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Developments in Design and Execution in Grouting Practice[1]

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

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.

The fist part of the paper appeared in the last issue of the journal 40(1), 2010. The first part dealt with classification of grouts, various types of grouts and their applicability and few testing methods. The second part of the paper deals with the plant and machinery used for grouting, monitoring efficacy of grouts, efficacy of grouted mass, deep mixing techniques, get grouting methods and applications, compaction grouting technology, etc.

Case studies in deep mixing (jet grouting) and compaction grouting technology adaptations are discussed briefly in this part. The paper also deals in details the permeation grouting technology and it's applications. Case studies of curtain grouting are briefly discussed.

Part 2
Grouting Plants and Specifications

The mixing plants and delivery systems for suspension and solution grouts differ mainly in their storage and mixing configurations. A typical cement suspension grouting plant is shown in Figure 75.

The basic items of a grouting plant and their functions are as follows:

> Mixer to mix the grout ingredients and then discharge them to agitator

> Agitator that stir the grout and keep it ready for the pump

> Pump that draws the grout from the agitator and pumps it through the line

> Circulation line that commences at the pump, leads to the grout hole and then discharges unused grout back into the agitator

> Control fittings located at the grout hole, these control the injection rate and pressure so that the hole can be regularly bled of water and thin grout.

For all the three types of grouting, drilling methods, the depth of holes, grouting pressures and mixes might vary but the grouting equipments and techniques remain the same.

For relatively small jobs, the individual items can be combined into a single compact unit. Automated, computer controlled and monitored equipment is economically feasible in large scale projects.

Grout Mixer

There are high speed mixers and paddle mixers. The basic differences between them are the rotation speed and the type of mixing impeller. The high speed mixers operate at 1500 to 2000 rpm while paddle mixers usually rotate at a speed of 100 rpm or less. The paddle mixer uses a stirring action while the high speed mixer uses a violent shearing motion in conjunction with a vortex wetting each cement grain promoting thorough hydration. The resultant grout is commonly called 'colloidal' and can penetrate well. High speed mixing is an essential requisite for obtaining effective cement grouting in rock. The applications where the paddle mixer is superior include compaction grouting, where the mix is relatively thick. There are colcrete mixers manufactured in India.

Grout Agitator

During the period between the time of grout mixing and the time it enters the pump, it must be agitated continuously to prevent setting. This is accomplished by an agitator sump between the mixer and the grout pump. An agitator sump is usually a cylindrical tank approximately 1 meter high and 0.75 m in diameter. Within the tank is an agitating mechanism consisting of a vertical shaft to which horizontal blades are connected, which rotates at a speed of 30 to 100 rpm.

1 31st IGS Annual lecture delivered at IGC 2009, Guntur. Concluding Part.

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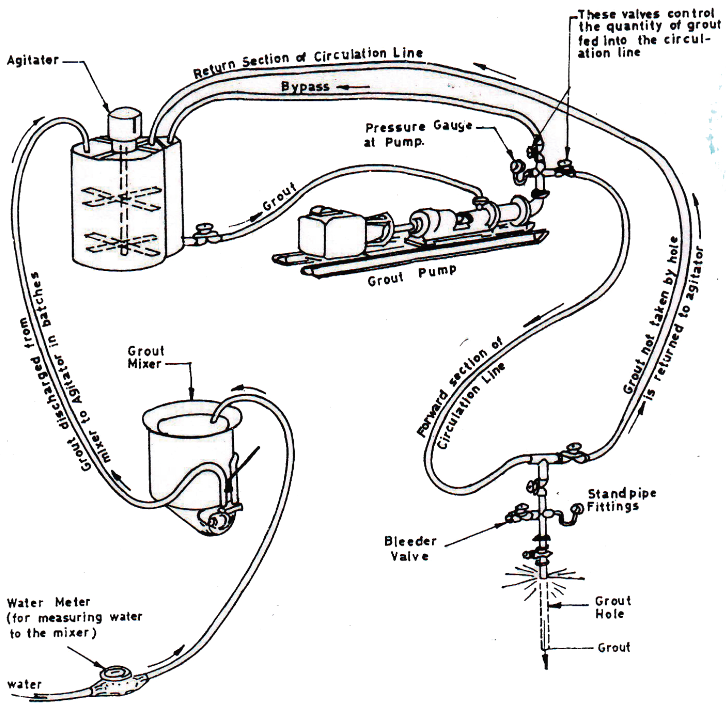


Fig. 75 Layout of Grouting Plant

Pumps for Cement based Grouts

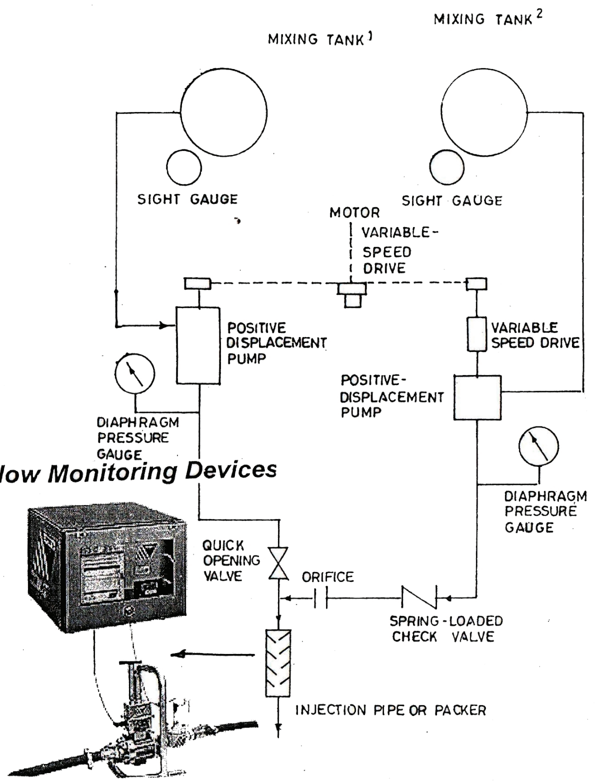
Helical Rotor Pump that has a chrome plated rotor within double internal helix provides a pulse free flow of grout with the help of a resilient stator. Duplex and Triplex pumps have less pronounced pulsation. But for some applications an accumulator is used to further 'smoothen' the output.

Hydraulic Pumps are the best suited for foundation grouting works with the following advantages. (i) these pumps have independent control of pressure and flow. (ii) These can be set at a predefined pressure and the output pressures can exceed 100 bars.

General Observations on Grout Pumps: For applications involving the injection in fissured rock, such as curtain grouting for dams or contact grouting program in tunnelling, there is a distinct preference for ram type pumps. It is believed that the pulsating flow characteristics mentioned above give better results by allowing a longer period of injection in fine fissures before eventual refusal.

Control Units

Metering System: Figure 76 presents a line diagram of the major components of a chemical grout metering system along with instruments to indicate actual discharge flows from each pump and a pressure actuated shut off system to prevent inadvertent over pressuring of the formation. Glass tube flow meters or computerized flow monitoring devices (Shroff, 2004) are used on the discharge end of each pump to give continuous record of discharge volumes. Metering is accomplished by adjusting the pump outputs to the desired values or ratios. For electrically driven pump units, similar controls are available to shut off and restart the motor. More sophisticated mix/metering equipment may be used to allow the resin and hardener elements to be pumped separately to the injection nozzle or to special mixing heads.



Flow Monitoring Devices

Fig. 76 Line Diagram for a Computerized Metering System

Grout Valves

Lubricated plug cocks can be used for pressures above 200 psi (14 kg/cm^2) while Saunders type valves are in common use for pressures below 14 kg/cm^2. The valves and fittings should be of stainless steel on the discharge side of the catalyst pump used for chemical grouts. Plastic valves and fittings (compatible with the catalyst) may be used at the suction end.

Standpipe Fittings with Manifold: Standpipe fittings are used for controlling the grout pressures at the hole, measuring the grout pressures, closing off the hole temporarily when grouting is completed.

Upheaval Gauge

The foundation has a tendency to heave upward under high pressure grouting. Continuous observations of the ground heave are made and the grouting is immediately stopped if any significant upheaval is observed. The allowable upheaval is 5 mm in earth dams, while 0.25 mm for the formation of masonry/ concrete dam. Figure 77 presents a system of measuring upheaval.

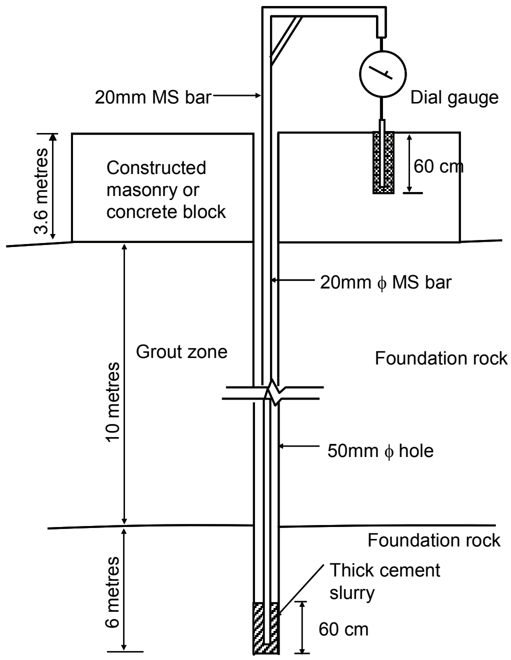


Fig. 77 Upheaval Gauge- Typically Suitable for Dam

Monitoring and Efficacy of Grouting

Refusal Criteria

Refusal is considered to have been reached when the grout flow is less than 2 liter/minute averaged over a period of 10 minutes at the desired limiting pressure greater than 3.5 kg/cm^2. The refusal flow rate is one liter/minute when the pressure is less than 3.5kg/cm^2.

Grout Project Control Curves (Soil or Rock Response to Injection)

During injection, the flow rate of a properly mixed grout should be constantly monitored and plotted against the grout pressure to understand what is happening below ground. Typical response curves in the form of flow rate versus pressure and flow rate versus time are shown in Figure 78.

1. For permeation or penetration grouting, the flow-rate versus pressure and flow rate versus time curves are parallel to each other (resulting in a nearly linear pressure versus time response). The flow rates depend upon the porosity of the soil or the sizes of the crack openings in the rock to be grouted and always decreases with time since the voids are being gradually filled with the grout (Figure 78(a)).
2. For compaction or controlled displacement grouting, the response is very similar except that generally lower flow rates are to be expected. These rates decrease with time as the in-situ formation being grouted becomes denser (Figure 78(b).
3. For hydro fracturing or uncontrolled displacement grouting, the responses are markedly different from the other two functions. Here a limiting pressure is always arrived at where the tensile strength of the soil or rock fails and a lens of grout shoots into the vacated space (Figure 78(c)).

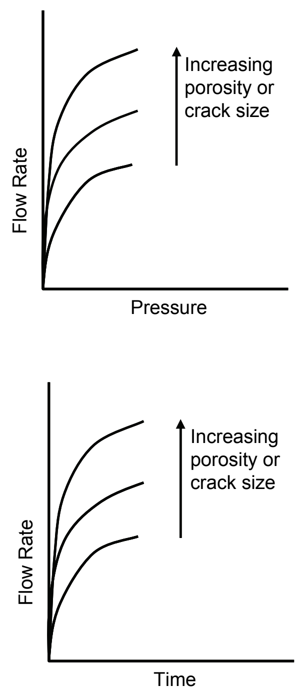


Fig. 78 (a) Response of Soil or Rock to Injection - Permeation Grouting

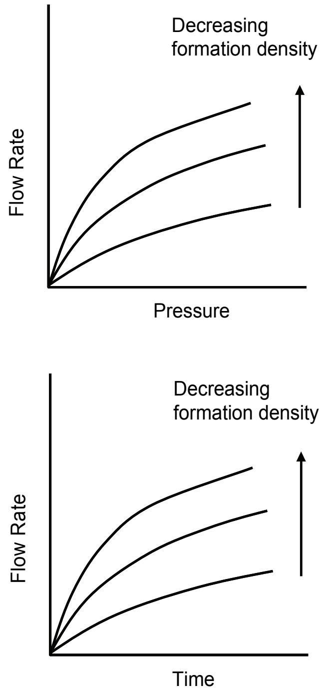


Fig. 78 (b) Response of Soil or Rock to Injection - Compaction Grouting

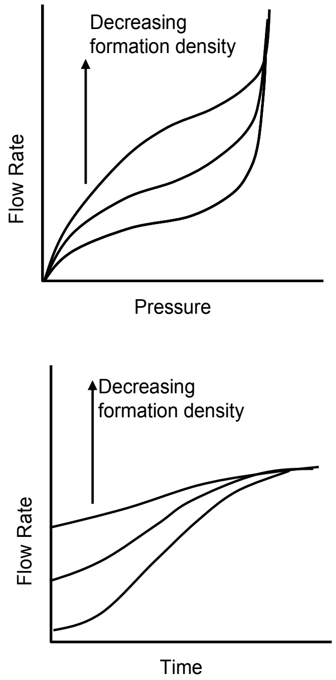


Fig. 78 (c) Response of Soil or Rock to Injection - Hydro-fracturing

Time Pressure Consumption Graph (TPC Diagrams in Figure 79)

The behaviour of a grout hole has been classified in six broad categories based on time-consumption and time-pressure data (Mistry, 1988, Shroff, 1999).

The classifications are as follows.

1. *Consumption drops and pressure remains constant after peak (Figure 79 a)*. After reaching the maximum pressure, a few minor cracks might open out which are again filled up. The pressure curve is approximately parallel to the time axis while the consumption at a nearly constant pumping energy goes on decreasing. This is the ideal pair of curves.

2. *Pressure increases and consumption remains constant after peak (Figure 79 b)*. This pair of curves shows that the pressure slightly falls due to opening of the cracks. After filling in the cracks, the pressure rises and the rate of consumption remains constant. If the rate of consumption is within permissible limits, the grouting may be stopped. If the pressure achieved is more than specified, the operation may be continued at a suitably reduced pump speed. This is also an ideal pair of curves.

3. *Pressure and consumption remains constant after peak (Figure 79 c).* At the peak pressure, the grout continues to ravel in the cracks unchecked. In this case the grouting operation may be stopped after injecting a certain quantity of grout, say 50 to 75 kg of cement/meter. The injected grout left to set. Grouting may then be resumed after a lapse of about 48 hours, when the cracks will have been partly sealed by the setting grout.

(a)

(b)

(c)

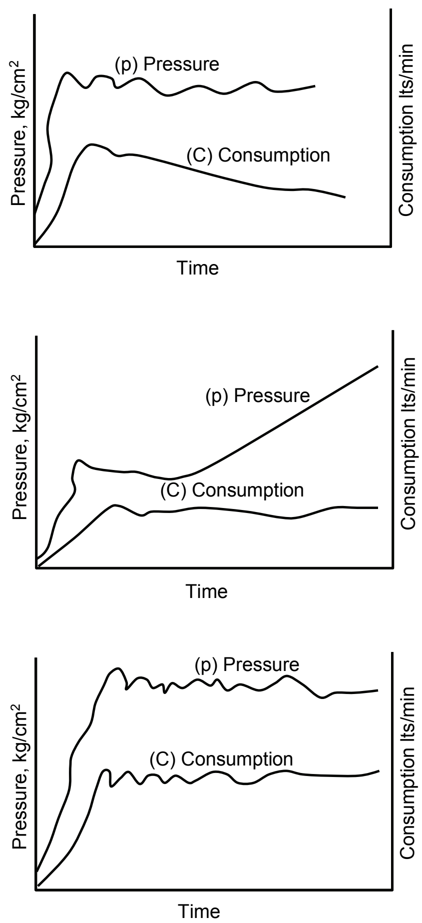


Fig. 79 (a), (b) & (c) Time-Pressure-Consumption (TPC) Monitoring Diagrams-Behaviour of Grout Holes

4. *Pressure and consumption rise to some value and then fall rapidly (Figure 79 d)*. This pair of curves indicates the opening out of new cracks at peak pressure. On filling of these new cracks, the pressures and consumptions remain constant throughout, indicating that at the second peak pressure the grout travel is continuing uncontrolled. In this case also, the grouting operation may be stopped after injecting a certain quantity of grout and then resumed after 48 hours.
5. *After a rise in pressure to a certain value, the pressure remains more or less constant, while the rate of consumption goes on rising (Figure 79 e)*. This pair of curves reveals abnormal behavior in the hole. They indicate that there may be leakage of grout through natural strata or along the hole or some upheaval in the rock strata. Immediately on locating the leakage point, it should be plugged by excavating a small pit around it an filling this with lean cement/concrete. If there has been any upheaval; grouting should be stopped forthwith and grouting resumed after 48 hours with pressure sufficiently reduced.
6. *After reaching to a certain value, the pressure quickly drops while rate of consumption remains constant (Figure 79 f)*. It also shows abnormal behaviour of hole. There may be leakage of grout through natural strata or along the hole or same upheaval in rock strata. The procedure outline in (5) should be followed.

