NEW HORIZONS IN GROUND ANCHORAGES, PINPILES AND CEMENT GROUTING

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INTRODUCTION

Developments in the specialty geotechnical engineering processes used in ground treatment, improvement and support continue at a fast pace in the United States. Much of the impetus comes from specialty contractors, usually linked with foreign partners, who are promoting new concepts and technologies for commercial edge. This is being facilitated by an increasingly wider acceptance of design-build concepts and the related contractual developments such as Partnering (Nicholson and Bruce, 1992) which are helping to create a less litigious and more equable contracting environment. A third major reason is simply the market demand: urban redevelopment and infrastructure development and rehabilitation are posing consistently difficult challenges to the foundation engineering community. A frequent consequence is the need for original, innovative solutions involving both new technologies, and modified or improved older methods.

This paper reviews developments in three major techniques, namely prestressed anchorages, pinpiles (used as in situ earth reinforcement) and cement grouting. Attention is focused only on a limited number of topics believed to have the best chance to impact practice for many years to come. The paper restricts its review to these three techniques as other methods used in ground treatment and improvement are described elsewhere in this seminar (e.g., Welsh, 1992).

GROUND ANCHORAGES

Prestressed ground anchorages have been used in the United States for almost thirty years. There still remains, however, a wide range in regional constructional methods, and a considerable unevenness in the knowledge and understanding of the fundamental engineering mechanisms. Several technical papers have helped to ease this situation (e.g., Bruce, 1989a, 1991), while a recent textbook by Xanthakos (1991) was similarly conceived. The topic has also been explored at depth in conferences (e.g., ASCE Cornell, 1990; ASCE New Orleans, 1992) while renewed efforts are being made by committees within bodies such as the Post Tensioning Institute and the Association of Drilled Shaft Contractors. The Federal Highway Administration remains a focal point for national research efforts (e.g., "Demonstration Projects Program", 1990). While overseas, a wealth of information is contained in the best of the new national standards such as the British Code of Practice (1989) and the revised international FIP recommendations (1992).

For convenience, developments can be considered in each of the two basic groups: anchorages in competent rock, and anchorages in weaker rocks and soils.

Anchorages in Competent Rock

A recent, major rock anchorage project, executed to stabilize Stewart Mountain Dam, Arizona, illustrates many significant and novel features (Bruce, et al., 1991a, b; Scott and Bruce, 1992; Bianchi and Bruce, 1992). Although many concrete gravity dams have been post tensioned to improve their stability, this was the first application for a multicurvature thin arch structure.

Background

Stewart Mountain Dam was constructed from 1928 to 1930 on the Salt River. It is 583 ft. long, a maximum of 212 ft. high, 8 ft. thick at the crest and 34 ft. thick at the base. There are concrete gravity thrust blocks at each abutment, from which wing dams extend into the abutment (Figure 1). The 10 MW powerplant is fed by a 13.5 ft. diameter penstock through the dam. There is also a 7 ft. diameter opening through the dam for bypass outlet works.

Unbonded horizontal planes within the arch concrete were the main cause of the dam's instability. At the time it was built, the importance of good cleanup on the horizontal construction joints was not recognized, so the joints were left untreated. This resulted in a layer of laitance on these joints, which compromised the bond across them.

A three-dimensional finite element analysis of the dam's performance during seismic and other loading conditions indicated that the dam would lose arch action during the maximum credible earthquake of 6.75 on the Richter scale occurring within 9 miles of the dam. This would leave vertical cantilever sections to support themselves. Because the horizontal lift lines were unbonded, the blocks in the upper portion of the dam would then be free to displace.

To stabilize the arch, 62 tendons were installed at about 9 ft. centers, with free lengths ranging up to 216 ft. and bond lengths from 30 to 45 ft. Their inclination varied from vertical to 8 deg. 40 min. from vertical. All but seven tendons, located immediately above the outlet works, were anchored in the dam foundation: these other seven anchors were bonded in the dam itself. Each arch tendon was composed of 22 epoxy-coated strands, each 0.6 in. in diameter. Design working loads averaged about 665 kips (a range of 545-740 kips) per tendon, equivalent to about 50% guaranteed ultimate tensile strength (GUTS).

