tensile strength (3275kPa) among other variation of slag with OPC and w:c ratio (Table 5). It is observed that maximum deviator stress of triaxial compression strength is about 1.6–1.9 times indirect tensile strength.

Group-2

Time-viscosity of MCIII grout with activator-NAF (0.1%) tends towards ideal curve remaining watery up to 60 min & thereafter rate of viscosity increases immediately after limiting injection 140 min leading to solid mass (Figure 35).

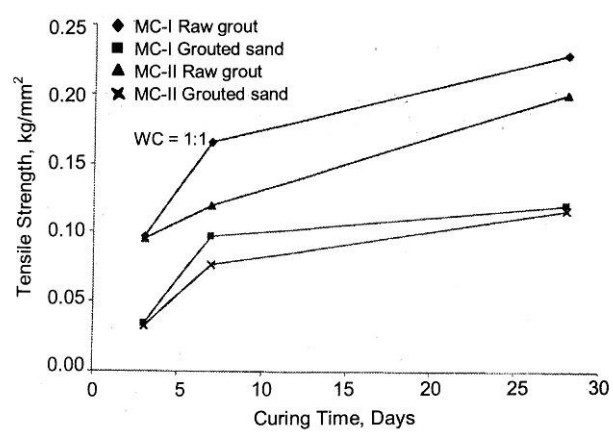


Fig. 34 Indirect Tensile Strength-Time Relation of MC-I and MC- II Grouts

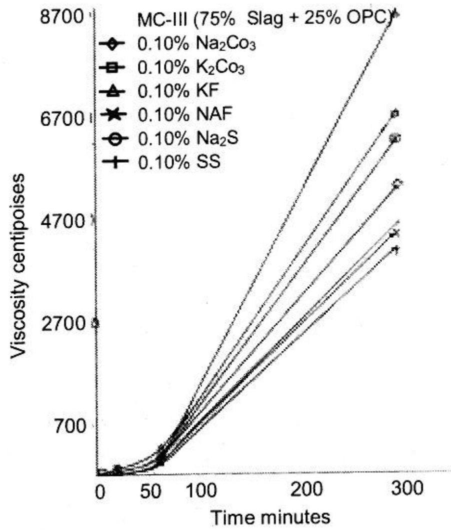


Fig. 35 t-η Relation-MC-III with Activators

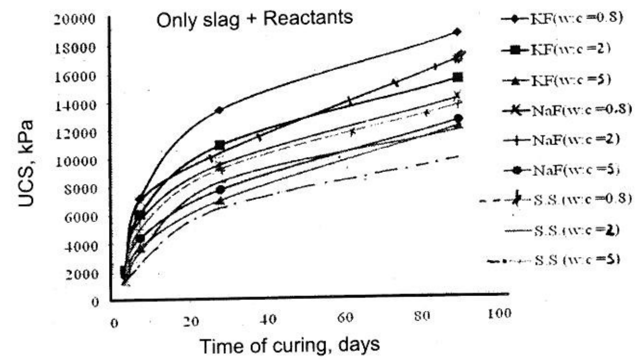


Fig. 36 q_c-Time Relation-Slag with Activator

When only slag grouts are considered with chemical activators, three days strength ranged from 127 kPa to 990 kPa whereas 90 days strength are in the range of 1995 kPa to 8029 kPa. For grouted sand, slag with 0.1% NaF activator has 33% more strength at 90 days with w: c = 0.8 and 240% more strength with w:c = 0.50 compared to slag with OPC grouts (Figure 36). Amongst the alkaline activators with slag, 0.1% NaF is found to be best activator.

Slag with 0.1%NaF gives highest flexural strength (6279Kpa) amongst other reactants while activator o.1% K_2Co_3 gives higher indirect tensile strength of 3343 Kpa which is almost the same value of NaF activator. In slag with alkaline activators UCS is 1.3 times flexural strength (Table 6).

Group-3

The pattern of t-η curves of MC-III with activators remain same irrespective of any w:s ratio giving initial time 50 min and gel time 240 min (Figure 37) In the third group of grouts (MC-III) i.e. 75% slag + 25% OPC which is found to be weak, 0.1% activator is added. 3-days strength of grouted sand ranges from 130kPa to 1020kPa for variation of w:c 5 to 0.8 and respective 90 days strength are in the range of 2017 kPa to 8278 kPa. For the same grouts, strength of neat grout ranges from 5412 kPa to 18379 kPa for variation of w:c = 5 to 0.8 after 90 days of curing (Figure 36). Considering strength of grouted sand after 90 days of curing, MC-III + 0.1%NaF is found to be the best grout amongst all slag based grouts. For all the slag based grouts strength of grouted sand is found as high as at 40–50% of neat grout in majority of cases when grouted in loose sand. (Table 7).

MC-III grout with 0.1% NaF gives highest indirect tensile strength (4369kPa) as well as flexural strength (6827kPa) amongst other variable with w:c = 0.8 at 90 days. In this case UCS is 1.2 times flexural strength with w:c ratio 2 at 90 days. Slag with sodium activator gives higher cohesion value and ϕ value compared to MC-III at any water cement ratio.

Group-4

MC-SS as component grouts and MC + Bentonite & MC+ Silica fume (SF) admixture (Shroff & Joshi, 2005):

Time viscosity relationship: MC grout with silica

Table 6 Effects of Alkaline Reactants on Ground Granulated (Microfne) Blast Furnace Slag Grouts (Shroff, Joshi, Sinroja, Patel, 2006), Group-2

Slag+ React- ants	*Opt %*	*Unconfined Compressive Strength, kPa*												*Indirect Tensile Strength, kPa*			*Flexural strength kPa*	*Adherent washout strength, kPa*
		Raw gel						*Grouted sand (RD=30%)*						*Grouted sand*			*Grouted sand*	*Grouted sand*
		0.8W		2W		5W		0.8W		2W		5W		0.8W	2W	5W	2W	5W
		3*	90*	3	90	3	90	3	90	3	90	3	90	90	90	90	90	7
Na_2CO_3	0.1	1319	14663	1233	12514	1110	11122	373	5943	193	5433	127	4019	2708	2591	2080	4036	700
K_2CO_3	0.1	1356	14707	1203	12614	1097	10991	575	6184	198	5822	148	4943	3343	2657	2335	4916	835
NaF	0.1	2101	16126	1923	13515	1717	12303	591	8029	464	7835	447	5917	3029	3014	2007	6279	790
KF	0.1	2152	18255	2068	15450	1411	11809	522	8016	497	7315	476	5965	2839	2627	2138	5468	817
Na_2S	0.1&2	1947	9629	896	13776	1006	5348	990	4375	573	5912	479	1995	2474	2109	1313	1794	660
NaOH	5	825	4615	185	2449	-	-	290	2915	180	2400	--	-	1184	575	-	1510	-

Table 7 Effects of Reactants on 75% Slag+ 25% OPC Grouts (Shroff, Joshi, Ketki, 2007), Group-3

75% slag+ 25% OPC+ reactants	Opt %	*Unconfined Compressive Strength, kPa*												*Indirect Tensile Strength, kPa*			*Flexural strength kPa*	*Adherent washout strength, kPa*
		Raw gel						*Grouted sand (RD=30%)*						*Grouted sand*			*Grouted sand*	*Grouted sand*
		0.8W		2W		5W		0.8W		2W		5W		0.8W	2W	5W	2W	5W
		3*	90*	3	90	3	90	3	90	3	90	3	90	90	90	90	90	7
Na_2CO_3	0.1	1336	14869	1164	12761	1121	9368	1396	6015	195	5498	131	4059	3520	3302	825	4519	490
K_2CO_3	0.1	1373	14928	1215	2791	1109	11123	581	6258	200	5886	150	4998	2989	2682	797	5020	660
NaF	0.1	2206	16641	2020	13907	1802	12343	621	8278	488	8070	469	6066	4369	4146	1439	6827	513
KF	0.1	2217	18379	2180	15238	1430	11927	538	8138	507	7389	480	6025	4185	3902	1046	6117	547
Na_2S	0.1	2205	9797	1087	7135	1033	5412	1020	4441	596	2516	492	2017	2842	2790	811	1981	509
S.S	0.1%	1236	13435	1069	11639	965	9574	494	5507	176	5298	130	4248	4021	3858	955	4480	722

*days, Na_2CO_3- Sodium carbonate, K_2 Co_3- Potassium carbonate, NaF- Sodium fluoride, KF- Potassium fluoride, Na_2S- Sodium Fluoride

fume (Figure 37) and bentonite (Figure 38) additives reflect pseudo plastic behaviour with deflection of flow curve increases towards shear stress axis with progress of time. Time viscosity curve of MC-SF grout remain towards viscosity axis compared to that of MC-bentonite showing elastic dominancy while bentonite curve shows more zero displacement and gel time.

Time-strength relationship: Stress-strain curve of MC bentonite and MC-SF grout illustrate more elastic-plastic behaviour. Increase of % bentonite and silica fume content shift the stress-strain curve towards strain axis and peak stress and corresponding strain decreases. Also MC-bentonite and MC-SF grout offer

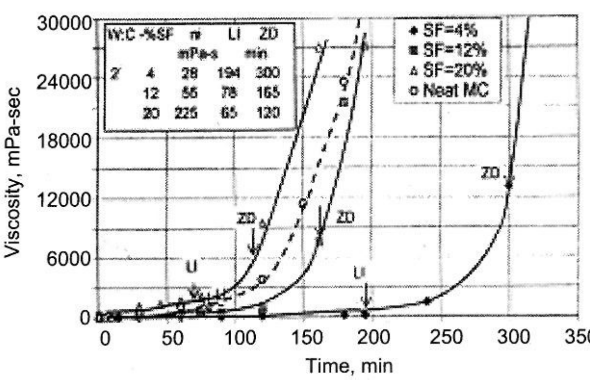


Fig. 37 t-η Relation of MC-SF Grout

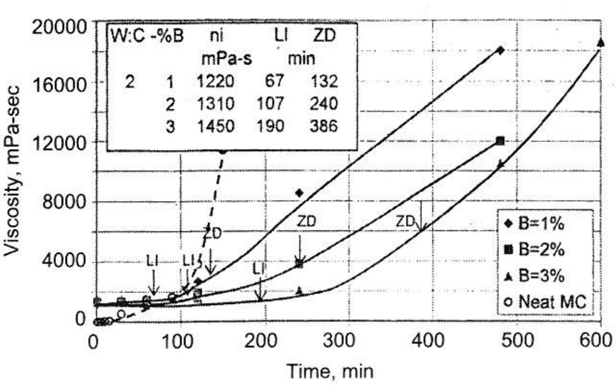


Fig. 38 t-η Relation of MC + Bentonite Grout

residual stress to some extent for further increase of strain. MC with bentonite and silica fume grouts exhibit elastic-plastic failure unlike neat MC grout.

MC-SF failed samples indicate more towards elastic failure tending to elastic-plastic failure while MC-bentonite samples exhibit more towards plastic failure.

Cohesion and ϕ-value decreased with increase of bentonite/silica fume content. Cohesion decreases continuously (19% drop with W:C = 2) with addition of 1 to 3% bentonite while ϕ-value decreased to 17% at W:C = 2 and 59% with W:C = 5. When SF content increased from 4% to 20% cohesion reduces 33% with w:c = 2 and 55% with w:c = 5.

UCS of MC with SF grouted sand shows increase with curing time while MC with Bentonite, though it is increasing with time but remain less than MC-SF and neat MC grout (Table 8).

Bentonite addition hampers the process of hydration of MC grout. The increase in unconfined strength or peak deviator stress up to optimum silica fume is due to pozolonic activity of silica fume.

MC with sodium silicate (MC-SS): In MC-SS, UCS of 60%ss increases from 60kPa @ 1day to 680kPa at 28 days with w:c = 5.

Specific gravity and time of extraction decreases with increase of w:c ratio and decrease of silicate content. Gel time of MC-SS grout is within few seconds which can be lengthen up to 30 min by adding 1 to 3% phosphoric acid (by wt of MC). The grout is most stable with minimum bleeding in very lean mixes. Depth of true permeation increase with increase of permeability of sand, W:C ratio and decrease of SS content.

Time-viscosity relationship (Figure 39a): Viscosity of MC-SS grout increase with increase of time, % SS concentration and decrease of W:C ratio. Time-viscosity curve of MC-SS grout is extreme left of the cement based grouts. The addition of phosphoric acid (PA) shift these curves towards right side showing distinct behaviour conforming to ideal grout as compared to OPC, MCD and MC grouts.