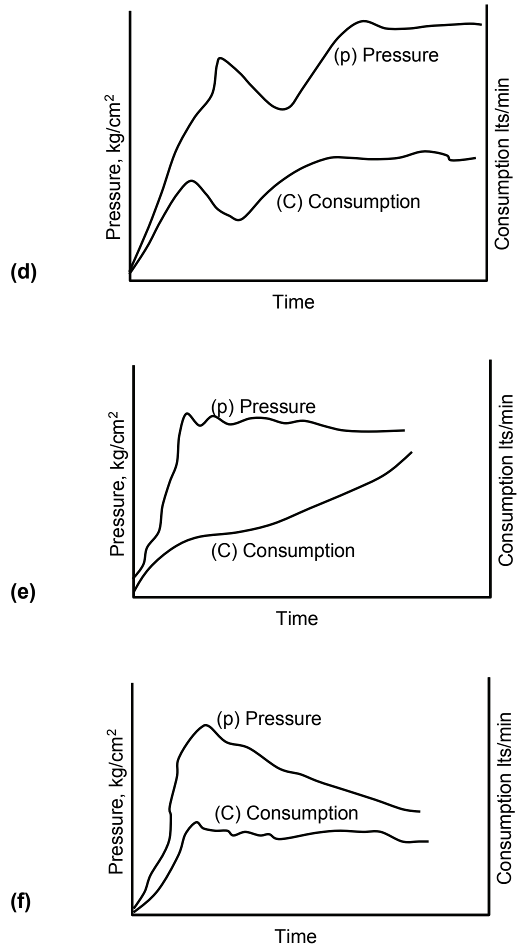


Fig. 79 (d), (e) & (f) Time-Pressure-Consumption (TPC) Monitoring Diagrams-Behaviour of Grout Holes

GIN (Grouting Intensity Number) Process

GIN is a grouting method for rock as suggested by Lombardi *et al.*, 1993. This method is based on modeling the rock by the FES system (fissured elastic saturated rock system). In this model, the rock body is characterized by joint opening in the direction perpendicular to the principal discontinuities, and by the effective stress field at these discontinuities. This makes it possible to represent the permeability of the rock to be grouted by the spacing and properties of these fissures and the general state of stress. The GIN method defines a number, called the Grouting Intensity Number, which is the end injection pressure multiplied by the injected grout volume. This co-efficient is proportional to the energy employed in grouting the rock. When the GIN coefficient is kept constant during grouting, grout penetration is steady and the volume in open fissures is automatically limited, while the pressure can rise in finer fissures that are harder to penetrate. GIN indicates that limit pressure is lower when the volume of unset grout injected is large. Figure 80 shows GIN limit (pressure - volume) curves.

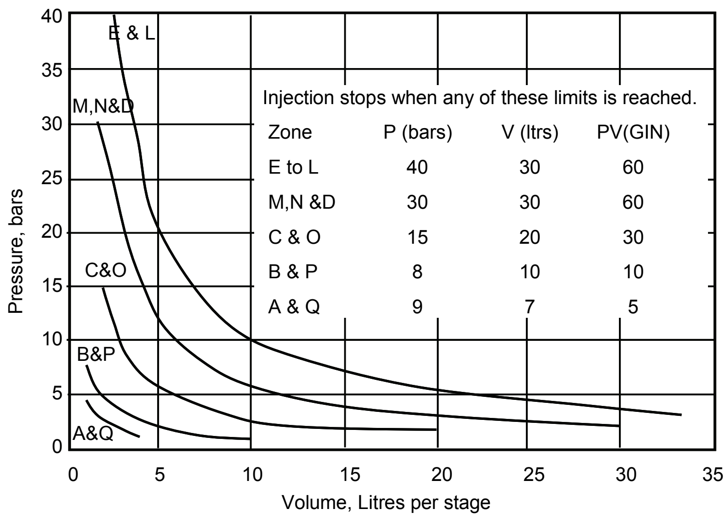


Fig. 80 GIN Limit Curves (Pressure Volume Curves)

From grouting field practice, it seems that time parameter is very significant while considering pressure-grout consumption relationship.

New TPC (Time-Pressure-Consumption) Number/System (Shroff *et al.*, 2004)

Time-Pressure-Consumption diagrams during grouting conceived by Mistry 1985 and further modified by Shroff (2004) bears several important features of grout hole behavior.

TPC Number is devised from field studies at several grouting sites. It is defined as a number, called time Pressure-Consumption (TPC) number which is the product of injection pressure during or end of injection and the injected grout (Volume) at that particular time divided by the time of injection. Its unit is kg. m. per unit time for unit depth of hole. This co-efficient is proportional the power (energy) used in grouting the rock or alluvium at that time. This may vary as per joints fissures orientation and their width in rock and tortuosity of grout flow in alluvium. When TPC value is reached, grouting is completed satisfying refusal criteria. When TPC is remaining constant during the grouting, grout penetration is steady and the volume in open fissures is automatically limited, while the pressure can rise in finer fissures that are harder to penetrate, if one crosses this number, the hydro fracture may initiate. It signifies that a limit pressure and limit volume must be with reference to rock joints or pore size.

Micro Fine Grouting along with newly devised TPC system author has put forth micro fine grouting process. Micro fine process consists of grouting the soil in a single operation with very thin mix comprising micro fine cement, or colloidal Silica components which combine with the in situ soil to form hard set mass. Time-viscosity and Time strength study of above grouts are co-related to TPC number which is discussed earlier.

Dam Grout Data Processing System

The Dam Grout Data Processing System is an advanced grout control systm that comprises flow and pressure detection unit, flow and pressure recorder unit, data processing unit, daily report preparing unit along with grout mixer and grout pump (Figure 81).

Flow and Pressure Detection Unit

This control system is designed to return the injection flow when the pressure or flow rate exceeds the limit values in order to avoid any dangerous effects on the base rocks or structures. The main features of this unit are:

- This unit is light weight compared to the conventional ones.
- Ceramics are used for materials of the return valve head and sheet so that abrasion resistively has been much improved.
- Throttle can be made arbitrarily on either on the bore hole side or on the return side so that the flow control in low flow rate is easier.
- The mixer can be installed either at a higher or at a lower position than the entrance of a borehole, depending on the layout of the site.
- The return valve is easily decomposable by the use of a one touch coupler.

Data Processing Unit

Features:

- Rapid and reliable data management is possible and directions for operation are available.
- Real-time calculations and display of PQ curve and Lugeon-value are possible at the construction site.

Daily Report Preparing Unit

Features:

- It generates operation instructions.
- Immediate generation of a stage wise daily report in the form of tables.
- Rapid generation of various statistical tables such as correlation, probability etc.
- Generation of various analysis diagrams including Lugeon maps.

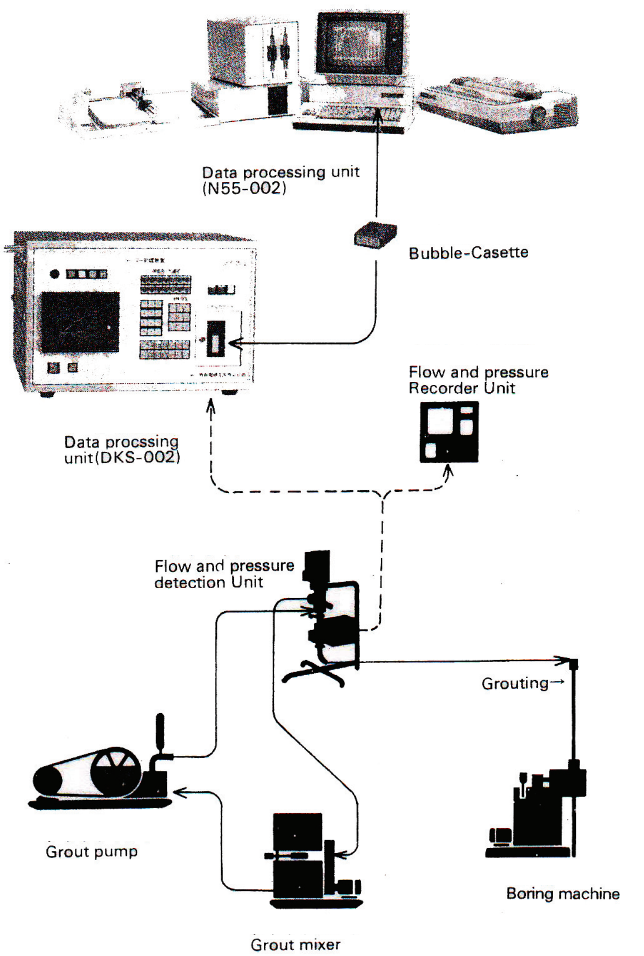


Fig. 81 Dam Grout Data Processing System (Computerized Grouting Plant)

Software Developments

The system uses soft wares like Saphyr for water tests, Enpasol for reconnaissance, Castaur for drilling pattern design that produces worksheets, Sinnus 3 E for grouting control and finally Chairloc for job analysis.

The real time display of grout pressure versus flow rate is of great assistance because the pressure can be increased monotonically provided the x–y plot remains linear or the pressure increases faster than the flow rate (turbulent flow). The limiting pressure desired is just prior to the hydraulic fracture, which appears as a line almost near parallel to the flow axis on the x–y plot. Grouting Engineer can control injection pressure to 80 per cent of the fracture pressure for normal injection by this system. Moreover, interpretation of the time plot of the flow rate record on the two channel chart recorder allows the grouting engineer to determine the required grout mix. Altas Copco, Haeny and Ciolcrete Eurodril are some of the manufacturers of the total grout controlling systems.

Efficacy of Grouted Mass

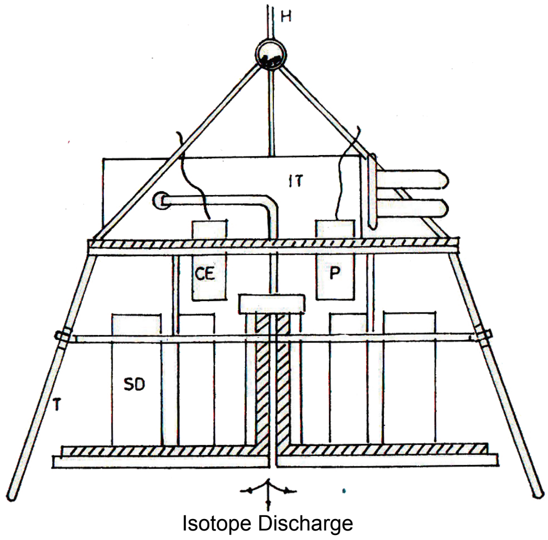
Consideration should be given to the effect of three main variables (i) depth of penetration (ii) degree of imperfection and (iii) thickness of the cut-off. Some new methods with compensatory accuracy and ease have been devised. Notable amongst them are (i) radioactive tracer, (ii) earth probing radar and (iii) cross-hole acoustic system.

Radioactive Tracer

It is possible to inject readily absorbable radiotracers through a tube inserted near the bottom of reservoir. The accumulation of adsorbed tracers at any point on the bottom is proportional to the amount of water which passed through that point. The detectors are deployed at points where seepage water has emerged. The electronic recorders give the number of isotopes collected at such points. If the grout curtain is watertight, the number of isotopes collected at a certain point will be less. A deep-water isotope current analyzer (DWICA) may be used for discharging isotopes, as shown in Figure 82 (a). It consists of a central isotope tank and is associated with a number of scintillation counting detectors placed peripherally around it. The entire device is lowered into the water by means of a hoist and the device measures both the direction and the velocity of currents.

Borehole Earth Probing Radar System

The system to be used for grout monitoring is the trans illumination mode in which the transmitter and the receiver are in separate boreholes. With the transmitter continuously sending pulses to the receiver, both are raised keeping them at the same level and recording the received signal and the depth of the two instruments continuously. The radar signal will be greatly attenuated, since most grouting materials are excellent absorbers of electromagnetic radiation. Areas which are poorly grouted will permit more signal to pass through the receiver. Now, quantitative assessment of the grout between two boreholes at any given level has become possible (Shroff *et. al.* 2008). (Ref Figure 82.b)



H-Hoist | CE- compass & electronics
SD-scintillation detector | IT-isotope tank
P-power supply | T-tripod

Fig. 82 (a) Deep Water Isotope Current Analyzer

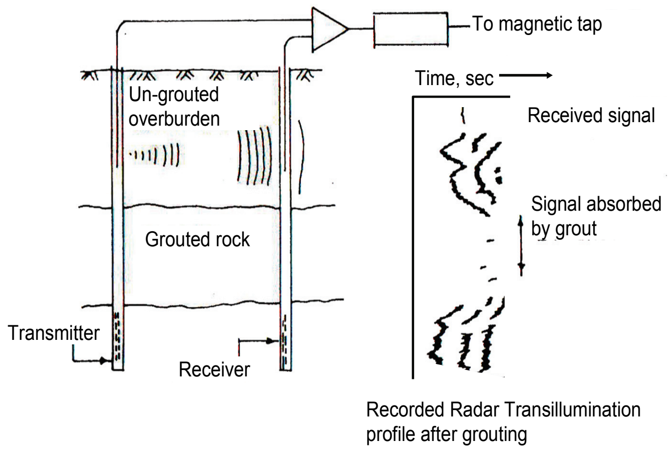


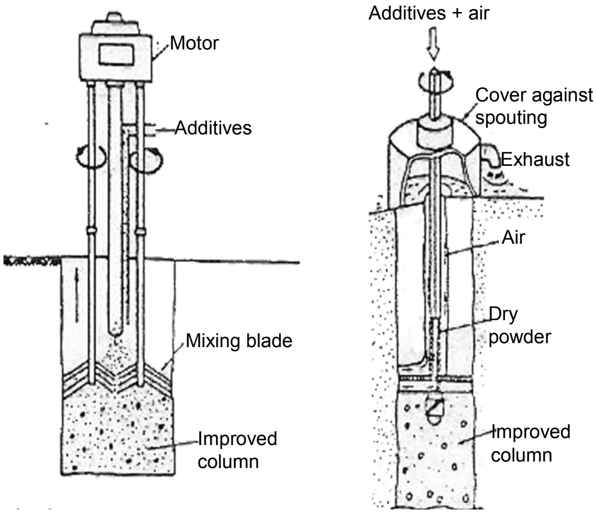
Fig. 82 (b) Pulsed Bore Hole Radar

Acoustic Cross-hole Shooting System

It is based on profile of acoustic velocity of the medium between two boreholes as a function of depth. Since acoustic velocity is a function of $\sqrt{E/\rho}$ (where E = elastic modulus and ρ= density of medium). Acoustic profile where a wave may travel indistinctly a curve path helps in assessing primarily the stiffness of the grouting medium, it may be desirable to use both systems to obtain the best possible indication of location and character of the grouted zone. Radar and acoustic velocity are techniques by which the grouting effort may be evaluated independent of the grouting operation itself.

Deep Mixing Method (Jet Grouting) Technology

The in situ mixing of stabilizers with soft soils to form columns, walls, grids or blocks in the ground has been developed and applied extensively in different construction sites since the 1970s. This method is abbreviated as Deep Mixing Method, DMM. Earlier, quicklime was used as the stabilizing material, but now Portland cement is more popularly used in Japan and other countries. The cement is used in both slurry state and dry powdered state. The two types of mixing methods, namely, deep mechanical mixing (DMM) and high pressured grout mixing (Jet Grouting) have been developed for mixing the soil with stabilizer under deep ground condition. These are shown schematically in Figures 83 and 84.