In addition to the arch tendons, the design called for 22 tendons to be installed in the left thrust block of the dam to stabilize it against failure at or just below the structure/foundation contact. The free length of the thrust-block tendons varied from 40 to 125 ft., plus a 40 ft. bond length, and each was composed of 28 strands. Design load for each tendon was 985 kips (60% GUTS).

Most of the arch dam foundation consisted of hard, pre-Cambrian, medium-grained quartz diorite. The diorite was cut by irregular dikes of hard, medium-grained granite that varied in orientation and thickness. A fault divided the arch dam foundation into three distinct zones with unique mechanical properties, joint systems and permeabilities: (1) to the right of the fault, (2) to the left of the fault and (3) in the fault zone itself.

The rock underlying the right portion of the dam was hard, slightly weathered to fresh and generally of excellent quality. To the left of the fault, including the left thrust-block foundation, the rock was slightly inferior, being more fractured, sheared and weathered. The fault and the surrounding zone contained very intensely fractured and moderately to slightly weathered.

During the design phase, it was assumed that 32 of the arch tendons would be anchored in the right foundation zone (excluding the seven tendons anchored in the dam concrete), 15 in the left foundation zone and eight in the fault zone. All 22 thrust-block tendons were founded in the left foundation zone.

Test Anchor Program

Pairs of vertical "research" anchors were installed 12 ft. apart in each of three test sites representative of the three major rock zones expected to underlie the dam (Table 1). The nominal bond lengths at each site were 10 ft. and 20 ft. Each anchor was cyclically tested in 25% design working load (DL)

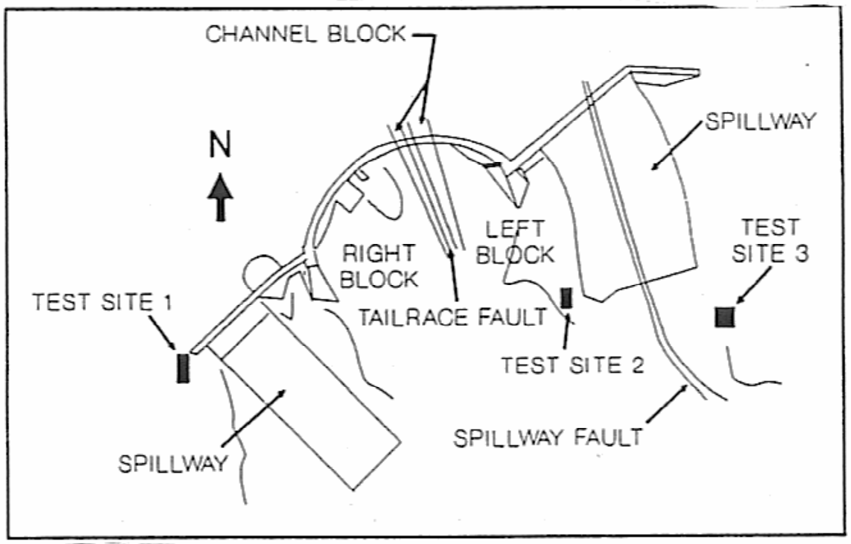


Figure 1. Simplified plan of Stewart Mountain Dam, AZ.