Tablle 8 MCIII+SS as Component & MC with Bentonite & Silica Fumke Admixture (Shroff & Joshi, 2005), Group 4

C:SS$ =1:1 MC + Bentonite & Silica Fume as Admixtures	%	*Unconfined Compressive Strength, kPa*									*Triaxial compression test kPa 28 days σ3=196.2*											
		Raw gel					Grouted sand (RD=30%)				Raw grout									Grouted sand		
		0.8W		2W		5W	0.8W		2W		0.8W			2W			5W			0.8W		
		7*	28	7	28	28	7	28	7	28	σ_d	C	φ	σ_d	C	φ	σ_d	C	φ	σ_d	C	φ
Bentonite	1				3250	1900								8200	600	62	1900	360	51			
Bentonite	2				2400	400								5100	550	55	1175	110	34			
Bentonite	3				2000	380								5000	490	51	660	130	21			
Silica fume	4				4810	4075								8225	750	61	7400	560	61			
Silica fume	12				4250	2750								7250	500	56	6100	490	58			
Silica fume	20				4000	1480								5750	700	61	3500	250	53			
sodiumsilicate	20	2225	2950	2400	2250		2200	3800	1200	1800	5000	820	54	3300	505	43				4500	955	38
sodiumsilicate	40	4100	4900	2200	3150		3400	4800	1800	2000	8100	920	55	4200	491	50				7250	1090	49
sodiumsilicate	60	4400	6500	2300	2850	458	4500	5500	2000	2200	7500	950	55	4500	600	46	825	150	26	6550	1175	48
Sodiumsilicate	80	5500	6780	2500	4150	820	3300	3350	2400	3110	7500	1120	50	4750	1005	40	680	220	25	4400	-	-

SS$ = Sodium Silicate; Adherent wash out strength of MC-III with 20% SS grouted sand at w:s=2 is 660kPa,while at w:s ratio 5 give 171 &231 at 40% and 60% SS

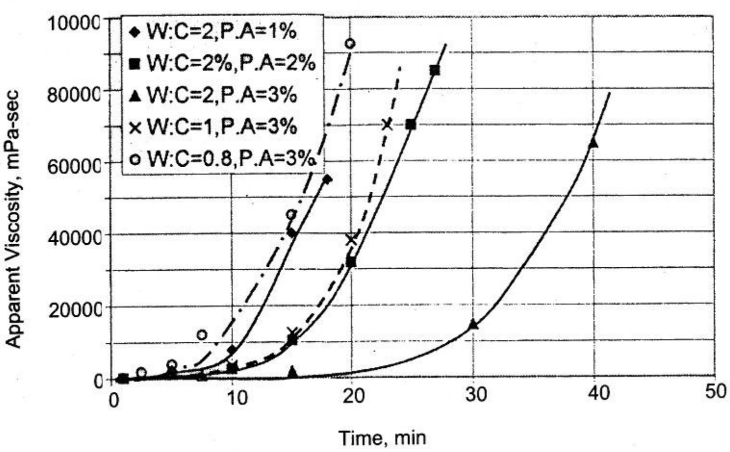


Fig. 39 (a) t-η Relation of MC-SS Grout

Time-strength relationship (Figure 39b): Sudden increase of unconfined compressive strength and E-value up to 7 days, thereafter it remains constant in thin mixes and increases gradually in thick grouts. The unconfined compressive strength of MC-SS grout is 4211 kPa with 0.8W at 80% SS concentration after 3 days of curing (Figure 39b). The above values for neat MC grout are below 1000 kPa which indicates early high strength gain due to addition of sodium silicate in MC grout.

Peak deviator stress at 7 days, with 40% SS increases from 665 kPa to 6529 kPa as W:C ratio decreases from 5 to 0.8. For 2W grout at 7 days, it increases from 3204 kPa to 4186 kPa as % SS increase from 20 to 80% (Table 8). MC-SS material follow Mohr-Coulomb failure criteria.

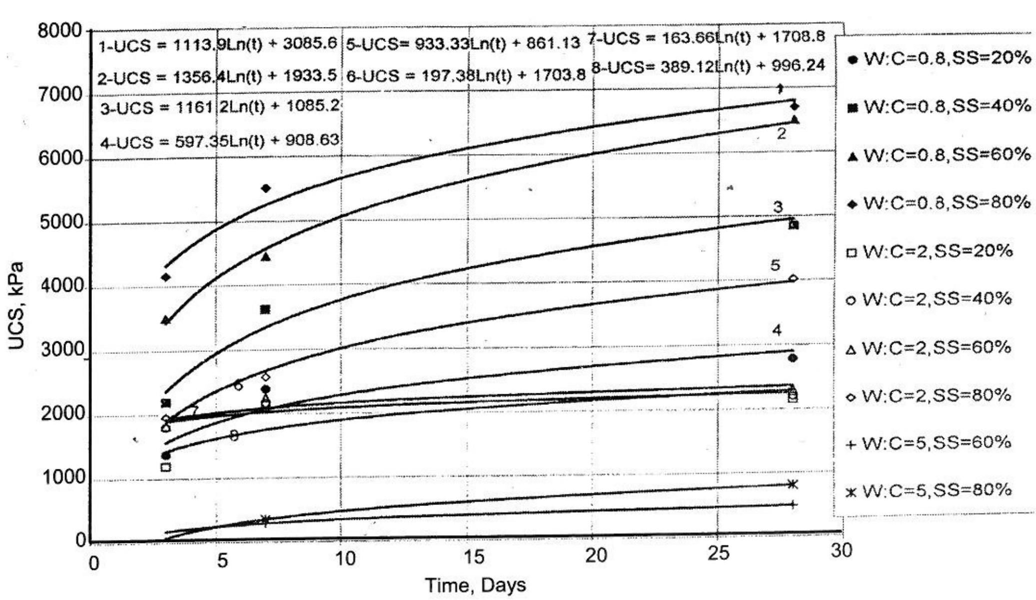


Fig. 39 (b) q_c-Time Relation of MC-SS Grout

Cohesion of raw MC-SS grout increase at faster rate up to 7 days and increases gradually or remain constant afterwards. Per cent increase from 0 to 3 days is about 70% and between 7 to 28 days it is 21%. ϕ value of raw MC-SS ground ranged from 25° to 55°. The increase in ϕ–value between 3 to 7 days is 15-25% and between 7 to 28 days, it is 0 to 15%. ϕ-values are 35° and 55° for 2W and 0.8W grouts.

More content of sodium silicate help shifting stress-strain curves towards stress-axis of MC-SS grouted sand. MC-SS grouted sand exhibit average E-value of 443 MPa when grouted with thick grout while average value of 162 MPa for thinner grouts. Strength of MC-SS grouted sand increase with curing time and % SS concentration (Table 8). MC-SS grouted sand exhibits 1.5 time strength than MC grouted sand.

After 7 days of curing, cohesion of 0.8W grouted sand with % SS = 20, 40 and 60 is 675 kPa, 970 kPa and 1050 kPa respectively. Increase in cohesion from 7 to 28 days is around 7 to 41% while % increase in cohesion from 3 to 7 days is between 24 to 35%. SS component impart cohesive bond bridging between micro cement flocs while MC contributes overall stiffness of MC-SS matrix. ϕ-value of MC-SS grouted sand ranged from 37° to 49°. MC-SS grouted sand resist the threshold hydraulic gradient of 236 (660 kPa) against washout forces with W:C-2 and 20% SS concentration.

Rheology and Strength Aspects under Dynamic Loadings-Solution Grouts

The cyclic behavior, estimation of the fatigue life of grouted sand with 50% sodium silicate, 40% water, 5% formamide and 5% ethyl acetate (by volume) is useful in Earthquake & machine foundations applications.

Damping (attenuation) characteristics combined with stiffness characteristics have significant potential to reveal considerably more information about grouted sand. The logarithmic decrement method was used to evaluate internal damping of grout and grouted sand. S-N-P(s- stress level, N -number of cycles to failure, P-probability of failure) relationship to ensure better predication of the fatigue life of grouted sand is used in probabilistic analysis. The McCall model based on a nonlinear relationship between maximum stress level S_{max} and the logarithm of number of cycles to failure N_f. In addition, the probability of failure P is introduced as a third variable.

Relation Between S_{max} N_f and P

$L = 10^{-a} S^{b}_{max} (\log N_f)^{c}$, where L = 1 – P = probability of survival: S_{bmax} = maximum applied stress expressed as a fraction of the static compressive strength (σ_{max}/σ_c) and ; a, b, c = material parameters. It was observed that almost all of the experimental data were within the probability of failure of 50 and 90%.

Stress-Strain Relation

To fully understand the performance of grouted sand under cyclic loads, a complete record of the changes in the stress strain characteristics is required. The major properties of concern are the variation of cyclic strain, secant modulus and damping ratio with the number of cycles at different stress levels. The test results selected for this analysis had a 50% probability of failure. It should be noted that confinement did not have any effect on the shear strength of grouted sand under the conditions tested. The typical stress-strain record for cyclic tests on a grouted sand specimen (cured for Tc = 28 days) loaded to a maximum stress level of Smax = 0.75, under a confining pressure of $\sigma 3$ = 207 kPa, is shown in Figure 40. It is therefore concluded that actual failure was assumed to occur at the time when the maximum stress recorded decreased from the initial preset value (σmax). Hence, in this case the number of cycles-to-failure was indicated as Nf = 1,673.

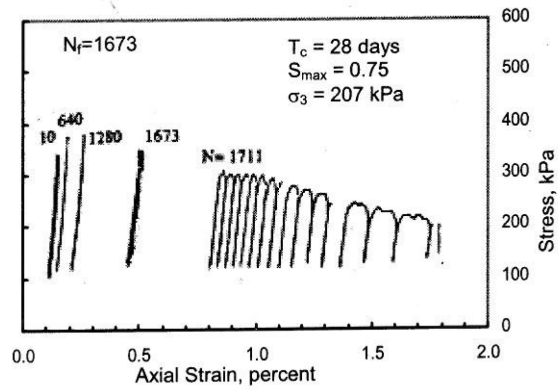


Fig. 40 σ–ε Hysteresis Relation for Silicated Sand at Stress Level 0.75

Hysteretic Characteristics

The loading branch of the stress-strain relationship, which was originally convex toward the stress axis, becomes concave upward after the first loading cycle and continued to maintain the same shape until the last few cycles before failure. This is typical silicate grouted sand specimens loaded to stress levels lower than S_{max} = 0.90. For stress levels equal to or higher than 0.90, the loading branch of the stress-strain curve remains convex toward the stress axis until failure.

Grouted Sand failed under an axial strain of 3%.The variation of axial strain & secant modulus with cycles at high stress level characterized in three stages: (Figure 41) In the first stage, between 5 and 10% of the

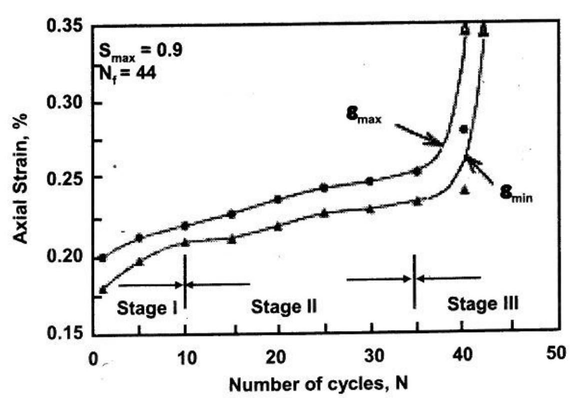


Fig. 41 Variation of ε with no of Cycles for silicated Sand at Stress level 0.90

number of cycles-to-failure, the strains increased with the number of cycles at a decreasing rate. In the second stage, between 10 and 75% of the number of cycles-to-failure, the strains increased at a constant rate. The third stage is identified by the said increase of strains up to failure. The strain values at the initiation of failure in cyclic tests (at N/N_f = 0.75-0.9) are very similar in magnitude to the strain at peak stress under static loading conditions.

Secant Modulus

The secant modulus E_{max} determined as the slope of the straight line joining the maximum and minimum points of the hysterias loop, decreased in three stages as observed for the axial strains. The ratio of the secant modulus E max to the initial tangent modulus E_i versus the life-cycle ratio N/N_f is presented in (Figure 42 a). The modular ratio varied between 65 and

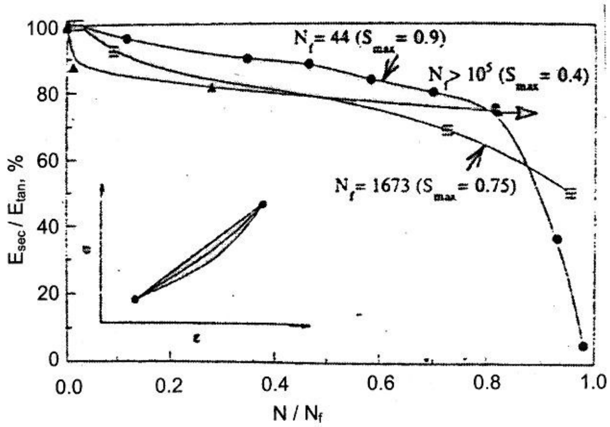


Fig. 42 (a) % Reduction in E_{sec} with Life Cycle Ratio for Silicate- Grouted Sand

75% of the initial modulus. At low stress levels (no failure) the secant modulus remained in the range of 75–80% of the initial modulus, even beyond 100,000 cycles.

AL is defined as the area between the loading and unloading branches of cycle and expresses the loss of dissipation of energy. The secant area A_s, defined by the area between the loading branch and the secant, serves to quantify the deviation of the stress-strain relationship from the linear path. With increased number of cycles, the hysteretic area AL was <0.005 kPa for grouted sands (Figure 42 b).