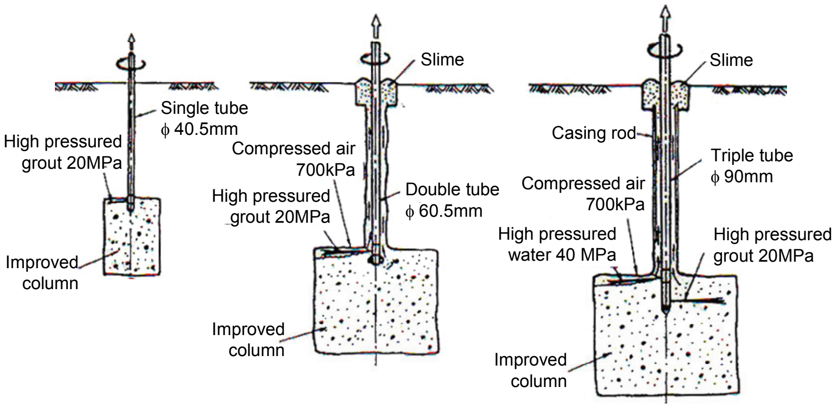
(a) Slurry State Supplying (b) Dry Powder State Supplying with Air

Fig. 83 Deep Mixing Method

Jet Grouting Technology

The technique termed as 'jet grouting', was first developed in Japan in the late '70s and was introduced for the first time in India in 1983. In jet grouting, the soil is mixed in-place with a stabilizing mixture under a very high nozzle pressure. It is possible to treat a wide range of soils by using this technique. Successful applications in weathered rocks by the use of simple cement grout mixed in place under very high nozzle pressure of 300–400 kg/cm^2 are reported.

The advantages of deep mixing method are the application to wide range of soils, capability to obtain Jet-grouted columns of consolidating soil with diameter ranging from 60 cm to more than 200 cm by using much smaller diameter drilled holes, (usually 100 to 140 mm), capability to overpass pre-existing foundations, boulders, rocky layers, etc. and the use of light weighing and small sized drilling rigs. The major applications of the method are underpinning, diaphragm walls, tunnel consolidation, bottom plugs, slopes consolidation, impervious Cut-offs, sealing of diaphragm wall gaps and break-in and break-out for TBM.



(a) Mixed by pressure grout (b) Mixed by pressure grout with compressed air (c) Mixed by pressure grout with compressed air and pressured water

Fig. 84 Jet Grouting Methods

'Jet grouting 1' implies the use of single fluid (the grout) as a fracturing medium and stabilizing agent, when single phase system is adopted. (Figure 84 a)

'Jet grouting 2'is the double fluid consists of the simultaneous high-pressure jet injection of grout and air. In the process, the *in-situ* soil structure is broken up completely by the very high-pressure grout jet, assisted by an enveloping air jet; the air jet increases the cutting radius and improves the workability of the soil grout mixture.

'Jet grouting 3' means the use of 3 fluids, air and water as the fracturing and washing media and grout as the stabilizing agent, when three phase system is used. In 'Jet grouting 3', soil is cut by cutting action of air-water jet from upper nozzle (4mm dia), at pressure of 300 to 400 kg.cm^2 and removed by water flow and is simultaneously replaced by cement grouting lower nozzle (7 mm dia) (at 40 kg/cm^2) pressure. This system is known as the three phase (air, water and grout) procedure (Figure 84 b.)

In three phase jet grouting process, the monitor, which comprises a triple phase fluid drill pipe, conveying three process elements of air, water and cement grout, is lowered in to predrilled hole. There are two nozzles (Figure 84 c) one above the other, separated by a spacing of 500mm. Generally, the cutting action of jet is effective up to a distance of 1.5m from the nozzle. The high pressure jetting grout in-situ soil structure breaks up and pushes out the destroyed materials through the annular space between the rods and the borehole while the grout mixes with part of the disturbed soils. The soil cement mix sets after some time to form a stabilized pile with nominal diameter of about 600–1400mm.

For grout injection rates above 500t/m, the triple-fluid system can form large diameter columns with less spoil generated than the single-fluid system. The excavating efficiency of the triple-fluid system is more because of the airlifting action of the compressed air/water cutting jets assisting the removal of cuttings from the drill hole.

The unconfined strength versus depth data plotted in Figure 85 indicates that the single-fluid system results in higher strengths than the other two jet grout methods. Below 6m, the single-fluid system produces soil Crete comparable to structural concrete (400 kg/cm^2). The triple-fluid creates the second strongest soilcrete (150–300 kg/cm^2. The double-fluid system produces the weakest soilcrete with most strength between 75 and 150 kg/cm^2.

Jet grouting is usually applied below the water level. Direct jetting in water results in significant attenuation which is not preferable. In an actual work, a longer cutting reach is of vital concern. Surrounding the water jet with a cylindrical film of air has proved to be solving the issue lowering the attenuation of jetting power.

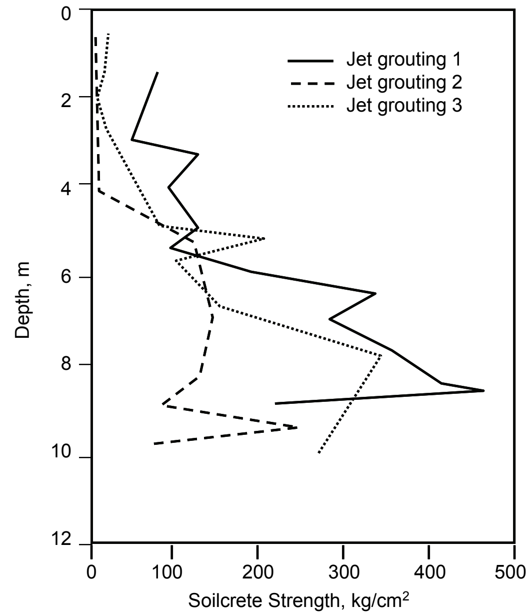


Fig. 85 Soilcrete Strength for Three Jet Grouting System

Deep Jet Grouting in Clay

During Jet grouting in marine clay (more than 35 m) to form a slab, the production process is separated in two phases. The first is cutting the clay by modified double system (super jet) with a grout of low cement content and afterwards filling the volume with a grout of high cement content which will satisfy the requirement of column with high strength in the treated soil. The permanent back flow of spoil (waste slurry) & its viscosity should be monitored to prevent heaving or settlement of the adjacent buildings.

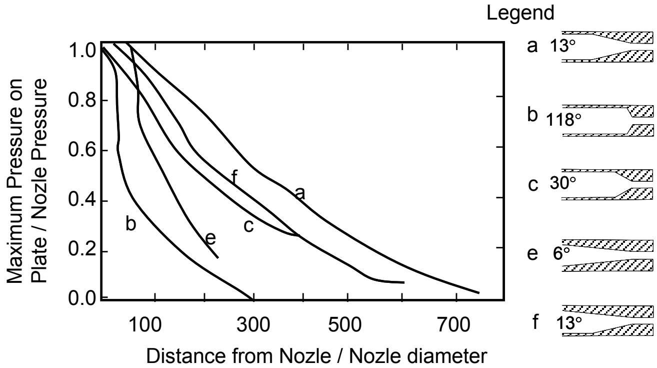


Fig. 86 Effect of Nozzle Shape on Performance

Theoretical Considerations

Jet cutting is predominantly affected by the nozzle characteristics. Subsequent research at M.S. University, Baroda, India made it possible to publish a general nozzle specification based on the following: (1) The nozzle should be made of the hardest material possible, e.g. tungsten carbide. (2) The internal surface should be mirror-finished. (3) The contraction angle should be 13° (4) The straight portion should be 2.5 to 3 times larger than the nozzle diameter (Ref Figure 86).

A nozzle satisfying the above conditions can maintain good efficiency. The upper stream condition towards the jetting nozzle is also of vital importance for achieving ideal jetting performance.

Both the experimental and theoretical results verify that the length of the straight portion generating a laminar flow in the upper part of the nozzle, significantly affects the jet performance. Specifically, the L/D should exceed 50.

New Concepts: A new concept was introduced in which two water jets are aligned to collide with each other and then artificially diffused to attenuate the kinetic energy which would otherwise continue.

By diffusing the kinetic energy of the jets, the concept succeeded in controlling the reach of the jets within the desired distance of improvement. The principle concern of conventional methods was only to enlarge the improved body, not to improve homogeneity which is dependent on the characteristics of the soil. Because two jetting streams always cross at an exact point, (Figure 87 a) the grout front constructs a nearly straight configuration as if within a

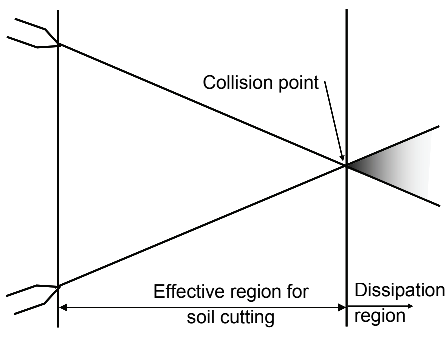


Fig. 87 (a) Collision Jet for Soil Cutting

virtual formwork. The many diamond shapes formed by intersection stream lines promise a better mixing of soil with cement slurry. Auxiliary equipment has been specially developed to generate a highly focused flow (Shroff *et al.,* 2007). The research concludes that increasing jetting energy alone will not always prove successful in meeting the improvement goals. The increase in cutting reach and acquiring an exact diameter and homogeneity are dependent on principles of fluid dynamics.

A combination of mechanical and jet mixing is recently introduced in practice along with collision of jets. Figure 87 (b) illustrates the concept of this method. The apparatus has grout nozzles at the tips of the mixing blades. The combination of mechanical mixing and dual colliding jet technology subsequently became practicable.

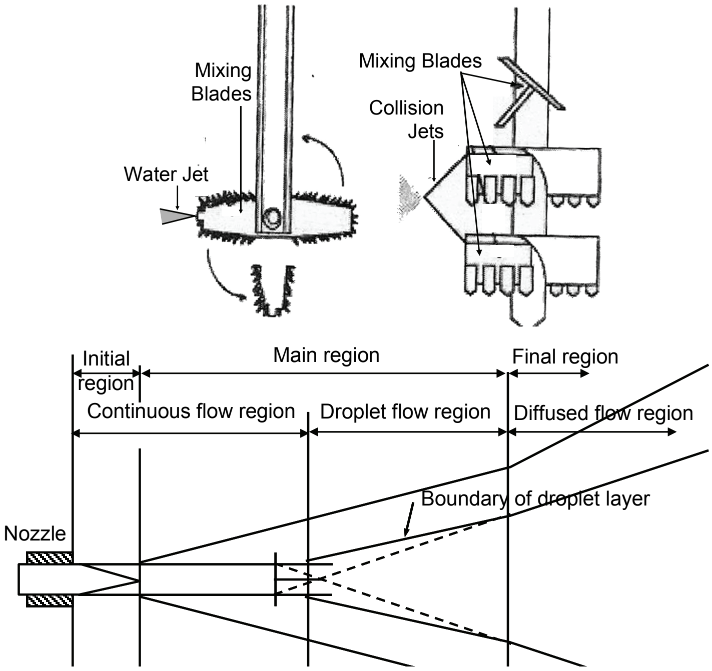


Fig. 87 (b) Mechanical and Jet Combined Method and Jet Structure

Case 1 Jet Grouting in Tunnel Construction with Weak Overhead Crown Materials

South Eastern Railways (S.E.R.) have undertaken the work of laying a 164 km single track connecting Koraput and Rayagadha in Orrisa State. The tunnel has a diameter of 6.09 m in heading portion and the overall height is 6.606 m. The total cross section area is 41.57 m^2.

While excavating the benching portion at Ch. 361, the material lying above the crown started collapsing into the tunnel and a chimney formation took place above the tunnel. A pit of almost 10m diameter at the ground surface and for a depth of 6-8 m was formed above the crown. There was heavy ingress of water into the tunnel from this pit. The tunnel was filled up with mud, boulders and soft rock which had fallen from the pit. Cement grouting was attempted to tackle the problem, but this was not successful.

The material above the crown was predominantly soil overburden, consisting of hard/dense sandy clay/clayey sand, which used to get fully saturated with ground water during the monsoon. At ch. 151, the material overlying the crown consisted of completely and highly weathered rock. When the ground got saturated with water, these formations yielded, resulting into puncturing of the crown portion causing a mud flow carrying with it approximately 3500 cum of debris. The maximum ground subsidence at this location was approximately 12m.

Truck mounted Casagrande C-6 hydraulic drilling rig that can be easily maneuvered in all types of ground conditions was used for drilling. 115 mm diameter boreholes were drilled with the help of a T-150 power swivel using special type drill rods and tricone roller bottom bits. The drilling was done at rate between 80 to 100 rpm. Figure 88 illustrates the jet grouting scheme adopted in this stretch.

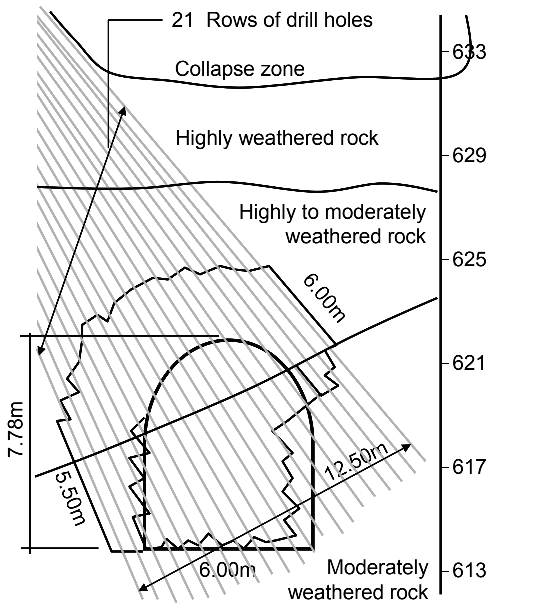


Fig. 88 Jet Grouting Treatment -Ch. 151

Case 2 Tunnelling in Soft Ground Using Shield and Jet Grouting: Sinking of Shield in Soft Ground

Eastern Railways have constructed 17 km of underground railway known as 'Calcutta Metro'. The job required construction of Underpass beneath an embankment supporting six railway tracks without disturbing the rail traffic. Two tunnels of 5.1 m inside diameter (one for up-line and other for down-line) were to be constructed for this purpose.

After construction of about 30 m length of up line tunnel, construction progress was hampered on account of very soft nature of strata. The shield started sinking in soft ground underneath and could not be maneuvered. At this stage, the shield nosedived by 87 cm that was

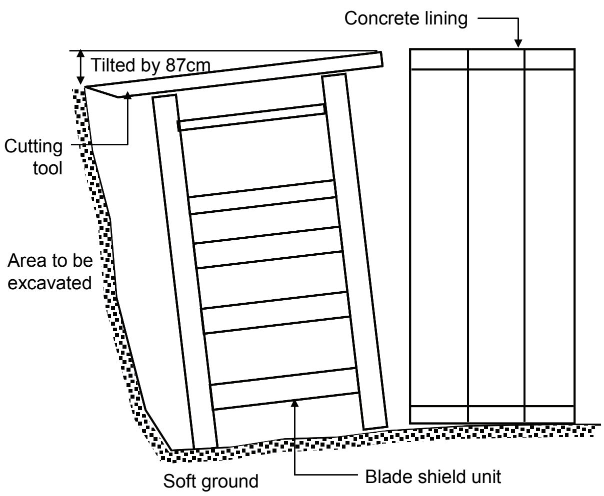


Fig. 89 Tilting of Shield in Soft Ground

very much in excess of specified tolerance limit of 50 cm (Figure 89). Further construction of the tunnel was then suspended and it was decided to use jet grouting technique using cement to improve the soft clayey silt beneath the base of tunnel.

Jet grouting was carried out from the surface. For this purpose, jet grouting rig was kept on the railway embankment and inclined holes were drilled up to required depth (Figure 90). After completion of drilling only 2 to 3 m portion below the tunnel base was jet grouted. The grout holes were arranged in fan array so that the spacing between adjacent drill holes was 50 to 60 cm when it reached the tunnel base. Diameter of each jet grout column was of order of 1.5 m–2.0 m touching each other forming a composite envelope beneath the base of the tunnel. The grouting was done using single phase process i.e. water cement slurry only. Cement consumption varied between 300–400 kg/m of jet grout column. After treating a length of 8-10m, excavation in that portion, was carried out using blade shield unit. By providing jet grouting beneath the base, the nose diving tendency was totally restricted and tunnel shield could be maneuvered in correct alignment.