Hole	Strength (MPa)[8]	Dilatometer tests		Empirically derived		Settlement modulus (GPa)[3,4]
		Number of tests	Modulus (GPa)[4]	RMR[1]	Modulus (GPa)[2]	
1A	137	1	2.76	59	16.77	n/a[7]
1B	222	4	7.58	55	13.34	n/a[7]
2A	114	0[5]	-	49	9.44	n/a[7]
2B	143	4	2.07	46	7.94	n/a[7]
3A	n/a[9]	1	0.07	-	n/a[6]	0.10
3B	n/a[9]	0[5]	-	-	n/a[6]	0.17

[1]Rock mass classification according to Bieniawski (1984)
[2]Mass modulus using equation of Serafim and Pereira (1983), $E = 10^{(RMR-10)/40}$
[3]Inferred using influence factors from Poulos and Davis (1974)
[4]Poisson's Ratio assumed to be 0.2
[5]Obstruction in hole prevented testing in bond zone
[6]Out of data range used to develop empirical relationship
[7]Surficial fill at sites 1 & 2 precluded evaluation
[8]Compressive strength based on average of 5 point load tests (on core)/hole
[9]Rock core at site 3 was too fractured for point load tests

Grout age (days)	Compressive strength (MPa)	ASTM modulus (Gpa)
7	35.2	13.1
14	40.7	14.5
28	45.5	16.5

Table 1. a) Rock mass modulus and core strength estimates; b) Grout test results (Scott and Bruce, 1992)

increments to the safe maximum test load - or failure. All achieved the maximum test load of 133% DL - 1,310 kips (80% GUTS) - with relative ease, with one exception: Anchor 3A, the shorter anchor in the worst rock, demonstrated grout/rock failure at 968 kips.

The relative amounts of apparent tendon debonding were exactly in line with the quality of the rock mass (Table 2): basically, this proved that the more competent the rock mass, the less the extent of apparent debonding and the higher the bond stress concentration at the proximal end of the anchor - and the more erroneous the conventional approach of designing on "average" bond values.

Permanent bond zone movements were smallest for site 1 anchors and greatest in site 3 anchors, reflecting the overall quality of the rock mass (Figure 2). In addition, the second anchor stressed at each site had smaller permanent movements (as well as less debonding and creep) than the first, strongly indicating some type of rock mass improvement during the loading of the first anchor. Clearly demonstrated, this phenomenon is easy to accept and understand, but had not been previously documented. This is significant when assessing relative production anchor performance, since later anchors may exhibit better stressing performance than those installed earlier.

Creep was not significant at sites 1 and 2. Interestingly, however, while creep generally increased with load, the highest amounts were at 75-100% DL, decreasing at higher loads (Figure 3). Also, while test anchor 3A showed the classic progressive failure pattern, 3B showed creep values at 133% DL lower than at 100% - 0.057 in. in 10 min as opposed to 0.064 in the same period. When restressed to 133% DL a second time, the creep was lower still (0.045 in. in 10 min.).

These data point to an irregular "ratchet"-type rock mass response at odds with the smoother, more predictable performance assumed in theory and usually found in soils. This rock mass improvement was probably due to a tightening up of the fissures and joints in the mass, in the region around and above the bond zone: crushing of the rock itself was not feasible, given its material strength.

Overall, the test verified that the originally designed bond lengths had satisfactorily high safety factors in the rock at sites 1 and 2, but merited a slight increase when installed in the poorest-quality site 3 material. Work on the production anchors proceeded accordingly.

Production Anchors

Under a previous contract, 4 ft. 9 in. square recesses, approximately 2 ft. deep, had been formed in the dam crest. At the precise location, bearing and inclination, a 12 in. diameter hole was cored about 5 ft. deep at each anchor position. A 10 in. diameter steel guide tube was then surveyed and cemented into this hole to ensure the anchor-hole drilling would have the exact prescribed starting orientation. Angles were measured by independent state-of-the-art methods to within minutes of accuracy.

A down-the-hole hammer mounted on a Nicholson Casagrande C12 diesel hydraulic track rig then drilled the 10 in. diameter anchor holes. Special hammer and rod attachments promoted hole straightness. In accordance with the specifications, the position of each hole was measured at 10 ft. intervals in the upper 50 ft. and every 20 ft. thereafter to final depth - a maximum of 270 ft.