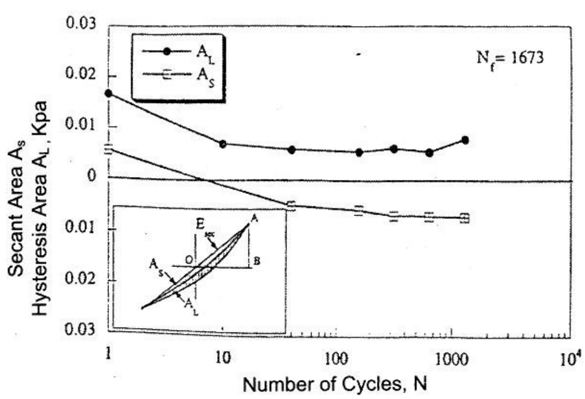


Fig. 42 (b) Variation in as and AL with N for Silicate Grouted Sand

Damping Ratio

Damping ratio, commonly expressed as D%, is often used in geotechnical engineering as a measure of attenuation or energy dissipation. The damping ratio is defined as the ratio of the energy dissipated per cycle of harmonic excitation in a certain volume to the peak elastic energy stored in the same volume and is expressed as (Richart *et al.* 1970). $D = A_L/4\pi A_r$, where A_r = area of triangle OAB as show in Figure 42(b) (insert). The damping ratio during the first few hundred cycles varied between 2 and 4% and increased with the increase in the number of cycles.

Liquefaction Potential of Sand Grouted with Micro Fine Cement (MC) and Chemical Grouts (Shroff & Joshi, 2004)

Soil liquefaction during earthquakes can cause severe damage to all types of overlying or buried soil structure systems.

Among the treatment methods used for reduction of liquefaction potential, chemical grouting may be the most effective in cases wherein the soil to be treated is difficult to reach, as in the case of soils under existing foundations (Maher, Ro and Welsh, 1994).

The undrained cyclic test results were used to evaluate the effect of grouting on the potential for initial liquefaction (i.e. pore pressure generated equals confining stress and effective stress becomes zero) and cyclic mobility by measuring the cyclic stress ratio and number of cycles required to reach initial liquefaction and 5% strain cyclic mobility. There are basically two approaches in the literature for defining liquefaction (1) flow failure liquefaction and (2) initial liquefaction and cyclic mobility.

1. Both sodium silicate and acrylate polymer grouts (with concentrations ranging from 20 to 60% in the grout solution) increase the resistance of sand to both initial liquefaction and 5% cyclic mobility. The increase was three to six folds with sodium silicate and two to four fold with acrylate polymer respectively.

2. Addition of microfine cement (MC), MC-SS, MC+bentonite and MC + silica fumes grouts. In water cement ratio ranging from 3:1 to 5:1 caused the sand to experience no initial liquefaction at 5% cyclic mobility for the ranges of stress ratios and cycle numbers tested. The standard vibration triaxial compression testing machine is used for the investigation. Figures 43 (a, b) shows the results for sand grouted with MC-SS grouts with one day and three day curing time which shows less susceptibility to liquefaction with increase in percentage of SS and decrease in w.c. ratio sand samples grouted with MC grout and MC+SF grout were completely resistant to liquefaction and were rock hard even at one day curing and could not be liquefied. The susceptibility of sand decreases with increase in curing time since the grout has gained more strength.

The fracture surface in HCP containing slag suggested that crack propagated preferentially through the C-S-H and calcium hydroxide. The fracture toughness accompanied with instantaneous fracture was probably dependent on mechanical properties of hydrates. While, the fatigue behaviors was mainly dependent on the property of C-S-H.

Liquefaction flow failure in grouted sand, a higher load or excitation is needed to "push the sand over the peak" of the undrained stress-strain curve and reach steady state levels. Cyclic resistance ratio(CRR) at W:C = 10 is above 0.4 of virgin sand indicating safe state of grouted mass after 1 day and 3 days curing period.

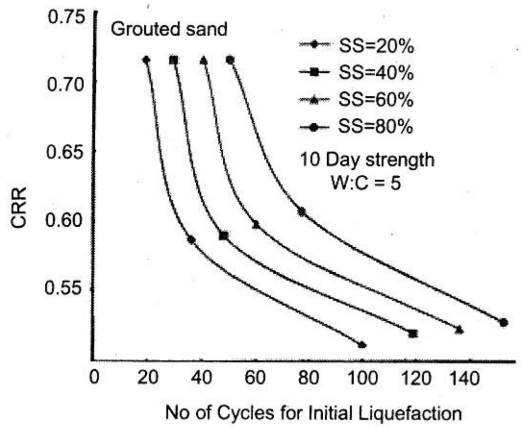


Fig. 43(a) Liquefaction Potential for Grouted Sand–1 Day Strength

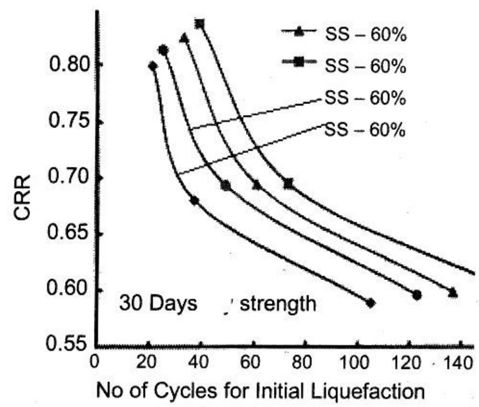


Fig. 43 (b) Liquefaction Potential for Grouted Sand–3 Day Strength

CRR vs W:C Ratio with Bentonite

Figure 44 show that for same W:C ratio and cycle number, as bentonite concentration increases, CRR decreases, though CRR remains higher than that of pure sand. At low and high % of bentonite, in very thin grout exhibit about 0.65 and 0.57 cyclic resistance ratio respectively during strong ground motion (N = 30). With W:C ratio of 10 and 3% bentonite minimum CRR of 0.54 is observed while at W:C = 2 with 1% bentonite maximum CRR of 0.83 is observed when samples cured for 3 days.

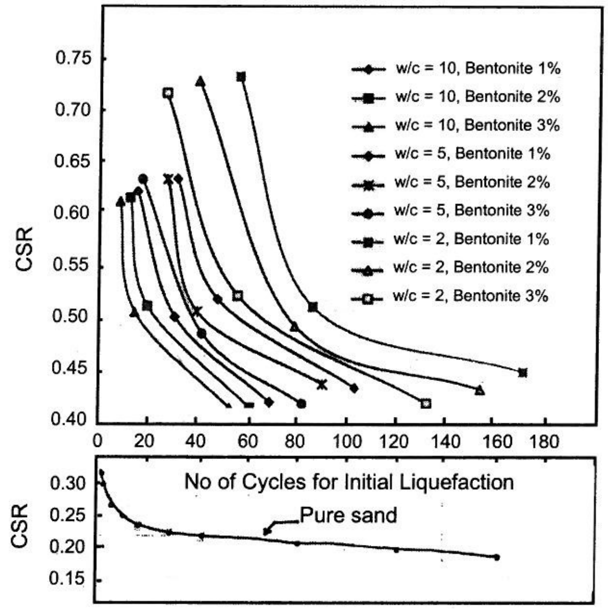


Fig. 44 Liquefaction Potential of Grouted Sand (1 day strength) and of Pure Sand

The pattern of the curve indicate that irrespective of % SS concentration, initially CRR reduces significantly with little change in cycles required for liquefaction which alter wards become gradual. At 30 cycles, CRR values are above 0.6 at 1 day and 3 days curing time which indicates safe state of grouted mass with reference to liquefaction. Compared to pure sand, the cyclic strength of MC-SS grouted mass is more than 3 times after 1 day curing time.

As the percentage SS concentration decreases, CRR decreases for same W:C ratio. The cyclic strength of grouted mass increases by about 10 to 15% as % SS concentration increased from 20% to 80%. This increase in strength is attributed to bridging of sand particles with network of silica gel preventing any contraction of sand mass.

Design of Grouted Factor of Safety and Improvement Index of a Soil at Gandhidham, Kutchch Area, Gujarat (Hospital Building)

Soil data: moderately stiff uniform fine sand deposit, other input data: earthquake magnitude of 6.5 on Richter scale with 5 km distance from seismic energy source, input peak horizontal ground acceleration, a_{max} = 0.998 m/s^2 (from Gandhidham meteorological laboratory), Unit weight of saturated sand, γ = 20.2 kN/m^3, Depth below ground level, z = 5 m.

Stress reduction factor computed as S_d = 1-0.00765 z (refer paper T L Tyoud *et al.*, 2001), therefore S_d = 0.9617

$\sigma_o = \gamma h = 20.2 \times 5 = 101$ kPa, $\sigma'_0 = \gamma' h = 10.2 \times 5 = 51$ kPa, Induced-CSR (Cyclic stress ratio) = 0.65 a_{max}/g) / ($\sigma_o/\sigma'_0)S_d$, g = gravity

$= 0.65 \times (0.998/9.81) \times (101/51) \times 0.9617$

$= 0.1259$

From Seed & Idriss Table, equivalent initial liquefaction cycle (N_i equi) at earthquake magnitude of 6.5 is 8.33

CRR value from cyclic triaxial test on virgin uniform sand = 0.251 (Figure 44) for equivalent cycles of 8.33.

Applying correction factor, Cr = 0.642 for effective confining pressure 400 kPa, corrected value of CRR,

CRR_c (Corrected Cyclic Resistance Ratio) = $0.251 \times 0.642 = 0.161$

Factory of safety = CRR/CSR = 0.161/0.1259 = 1.28, this is low, which is required to be increased.

When same soil grouted with MC-SS grout with W:C =5 and SS = 20% after one day, CRR increased to 0.81 at 8.33 cycles (Fig. 47.a). Applying same correction factor C_r = 0.642,

$CRR_c = 0.642 \times 0.81 = 0.52$

$\therefore$ Grouted factor of safety after one day curing = 0.52/0.1259 = 4.1

The improvement index is the ratio between CRR_c after grouting and CRR_c before grouting

$= 0.52/0.161 = 3.23$

Factor of safety with respect to liquefaction increased from 1.28 to 4.1 with improvement index of 3.23, when pure sand is grouted with MC-SS grout.

When same soil grouted with MC-bentonite grout with W:C=5 and bentonite=1% after one day, CRR increased to 0.99 (extrapolated value) at 8.33 cycles (Figure 44).

$CRR_c = 0.642 \times 0.99 = 0.635$

$\therefore$ Grouted factor of safety after one day curing = 0.635/0.1259 = 5.04 and improvement index = 0.635/0.161=3.94

Factor of safety with respect to liquefaction increased from 1.28 to 5.04 with improvement index of 3.94, when pure sand is grouted with MC with 1% bentonite grout.

In brief, the danger of liquefaction in grouted sand is too remote as set grout fills the pores and adheres to surface contact of void space which displaces the pore water imparting cohesive component.

Cyclic Plate Load Test

For sand grouted with MC (w:c = 5), the ultimate load intensity obtained from load settlement curve that after 28 days of curing is 50kPa which 4.67 times in dry sand and 5.53 times that of 30% saturated sand. In grouted sand (MC-W:C = 2, 14 days curing) E_i, E_n & c_u increased from 44.4kPa/mm, 77kPa/mm, 65200KN/m^3 at dry sand to 50kPa/mm, 93.3kPa/mm and 90000KN/m^3 respectively, which gives improved E_i ratio, E_n ratio and c_u ratio equal to be 1.31, 1.21 and 1.44 respectively. (Figure 45 (a, b)).

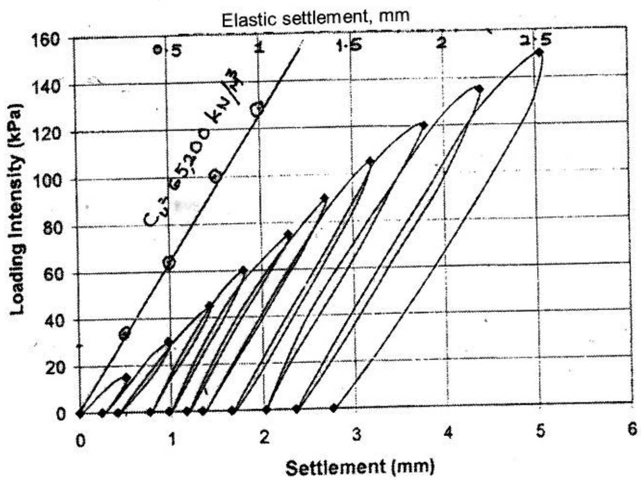


Fig. 45 (a) Static Cyclic Plate Load Test Results on Dry Sand (RD = 40%)

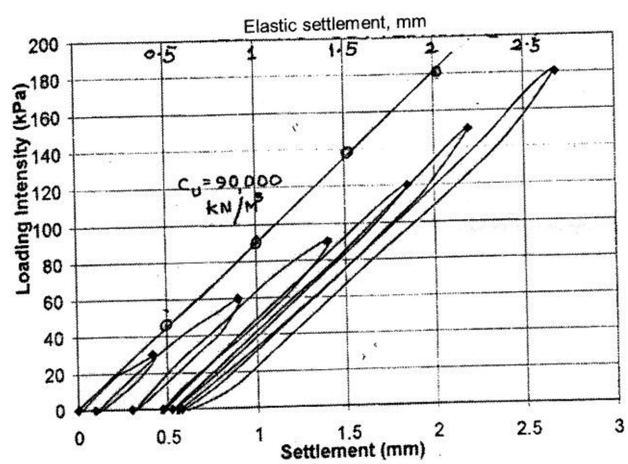


Fig. 45 (b) Static Cyclic Plate Load Test Results on Dry Sand (RD = 40%, MC Grout, W:C=2, Curing time = 14 days)

Optimization Considerations along with Theoretical Impositions

The flow of grouts in the pore space of the soil or any discontinuity of rock is resisted by drag at the interface between the grains and the fluid. This drag is proportional to the viscosity of the grout, mean flow velocity in pore space, its geometrical characteristics, tortuosity of alluvium (Soil structure) and Rock Geology.