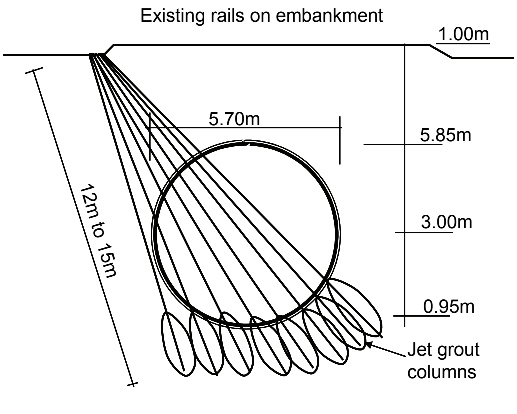


Fig. 90 Jet Grouting in Operation at Site

Case 3 Jet Grouting for Well Foundation at Jogighopa Bridge

A major rail-cum-road bridge was recently completed across river Brahmaputra connecting NH31 at Jogighopa with NH37 at Pancharatna (M.P.). Large diameter wells were sunk up to 67m depth below the river bed through predominantly sand/silt strata. However, a hard rock stratum was existing at a depth of 40m at the well locations 17 and 18. The rock surface was having a steep slope. A section through the well is shown in Figure 91(a).

Among various alternatives jet grouting was selected as to be the most optimal solution in view of its inherent advantages of geometric flexibility and cost-effectiveness.

The gap between well cutting edge and rock was sealed by two rings of overlapping columns as shown in Figure 91(b).

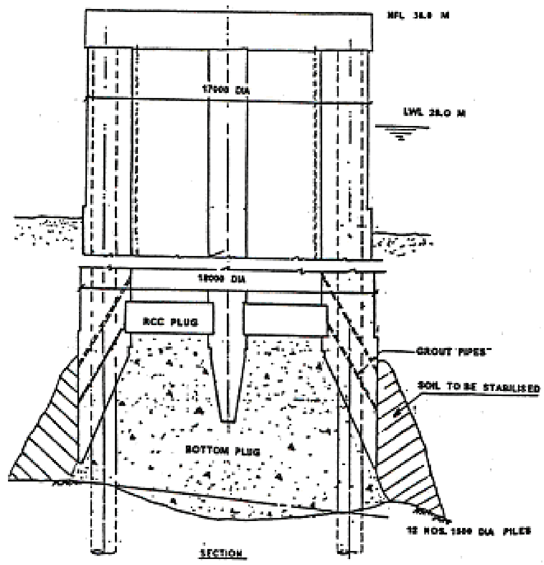


Fig. 91 (a) General Arrangement of Well No. 17 & 18

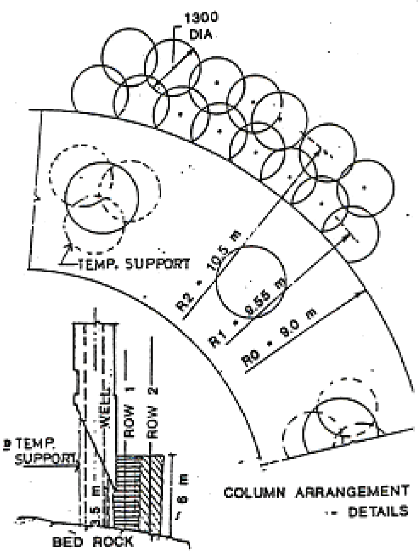


Fig. 90 (b) Layout of Jet Grout Columns

Case 4 Jet Grouted Cutoff Below Cofferdam

In construction of Omkareshwar Hydroelectric Project (520 MW) an effective curtain was to be provided to prevent seepage from river bed to the dam seat. Riverbed material consisted of large size boulders. Jet grouted columns acting as cut off below the cofferdam were found to be cost effective and execution friendly in this site.

Grout columns of 1200mm diameter at a regular spacing of 1000mm were provided to cover 1169m length of the cofferdam. The depth of columns was roughly 14m. Bi-Fluid system was followed for making the grout columns (Refer Figure 92 a and 92 b).

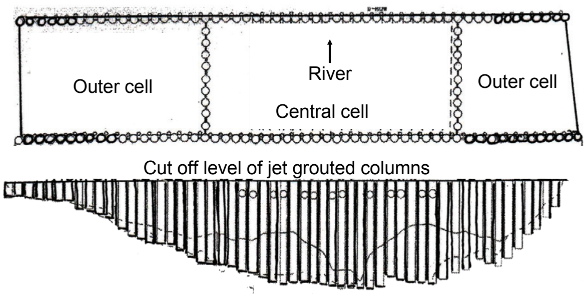


Fig. 92 (a) Scheme for Curtain Grout as Cut Off for D/s coffer dam

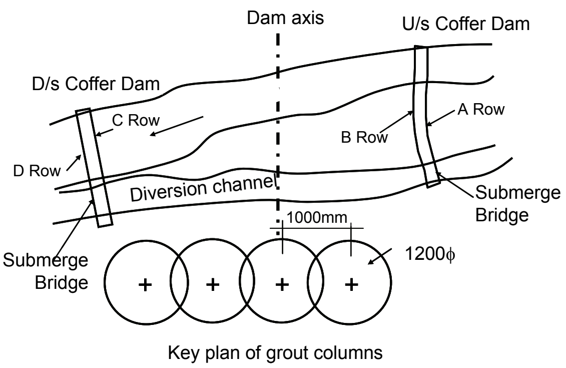


Fig. 92 (b) Layout of Jet Grouted Column

Rotex N-116 bits with 152mm diameter steel casing pipes or Odex bits using Panthar Hammer and 141.2 mm steel casing pipes were used for pre-drilling. The pre-drill holes were filled with cement bentonite (1 : 0.5) mix.

The drilling rig was setup at the pre-drilled hole location for re-drilling and for the final soil treatment with the high pressure jetting. Once the borehole was reached the cutting edge level, the valve is opened for jet grouting. High pressure grout consisting of cement with w/c ratio 1:1 is allowed to jet horizontally from the nozzles in an air envelope. The jet grouting monitor was raised and rotated at the designed rates according to the operation parameters. Simultaneously, the air and cement grout at the design pressure was injected into the surrounding soils. The jet grout column was formed at the monitor.

As the monitor was raised, the column was formed upwards until the designed cutoff level was reached. During jet grouting, the pressure of respective fluid, grout flow rate, air pressure, the withdrawal and rotational speed were monitored closely. The grout jetting pressure, the rotational speed and withdrawal rate were adjusted according to the in-situ conditions.

Case 5 Jet Grouted Columns at Intake Well and Caisson Junction at 2 × 600 MW Mahan Thermal Power Plant, Madhya Pradesh, India

A circular intake well designed to draw water for generation of electricity was constructed as part of a 2 × 600 MW power plant at Madhya Pradesh. The depth of intake well was 30 m below the bed level and from the reservoir level it was 33 m. The bore log data indicated fine sand up to 7 m from river bed level followed by black/white quartz for the next 10m. Highly fragmented fractured rock was found below the quartz layer.

Detailed design suggested that there could be over tuning of intake well towards reservoir side and as a remedial measure, a rectangular caisson was designed to provide passive resistance to intake well. Water from the channel was collected in the caisson and taken into the intake well through sluices. The stress concentration study by FEM & Photo elastic model of the junction of caisson and intake well revealed that the stability provided by caisson to intake wall was insufficient for realizing the full passive resistance against overturning. Observations of the model testing by centrifuge model set up at GERI also confirmed the above apprehensions.

Among the various alternatives reviewed, improvement of silty fine sand and quartz layer by jet grouting was selected in view of inherent advantage of geometric flexibility and cost effectiveness. Two rows of jet columns overlapped with one another to act as barrier against seepage and capable of withstanding over tuning moment of well were considered (Figure 93). Design calculations worked out from the over tuning forces indicated that 20 jet grouted columns on either side of the caisson - intake well connection was sufficient to offer the necessary factor of safety.

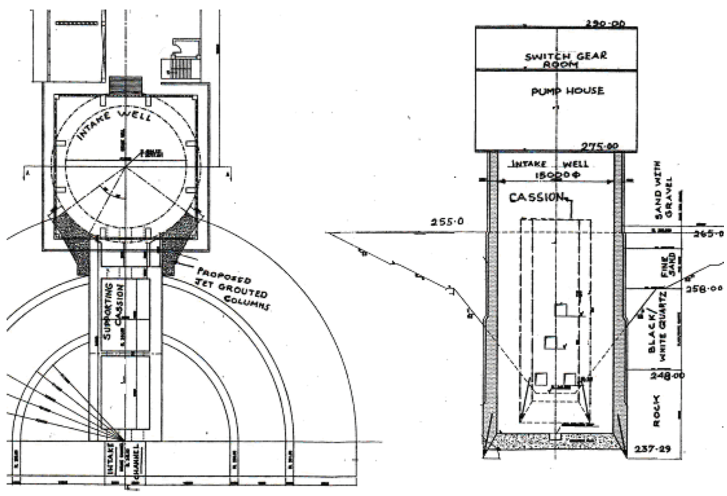


Fig. 93 Jet Grouted Columns for Strengthening In-Take Well–Caisson Junction

Case 6 Comparison of the Effectiveness of Deep Soil Mix Columns Using 2-D and 3-D FEM

Slip Descriptions, Topography and Ground condition

A road slip occurred in State Highway 11, in Himachal Pradesh in a section where the road was constructed on a natural valley feature. The upper slope of the valley above the road is hummocky and showed a number of head scarp, creep and slump features. The area below the failure was highly vegetated with trees and bush, with the lower flanks of the valley also containing houses. Borehole logs indicated that the sub surface condition consisted of soft to firm silty clay sandwiched between fine to coarse grained gravel and firm to stiff silt clay. Mudstone/sandstone forms the bedrock.

Figure 94 (a) shows a section of the failure. Site observations suggested that the failure should have been resulted from a circular slip induced by saturation of the slope during a heavy rainfall event. The slip was approximately 50 m long in the longitudinal direction. Creep movement was noticed below the toe of the slip, which would have contributed to the failure to some extent. A secondary deeper block failure was suspected to be responsible for the crack close to the road centerline. The strength parameters of the soil and rock are summarized in Table 9.

Fig. 94 (a) Slip on the Shoulder on the Road

Table 9 Strength properties of soil/rock

Soil/Rock	*C, kPa*	*ϕ Degree*	*E, mPa*
Fill	0	30	12
Backfill	2	30	20
Soft to firm clay	5	28	25
Stiff silty clay	30	30	50
Mud stone	2000	30	1000

Suitability of deep soil mixing using jet grouting was studied using 2D and 3D finite element model. Once the actual slope failure was successfully simulated, deep soil mixing columns are introduced into the model as shown in Figure 94(b). The columns were modelled as soil using volume elements. Typically the design unconfined compressive strength of a deep soil mixing column was about 1.5 MPa. In reality, the achieved strength was much higher in most of the cases.

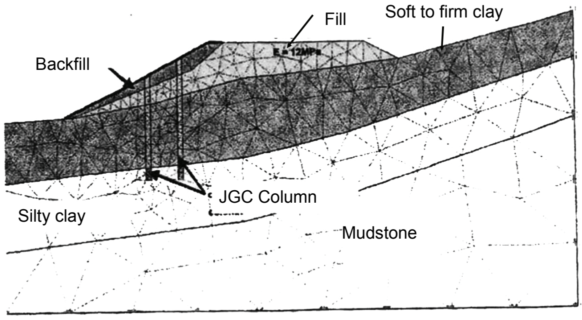


Fig. 94 (b) Two Dimensional Model

In 2D models, the column were modeled using a replacement ratio method. The columns were modeled together with the surrounding soil as a block of composite material. This takes the column spacing into account by assigning appropriate average composite properties.

The 3D model uses the design properties of the deep soil mixing column as input. The columns are positioned in such a way that a block of improved soil bounded by the columns is created in the pavement and the slope below the road.

Two rows of 5.0m to 6.0m long columns of 0.30m diameter were installed. The numerical model studies were conducted varying the column spacing from 0.6 m (2 times diameter spacing), 2.5 m (8 times diameter spacing) and 4.0 m (13 times diameter spacing) in the longitudinal direction (parallel to road). The 2.5 m spacing is the practical maximum spacing we have adopted based on field observations of completed projects. The study also considered two soil models namely Mohr Coulomb Soil Model and Hardening Soil Model.

The output of Plaxis analysis with regard to factor of safety and shear stress acting in column are given in Table 10.

Both 2D and 3D models produced more or less similar failure mechanism. The shear stress mobilized by the columns were determined from the deviator stress given in the Plaxis output. In 2D models, this needs to be converted to an equivalent shear stress as the columns

Table 10 Comparison of Factor of Safety Using Plaxis 2D Version 8.2 and Plaxis 3D Tunnel Programme

Column Spacing (m)	*Factor of Safety (FoS)*							
	Mohr Coulomb				*Hardening Soil Model*			
	2D	*τ_{max}*	*3D*	*τ_{max}*	*2D*	*τ_{max}*	*3D*	*τ_{max}*
No Columns	1.10		1.21		1.10		1.19	
0.6	1.54	142	1.55	70	1.53	111	1.54	64
2.5	1.53	326	1.45	73	1.52	252	1.45	54
4.0	1.54	388	1.43	70	1.51	287	1.44	50

τ_{max} = Max Shear stress (kPa)

have been modeled as a block of composite material instead of an individual column as in the case for 3D. The conversion is based on the assumptions that the shear stress taken by each column is proportional to the stiffness of surrounding soil and takes into consideration of column spacing. This aspect needs to be refined to maximize the effectiveness of the columns.

Factor of Safety

The computed factor of safety in 3D models were lower than 2D models by 5% to 7%. The effect of stress distribution for different column spacing have not been taken into account in the case of 2D model.

Shear Stresses in Column

2D models show significantly higher shear stress compared to 3D model because the shear stress in 2D model is the equivalent shear stresses acting in columns which is calculated using the equivalent stiffness assumption whereas the shear stress in 3D model is the actual stress determined from the finite element analysis.

Mohr Coulomb models show higher shear stresses in the column in comparison to Hardening Soil Model, if E of the Mohr Coulomb model is equal to E50 of the Hardening Soil Model. However, the impact of the columns being stiffer than the surrounding soil modifies the stress regime and hence the comparison between 2D and 3D models is difficult.

Dry Slope versus Wet Slope

The ground water was approximately 2.0 m below road embankment. Comparison between the modified dry model and the original saturated model confirms a similar failure mechanism on the embankment slope for both models. On the saturated model, a deeper secondary failure mechanism was noted. The reported factor of safety for both models however were the same since the main failure was on the slope. It was noted that installing deep soil mixing columns have increased factor of safety by 20% to 40%.

The remedial work performed satisfactorily and has undergone events of heavy rainfall without developing any signs of further instability.

There is a need to understand the interaction of deep soil mixing columns with surrounding soil mass. Columns need to be designed so that they create an arching effect, and change stress distribution. Such change in stress distribution should be readily visible in model output and should in most cases be reflected in computed factors of safety. 3D effects of column interaction play a key role in the field performance of deep soil mixing columns in road remedial work.

Compaction Grouting Technology

Compaction grout is a silt/sand grout, with or without Portland cement and/or aggregate, pumped with a maximum 50 mm slump, preferably 25mm or less slump, characterized by high internal friction and minimal plasticity, injected into the soil under pressure to form a mass of grout that radially displacing the surrounding soil without hydro fracture and damage to adjacent structure or permeation of the soil. The concept of compaction grouting and its range of application with respect to the grading of soil particles are presented in Figure 95 (a) and Figure 95(b) respectively. A typical relation between slump and the flow rate is presented in Figure 95(c) and an exhumed compaction grout bulb is shown in Figure 95(d).