This frequent measurement and the precision required - to within 3 in. in 100 ft. - demanded very special attention. Eastman Christensen, Bakersfield, Calif., adapted their Seeker 1 rate gyro inclinometer, normally used in oil-field applications, to this project. The Seeker could accurately measure the drill bit's position through the drill rods. Modification of its computer

Anchor	Apparent Debonding at 133%	
	Actual	Site Average
1A	21"	23"
1B	25"	
2A	40"	42"
2B	44"	
3A 3B	Failed 142" bond 73"	Possibly 108"

Table 2. Calculated apparent tendon debonding lengths, Test Anchors, Stewart Mountain Dam, AZ

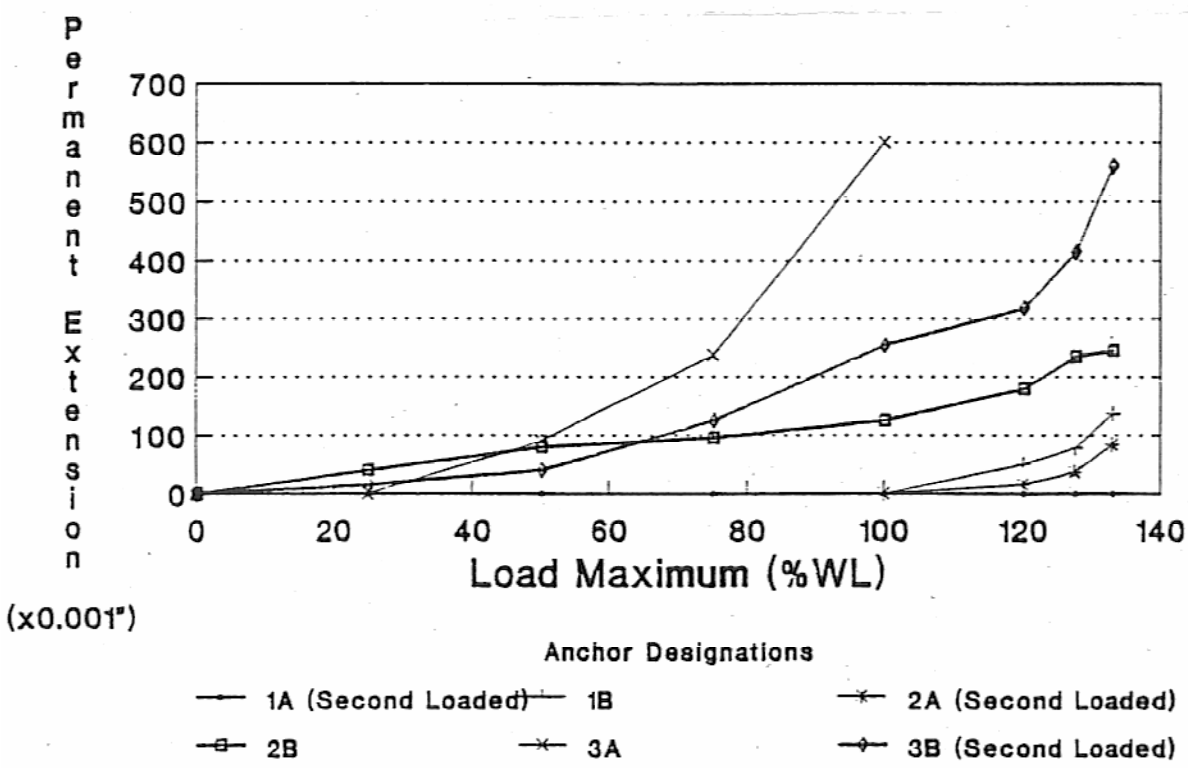


Figure 2. Net permanent displacements, Test Anchors, Stewart Mountain Dam, AZ

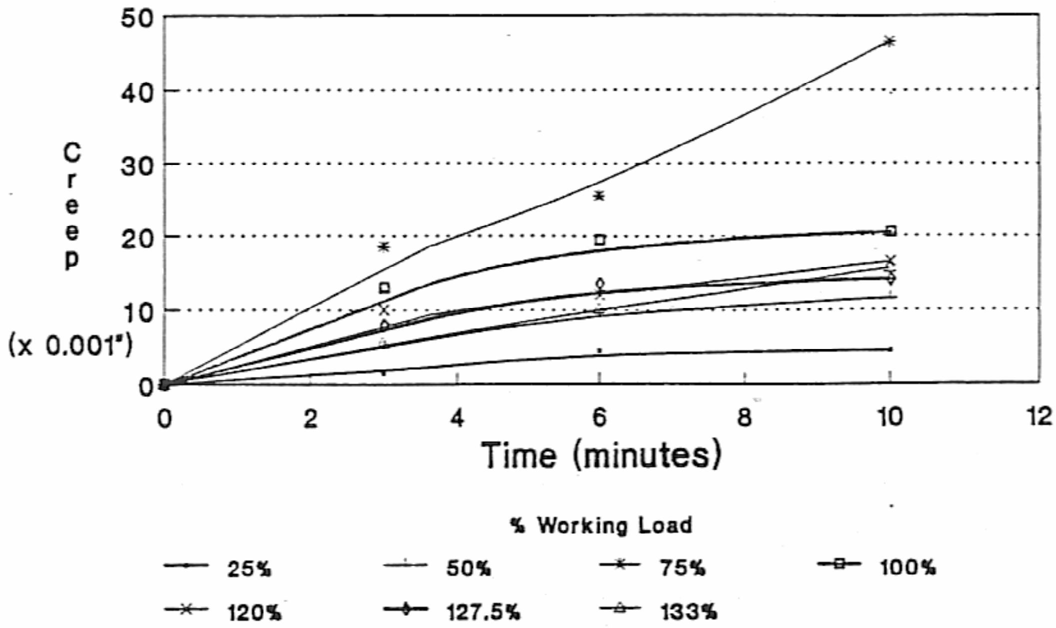


Figure 3. Creep data at each cycle maximum, Test Anchor 2A, Stewart Mountain Dam, AZ

software demonstrated the acceptability of the hole's progress within minutes at the hole collar. This minimized downtime in the construction cycle.

As a further check, government personnel ran independent precision optical surveys on randomly selected holes. These confirmed the immaculate straightness of the holes, and their acceptable bearing and inclination.

Another series of tests was run during early drilling operations, in which geophones and crack meters were fixed at the downstream face of the dam immediately adjacent to the drill hole and constantly monitored during drilling. They proved that the maximum fissure apertures and vibrations induced by drilling were tiny - barely of the order induced by natural temperature fluctuations (Table 3 and Figure 4). This has major significance for dam engineers. Even on a "delicate" dam structure, drilling a hole by rotary percussion within 5 ft. of a free face had minimal effects. This drilling method is extremely cost-effective, helping keep anchors an economical solution for a variety of dam stabilization problems.