The theoretical works of Maag and Raffle, 1961 relate the pumping pressure, density and viscosity of grout, rate of injection, permeability of soil and geometry of the flow. The expression is $t = (\alpha n/3khr) \times (R^3-r^3)$, where R = radius of the grout front after time, t; r = radius of injection pipe (sphere of origin); n = porosity of soil; k = permeability of soil; α = ratio of viscosity of grout to that of water; h = piezometric head in the grout pipe which can be related to the pumping pressure and the density of grout. The above formula is limited to that situation wherein the grout front is far from the injection point. Vaughan *et al.* (1963) stated that the flow rate under a differential hydraulic head is directly proportional to the (fissure width)^3 and inversely proportional to the viscosity of the injecting fluid. For a uniform flat rock fissure of width b penetrated by a hole of radius r and connecting with an open reservoir or sink at a radial distance R from the hole, the estimation of flow rate q either for grout flowing through a fissure into the formation: $q/H = (\pi \Upsilon_w g b^3)/6\mu(\log R/r)$, Where, g = acceleration due to gravity, Υw=density of water, μ = viscosity of injected fluid, and H=differential hydraulic head. When fissure merges with open structure or multi connected fissures and joints flow is largely determined by b^3/μ. The flow resistance of chemical grouts in fissured rock by a parameter is dependent on the characteristic width and mean spacing of the fissures. Shroff *et al.* (1987) computed the radius of penetration of grout from the data on its consumption and studied the effect of flowing water through stratification and of short gel time on shapes of stabilized mass (Figures 46a & b).

Further, Shroff and Shah (1992) studied the flow interaction of grout along by injecting through layered sand samples having different permeability, density and stage length. To achieve a uniform stabilized mass while grouting from top to bottom or from bottom to top, the optimum stage length should be maintained. Figure 46 (b) also depicts a non uniform stabilized mass having two spherical bulbs with no grout in between. A spherical model of the injected mass, stating that upon injection the grout appears to flow progressively into the soil, with fresh grout displacing the older grout towards the exterior of the grouted mass.

The analytical approach of spherically radiating displacement flow is as follows: If the grout has reached a radius R at time t, the volume flow rate Q is related to the hydraulic head h at the source of radius *r* by:

$$h = (Q / 4\pi k) [\alpha (1/r - 1/R) + (1/R)]$$

where α is ratio of grout viscosity to that of surrounding ground water. If the void ratio of the soil is e, the time for grout to reach a radius *R* is given by:

$$t = (er^2/kh) [\alpha/3 (R^3/r^3 - 1) - (\alpha-1)/2 (R^2/r^2 - 1)]$$

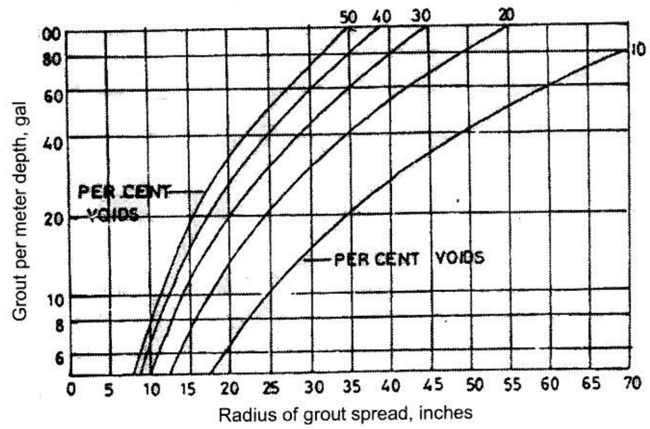


Fig. 46 (a) Penetration Relation

Fig. 46 (b) Shapes of Grouted Mass (sand)

The specific value of t can be found from Figure 47 in which kht/er^2 versus R/r for three values of viscosity ratio is given.

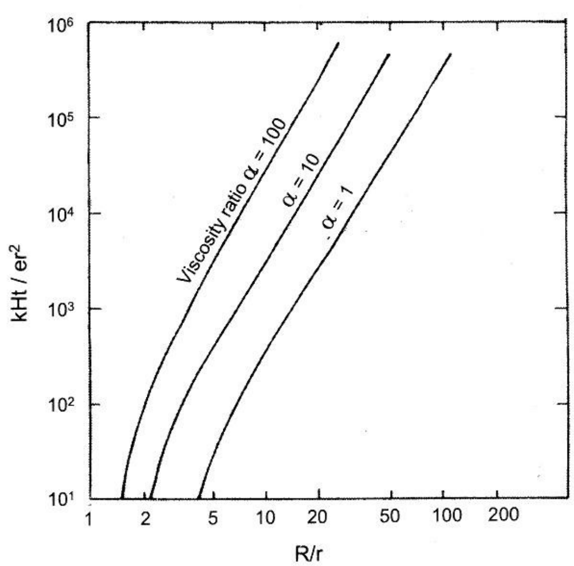


Fig. 47 Penetration Time on Viscosity Ratio

Optimization of Penetration Rate for Grouts

At the later stages of injection, by which time the grout-water interface has moved well away from the injection hole, a much simpler and yet reasonably accurate estimate of penetration rate can be made using the reduced coefficient KG in place of the conventional water permeability coefficient k. Flow rate : $q/H = 4\pi k_G L/\alpha_g$, where α_g depends upon cell dimensions and extent of confinement of the aquifer. α_g = 2 \log(L/a) is for uniform ground and $\alpha_g = \{2L/(L^2-4a^2)^{½}\}$ [log $\{L + (L^2 - 4a^2)^{½}\}/2a$] is for stratified aquifer (Figure 48).

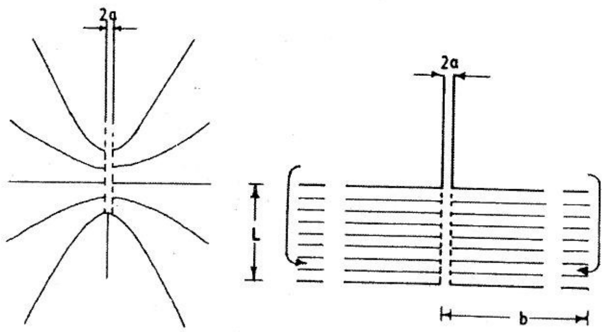


Fig. 48 Grout Flow Pattern-Uniform / Layered

Criteria of Blockage

For soils that are not uniformly graded, a useful estimate of the condition for blockage by a particulate grout can be gained by applying the Kozeny relationship: $R = (8\mu k/\Upsilon_w gn)^{½}$ to find diameter 2R of the average pore passage, where μ is viscosity, *k* permeability, Υ_w density of water, *g* gravity and n the soil porosity. If the average pore diameter is taken as equal in size to the 100 μm particle of cement, positive blockage can be expected on this basis for a soil permeability of $10^{-2.5}$ m/sec.

Consolidation of Clay Gauge Material

The volume of grout injected after time *t*, i.e., volume of water displaced from the voids to consolidate the soil is given by $V= \int (U_o - U)m_v \times 4\pi r^2 dr$, (limit of integral is from o to a) Where V=volume of water expelled (equal to volume of grout injected), a=radius of sphere of influence, U_o=injection pressure, m_v= volume compressibility of the soil, U=pore water pressure as a function of time and position within the sphere, r = radius of sphere of influence between 0 and a.

Integration of equation governs the rate at which consolidation can occur. This approach was adopted for compressing silt or clay existing in karstic lime stone at Kharun III River at Iran. Also it is helpful in jointed rock with gauge material.

Hydro Fracturing by Grouting

The values of hydro-fracture pressures in terms of their magnitudes and orientations are being actively investigated. Theoretically, they can be determined by the relation: $P_f = \sigma_v\gamma(1+\sin\phi)$, where Pf = hydro-fracture pressure, σ_v = effective vertical stress, γ = Poisson's ratio and ϕ = angle of shearing resistance (Figure 49).

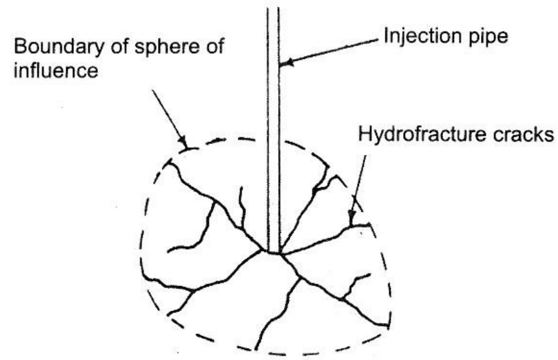


Fig. 49 Hydro-fracture Pattern

Relation:

$P_f = (q_r/2\pi k)\ (\gamma_g V_g/H \times V_w)$ (log r/r')

where P_f = hydro-fracture, lb/ft^2

q_r = rate of grout flow; γ_g = unit weight of grout, lb/ft^3;

k = permeability, ft/sec; v_g = viscosity of grout, cP;

v_w =viscosity of water, cP;

H = thickness of zone being grouted, ft;

r = radius of zone being grouted, ft;

r′ = radius of grout hole, ft.

Time Viscosity Equations

Shroff and Shah (1977) developed a theoretical concept and suggested a modified power model law for the viscosity of a grout at any time after mixing to any stage of permeation in the formation. The free volume of the liquid grout changes with time either in a gradual fashion or instantly; the viscosity must necessarily mirror the change of free volume at any time (Shroff and Shah 1977). Power model $\eta = s/D^n$ = shear stress (s)/shear rate (D).

This power model will not follow the entire curve of time-viscosity, as the total behavior from liquid to gel of a grout system consists of various rheological changes. Shroff and Shah (1988) putforth the mathematical model. Viscosity of grout ,η_g = exp $(0.013)\{(S)^{0.395}/D\}$. The above equation accounts for all the rheological changes in a chemical grout system and follows the time viscosity curve reasonably well.

Mechanical Requirements of Grout

The grout pumped into fissures has to fulfill certain mechanical requirements in order to be suitable for consolidation or tightening. The allowable average compression stress p as found by Prandtl for a perfectly plastic medium with a cohesion c in a fissure of length l and width e will be equal to $p = c[(\pi/2)+(l/2e)]$, as l/e is always large, even very small cohesion value is suffice for the grout to resist high compression stresses (Figure 50).

Similarly, the resistance against being pushed out by the water head (Figure 51) was found Mandel to be: q = 2c(l/e) (for purely cohesive grout) an $q = (c/tg\rho)\ (e^{k.l/e-l})$ (for medium with cohesion c and internal friction ρ) With

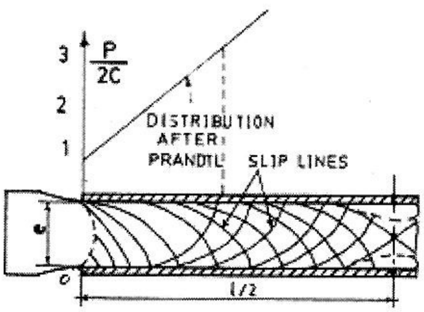


Fig. 50 Allowable Compressive Stress

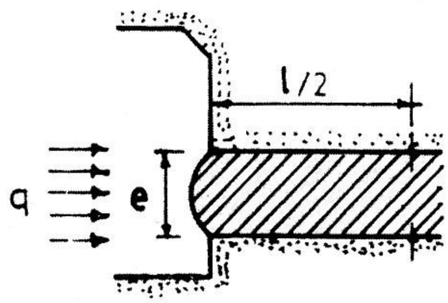


Fig. 51 Resistance against Water Head

$k = 2tg^2\rho\ (\cot \rho + \rho + \pi/2)$.

In a fissure with a large value of l/e, even a very poor grout is able to resist high hydraulic pressures.

A smooth grout-filled fissure may be solicited to a shear yield at $\Gamma = C + p \,.\, t \,.\, g \,.\, \rho$,where C and ρ are either the values of the grout itself or of the contact plane between the grout and the rock (Figure 52). It is evident that a consolidation grouting which has to confer to the rock mass of high shear strength, must be done with resistant grout, generally based on cement. This is not absolutely necessary, however, if the fissures are rough and well indented, so that the shear strength is furnished by the rock (Figure 52).

Flow Equation of Grout through Rock Fissures

Contrary to the grouting of loose soil, where the volume of grout flows through voids of largely varying

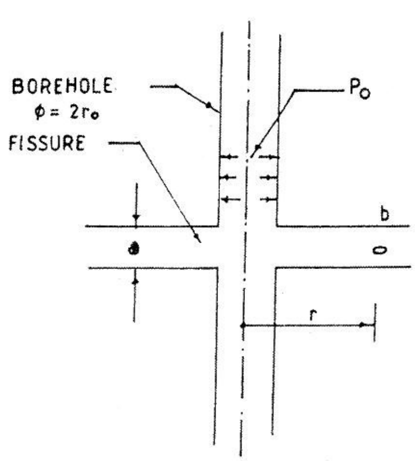


Fig. 52 Contact Plane: Grout-Rock

sections, in rock grouting the mix flows essentially through fissures (Figure 53). Baker derived

$$p_o\text{–}p=\eta Q\Upsilon/\pi e^3)\ (\log r/r_o) + 3\Upsilon Q^2/20g\pi^2e^2)(1/r_o^2 - 1/r^2),$$

where, p_o = pressure in borehole of radius r_o, p = pressure at distance r

η = coefficient of viscosity, Q = flow rate, Υ= density of the mix

The first term allows for the pressure loss due to friction, and second term due to the modification of kinetic energy in radial flow.