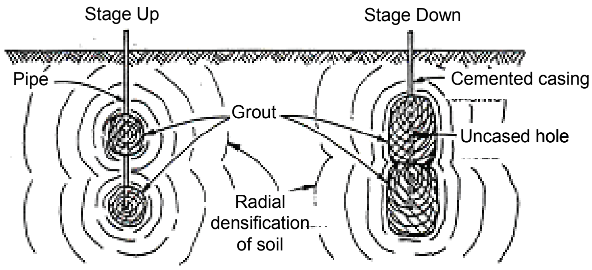


Fig. 95 (a) Concept of Compaction Grouting

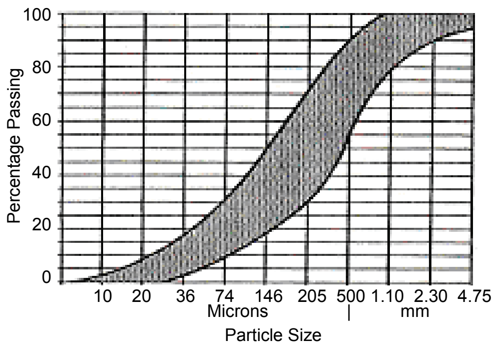


Fig. 95 (b) Grout Material Preferred Gradation

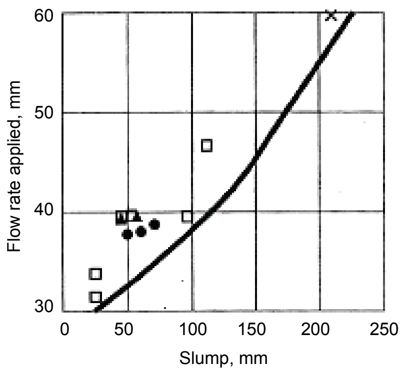


Fig. 95 (c) Correlation-Slump and Flow Rate

Fig. 95 (d) Compaction Bulb

Range of Applications

Compaction grouting has been successfully used in most types of soil, and to a deeper depth. Injection into saturated clays is dependent upon their ability to drain and very slow injection rates are usually required. This increases the amount of time required to perform the work limiting its usage. The procedure also has limited effectiveness in clean course sands and gravel.

The work is always done in stages, that only a few meter of the grout hole are injected at any given time. The staging can be from the top to down (downstage) or from down to top (upstage). The upstage method is found to be faster and economical, and thus the most frequently used procedure, especially for deep injection. For shallow work (less than about 4.5 m) working downstage has the distinct advantage of additional restraint offered by the injected length. When working downstage, each stage is allowed to harden before proceeding to the next stage below. The upstage injection is nearly always accomplished in one continuous operation.

Grout holes are usually spaced on a grid of 1.8 m to 3.6 m although closer spacing is occasionally used. Alternate primary holes should be injected before the intermediate secondary holes. Where inclined holes are used they should generally be not more than about 20 degrees to vertical.

Field Applications

The most common use of compaction grouting is in connection with the repair of settled buildings and other structures. Compaction grouting is also used for site improvement prior to new constructions. It has applications for improving the density of liquefiable soil under existing structures.

Travel Index

One measure of regularity of the obtained grout mass is the Travel Index, (TI) of the grout. The TI is the maximum radial travel of the grout from the point of its injection, divided by the minimum radial distance to a grout-soil interface, and is thus representative of the grouts propensity to remain in a controlled mass and at the intended location. Grouts with a low travel index (less than about 3) remain as relatively symmetrical masses that have clear interfaces with the surrounding soil. Travel indices exceeding five indicate hydraulic fracturing of the soil. In addition to the grout Rheology, the rate of injection must be carefully controlled. While it will vary with the individual soil, it will most often fall within a range of 0.027 to 0.216 m^3 per minute. Excessively high injection rates can result in loss of control of the grout and hydraulic fracturing of the soil.

Limiting Grouting Pressure

The potential conical shear failure surface is assumed to be at an angle θ to the horizontal. Then the allowable maximum grouting pressure is computed as

$$p_g = [\gamma h\{(h/a)^2+3(h/a)\tan\theta+3\tan^2\theta\}/3\tan^2\theta]\ [1+2(1-\sin\phi)\cos(180-\phi+\theta)\}/\cos\phi\cos\theta]$$

in which γ = total soil bulk density and ϕ is the angle of friction of the soil. The maximum allowable grouting pressure p_g is equated to the total weight of the truncated cone of soil plus the downward shearing resistance (calculated from Mohr-Coulomb criterion) of soil along the potential failure surface (See Figure 96.a). It can be seen that design curves, such as those in Figure 96(b) offer a very convenient means of predicting the order of magnitude of the maximum allowable grouting pressure in actual compaction grouting since the γ and h values are normally available. As for the diameter of the grouted mass, 2a, this can be estimated by measuring the volume of grout that has been pumped in (Graf, 1992).

Rational analysis indicates that the soil immediately adjacent to the grout bulb becomes dense due to spherical bulb cavity of radius R_i expanded by uniformly distributed internal pressure p. The pressure reaches and ultimate value P_u, the cavity will have a radius, R_u and the plastic zone around the cavity will expand to a radius R_p.

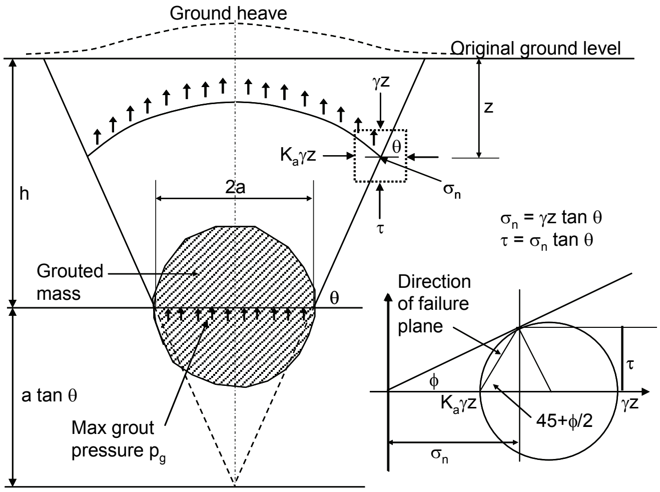


Fig. 96 (a) Ground Stresses Under Compaction Grouting

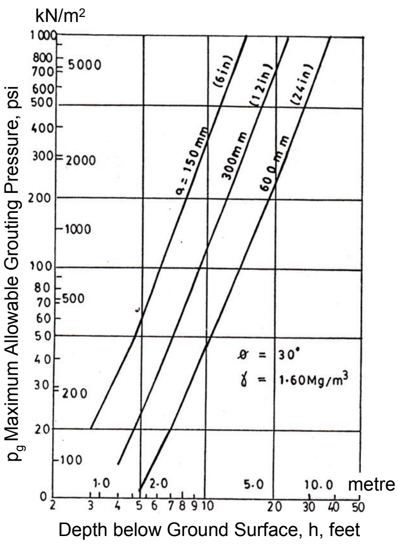


Fig. 96 (b) Maximum Allowable Grouting Pressure

$$R_p/R_u = \sqrt[3]{I_r/(1+I_r\Delta)}$$

where Ir = E/2(1-μ)(C + q tan ϕ) = G/S, quantity Ir is introduced as "Rigidity index" which can be represented as the ratio of shear modulus 'G' of the soil surrounding bulb to its initial shear strength, S = C + q tanϕ. (Shroff, Shah, 1991). The maximum compaction is achieved in the model analysis up to 'R/r' ratio of 3.8; moderate compaction is achieved up to 'R/r' ratio of 6.3 and it diminishes onwards. R is the radial distance from injection pipe of radius 'r'.

Grout Mix

In order to assure quality compaction grout, it is important to carefully control the grain size distribution of the grout. An envelope of acceptable aggregate distribution is provided in Figure 94.(b). The aggregate is usually mixed with about 10% cement and just enough water to result in a very stiff mortar-like consistency.

Silty sand is the best compaction grouting material. Clays add plasticity that may lead to hydro fracture and hence the clay content shall be less than 1%. Silts are necessary to give the required water holding consistency that allows the grout to be pumped. The silt content is normally limited to 10% to 25% of the sand but can be as high as 35% if the silts are coarse. 25 mm slump or less is desirable for preventing hydro fracture. 50kg to 150kg Portland cement per cubic yard (0.73 m^3) of mix is found adequate for most purposes.

Equipment

'Chopping' pug mixer that provides thorough and uniform mixing of the plastic materials is preferred for preparing the grout for compaction grouting. A feed system to the piston pumps which effectively shuts off air-sucking channels is necessary. Hydraulic piston pumps modified to achieve grout pressure of 7 to 11 MPa (70 to 112 Kg/cm2) are the most suitable.

Placement of Injection Points

Injection points have usually been placed by drilling inside and ahead of a 3.8 cm or 5 cm pipe while simultaneously driving the pipe. When conditions allow, a modified wagon-drill is used. A jack hammer or air hammer can be sued to drive the points. The grout point layout might be similar to that shown in Figure 97a as the effective radius of compaction of a grout bulb substantially increases with the depth. Another variation frequently used is angularity (inclination) of the grout points so as to grout beneath a structure without going through it (Figure 97b).

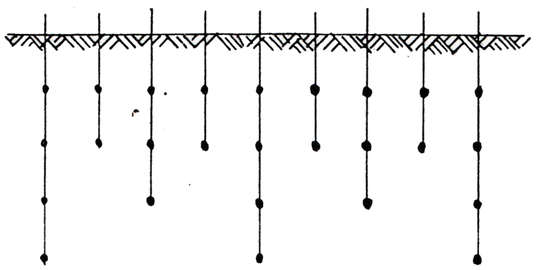


Fig. 97 (a) Grout Point Positioning w.r.t. Depth

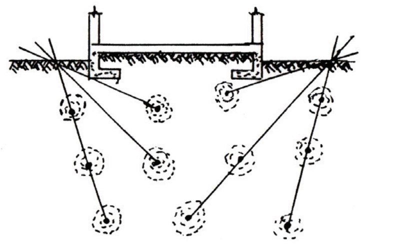


Fig. 97 (b) Angular Positioning of Grout Points & Bulbs

Radius of Compaction

Field observations indicate that the radius of compaction at grout point is a function of the following.

- The restraining pressure of the soil which is the sum of (a) the weight of the inverted cone of soil above the bulb and (b) the shear strength along the shear surface of the restraining cone of soil.
- The weight of structure above the grout bulb. When even a small amount of lift is made, it may involve up to 50% of the weight of the entire structure when the lift is made at one small area of the structure (Figure 96a).
- The surface area of the grout bulb (total radial force) directly proportional to about the 2/3 power of the volume.
- The grout pressure at the bulb.

Lifting of Structures

Controlled lifting of structures with compaction grout can be accomplished under almost all soil conditions. Many structures have been lifted in soft clays. The limiting weight that can be lifted using compaction grouting has not yet been demonstrated. Structures have been lifted by more than 0.50 m with compaction grout. Machinery foundations supported by shallow and deep foundations have been leveled using compaction grouting.

Laboratory & Quality Control Field Tests

There shall be nearly continuous observation of the mix at the mixer with frequent slump tests and the measurement of apparent shear strength of grout by custom penetrometer. M S University, Baroda, India had fabricated a special cone and horizontal flow meter that is directly related to the slump. Slump value is correlated to value of horizontal flow meter as illustrated in Figure 95c.

Rheological parameters such as initial shear strength and plastic viscosity should be measured routinely to classify the fluid properties of the grout. Flow through a 5.08 cm hole under at a 0.067 m3/min pumping rate the pressure drive of about 0.07 Kg/cm2 open ended; that is pretty good grout. A filter press has the advantages of allowing much bigger pressure to be applied, and of being easily standardized. The test is same as squeeze test.

The degree of compaction in the field can be controlled by the following parameters.

- Deformation measurements at site surface or structure
- Soundings (CPT, SPT) before and after the compaction grouting process
- Continuous record of pressure and grout volume.

Performance

The results of the relative deeper injections are more consistent than those of the relatively shallower ones. The injections into soils shallower than about 4.80m were associated with significant upward displacement of ground surface. Such displacements are reported by others also (Yamaguchi *et al.* 2000). On the other hand, no significant changes of ground surface elevations were observed due to the deeper injections. The depth of 4.8 m is found to be sort of critical depth for the injected piles. The gradual improvement in the correlation between the grout pressure and volume while going deeper around is explained by the gradual transition from the shallow mechanisms to the deep ones. The overburden pressure and shear strength of overlying soils are the main factors governing the shallow injections.

Case 1 Compaction Grouting for Correcting Building Settlement

Two buildings with four floors in each and supported on shallow spread footings placed at a depth of 1.0m suffered settlements 20 mm to 44mm during construction of an underground work adjacent to these buildings. Figure 98 shows the position of one of the buildings with respect to the underground works. The settlements were attributable to the loosening of soil during diaphragm wall construction and subsequent excavation. Compaction grouting was then carried out below the foundation of these buildings to strengthen the loosened soil.

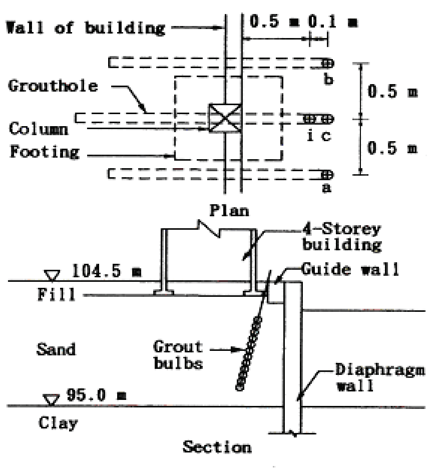


Fig. 98 Position of the Building w.r.t. the Excavation and the Compaction Grout Hole Locations

At each column, 4 grout holes, with series number of 'i', a, b and c, were installed and injected sequentially. According to the sequence of grouting, grout hole 'i' is the primary, a and b are the secondary and c is the tertiary. The grout pipes were formed by drilling and reaming 100 mm outer diameter steel casings to depth of 5 m to 7 m. Grout holes 'i' were inclined at 12° and grout holes a, b and c was at 15° from the vertical. Figure 98 also shows the positions of the grout holes with respect to a footing. Grout injection was conducted at 0.5 m intervals between depth of 8 m and 3 m, working progressively upward from the maximum depth of the grout hole. Injection quantity was measured by counting the number of the piston strokes. The rate of injection was about 42.5 liter/min. The peak pressures were recorded. For every cubic meter of grout material the mixture was composed of 320 kg per gravel, 1040 kg silty sand, 160 kg cement and 425 to 526 kg water. The slump ranged between 30 mm to 40 mm.

Results of Grouting: A criteria for injection for grout holes a, b and c was revised to limit the peak pressure to 3.5 MPa instead of 4.5 MPa to avoid heave of 26 to 30mm as recorded. Totally 268 and 90 bulbs were injected beneath 9 columns and 3 columns at building A and B, respectively.

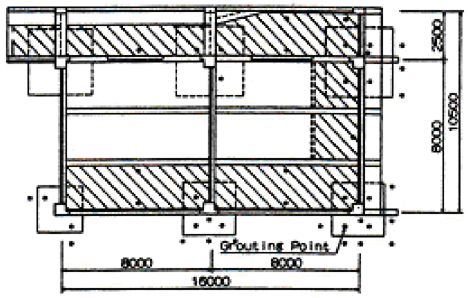
Ground Response: Settlement readings has indicated that uplifting of the columns were induced when the grout bulbs at or above 4.5m depth of the nearest tertiary grout holes c were being injected. Although, the injection quantities and pressures are similar between each grout holes at depths between 5 m and 3 m. The maximum heave induced at column 7 was 24 mm. The inclinometer profiles indicate that the maximum lateral displacement of 17 mm was mainly attributed to the injection of grout holes 6i and 7i, which were distanced 3.0 m and 1.8 m from the inclinometer casing respectively.

Soil Densification along with Lifting Up: The average peak pressure for the tertiary grout holes c were 20% higher than those for the secondary grout holes a & b. These comparisons indicate that stiffer soil conditions had been achieved by compacting grouting. The soil densification effects are further supported by the results of inclinometer observations. The ratio between radial displacements to bulb radius represents the radial strain caused by compaction grouting. The radial strain response to grout holes i was about 2 times that of the grout holes c.