For the thrust-block holes, a massive frame was erected up the face of the structure. The Casagrande drill mast was affixed to platforms carried on the frame. Again, special precautions were taken to ensure hole correctness and direction.

Every hole was water pressure-tested, and pregrouted and redrilled if necessary, prior to the final acceptance survey. Most test stages, which ranged in rock and concrete from 50 to 130 ft., proved tight, but other stages required as many as three pretreatments to meet the specifications of 0.02 gpm per foot of hole at 5 psi excess pressure for the free length and half that for the bond length.

Epoxy-coated strand tendons were placed in reels on special uncoilers and transported to the holes. Extreme care was taken to prevent abrasion of the epoxy coating, as each tendon was placed to full depth and tremied with a specially researched, high-strength, plasticized grout into each hole to provide the exact bond length. Fluid and set grout properties were rigorously recorded as routine quality control.

Elevation of Crackmeter (feet)	Maximum Recorded Movement (in)	Typical Daily Movement due to Temperature Effect only (in)	Approx. Distance from Meter to Hole (ft)
1520.39	0.00239	0.00284	5.0
1510.45	0.00218	0.00284	5.5
1500.34	0.00409	0.00432	5.8
1490.43	0.00510	0.00348	5.8
1480.56	1	0.00353	5.5
1470.17	1	0.00376	5.5
1460.21	1	0.00459	5.5
1450.23	1	0.00440	5.5

[1] No discernable movement was detected during the drilling operation.