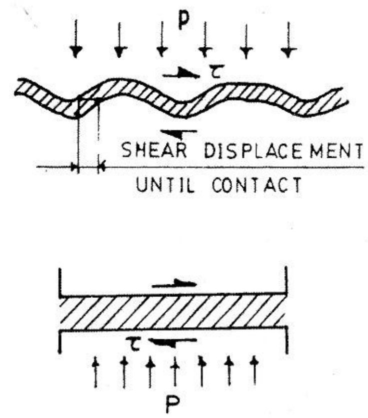


Fig. 53 Grout Flow in Fissure of Constant Width

Assume that at a distance *r* (radius of action) the pressure *p* = O, and obtain for a given fissure and grout type, a diagram of the type given in Figure 54, which has to be slightly modified for greater values of Q when the laminar flow changes into a turbulent one. The phenomenon of widening the fissures by hydraulic pressure makes the quantitative interpretation of water test very difficult.

The setting time of grouts must meet the time needed for injection, corresponding to the design spreading radius. The spreading radius of grout R in cm can be evaluated by the Jiacai *et al.* (1982) formula:

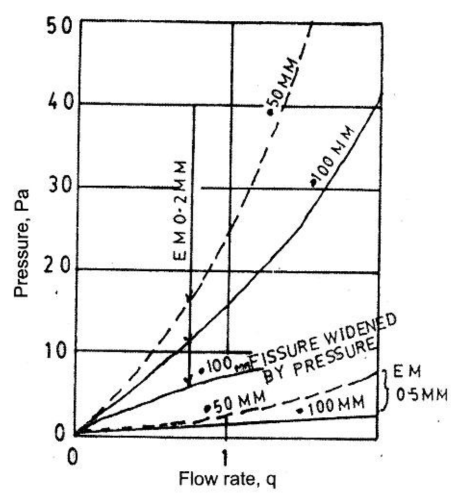


Fig. 54 Pressure-Flow Rate Relation

$t = [1.02 \times 10^{-7} \eta \, (R^2 - r^2) \log R/r] \, / \, (p - p_o) \, b^2$

where; t = time needed for injection, min; η = viscosity of grout, cP;

R = spreading radius of grout, cm; r = radius of grouting hole, cm;

p = pressure in the grouting hole, kg/cm^2;

p_o = hydrostatic pressure of the grouted water in the water bearing fissures, kg/cm^2; b = thickness of the fissure, mm.

Geological Considerations in Optimization of Grout Hole and Technique

An explanation/description of each factor has been given to assist the field engineer engaged in a particular grouting project with respect to orientation, spacing, when to start the grout mix, types of field techniques to be used, precautions to be undertaken during grouting, decisions regarding stage length, use of grout cap etc. These considerations are only the commonly observed. There are many others and vigilance is necessary in finding them before grouting is undertaken to enable a grout design that is intelligent and appropriate.

Spacing of Open Joints

Open joints may be widely or closely spaced. More grout holes will usually be needed than for wide spacing. In widely spaced joints, just the primary hole serves the purpose of grouting. In closely spaced joints, the sequence of grouting may require not only a secondary hole but even tertiary and quaternary holes in extreme cases. When joints or cracks are uniformly distributed, one may initially double the spacing, and then reduce it as per the requirements seen from water testing (Houlsby, 1982). If the gouge material in the seam or fault plane is clayey, it is essential to wash it out by a calculated dose of chemical dispersant under pressure and then grout with a cement sand mortar to avoid sliding of the dam due to shear failure of the clay gouge (Parikh & Shroff, 1978) or squeeze it by pressure grouting (Kadana Dam Gujarat, Shroff, 2003).

Size of Open Joints

Joint openings wider than 2 mm (Figure 55) enable the grout to penetrate easily, although if they are very wide, such as 6 mm, the grout might penetrate so easily that proper tightening up to refusal requires inhibition of excessive travel by using multiple applications of grout. A thick trial grout mix might also serve the purpose. In wider openings a coarser grout mix, such as sand cement, might help. The use of chemical admixtures in a cement grout or Newtonian (watery) resin grouts might solve the problem of limits of injectability. Theoretical imposition earlier will provide clue to fissure width and viscosity of grout.

Direction of Open Joints

The direction of open joints influences the orientation of the grout holes and the possibility of rock movements during grouting. Joints which dip between about 30 to 60 degrees are easily intercepted by vertical holes and are less liable to permit rock movements than those of near horizontal or near vertical dip. The latter require inclined grouting holes. The optimization of orientation of grouting holes with respect to dip and strike direction of bedding plane will be discussed later on. In addition, the direction of open joints may also influence the techniques of grouting to be adopted as well as the decision regarding permissible grout pressures.

Rock Strength

In the case of weak or weakly embedded surface slabs liable to movement during grouting, precautions against rock movement become necessary. This is apt to slow down the grouting and involves intensive placement of holes. In such cases, in addition to grouting, the provision of rock anchors at various spacing provides excellent assurance against rock movement.

Rock Soundness

When holes collapse frequently during or after drilling, the upstage method is totally unsuitable. Only downstage methods without packers or, in extreme cases, circuit grouting will serve the purpose. If there are many surface cracks in the rock, stand pipes help to control surface leaks. A grout cap helps to improve the grouting by reducing the surface leaks.

Rock Stresses

The stress in rock is usually of tectonic origin and can be severe in massive strong rock. The strain energy released can frequently project the detaching rock away from the bedrock and leave rebound joints that are open up to a few centimeters. The grouting practitioner needs to be able to recognize the presence of stressed rock because of its predominant effect on grouting procedures. The other extreme is the soft, intensely cracked foundation of the type which has already relieved stresses itself and will present no difficulties on this score during grouting.

Uniformity

Uniformity of the foundation assists in the layout of grout holes whereby they can be evenly spaced and all drilled at the same inclination. Irregular jointing, variable rock type, intrusions, faults etc. may require the holes to be at various inclinations and weak features may need especially intensive grouting.

Proneness to Piping

If the material in joints is liable to be removed by seepage, the grouting may need to be more intensive than otherwise to virtually eliminate seepage in the piping-prone areas.

Geological investigations

The following information should be obtained and shown in either a topographic or geologic scheme to optimize position and orientation of grout holes. (a) boundaries and contacts of the different geologic units to be treated, (b) altitude (dips and strikes) of sedimentary and metamorphic rocks, (c) prominent joint system, their spacing, opening and character of the material of infilling

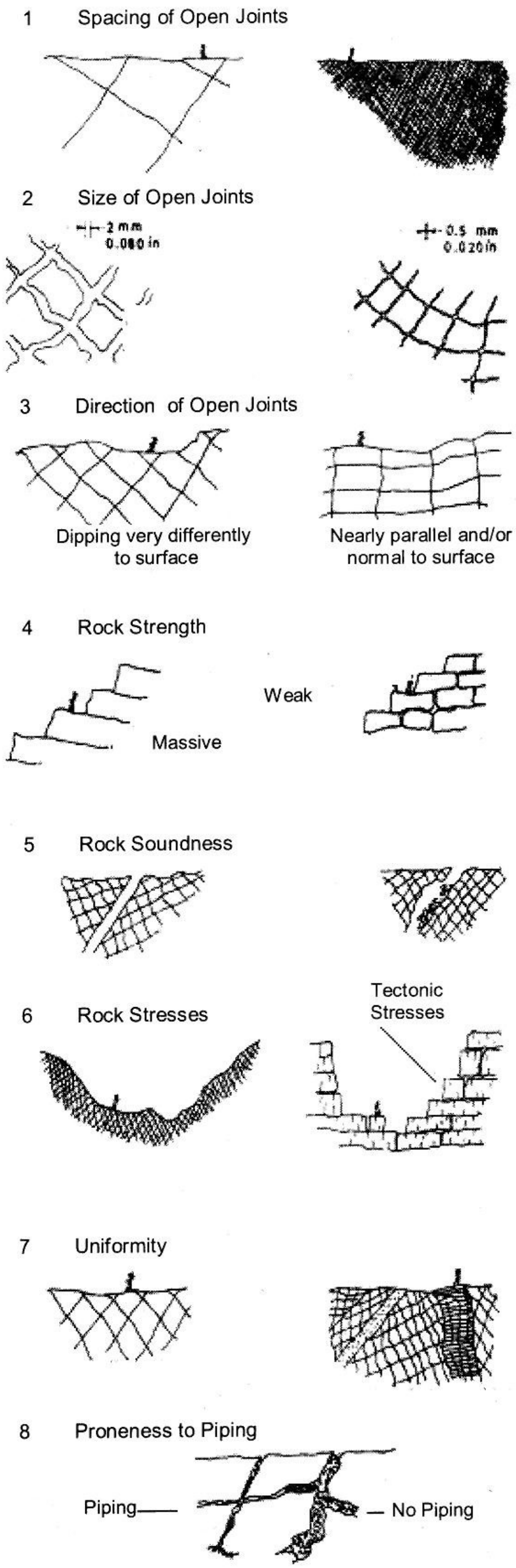


Fig. 55 Geological Considerations in Optimization of Grout Holes and Technique

if any; location and altitude of prominent faults, shear zones faults etc. (d) lines of geologic cross section and (e) location and logs of drill and auger holes, exploration tunnels and shafts.

Geophysical methods may be used with advantage to locate boundaries between different elements of the sub rocks. Rotary drill cores are also a useful source of information.

Grouting in Relation to Lugeon Value

One Lugeon (1 lit/m of hole per minute at 10 bars pressure) is the degree of permeability encountered in those nearly tight foundations, which require almost no grouting. Ten Lugeon warrant grouting for all most all types of dams. One hundred Lugeon are encountered in heavily jointed sites with relatively open joints.

Water Testing

Water testing before grouting has the purpose of indicating the permeability at a given stage (or hole). It helps in determining the starting mix and whether any special measures are necessary.

It also indicates the effect of previous grouting of closure sequences. Water testing is not usually required after grouting. Water test pressure = 1 bar (15 psi), (1.05 kg/cm^2); Test time – 15 minutes comprising 3 to 5 minutes per run. The water test measures the amount of water taken in each of the three 5 minute runs.

The water testing is preferably carried out 'downstage with single packers' rather than a 'double packer' in a hole which has been previously drilled to full depth. The length of the test stage is usually about 6 m.

New Developments: The Lugeon values for the stage can be calculated from the electronically acquired data chart flow rate versus pressure curve recorded on the x–y plotter, as the gradient of the plot is proportional to the Lugeon permeability, the shape of the x–y plot indicates the various flow patterns as described by (Shroff, 1999). The gradient of the x–y plot at the higher pressure stages is used to calculate the Lugeon value.

Some techniques have been reported in the literature for Lugeon test, such as the air pressure test (Kayakin *et al.*, 1985), the free oscillation test (Aidel and Krauss-Kalwait, 1985), and the radioactive tracer dilution method (Monev *et al.*, 1985). Whereas the air pressure test is limited to special circumstances, the other two tests seem promising and need further development.

Optimization of Field Grout Mix for Alluvium

There are certain considerations to be observed in the use of cement bentonite grout for alluvial grouting; in general, the quantity of bentonite or clay which can be incorporated in a mix is dependent for the purpose, it is to be used.

Triangular Chart for Cement - Bentonite (Suspension) Grouts

Considerable economy in cement consumption per unit volume of grout mix can be realized, particularly when the final strength is of secondary importance. A cement bentonite grout for impervious grouted mass should contain hardly more than 175kg of dry material for 1 m3 grout and 300 kg of dry materials for moderate consolidation resistance (20 kg/m^2).

To determine the field utility of a phosphate and silicate solution in bentonite slurry, the mixes shown in the triangular chart (Shroff, Shah, 1999) depicted in Figure 61 can be consulted.

In the Figure 56, 'a' unstable suspension settles; 'b' temporarily stable suspensions settle before setting; 'c' clay cement gels of low compressive strength; 'd' free-flowing stable and pump able suspensions; 'e' stable putty like suspensions and 'f' solid unworkable mixes, normally powder. A weak cement bentonite grout with 10 per cent bentonite, 2.5 per cent cement and 87.2 per cent water (1 B : 0.25 C : 8.72 W) falls in zone 'd' of the triangular chart, and constitutes a free flowing, stable and the pump able grout. The strength of the grout can be increased by adding sodium silicate and monosodium phosphate in the proportion of 1:1.