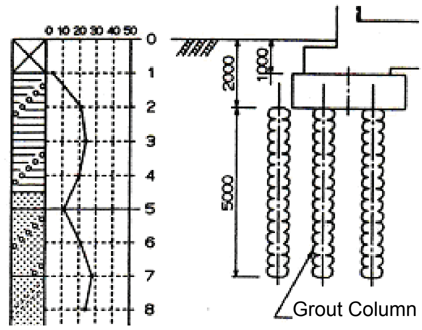
Case 2 Rectification of a Tilted Elementary School Building at Latur at Maharashtra, India

One elementary school building with two floors experiences tilt as the result of uneven settlement caused by earthquake. The building is supported on column footings as shown in Figure 99a. The maximum uneven settlement recorded was 68mm. The subsoil comprised layers of gravely clay and gravely sand Compaction grouting was adopted with the objective of lifting the foundation as well as to achieve ground improvement.

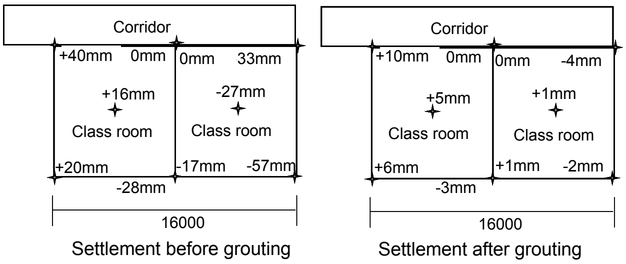
Figure 99b shows the values of settlement before and after rectification. The settlement was rectified to within + 7mm. The rectification work period was about one and a half months, and 42 columns were formed using a total volume of 160 m^3 of grout. Figure 99c shows the relation between grouting quantity and amount of correction achieved.



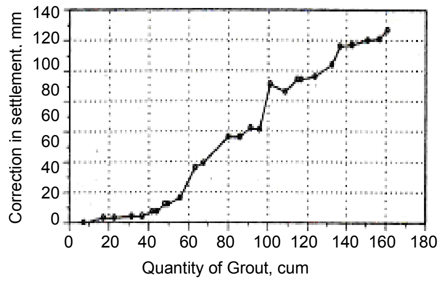
(a) Foundation-Grout Holes Position



(b) Boring Log, Grout Bulbs



(c) Settlements before and after Grouting



(d) Grout Quantity vs. Correction

Fig. 99 Correction of Tilt of an Elementary School Building using Compaction Grouting

Case 3 Compaction Grouting for Alleviation of Settlement and Tilting of Roadway Retaining Wall

Shortly after construction, post-tensioned twin-tee concrete retaining walls with toe pressure in the range of 115 to 140kPa supporting the main line of interstate road through Bhuvneshwar, Orissa Glenwood canyon began to settle and tilt. Typical soil profile along with retaining wall with backfill is shown in the Figure 100a.

the footing has been evaluated based on elastic theory. A graphical representation of the stress distribution under the retaining wall foundation is shown in Figure 100 (b). The spherical and cylindrical grout pressure distribution are also shown in Figure 100(b) and the actual stress distribution is expected to be somewhere in between. Also the actual degree of improvement within these zones will vary as the soil densification by compaction grouting is not only the function of stress.

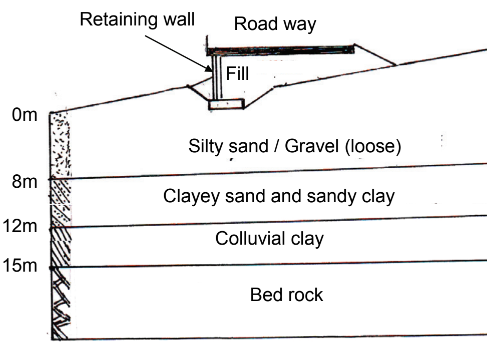


Fig. 100 (a) Retaining Wall with Road Way

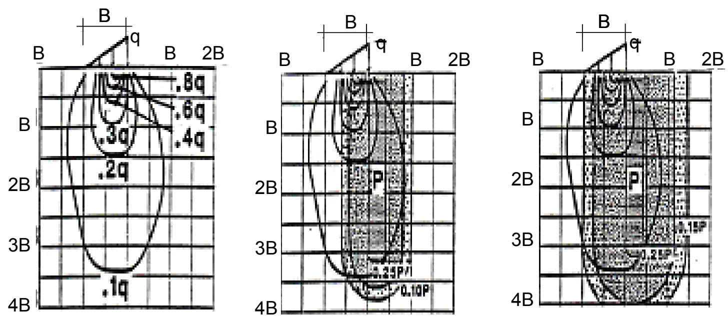


Fig. 100 (b) Triangular Stress Distribution with Spherical, Cylindrical and Grout Pressure Distribution Respectively

Execution problems: Monitoring of the movement with level and inclination surveys indicated erratic and inconsistent, but progressive, settlements of about 12mm every year. A significant 80% of this settlement is attributed to the compression of the top 6.0m layer. The maximum pressure is exerted in the toe area with pressure near zero at the heel of the footing. Since the excessive toe pressure was causing the tilt, a single line of compaction grout injections in the toe area of the walls would provide sufficient densification to alleviate the continuing settlement. The stress distribution beneath

The compaction grouting program started with 'top down' method in a single row of injection points at the toe of the wall. Spacing between injection points was set at 1.5 m. This allows two injection points per panel and would provide a means for levelling if heave was to occur during an injection.

The grout stages were kept to 1 m or less to limit the potential for uncontrolled movement. The cut-off criteria for each stage were based on a pressure limit of 4 MPa with maximum grout take of 0.08 m^3/m at minimum 690 kPa. The maximum pressure criterion was included for reasons of safety and also because of equipment limitations. Continuous pressure and volume measurements were carried out apart from the monitoring of the walls and culverts near injection points. The grout injection program was staggered in a pattern of alternating primary and secondary holes. The primary holes were injected first to act as reaction for the grouting of the secondary holes.

Grout Mix: The grout mix consisted of sand, cement, fly ash and water in approximate proportions 12:2:3:8 by weight with 25mm slump. The plasticity index was less than 10% and the grout had more than 30% passing 75 micron sieve. A set retarder was added at a rate of 59m per 0.76 m^3 of grout to keep the setting time in acceptable limits.

A total volume of 443 m^3 of grout was injected into 230 drill holes with an average grout volume of 18 m^3/m hole. The compaction grout thus represents 8.2% of the compacted soil volume. The grout distribution along the grout depth was measured as 4.5% for the upper 2.40m and 10,7% for the remaining 3.70m. No significant settlements have been reported since the completion of the project.

Case 4 Compaction Grouting of Overburden Soil above Tunnel Alignment in Karstic Lime Stone Region

The SMART tunnel was constructed through Kuala Lumpur Limestone. The tunnel of 12 m diameter located between 10 m to 16 m below existing ground level. The TBM bored almost entirely within the rock mass. The soil overburden of 0 to 16 m arched around the cavity (slump zone) and a quasi-stable condition persisted for years (See Figure. 101a). However, the occurrences of ground subsidence and formation of sinkholes in soil overlain to karsted lime stone were frequently associated with construction activities.

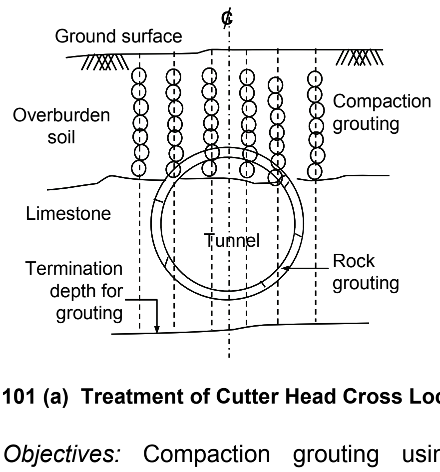


Fig. 101 (a) Treatment of Cutter Head Cross Location

Objectives: Compaction grouting using thick mortar grout was resorted mainly to reduce ground water lowering and minimize disturbance to the very loose soil overburden. The grout holes were arranged in a grid of 2m x 3m. Treatment points were added at appropriate distance wherever the grout takes were high.

Materials: The grout material comprised of stiff mortar that was mixed at batching plant and delivered to site. 100mm slump was adopted for the grout.

Compaction Process: The stiff mix was pumped into the soil under high pressure of 10 to 20 bar until a pre-determined termination criterion was met. The overburden soil was treated as the grout pipe was withdrawn in 0.50m steps upwards with an injection rate of 100 l/m. The end product was a homogeneous grout bulb or series of linked bulbs formed near the tip of the grout pipe as the pipe withdrawn in steps. The grout bulbs formed compact the surrounding ground by displacing loose soil and closing the voids within the soil, but without causing hydro fracture. Pre and post grouting SPT 'N' values are shown in Figure 101b. The displacement ability of the compaction bulbs also raised the subsided ground surface, thereby remedying any previous ground settlement.

Compaction Grouting Termination Criteria: Grouting at each step (depth) was terminated when one of the following criteria was achieved.

(i) Surface ground heave was observed exceeding prescribed limit

(ii) Refusal of further grout flow at pre-determined pressure (e. g. overburden stress + line losses + 5–10 bars)

(iii) Volume of grout exceeded pre-determined volume.

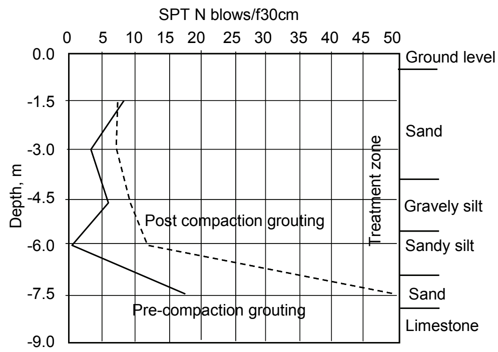


Fig. 101 (b) Pre & Post Grouting SPT Values

Permeation Grouting Technology

Permeation grouting is the oldest established and most widely used grouting technique. In soils, the procedure involves permeating and filling the soil pore spaces, without any significant disturbance to, or movement of, the individual soil grains.

As discussed earlier, two criteria must be met for achieving adequate level of water tightness, such as (i) the viscosity must not exceed 5cp and (ii) the powder grain size must be limited to 12 µm. The mathematical model discussed earlier simulates the propagation of the grout into the porous ground and aids determination of the required Rheology (viscosity) and strength of the grout and the grouted mass. Standardized operating methods have been developed to determine the optimum composition of the grout mix with reference to the structure of the ground.

Case 1 Seepage Control by Grouting During Sinking of Operation Shaft at LPG Underground Storage Project

This LPG Underground Storage Project work mainly consisted of the construction access and operation shafts linked with cavern construction for gas storage module. The storage module were kept at 190 m below GL so as to have adequate overburden.

Hydro Geological Conditions: In this particular site, it was revealed that prominent lineament was passing through the site and extended towards East side. Generally, lineaments/geological structures (such as folds, faults and joints) control the ground water flow. Disturbance to the existing aquifer due to construction activity of shaft led to loss of hydro static balance of discontinuities in general and formation of tortuous, but continuous, channels in particular. The triggering of seepage and gushing out of water through garnet foliated gneiss was characterized by three sets of joint-sub-vertical foliation joint, vertical joint and sub-horizontal joint in the vicinity of shaft from 26 m to 37 m. In general, the RQD was very low having Lugeon value of 4 at this level.

Grouting in the Access Shaft for Arresting Heavy Seepage at el. 26 m: Detailed records of core recovery, RQD, joint/m, degree of weathering, foliation direction and Lugeon measurement at various depths in the bore holes near the shaft location helped in formulating the grouting scheme. The depth, pattern of grout holes, grout material and procedure to be adopted were decided based on the above information.

Provision of Relief Well: Provision of relief wells with semi circular manifold at above identified locations as shown in Figure 102 were considered for releasing the pressure of gushing inflow of water. In this system percussive drilling and grouting were carried out by double tube without withdrawing drilling string, thereby time taken by replacement of grout pipe was saved. Grout cover no.1 was made effective from sump location-1 by 32 equally spaced grout holes and grout cover no. 2 from sump location 2 consists of two rings 48 equally spaced grout holes.

Double Tube Grouting Method under Artesian Pressure of Seepage by a Pressure Balance System: Total set up consists of pressure balance system, clonen bit system with drilling casings, packing between the drilling casings and the grouting pipe and water cut off apparatus.

Drilling Machine: The main part of the drilling machine was separated from the hydraulic power unit, and made it as small as possible for easy transporting and operation in narrow shafts. The machine was a multifunctional one designed solely for drilling so that it could be directed easily to the operating point and could respond to various drilling directions, including vertical, horizontal and diagonal. The drifter had sufficient drilling ability and had back hammer function. The power source was approximately 70KW.

Access Shaft Periphery Cement Grouting for Rock Fracture System: As grout injection rates are dependant upon the connectivity of rock fracture system

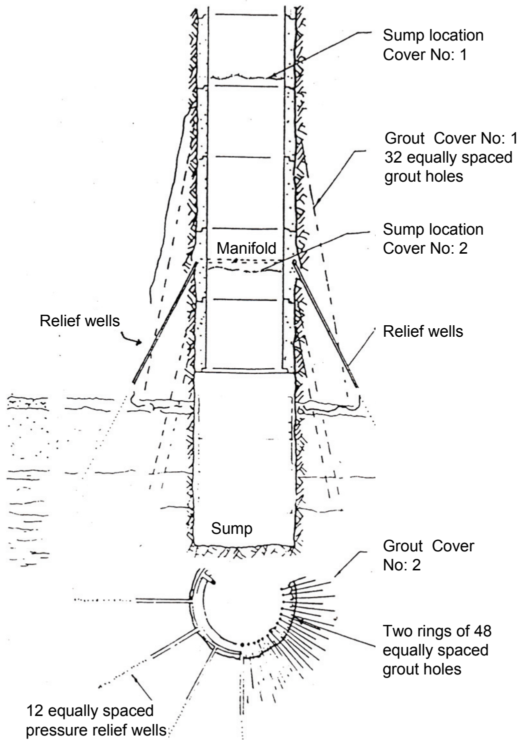


Fig. 102 Installation of Relief Well with Semicircular Manifold to Release Seepage Pressure

with grout holes, careful selection of primary holes to be grouted could result in significant savings of overall time and costs of project. In this case, selection of most appropriate hole(s) for the initial injection helped to achieve the most rapid seal of inflow area, thus appreciably shortening the overall grouting operation.

Grout 1: While treating the foliated gneiss under water table, double fluid silica sole based cement suspension grouts with short gel to long gel time was used. 200 litre liquid “A” (100 litre silica sole + 20 litre sodium silicate + 3.5 liter hardening agent + 76.5 liter water) is mixed with 200 liter liquid “B” (80 kg cement + 3 kg hardening agent + rest of water). Hardening agent of liquid “A” was 6-% dilute sulphuric acid while for liquid “B” it is sodium bicarbonate. For suspension grout, a pump with a capacity of more than twice the suction capacity was used to avoid the clogging in the pipe line or a suction port.

Grout 2: Gel times were kept as short as several seconds to avoid dilution by rapidly moving large quantities of water. Filling of large spaces was accomplished by using expansive foaming type polyurethane grouts.

Remedial Grouting Covers adopted for Encountering any Seepage at Higher Depths (Figure103): Further the correlation of RQD, Lugeon value and joint (crack) intensity established the location of an aquifer. Stable cement based silica soil grout was limited to relatively large inter connected cavities such as passages and fissures and open joints greater than 200 microns in width. Finer fissures and almost all inter

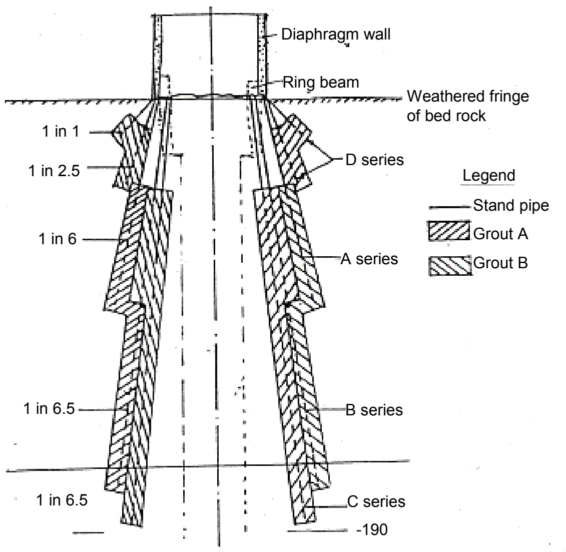


Fig. 103 Access Shafts Periphery Cement with Chemical Grouting in Various Stages

granular porosity left out in first stage grouting ("C" and "D1" series) required chemical grouting or micro fine cement based sodium silicate grout for 'A', 'B' and 'D2' series. The main components of both types of grout were pumped down to the shaft and mixed at bottom with reactive chemicals. The grout was then injected as a single solution. The grout hole pattern comprised a primary ring with 32 holes and an inner secondary ring with 16 holes. After injection of certain quantity of cement, a simultaneous inflow to four central test holes were required to be carried out. The last grout cover not only helped in preventing the inward flow but also reduced any seepage through first grout cover.