Table 3. Summary of crackmeter data recorded during drilling, Stewart Mountain Dam, AZ

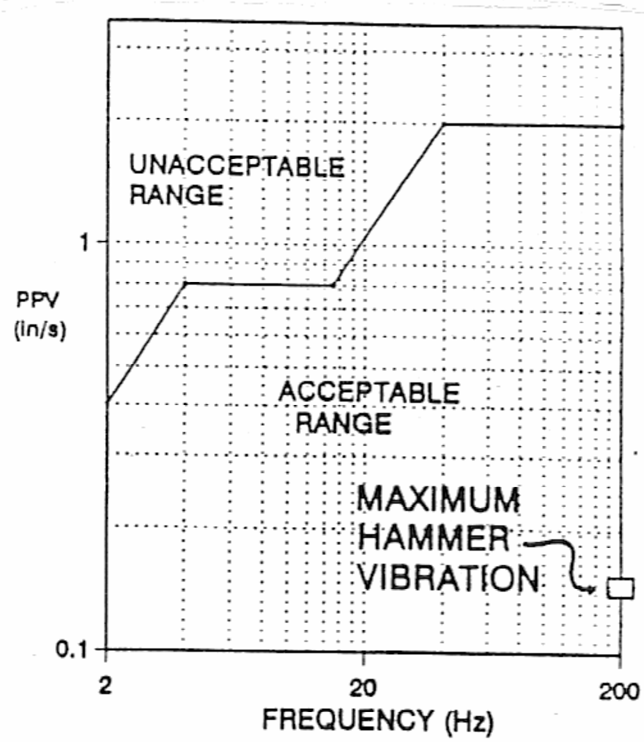


Figure 4. Hammer induced vibration monitoring, Hole 37, Stewart Mountain Dam, AZ

Stressing commenced a minimum of 14 days after grouting. To verify in detail the correct operation of the tendons, 12 cyclic Performance Tests were conducted. The remaining anchors were tested simply, under PTI Proof Test provisions. Given the high loads and long free lengths, net elastic extensions as long as 16.2 in. at the test load on the longest tendons were recored. Creep and lift-off checks rounded out the initial verification of the anchors. In all aspects, every anchor proved to have outstanding qualities, with details closely mirroring the results of the test program.

Each anchor was proved to 133% of design working load, prior to interim lock-off at 117% DL. Monitoring of the dam during stressing confirmed no significant structural deflections caused by this extra load. This was probably helped by building up the load gradually in each block of the dam to minimize any loading impact. Anchor 60 was followed by anchor 58, then anchor 6 by anchor 4, then 13 by 11, and so on. Final lock-off at 108.5% DL, and full secondary grouting of the free length, followed the 100-day observation period.

Lessons

Several features of the Stewart Mountain Dam project are unique and promise to make it one of the key dam rehabilitation projects of the decade:

- Application: The project represents the first use of high- capacity anchors to strengthen a double-curvature thin-arch dam to resist seismic effects.
- Research and development: The intensive test program confirmed many of the intricate theories of load transfer in hard rock anchors and - surprisingly - provided a clear reminder than even hard rock masses can be altered by prestressing.
- Drilling technology: Using appropriate planning, tooling, equipment and expertise, 10 in. holes can be drilled fast and extremely straight and accurately through both concrete and rock to depths of over 270 ft. Such methods appear to have absolutely no deleterious effects on the structure. And more - systems now exist to pinpoint this accuracy to within inches at this depth.
- Tendon technology: The relatively new product of epoxy coated strands appears workable in the field and seems to give excellent bonding characteristics.
- Anchor/structure interaction: If Stewart Mountain is typical of the current quality of such dams, then we can conclude that the application of tens of thousands of tons of prestress causes no structural distress to double-curvature thin arches.