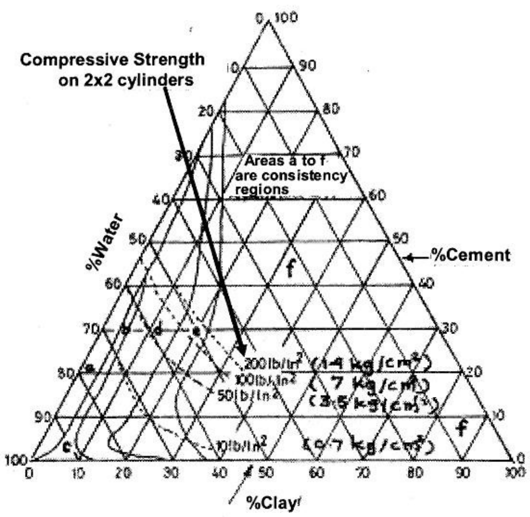


Fig. 56 ▲ Chart: Cement-Clay / Bentonite

Triangular Chart for Chemical (Solution) Grouts

Triangular chart of Aminoplast Grouts (Shroff, Shah, 1999): Within the frame work of gel time and stability, in this triangular chart, one side of the triangle consist of Urea-formaldehyde percentage from 0 to 30, other side is considered for the catalyst percentage which ranges from 0 to 30, and remaining side shows water dilution accounted from 70 percent to 100 percent. Various zones of grout mixes are shown in Figure 57.

Zone (a): It consist of stable grout mixes with high gel strength range from 0.15 to 1 kg.sq.cm, having gel time below 10 minutes. Zone (b): Grout falls in this zone having good stability, medium gel strength in the range of 0.05kg/sq. cm to 0.1 kg/sq.cm having gel time between 10 to 15 minutes. Zone (c): It contains the grout mixes which exhibits little low stability with low gel strength ranged from 0.005 kg/sq. cm to 0.05 kg/sq. cm having gel time between 10 to 20 minutes. Zone (d):

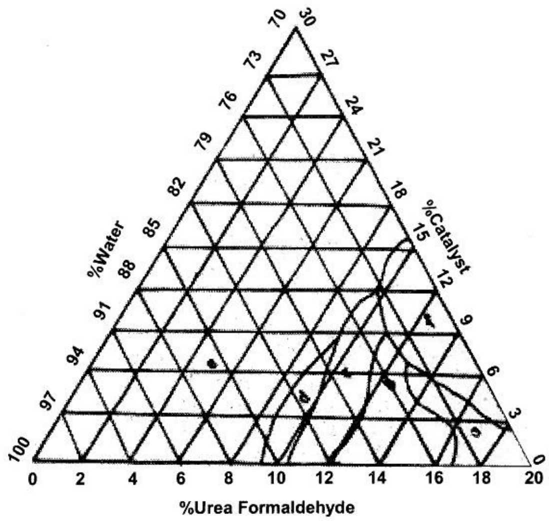


Fig. 57 ▲ Chart: UF Grout

Grout mixes of this zone produce unstable soft gel with gel time between 20 to 35 minutes. Zone (e): These zones shows grout mixes which are unworkable produces non homogenous gel or soft individual separate nodules at long time. Zone (f): The addition of catalyst in the grout system of this zone produces quick jellification within 1 to 2 minutes.

Similarly triangular Charts for optimization of other chemical grouts & various physical properties of Cement Bentonite are designed (Shroff, Shah 2002).

Optimization of Trial Grout Mix for Rock Grouting

Past experience and review during grouting are important for selection of mixes for an actual job. The process of deciding w:c ratios to be used for each grouting application involves the followings:

The starting mix based on information obtained from water testing carried out in the grout hole together with geological information such as size of joint opening, orientation etc. and experience from earlier holes. The majority of site experience has indicated that the w:c ratio 2:1 is usually suitable; obvious exceptions are (i) for relatively fine cracks such as 0.08 mm or less having permeability less than or equal to 5 Lu. Start the mix with w:c ratio 3:1 instead of 2:1; (ii) for usual crack dimension 0.1 to 12.5 mm having permeability between 5 and 30 Lu, use w:c ratio 2:1; (iii) if the cracks are fairly wide such as 12.5 mm to 25.4 mm having permeability more than 30 Lu, Start the grouting with mix having w:c ratio 1:1 instead of 2:1 and (iv) if the cracks are very wide such as 25.4 mm or more, start the grouting with mix having w:c ratio 0.8:1 or thick.

Often, however, crack sizes encompass such a wide range or are unknown and hence the above guidelines are of little assistance. In such cases, if experienced advice is not available, site tests may be under taken in accordance with the following flow chart.

Flow Chart for Optimizing a Starting Mix

Steps: start with a representative group of holes with cement grout w:c 2:1, then, assess the results as follows: How was the grout take in the first fifteen minutes? Did it generally compare with the water taken in the water test?

Less than the water take?

- Yes - Grout was too thick. Try a thinner mix next time

- No - This mix is suitable Try it again

More than water takes?

- Yes - Grout was too thin. Try a thicker mix next time

Optimization of Field Techniques of Grouting

For 'curtain grouting' (i.e. to form a relatively deep barrier to the passage of water through a foundation), holes are usually grouted in stages by one of the following methods.

Stage Grouting Method–Down stage

Downstage without Packer: In this method, grouting is done through the lower open end of the grout pipe in short stages of 1 to 2 m starting with the top of the grouted zone. The process involves repetition of a sequence of operations comprising drilling through the length of each stage and grouting followed by redrilling (Figure 58). This low output is observed in this method.

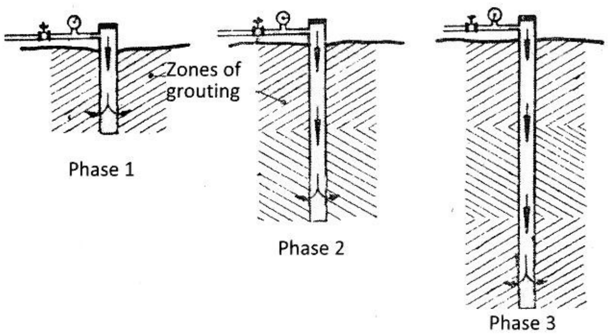


Fig. 58 Down Stage Method

Downstage with Packer: In this method, rubber packers are used which expand on pressure to about five to six times the diameter of the hole. Hence, grouting is facilitated in a particular zone by fixing packers on the top and bottom of it. This method facilitates the washing and grouting of seams at a particular level with the desired pressure established within permissible limits. Grouting thus becomes more effective than in the case of stage grouting. The addition of packers to the downstage method is rarely warranted. The downstage method with packers has one special advantage however, it delimits plastering of the walls of the borehole which extends to the successive stratum to be grouted (Houlsby, 1983).

Stage Grouting Method–Upstage Method

In this method, grouting is done through the casing which is driven to the bottom of the hole (Figure 59). The casing is withdrawn a short distance and grout is injected through the open end into the cavity left by the casing as it is raised. The process is repeated up to the entire depth. Grout leakage may occur along the grout pipe as some disturbance around the tube cannot be avoided when the grout pipe is pulled up. So it is difficult to treat the soil layers individually (IS: 4999-1978). The upstage method is useful in homogeneous strata, open gravel or boulders. In sites prone to rock movement, it is almost impossible to carry out grouting using the upstage method as the grout persistently travels upwards at high pressures through the as yet un-grouted upper stages. In sites where holes might collapse quickly, upstage grouting is definitely impractical.

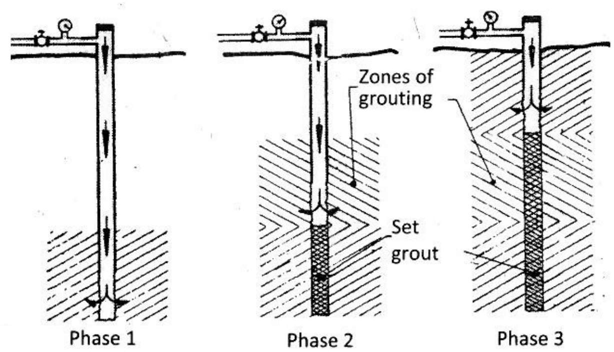


Fig. 59 Up Stage Method

Technique for Washing Out Grout Holes

Grout holes are washed out after drilling to ensure a clean hole for grouting. Washing out grout holes is also necessary in some methods grouting to prepare them for another application or stage. Washout equipment shown in Figure 60 is with polythene piping, with an open steel tube washout bit at its lower end (IS: 6066–1971)

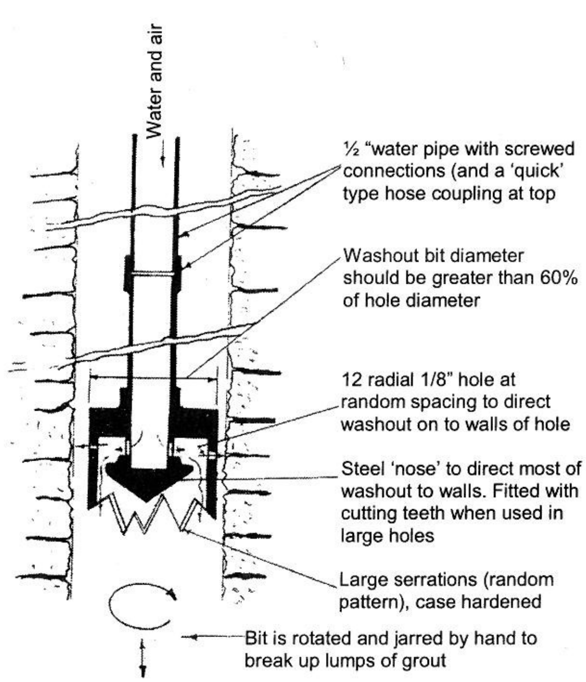


Fig. 60 Grout Hole Washout Equipment

Tube-a-Manchette (Sealed-in Sleeve Pipe Injection Method

In this system of grouting, a hole 12.5 cm to 15 cm diameter is drilled in the ground and a 6 cm diameter pipe with a circumferential row of holes at 30 to 33 cm is lowered inside it. These holes are covered with tightly fitting rubber sleeves. The 6 cm diameter pipe, called the tube-a-manchette, is sealed into the outer hole by a brittle clay-cement grout as the outer casing pipe is withdrawn. Grouting of the alluvium is then carried out from the bottom upwards through a smaller interior grout pipe lowered into the 6 cm diameter pipe. The inner grout pipe has rubber packers 30 cm apart which fit the 6cm diameter tube-a-manchette tightly and isolate a particular circumferential row of holes in the latter for grouting.

When the grout is pumped in through the inner pipe, its pressure expands the rubber sleeve and cracks the brittle grout to escape and fill the voids in the surrounding soil. Thus, both the problems of supporting the hole while permitting the grout to flow into the soil and of using packers are solved. This method is generally best suited to alluvial grouting. Though the procedure is expensive compared to other alternative methods of seepage control, this method enables grouting in any desired sequence. The procedure is illustrated in Figure 61.

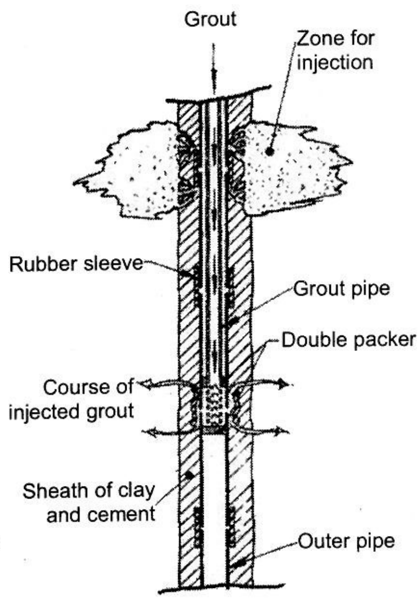
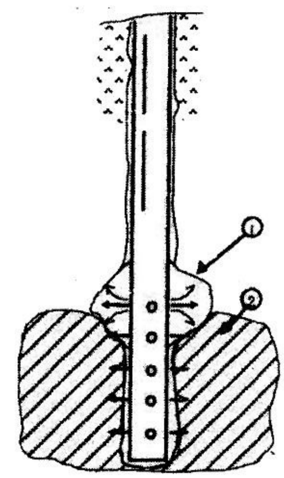


Fig. 61 Tube Manchette Grouting Method

Chemical Packer Injection (CPI) Method

For the purpose of attaining complete permeation, CPI method is devised. (Hoshiya *et al.* 1980, Shroff, 1999). A compound grout in which formation of a quick-setting chemical packer by a short gel time grout and permeation by a long-gel-time chemical grout are combined through a double pipe rod is applied in one continuous process. The short gel-time grout (gel time of several seconds) acts as a packer by solidification of the voids and coarse particle layers around the grouting pipe, and that the long-gel-time grout serves for permeation grouting. Development of this injection process utilizing the gelation characteristics of a non-alkaline silica solution has enabled application of a compound injection in a simple operation. Shroff (2008) developed dual mode grouting method as shown in Figure 62 where in a shorter gelling time zone acting as a cover against leakage of permitting grout following underneath.

Simultaneous Drilling and Grouting Method: As discussed earlier it allows for injection during rotary drilling. At a predetermined distance, the drill rod is withdrawn and grout is injected into the soil or material through a separate drill rod. The process continues from the top downward. It is only suitable for pervious, granular soils.