Case 2 Curtain and Consolidation Grouting at Arch Dam Site, Iran

Karun III hydroelectric Project is located on Karun River in Khuzestan province, Iran, 28 km away from Izeh city. It is a double curvature thin concrete arch dam having height 205 m and length 388 m with crest width 5.5 m and base width 29.5 m. Dam site is located in limestone, dolomites and mudstones of the Asmari and Pabdeh formations. There is a clear alteration of weaker, more easily erodible beds (marlstones and marly limestone) and stronger, more resistant beds of lime stones in the Asmari Formation. Local faulting are located immediately upstream from the dam site in the right bank, as well as crossing the dam foundation at the left bank. Typical cross section of the dam is illustrated in Figure 104.

Foundation Geology is described as jointed limestone with clay infilling in joints and slicken sides planes. The limestone is faulted and closely fractured with clay rich areas like karstic cavities generally filled with clay, de-calcified bodies (altered weak limestone), dilated jointed areas and weathered limestone.

The dam is founded totally on the Asmari limestone just downstream of the Asmari / Pabdch contact. On the left bank there is a massive thrust block transmitting forces from the dam into the rock. Initial

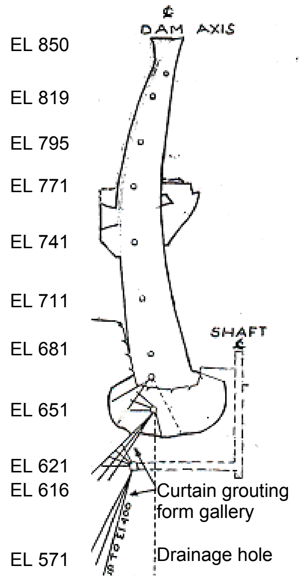


Fig. 104 Typical Section through Arch Dam Foundation, Iran

design shaping and stress analysis was performed using the ADSAS computer program based on the trial load method. Final design analysis, including the very important dam dynamic response to earthquake loading, was carried out with the finite element computer program AEDAP.

Remedial Measure at Left Abutment: The left abutment of the arch dam required local treatment. Unfavorable orientation of a major fault, sheared bedding plane joints in combination of very large downstream oriented dam forces at this elevation (651 to 700) made necessary to excavate some more rock and most other remedial works like concrete key plug (longitudinal & transverse directions) along low lying fault. Due to high degree of jointing and the presence of clay filled joints, the extend and care of this work was important for a satisfactory deformation behaviour of the dam without local cracking or joint opening.

Remedial Measures at Right Abutment: At the right abutment, the frequency of clay in filled joints is high, the degree of karstification is significant and the compressive strength of the rock is low between elevations 650 to 730m. This rock was be treated and several options were possible. Excavation in stages to reach acceptable rock quality in depth and backfill it with concrete in different stages so as to form an arch to transfer the load to stronger beds of lime stone. This "complete" solution provides best abutment condition of the completed structure but had severe impact on costs and schedule.

Consolidation and contact grouting were planned in blocks 11 to 15 just below and adjacent to dam seat. The grouting was carried out by drilling and grouting a series of fanned shaped holes from the down stream toe of the dam and from inside the gallery of the dam at el 651 m. (Figure 105).

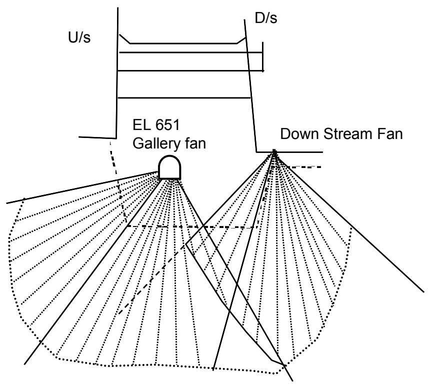


Fig. 105 Consolidation and Contact Grouting

Grout Materials: Cement grout with additives like (1) accelerator (i.e. Calcium chloride or approved equivalent). (2) super-plasticizer. (3) thickener (i.e. sand as specified) and (4) bentonite was used for contact grouting. Admixtures were not used for consolidation grouting.

Grout Hole Layouts: Grout holes were drilled in fan array as shown in Figure 105. The fan layouts consist of primary fans spaced at 3 m; secondary fans spaced at 3 m, (between primary fans); 35 to 37 holes per fan; grout hole inclinations of 0 to 80 degrees from vertical; hole lengths of 26 m to 47 m; holes collared in the el 651 m gallery and at the downstream toe of the dam.

Drilling and Grouting Sequence: Grouting used the 'Closure sequence" method. Series "A" fans were drilled and grouted before Series "B" fans. The entire grouting work was completed in minimum three series. Within each primary and secondary fan, drilling and grouting split spacing method was followed.

Series 1 comprised drill and grout of every fifth hole in the fan pattern, series 2 was to drill and grout holes to split space series 1 and series 3 consisted of drill and grout all the remaining holes to split space series 1 and 2 holes.

The closure take was suggested as 50 kg cement per metre of gout. Additional holes were suggested in the areas where this closure take exceeded in the final split space holes of secondary fans. In general upstage grouting was followed for most of the work. Down-stage grouting was carried out in areas of very high takes or where the holes experienced frequent water loss or artesian flow during drilling. A stage length of 5 m was used during the initial stages of grouting. This was increased to 10 m, as instructed in upper areas of the fans where takes were negligible during the later stages of grouting. Contact grouting of the concrete/rock interface was carried out during the final stage of grouting in each hole. The packer was set in the concrete about 1m above the concrete/rock interface for this stage of grouting.

The following criteria were recommended for grouting work.

Recommended grouting pressure criteria was 0.3 kg/cm^2/m of vertical depth and minimum pressure for holes drilled into base of dam was 4 Kg/cm^2. The minimum pressure to be maintained at holes drilled adjacent to dam foundation was 2 kg/cm^2.

Each grout stage was to be water pressure tested prior to the commencement of grouting. This consisted of the 5 minute single pressure "Abbreviated" test. Thickening steps followed were with C:W - 1:2, 1:1, 1.5:1 and 2:1 for L < 10, L between 10 to 20 and L > 20 respectively.

Grout Refusal was defined as a stage when no measurable injection occurs after a 15 minute period using the maximum grout pressure of the grout stage. The grout pressure was to be maintained for a further 15 minutes in order to "Pack" the grout.

Curtain Grouting

Factors and Characteristics of Design: The factors that determined the design of grout curtain were (a) a 180m head would exist across the dam and spillway structures, (b) The Asmari Limestone was Karstic and was locally riddled with solution cavities, (c) Ground water levels in the abutments were low, (d) The lower limit of practical rock grout ability was about 2 Lugeon and (e) The presence of silt/clay filled voids.

The design included following characteristics: (i) tied into Pabdeh formation in the left abutment, (ii) terminated in the Asmari formation in the valley bottom and ridge abutment, (iii) there were nine galleries and two shafts and most of the grouting was to be carried out from galleries located at elevations 850, 795, 771, 711, 651 and 621 m, (iv) in Asmari, triple line grout curtain to 75m depth from the base of the dam was needed in the areas of high take and single line grout curtain was adopted elsewhere, (v) single line grout curtain in Pabdeh and (vi) grout pressure should be equal to/or greater than induced pressures from the planned reservoir.

Objectives: The curtain grouting at the Karun III dam site was designed to provide a watertight barrier to prevent seepage of water through the rock mass around the dam. It also aimed at displacing as much of the infillings as possible and to replace these material by cement grout and to confine and compress the erodible silt clay material filling in the joints, bedding planes and Karstic voids.

Grouting Parameters: The general grouting parameters used to date are as follows:

- Pressure grouting in the range of 0.3 bars/meter. Original with a maximum pressure of 20 bars below 60 depth.
- Multiple grout mixes, starting at 2 : 1 water : cement ratio by weight.
- Stage grouting in 5 meters length intervals.
- Upstage grouting method whenever possible and downstage grouting in high take zones.
- The spacing of 8 meters for holes of series 'A', as a start for the split spacing method.
- Split spacing method for B, C, D and E series holes. Closet hole spacing is 0.5 m.

Acceptance Criteria: The general acceptance criteria for completion of a line of grouting is as follows: Split space the drilling pattern until grout takes of less than 50 kg/m are achieved in the final series of hole. Permeability of less than 3 Lugeon units must be attained in check holes.

Triple line curtain was required at Karun III because the significant amount of joints and bedding planes in filled with silt-clay material. As previously noted, one of the main objectives of the pressure grouting was to confine and compress the silt-clay material in the discontinuities and voids. The only way to achieve this objective was to use a triple line approach in order to 'encapsulate' the clay bearing rock.

It was planned to drill the middle line grout holes towards the end of grouting programme and inject it to a stable grout with higher pressure than the grout holes along u/s & d/s lines. In this concept, the higher pressure will be used to compress the clay seams in the centre of the curtain. Cement grout from centerline will travel in between the external lines ensuring that all the spaces are filled with higher pressure. The grout holes will be oriented so as to cross in all directions any type of joints and bedding. These procedures will make an efficient grout curtain.

In addition to the above criterion, it was decided that triple line curtains would extend for a minimum of 70 m from the portals of the galleries.

For saving cost of drilling, RQD values and Lugeon depth wise were compared and wherever primary takes were less than 600kg/m, two line grout curtain were recommended.

Use of "Grouting Intensity Number (GIN)" method, was proposed to decrease the time of grouting operations as the water test is omitted in this method. TPC system was used to control behavior of grout holes during injection.

Case 3 Grouting of TBM Diversion Rock Tunnel at Seymareh Arch Dam & Hydro Power Plant at Iran

The Seymareh dam is located on the Seymareh river, west of Islamic Republic of Iran, approximately 700 km south west of Tehran. Diversion system comprises u/s and d/s Coffer dams and two diversion tunnels of 870 m length and having diameters 10.5m and 8.3 m respectively.

Site Geology: According to the bore hole logs the weak limestone's sections (possible cavernous sections) were detected as "rock falls". Those sections were mostly concentrated between elevations RD 550 and RD 650m that was the existing river level. Only the boreholes in the river bed indicated some possibility of karst features or crushed rock mass 4.5m below the bottom of the gorge. At that elevation cavities were filled by clayey and sandy material.

Water springs were principally associated with faults of rock type boundaries consequently; few 65m wide grout zones were initially identified at geological features with the upper and lower Asmari lime stones to prevent inflow of water.

A three phase grouting system was developed to control the water seepage. The system consisted of:

- Face grouting through probe holes drilled ahead of the TBM
- Supplemental Grouting behind the TBM's but prior to final lining installation
- Formation Grouting through the final cast-in-place concrete lining.

Face Grouting: Original contract requirements included probing ahead of the TBM face to determine ground water inflow potential and direct grouting when probe hole inflows exceeded 175 l/m per 30 m of probe hole. In the grout zones, grouting was carried out when inflow exceeded 7 l/m per 30 m. Each TBM was equipped with 2 fixed hydraulic boart rotary drifters above spring line and a third drill was temporarily mounted below spring line. These were subsequently supplemented with rotary percussive drills in the karsted limestone.

Cutter head limitations reduced the available grout pattern to 4 horizontal holes drilled through cutter head buckets. To achieve desired grout travel, a fine ground slag cement referred to as micro fine was used except during major takes where type III Portland cement was used. Often Type III would meet with pressure refusal and a switch to micro fine cement would result in significant grout take in the same hole. Volumetric water/cement ratios of 1.5:1 to 3:1 were mixed and pumped at the face to pressure refusal of 3.5 bar above hydrostatic head. Verification holes were repeatedly drilled and grouted until the 7 l/min per 30 m of hole criteria was met. Face inflow was generally less than 35 l/m per 1.2 m (4') shove during excavation through the grouted zone.

Supplemental Grouting Prior to Tunnel Lining: In order to meet the ground water discharge limits, a supplemental Grouting program prior to final lining was developed and implemented .This involved drilling rings at 2.4 m spacing of six radial holes each 3.6m long and 50mm in diameter (Figure 106 a). Each hole was subsequently grouted either with micro fine or with Type III cement through mechanical packers at the hole collar. Refusal criterion was zero take at 14 bar. Additional holes were added to the pattern holes to target specific fissures and inflows as required. With relatively open fissures in the lime stone, grout takes per hole would often exceed 100 sacks (13.6 kg, each) of micro fine, prompting implementation of a volume criterion of 30 sacks of micro fine to determine a switch to Type III cement.

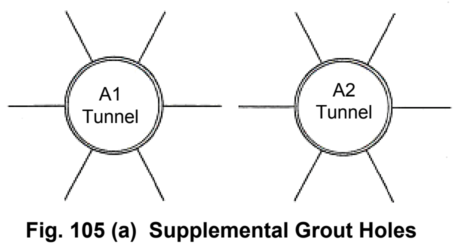


Fig. 105 (a) Supplemental Grout Holes

Formation Contact Grouting through the Tunnel Liner: Despite face and supplemental grouting efforts, a formation contact grouting program was developed to create a low permeability annulus around the tunnel. Radial ring patterns, on 2.4 m centers consisting of eight 3.6m long holes were planned. After the concrete lining of the diversion tunnels had attained sufficient strength, grout holes 40 mm diameter (Figure 106b) were drilled with percussion type rock drills through the 75 mm diameter pipe nipples (Figure 106c) already embedded in to the rock so as to expose the contact between rock and the concrete for effective grouting. As per

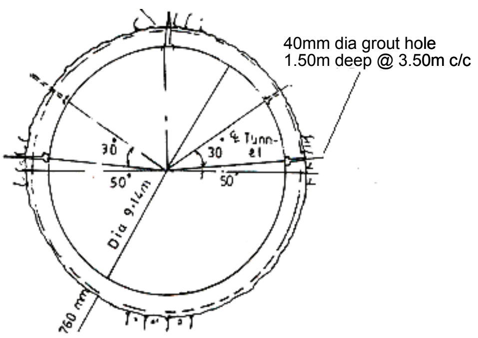


Fig. 106 (b) Pattern Showing the Location of the Grout Nipples in Tunnel Cross Section

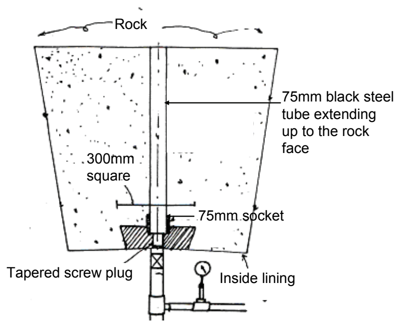


Fig. 106 (c) Details of Grout Nipples

supplemental grouting, micro fine cement was to be pumped to similar refusal criteria to further reduce annulus permeability. Formation grouting with the final liner in place was anticipated to be more effective in reducing the permeability than that achieved in the supplemental grouting (Figure 106d). Tunnel lining strains were monitored by strain gauge arrays to confirm the adequacy of allowable grout pressure.