Despite these technological conclusions, one of the lasting lessons of the Stewart Mountain Dam project may have been the procurement and contracting procedure. Far in advance of bidding, the Bureau of Reclamation researched current practices and experiences in all facets of the industry. As a consequence, the specifications, though by necessity very rigorous, were both eminently practical and right up to date.

The decision to invite separate technical and price proposals - independently assessed - assured that not only was the best qualified contractor for this job chosen but that it was also motivated to contribute "heart and soul" to every stage of the project's execution. As a consequence, the work was carried out virtually as an engineering joint venture, at site and head office levels, between equally committed parties.

Anchorages in Soft Rock and Soils

Littlejohn (1990a) identified four generic categories of ground anchorages, based largely on construction technique and in particular on the grouting methodology (Figure 5). Clearly rock anchorages are Type A, whereas most other anchorages are of Types B and C. The technique of underreaming

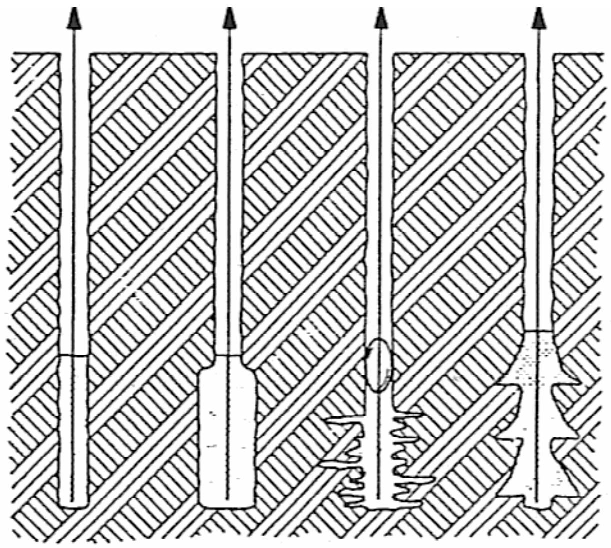


Figure 5. Main types of cement grouted anchors (Littlejohn, 1990)
Type A: straight shaft, gravity grouted
Type B: pressure grouted during installation
Type C: pressure grouted via a sleeved pipe after initial installation grout has set
Type D: underreamed, gravity grouted

boreholes to provide a large deformation in the bond zone (i.e., Type D anchor) is not common today, largely as a result of developments in other drilling and grouting methods designed to enhance pullout capacity. The following discussion concentrates on two major areas of current development and considerable future potential: grouting techniques, and corrosion protection.

Grouting

There are fundamentally four types of pressure grouting for soil (Figure 6), if the simple target of void filling is left aside. Void filling occurs when grout under its own head is simply tremied into the hole without the intention of permeating into the soil, densifying the soil or otherwise improving the soil at or away from the borehole interface. Such grouting is used in rock anchors or Type A soil anchors. Jet grouting, with the one exception of the field test run in England (Anon, 1988) is not typically a viable grouting method or concept, applicable for anchoring in the United States.

When grouting anchors in the soil, the aims are typically to permeate for some finite distance around the drill hole, to enhance the "effective bulb" diameter, and to cause some compaction of ground disturbed during the drilling process. Permeation will occur in coarse sands and gravels, but the phenomenon of "pressure filtration" will normally limit radial travel to a few inches in most cases using typical anchor grouts. This same phenomenon will squeeze out some of the integral mixing water leaving behind an anchor grout of water content considerably lower than that injected, and therefore considerably stronger than the corresponding cube results. For this reason, water/cement ratios used in cohesionless soils can be a little higher than those used for clays and tills and so on, without the drawbacks normally inherent with such mixes (Figure 7: reduced strength, significant bleed potential). Ratios for the former can be as high as 0.55 (assuming significant injection pressures are used), while it is prudent to limit water/cement ratios to 0.45 in cohesives.