(1) Injection of short gel as packer function

(2) Injection of long gel grout

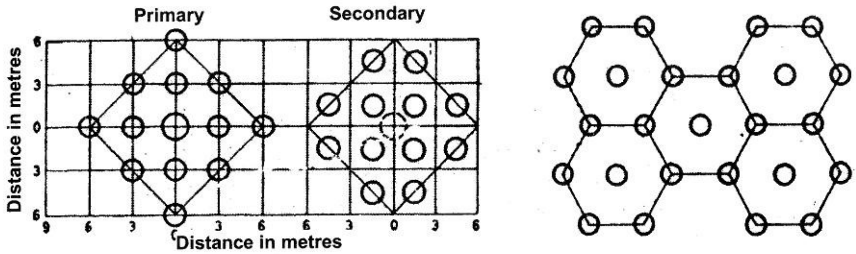
Fig. 62 Chemical Packer Grouting Method-CPI (Dual Mode Grouting)

Optimization of Pattern, Orientation and Depth of Grout Holes

A mathematical solution to the problem of orienting grout holes in the optimum direction with respect to formation, strike and dip for grouting a rock formation was devised by Singhota. The solution consists of a set of equations obtained with respect to the bedding plane, considering its interaction in the X–Y plane, Y–Z plane and X–Z planes, and evolving an equation for optimum inclination of grout hole with reference to a canyon profile.The variation of $(\theta + \phi)$ and α are shown graphically for different values of strike and dip. (Ref. Shroff & Shah, 1999). Where $(\theta + \phi)$ is inclination of plane of grout hole, θ is inclination of Canyon profile to the horizontal, ϕ is inclination of plane of grout hole normal to the x-z plane with Canyon profile and α is inclination of plane of grout hole with Canyon profile.

Consolidation Grouting

When the purpose is consolidation, the holes are arranged in regular patterns over the entire surface area required to be strengthened and the depth is determined



Square pattern

Hexagonal pattern

Fig. 63 Typical Pattern of Grout Holes for Consolidation

by the extent of broken rock as well as the structural requirements regarding the deformability and strength of the foundation.

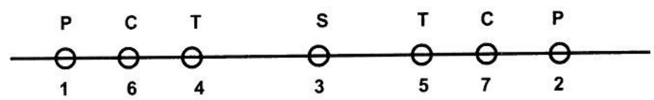
The choice of pattern of holes for consolidation grouting depends on whether it is necessary to wash and jet the hole systematically. When washing has to be carried out, a hexagonal pattern (Figure 63) is preferable as it allows for flow reversal. When systematic washing and jetting is carried out to remove all soft material in seams, it is generally not necessary to use primary and secondary systems of holes (IS: 6066-1984). When it is desirable to test the efficacy of consolidation grouting by comparing the grout absorption in primary and secondary holes, a rectangular or square pattern (Figure 63) of holes is preferred (IS: 6066-1984). This is generally the case when the joints are irregular and relatively free from in-filling or it is not necessary to remove the material filling the joints.

The most common difficulty experienced in consolidation grouting is surface leakage. It is therefore customary to pipe through the required height of concrete or masonry and carry out the grouting after the rock permits use of higher pressures so that even the smaller seams can be grouted effectively.

Curtain Grouting

In curtain grouting the purpose is impermeance; the grout holes are arranged in a series of lines to form a curtain approximately perpendicular to the direction of seepage. The depth of holes is dependent on design considerations as also on the depth of pervious rock and the configuration of zones of relatively impervious strata. For grouting with cement 38mm holes have been in common use. In long holes, the diameter at the top of the holes may have to be larger than the final diameter at the bottom of the hole to facilitate telescopic observations or to allow for the wear of the bit.

Single-line Grout Curtains: In this type of curtains, it is customary to drill a widely spaced system of primary holes, subsequently followed by secondary and tertiary holes at a progressively smaller spacing Figure 64(a). The initial spacing (of primary holes) usually varies between 6 m to 12 m based on the geological conditions and on



Single line of grout holes along a cut off trench (after Mistry, 1965)

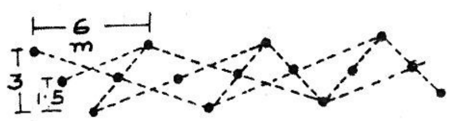
Legend:
P – Primary holes
S – Secondary holes
T – Tertiary Holes
C – Check Holes
1,2,3 – Order in which grouting may be carried out

Fig. 64 (a) Single Line of Grout Holes- Curtain

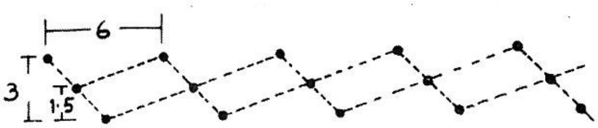
experience. Percolation tests should be carried out in a few test holes to ascertain whether further grouting of the area is necessary, or whether secondary or tertiary treatment is required. In addition to the systematic grouting of primary, secondary or tertiary holes, if may be necessary to drill and grout additional holes for treatment of peculiar geological features, such as faults, shear zones and weather rock seams.

Multiple-line Curtains: In rocks with a wide range of size of openings, cavities and discontinuities, which are also irregularly distributed, multiple line curtain is optimize selection for better seepage control. It consists of outer lines which are drilled and grouted initially with thicker grouts. After completing the grouting of the outer holes, the intermediate line or lines of holes may be drilled and grouted at comparatively higher pressures with thinner grouts. Grouting of the outer rows, which is carried out initially, facilitates confinement of the grout and thus ensures effective subsequent treatment of finer cracks at higher pressure through the central row or rows of holes (Figure 64b). Check hole, generally at the inter section of the diagonal formed by the square, rectangular or parallelograms. As a rule, the drilling of secondary holes in any zone of foundation should not be taken up until the grouting of primary holes is completed. In multiple-line curtains, the relative sequence of outer and inner rows should also be strictly followed. In the split-spacing or closer method, the sequence is as shown in Figure 69(b). If further check holes are necessary, then these will be along a line formed by a point joining the intersection points of these diagonals, as shown in Figure 69(b). In such cases, the grouting will be done as if for 3 rows (Mistry, 1988, Shroff 1999).

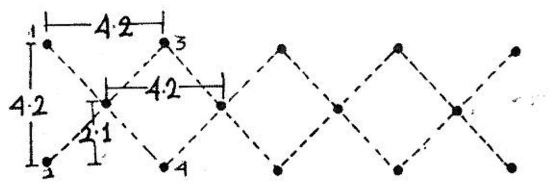
The typical patterns of the grout holes drilled at Ukai dam in Gujarat, India are shown in Figure 64(b) (Mistry,



(I) Three rows staggered with equal spacing in outer rows and half spacing in the central rows



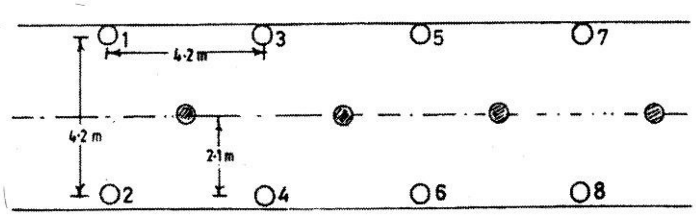
(II) Three rows staggered with equal spacing



(III) Three rows square pattern with equal spacing

Fig. 64 (b) Pattern of Holes-
Three Rows Staggered with Equal Spacing and Square Pattern

Shroff, 1977). The grouting pattern adopted included: (i) three rows staggered with equal spacing in the outer rows at 6m and half-spacing in the central row, (ii) three rows staggered with equal spacing at 6 m and (iii) three rows with a square pattern and equal spacing of 4.2 m. The distance between each row in (i) and (ii) was 1.5 m while in (iii) 2.1 m was adopted. The sequence of grouting adopted is depicted in Figure 64b. It can be seen that the square pattern with a check hole at the centre (Figure 65) was more effective than the staggered pattern.



Numbers denote grouting sequence; hatched circles are check holes

Fig. 65 Square pattern with
central check holes-Grout Curtain

Position of Grout Curtain

The type of dam exerts a strong influence on the optimal position of grout curtains occasionally; special geological considerations warrant a departure from the standard position or may warrant relocation of the dam to fit the grouting location. The standard positions for grouting curtains are at an embankment dam with an earth core (or of the homogeneous type) a curtain, if used, should be located upstream of the core axis, but not so far upstream that an insufficient length of the impervious fill foundation remains between it and the stored water influence (Houlsby, 1983).

The usual positions, ranging from a vertical curtain at the axis to a steeply inclined one commencing one-third of the way between the axis and the upstream toe of the dam. In a gravity dam, both are commonly drilled from a gallery within the dam. Some arch dams likewise make use of a gallery for this purpose. In small Arch dam grouting is carried out from the rock before concrete placement. At an arch dam the curtain must pass through that part of the foundation which remains more or less compressed during the flexure cycles of the dam.

Depth of Grout Curtain

The depth of a curtain is determined from considerations of the seepage characteristics of the foundation. When the fissure pattern is so erratic that no rational analysis is possible, the depth in rock is often established by empirical procedures, such as depth equal to 0.5 H to 1.5 H or 0.33 $H + C$, Where H = head on foundation and C is a constant equal to 25 feet (7.5m) in sound rock and 35 ft (10.5m) in fissured rock.

Blanket grouting with curtain: In blanket and curtain grouting for seepage control beneath dams on rock foundations, usually the depth of holes for blanket grouting is one fourth of design depth of curtain (6 to 12 m). Grout pattern typically used in fractured rock where upper surface is highly fractured is as per sequence shown in Figure. 64a with a difference that if Primary 'P' hole-depth is 'D', where , 'D' is design depth of curtain then secondary at 0.75 D, Tertiary at 0.5 D and Quaternary hole at 0.25D (Shroff 2003). Conventional closure pattern for curtain drilling and grouting is followed (Figure 65) (Shroff, 1999). Post grouting permeability should be 1/100 th of original permeability.

Optimization of Grout Curtain Dimensions-Cut off efficiency

Quantitative Estimation of cut off Efficiency Based on Piezometric Measurement: Cut-off effectiveness may be assessed quantitatively in terms of the head efficiency. Head efficiency $Eh = h/H$, where H = the difference between RWL and TWL or GWL at the d/s toe or piezometric level in foundation at the d/s toe; h = the difference between the piezometric level in a foundation just u/s and d/s of the COT bed. The piezometric level at the d/s toe ideally should be at such level as to ensure a safety factor of, say, 2.5, against blowout, boiling or heave.

Analytical Estimation Based on Flow Efficiency: The efficiency of an imperfect cut-off can be defined by the ratio $E_h = 100(Q_o\text{-}Q/Q_o)(\%)$, where Q = discharge per unit length through imperfect cut-off and Q_o = discharge that would have taken place had there been no cut-off. Zero efficiency means the cut-off is completely ineffective, while with an efficiency of 100%, there is no flow of water through the cut-off. The smaller the ratio of B/D, the greater the effect of the width of the cut-off efficiency, where B = width of the cut-off and D = thickness of the previous stratum below the dam. The value of the ratio B/D for earth dams is usually large and hence, if the cut-off below the dam is to be reasonably effective it has to be fairly wide.

Efficiency by Loss in Head Method: The loss in head due to the cut-off, expressed as a percentage of the total loss in head across the dam, was used as the measure of cu-off efficiency. For the two dimensional problem, the loss in head beneath the dam may be considered in the zones: (1) Upstream zone of essentially parallel flow, of length L_1 head loss ΔH_1; (2) cut-off zone of length L_2 parallel flow - head loss ΔH_2; (3) Downstream zone of essentially parallel flow of length L_3 -head loss ΔH_3; k – permeability of the material forming the pervious layer and k_1 the permeability of the grouted cut-off, both assumed to be isotropic, then the efficiency with respect to loss in head, h is equal to.

$E_h = I / \{I + (k_1/k)\}\ [B/L_2 - (1 + 0.88\ D/L_2)]$, numerical values can readily be inserted to indicate the magnitude of cut-off efficiencies likely to be encountered in practice.

Efficacy of Cut-off: Consideration should be given to the effect of mainly three variables-depth of penetration, degree of imperfection and thickness of the cut-off. The degree of imperfection is defined by the ratio of cut-off permeability (k_1) to the foundation permeability (k). The installation specification (represented by k_1 or k) required to achieve a specified level of performance becomes less demanding as the thickness of the cut off is increased. The interdependence of the cut-off parameter *B/D* and k_1/k_2 is therefore clearly apparent.

Compared to the first method, E_h gives only an approximate assessment since the quantity Q_o can not be directly established. One has to rely on flow measuring devices positioned downstream of a dam in assessing the quantity Q. Contrarily E_h is directly determinable on the basis of field piezometric data.

Cut-off Effectiveness Based on d/s Blow-out or Heave: Perhaps the measurement of uplift pressure upstream and downstream of the cut-off would be a more convenient method for expressing the efficiency of a cut-off in terms of its direct measurement. The magnitude of the maximum uplift gradient at the landward side of the dam and the factor of safety F against uplift failures can be calculated as the ratio of the downward forces acting on a column of soil to the upward forces: $D(G-1)/H(1+e)$, $h/D = i$, exit gradient at the toe, where D= depth of pervious foundation, h= head loss across cut-off measured at dam/foundation interface at d/s. When a weighted filter, loading berms, relief well are used, the factor of safety with respect to heave is increased because the total stress is increased whereas the pore pressure remains constant. Normally, the factor of safety should be at least 2.5 to allow for the uncertainty in predicting piezometric heads at critical points.