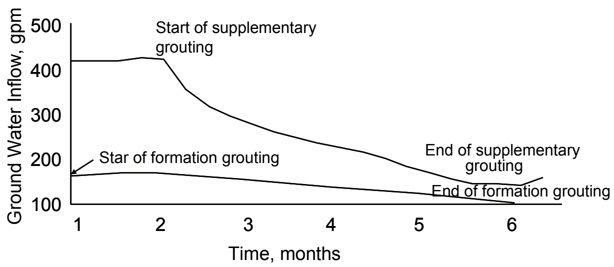


Fig. 106 (d) Ground Water Inflow v/s Time

Case 4 Foundation Treatment of Turbo Generator Unit Description of the Machine Foundation

Foundation of Turbo set 9K78 (1 × 75 MW KLTPS Extension, stage 2, Unit 3-TG, Gujarat) is a reinforced concrete frame structure consisting of bottom slab and of the frame structure. Foundation bottom slab is supported directly on the bed rock on level (-) 5.50m, slab dimensions in plan (-) 15.50 × 8.30 m and its thickness (-) 2.00 m. The base slab weighs 640 tonnes. Structure upper slab is supported on six columns supporting upper slab with three transverse and two longitudinal beams (Figure 107.a). The generator runs at a maximum RPM of 3000 and 50 Hz frequency.

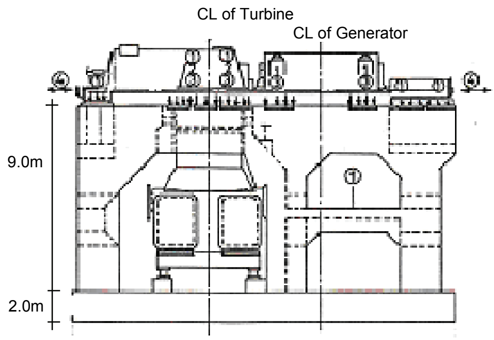


Fig. 107 (a) Turbo Generator Foundation

Immediately after installation of the generator, the operation exhibited unsatisfactory performance. A series of leveling measurements with the use of precise leveling method was made to measure the displacement of foundation and bearing pedestals. Measurements covered at several point at bearing pedestal, periphery of top deck slab, bearing casings at upper slab and at floor of lower plate (slab) of the foundation. The level measurements and velocity & amplitude measurements were taken at various measuring points with respect to fixed reference point in operating and static conditions.

These measurements suggested that raft slab and the columns of TG III foundation experienced moderate level difference during static conditions and significant amplitude difference during dynamic conditions.

The vibration measurements on a turbo generator foundation indicated that the natural ω_n for a rotational motion in the foundation was approximately 3500 rpm, while the operating frequency ω was approximately 3000 rpm. With the machine thus operating at near resonance, a solution of the problem appeared to lie in changing the natural frequency of the system, since the operating frequency could not be changed (Figure 107b). The amplitude of these vibrations were considered sufficient to cause fatigue and cracking of the joints of the rock and a reduction in the life of the bearings in the compressor. In addition the vibrations were being transmitted through the rock in tern to the joint's material-micaceous silt.

Permeate grouting was suggested for changing the natural frequency of foundation material. Four grout holes were drilled at where the horizontal motion of the turbo generator was causing the rocking motion in the

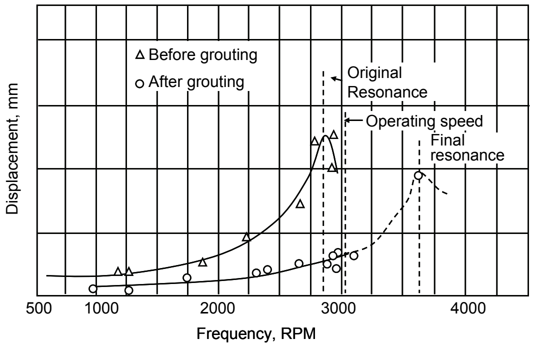


Fig. 107 (b) Vertical Displacement vs Frequency

foundation. After drilling these holes through the concrete foundation, a grouting tube along with surrounding casing was lowered to a depth of approximately 2.4m. Colloidal sodium silicate at 10% concentration, water and hardener (kcl in this case) at 50:50:30 proportion was used as the grout. The initial viscosity was maintained as 5 cp and the unconfined compression strength after 7 days was 400 N/mm2.

Under cyclic behavior, the grouted joints recorded damping ratio between 2% and 3%. Grouted rock failed at axial stain less than 3% under high stress level of cyclic loading. The variation of axial strain and secant modulus with increase in number of cycles characterized by three stages as described earlier.

The epoxy grout was forced into the interface between the floor slab and rock for ensuring complete contact between these surfaces.

The seismographic measurements made before and after grouting are shown in Figure 107c. The grouted foundation performed extremely satisfactory.

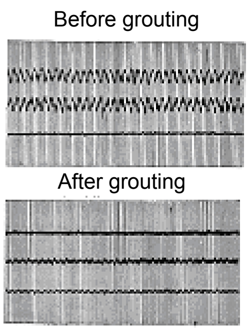


Fig. 107 (c) Seismographic Measurements

Case 6 Curtain Grouting and Treatment of Unconformity Zone at Almatti Dam on Krishna River

This case study deals with providing grout curtain of Almatti Masonry dam on Krishna river, Karnataka from Block No.1 to 52. Besides curtain grouting this study also deals with the treatment of weak zone (unconformity zone) of thickness up to 6 m which was existing in the foundation from Block No. 45 to 52 below the joint of base granite rock and overlying quartzite foundation. The single line grout curtain of permeability less than 3 Lugeon has been effectively formed below the foundation despite very poor strata having maximum pre-grout permeability of 90 Lugeon.

The main features of the grout curtain shown in Figure 108 a are as follows.

1) Single line grout curtain with primary holes spaced at 3 m c/c and secondary or tertiary holes till desired permeability is obtained.
2) Depth of grout curtain = 0.5 H, where H is Hydraulic Head from foundation.
3) Permeability limit of grout curtain < 3 lugeon.
4) The depth of drainage hole was kept as 0.75 times the depth of grout curtain and spacing of holes was 3m c/c. The location of grout curtain and drainage holes are also shown in Figure 108.

From Block No. 1 to 30 where the rock was very hard and abrasive in nature pre-grout permeability was normal and ranging from 3 Lu to 24 Lu. Drilling and grouting of primary grout holes at 3m c/c were taken up first. Secondary holes were drilled at 1.5 m c/c. Grout pressure in various stages were as follows.

a) I^{st} stage 10 m – 3.5 kg/cm^2
b) II^{nd} stage 10 to 20 m – 6.5 kg/cm^2
c) III^{rd} stage 20 to 30 m – 10.0 kg/cm^2.

A wide range of grout consistency with 1:10 to

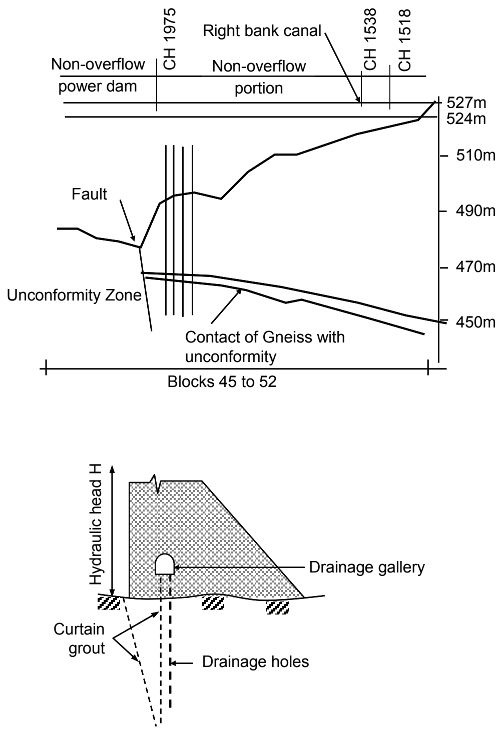


Fig. 108 Grout Curtain and Drainage Holes through Drainage Gallery and Unconformity Zone and its Treatment

1:1 cement to water by weight was adopted in the grouting. The permeability values have become less than 3 to 5 Lu in all the blocks from 1 to 30.

19 holes were drilled by core drilling at about 15 m c/c from Block No. 31 to 44. Grouting of primary and secondary holes were completed and the Lugeon value found to be 11.36 in the second stage.

From Block No. 45 to 52, a weak zone (unconformity zone) lying between Quartzite and the basement granite on the right banks had been established.

Thickness of this unconformity zone was varying from 0.5 m to 6 m in which 10 to 25% core recovery was recorded. The permeability was 20 to 90 Lu.

The work of curtain grouting in these blocks was also taken up in the similar way that followed in the earlier blocks. The primary holes were spaced at 3.0 m c/c initially. However, in most of the primary holes artisan conditions were met with and jets were rising up to the roof of the gallery. In primary holes, the maximum permeability to the tune of 90 Lu was found and the maximum grout intake was 3200 kg.

After completion of primary holes drilling and grouting, secondary holes drilling and grouting were taken up at 1.5 m c/c i.e. between the already done primary holes. The maximum pre-grout permeability of 78 Lugeon was observed in a hole in Block No. 48. However pre-grout average permeability of 22 Lugeon was recorded. The Dam Safety Review Committee recommended to take up tertiary holes in between the primary and secondary holes in these blocks, and to take up grout holes up to basement granite. It was observed that Lugeon value had come down well within limit. The transverse gallery had been provided in Block No. 45 for treating the geological faults existing across the dam. The DSRP recommended to drill grout holes along the fault line in the transverse gallery and grout them to effectively treat the major fault. As per the recommendations, two lines of grout holes along the fault zone were provided in the transverse gallery.

Case 7 Heavy Seepage Control at Old Railway Tunnel near Thane on Mumbai - Hawra Route

Physical Details and Location: The 1.2 km long and around 92 years old railway tunnel is located near Thane Railway Station on busy railway route Mumbai-Howra. The tunnel is approximately 10 m wide and 6m high and installed with down as well as up line tracks. (Figure 109) The tunnel that was built during the year 1916 did not record any instability problem.

However, about 150m tunnel length towards Mumbai end is having heavy water seepage problems. This portion of the tunnel is brick lined in the roof as well as in the walls. The thickness of the lining is 0.45m. The rock mass consists of compact and jointed basaltic rock. The rock mass in this tunnel could be classified as moderately jointed with essentially three regular joint sets with a random joint set. The basaltic rock is black in colour and resistant to weathering because of their hard and compact nature. Flow of ground water through the joints is more pronounced in the rainy season which was

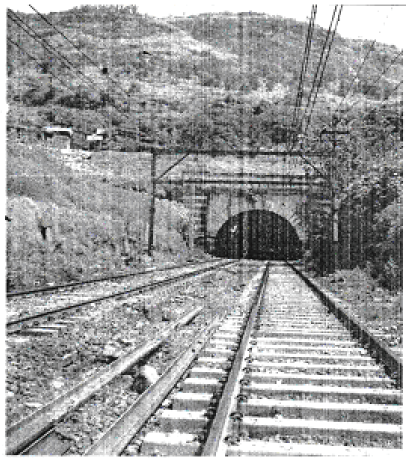


Fig. 109 View of Mumbai Howrah Railway Tunnel

increasing the seepage of ground water into the tunnel mostly towards Mumbai end.

Attempts were made by CIMIR, Nagpur, to control this water seepage problem by carrying out grouting in the wall and roof without much success.

A new set of grouting work were proposed to be carried out during dry season. The water testing of the holes were carried at an interval of 150 m to ascertain in-situ Luegeon value of rock. Trial grout mix was decided based on comparison of grout intake and water intake in the hole.

Pattern of grout hole as shown in Figure 110 were followed. One set of grout holes from the top of overburden rock at crown and other set of horizontal holes from inside tunnel and a third set towards junction between wall and crown were provided. The pattern was designed as per dip and strike direction of joints of the rock using CHAIRLOC software and Singota analytical work. The primary holes were provided at a spacing of 3m c/c and then the secondary holes at a spacing of 1.5 m c/c in-between primary hole grouting were provided.

Tertiary holes were also suggested wherever the seepage was not in within permissible limits. Cement bentonite grout was used in the primary holes and cement based silicate was used in the secondary holes. Sodium silicate with formamide and calcium chloride was recommended in the tertiary holes.

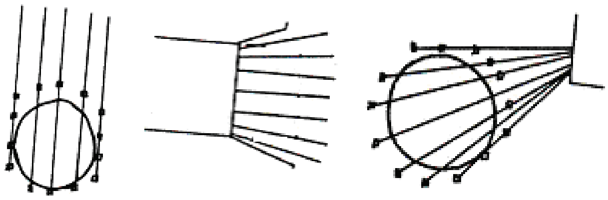


Fig. 110 Pattern of Grout Holes

(a) Grout Holes from Top of Overburden Rock at Crown
(b) Grout Holes Horizontal from Inside Tunnel
(c)Grout Holes from Junction between Wall and Crown from Pit

Concluding Remarks

The specific mechanical properties of each grout that are important factors in the selection of a grout for a specific job include mechanical permeance, penetrability and strength. Similarly, the chemical properties include chemical permeance, gel time control, sensitivity and toxicity.

The ratio of viscosity of grout to that of ground water, time viscosity relationships and the radius of the grout front are interdependent. Change of pore size of widening of fissures are the main features of hydro-fracturing with increasing grouting pressure. An analytical determination of the optimum orientation of a grout hole for a given geological bedding plane should be a prerequisite.

Ground water contamination by grouting chemicals is serious. Ultra fine particles below 4 μ and has very low viscosity, excellent penetrability, strength and durability. Several field applications of micro-fine grouts in India and abroad have proved the efficiency of these new grouts in the fine sand and silts in place of chemical grout. In this case, durable grout and precise injection methods are very important and in near future this application is expected to popular.

There is scope for improvement in drilling technology and for geological investigations aimed at a higher percentage of core recovery with minimal disturbance. A correlation needs to be developed between the type of drilling method to be selected and the type of rock. For a soft rock, the possibility of using a 5-bar nominal pressure may be considered a worthy modification of the Lugeon test. Optimization of alluvial grout mixes by triangular charts provides a ready 'Reckoner' for various basic properties directly required for an ideal grout. In selecting a trial grout mix for rock grouting, a flow chart based on water and grout intake seems to provide excellent optimization.

The selection of grouting techniques relies mainly on repetitive experience and on personnel options. Though new injection methods are available, the tube-a-manchette technique is best suited for grouting alluvium. The pattern and depth of the grout hole are governed primarily by the design requirement and the nature of the rock. The piezometric level relating to the dam foundation interface is the real measure of cut-off efficiency. Lower pressure permeation and critical pressure to hydraulic fracturing can be used for different purposes. Research is further needed to work out optimum pressure, correlating it with the radius of penetration, joint thickness and viscosity/strength of the grout mix.

The successful execution of compaction grouting requires through investigation and engineering analysis combined with continuous control of the actual work. Innovation in conventional upheaval test to determine the range of allowable pressures for grouting might be necessary. It is imperative to provide careful control of the injection and maintain good records, in order to ensure optimal results. The technique is bound to receive ever-expanding usage and lead to improved understanding of the engineering mechanics of injection. The understanding of the consistency and the modulus of soil and the grout is extremely important.

For cement grouts a hydraulically driven pump provides an excellent volume-pressure relationship with no pulsation. For most chemical grouts, proportioning pumps with the metering system are recommended to give the necessary degree of control over the components of a grout system.

Electric monitoring of cement pressure grouting with time-pressure-consumption provides a powerful tool for the evaluation of the behavior of grout holes during grouting. Important advances in *in-situ* monitoring to follow the progress of the grout through the ground and detect any flaws due to non-uniformity of the soil or rock are still awaited.

Case studies have shown that there is, apparently, no unique grout material, or process, or technique universally applicable to every project. The design and methods of grout injection for varying situations are different. Emphasis has been laid on how accurately grouting process are controlled importance of quality control measures are often ignored in grouting works.

Field trials are really essential for the correct execution of the jet grouting. In very soft cohesive soil the problems are related to excess pore pressure generated during execution of jet grouting which can cause heave and hydraulic fracturing when blocking of backflow occurs. In all cohesive soils, large volumes of back fluids must be disposed of and this causes environmental concern. Along with reduction in spoil or waste products resulting from the work, the promising new field in its application is treating industrial wastes.

Following points are the subject for future investigation. (1) expansion of yield stress/ yield locus (2) transition of failure criteria under residual strength (3) change of permeability with time (4) hardening and erosion for a long curing duration (5) relation between consolidation and effects of cementation.

I conclude with words of Karl Terzaghi: *"Man can choose how to use technology and by intelligent use can protect the environment without relinquishing progress".*

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