Optimization of Grout Holes for Tunnels and Underground Openings

Fan Array from Successive Headings

A fan type pattern of grout holes was so designed as to ensure the penetration of the grout mix in the space along each tunnel tube. To ensure the proper intersection of fissures the spacing of grout holes was so selected that the stage distances were not larger than 2 m. Similar pattern is used in subway crown and sides in Karchiya yard reaching to Reliance Petro-chemical complex, Vadodara, Gujarat (Figure 66a)

Parallel Array from the Surface

Vertical pattern of grout hole are used for the Prague subways in order to eliminate problems of settlement of structures. Different grouts were used in different soil strata. The pressure of grouting about 35 per cent of the grouted volume of original soil (Figure 66 b)

Inclined Array of Grout Holes

During the construction of the Delhi Metro, it is recommended inclined grout holes to intercept the inclined layer of pervious zone above the tunnel. The case is discussed later on part-III (Figure 66c)

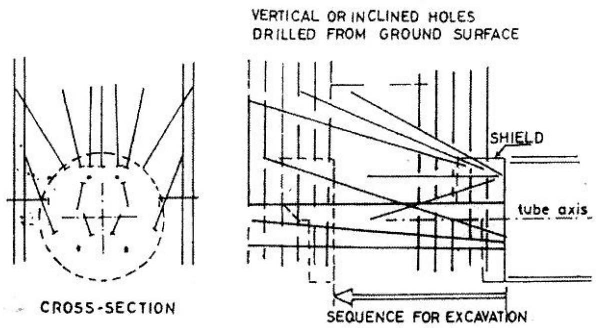


Fig. 66 (a) Fan Array from Successive Heading

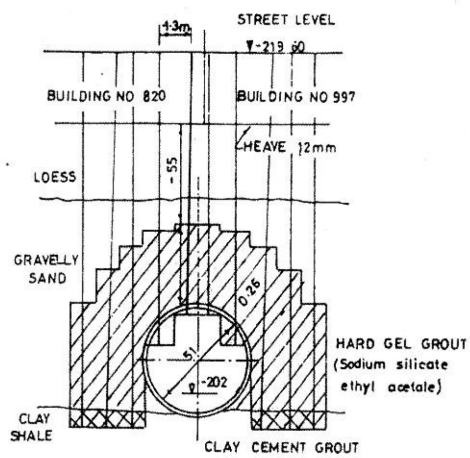


Fig. 66 (b) Parallel Array from Surface

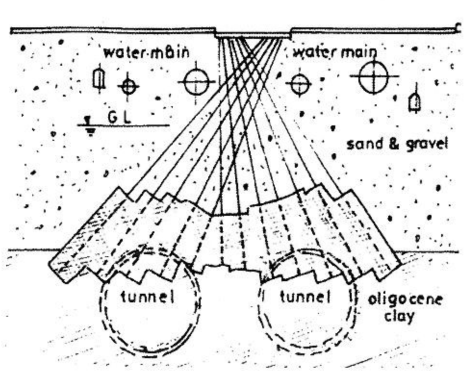


Fig. 66 (c) Inclined Array of Grout Holes from Pit

Staggered Array of Grout Holes

Grouting is from street level as well as from within the rail road tunnel. Surface holes can be arranged in two staggered rows on each side of the tunnel, spaced 1.5 m transversely and longitudinally, and a 38 mm diameter pvc pipe with grout ports spaced 0.75 m vertically can be inserted in the hole. The grout holes drilled from within the tunnels is generally of 51 mm diameter and fitted with 32 mm diameter pvc pipe. The tunnel holes are arranged in grout 'fans', with 'fans' spaced 1.5 m longitudinally along the railroad tunnel and likewise drilled to crown level or into the residual soils. Figure 66 (d). Any chemical grout with a stiff gel can be used. Similar pattern of holes is used at Baltimore Ohio rail road tunnel.

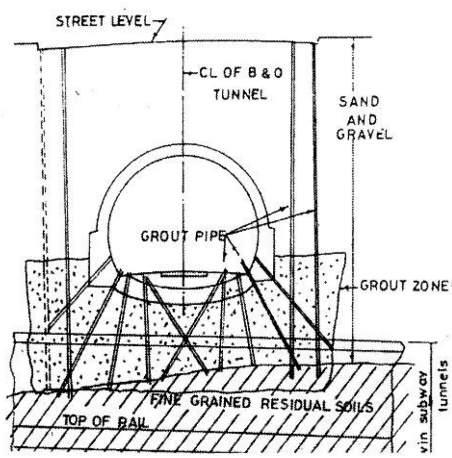


Fig. 66 (d) Staggered Array of Grout Holes

Optimization of Grout Holes during Sinking of Shaft

In tunnel grouting, to prevent caving often only half circle need to be grouted to solidify an umbrella above the excavation. In shaft grouting, complete circle must generally be grouted either for strength or water cutoff for seepage control. The low viscosity grouts in zones of high static ground water pressure as well as the use of two- row grout pattern are the optimal requirements Figure 67(a). Grouting pattern with holes on 1.2 m centres with a volume of grout to provide a 1.5 m stabilized diameter on even numbered holes and 1.2 m stabilized diameter on odd numbered holes. Holes (were not vertical) were drilled with radial dip of 1 in 10 and spin of 20° Generally three series of grout holes radially at different radial distance from sump is carried out with different grouts to prevent heavy seepage from fissures of rock at some level from ground level during of shaft sinking, Figure 67(b) (Shroff, 2006), enveloping of three series of set grouts help prevent seepage with properly grout design.

Grout Pressure

Various Imperial Concepts

> Allowable Injection Pressure = Overburden above the section being grouted,(Grundy 1957, North American Practice)

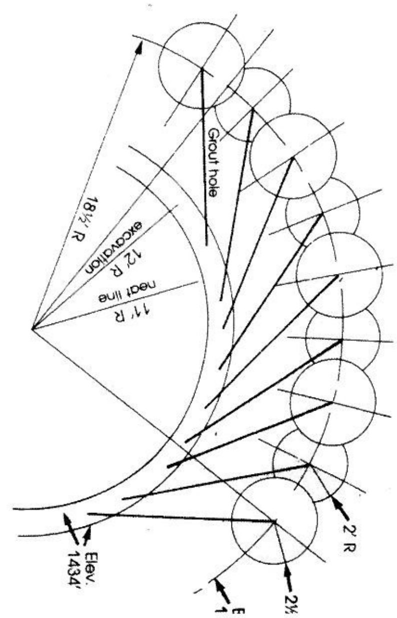


Fig. 67 (a) Shaft Grouting Pattern

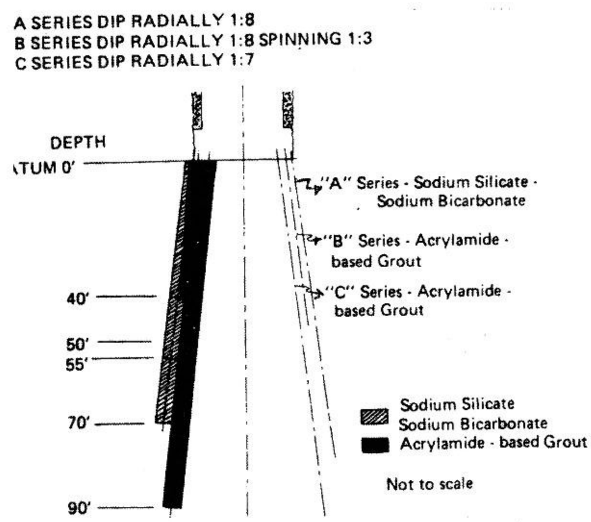


Fig. 67 (b) Shaft Grouting Pattern with Stages

> Allowable Injection Pressure = ¾ psi per foot of overburden (Lippard 1958)

> Allowable Injection Pressure = 4 times overburden (Italian Dam Practice)

> Allowable Injection Pressure = 5 times overburden (European Practice)

> Allowable Injection Pressure = $P/\gamma h = (1 + c' / \gamma h) \cot \phi'$ if horizontal planes are the dominant planes of weakness Zaruba *et al.* (1962).

Indian Standard Practice

It is always advisable to begin with a low initial pressure, say 0.1 to 0.25 kg/cm^2 per meter of overburden, and build up pressure gradually.

Control of Pressure: It can be exercised by adopting the following means singly or in combination: The results of trial grouting, along with observations of upheaval by an uplift gauge. Figure 68 (a) may be used as a guide, subject to verification by trial grouting. Pressure limits can be decided by analysis of the results of cyclic percolation test. Limiting pressure can be decided by continuous review of the trends of pressure and the rate of intake during grouting operations along with observation of leaks and movement of uplift gauges.

Australian Practice

The well known rule of thumb of 1 psi per foot depth is usually a happy compromise for average to weak rock. Pressure can be doubled for sound rock (Houlsby, 1983).

The hydrostatic aspects of grout pressure are explained by Houlsby (1981) as shown in Figure 68(b).

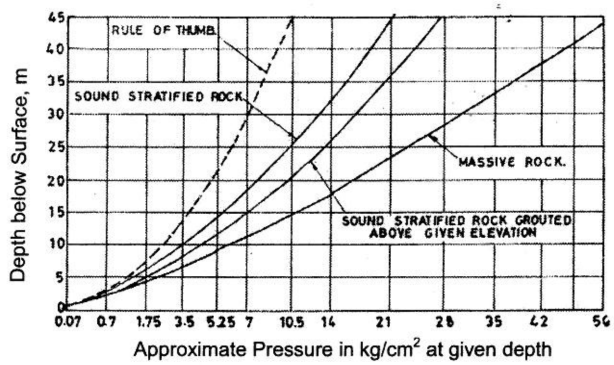


Fig. 68 (a) IS Guide For Grout Pressure

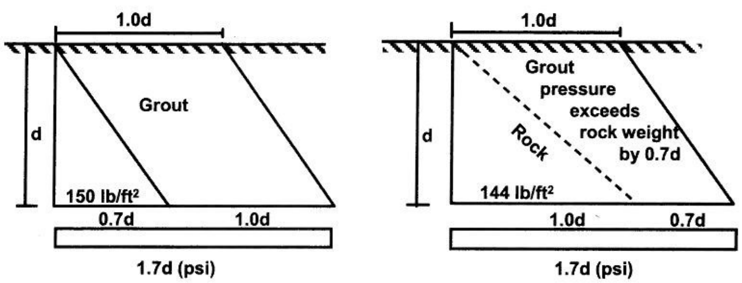
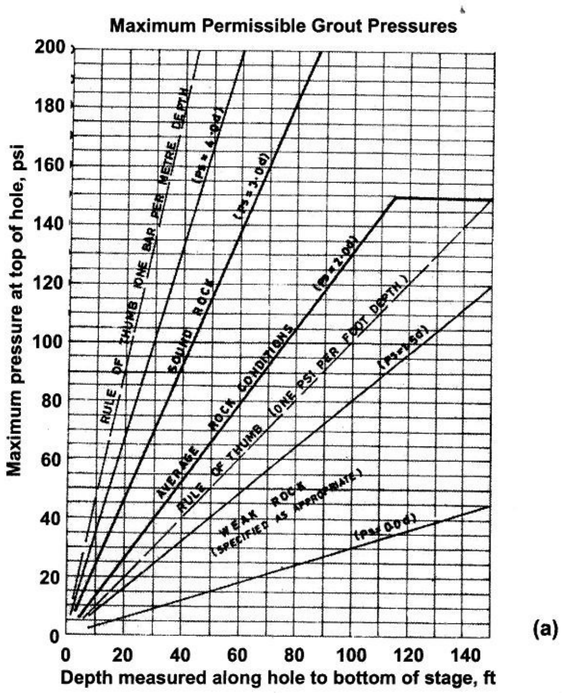


Fig. 68 (b) Hydrostatic Aspect of Grout Pressure



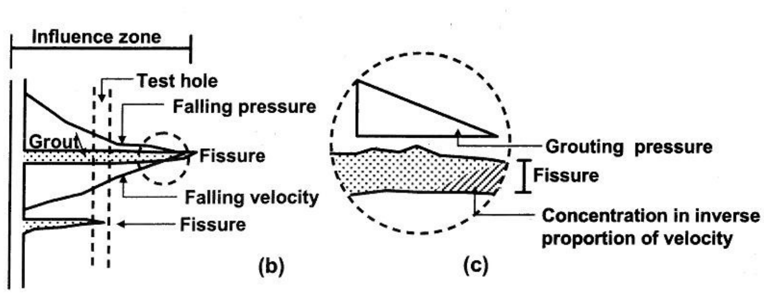


Fig. 74 (a) Maximum Permissible Grout Pressure Australian Practice (b) Grouting with Unstable Grout (c) Enlarged View of Falling Grout Pressure

Figure 74(a) has been adapted with an upper limit of 150 psi (10 bars) applied for average conditions. The critical injection pressure would be given by $P = P_e + \gamma_w h_w$ The common specification of 1 lb/in^2 per foot depth applies to a material with no tensile strength and a principal stress ratio of unity. The pump cannot exceed the overburden pressure unless the material has some cohesion. The existence of stratification, jointing, and changes in tensile strength with depth can be considered. Figure 74(b) and (c) depict diagram of the drop in pressure with distance from the grouting hole in the fissure & it reduces to zero towards the end of the influence zone. Since no useful approach can attempt to account for all these factors, the most reliable method for finding the allowable injection pressure will be based upon field tests.

Such tests are discussed in a subsequent section. The hydraulic fracture test is a useful method for determining the allowable grouting pressure if fracture is to be avoided.

The concluding part of this paper and the references will be included in the next issue 40(2) 2010.