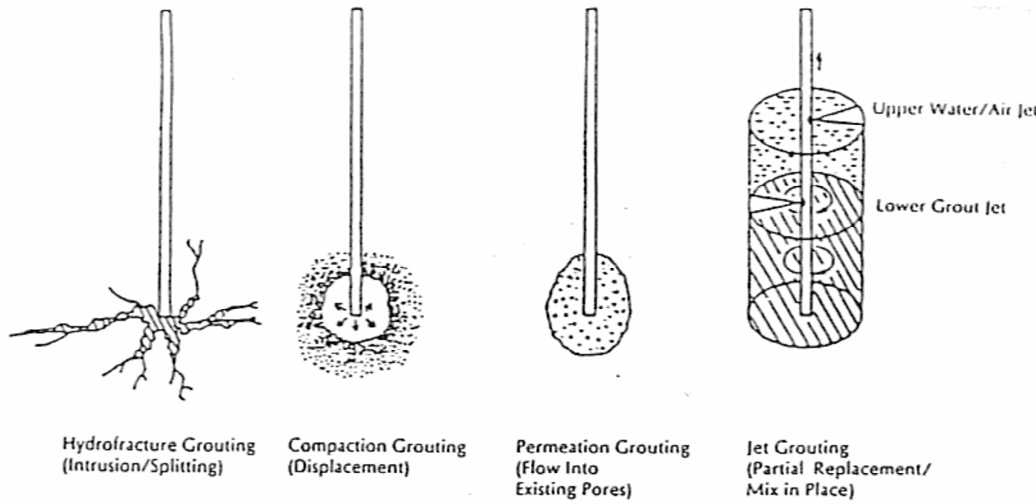


Figure 6. Basic categories of soil grouting methods

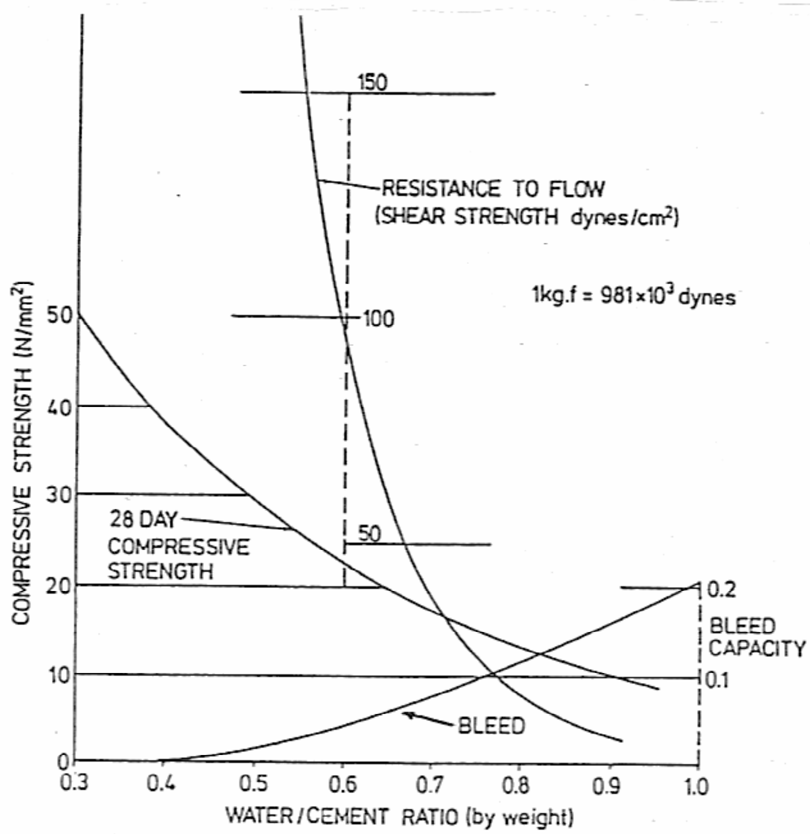


Figure 7. Effect of water content on cement grout properties
(Note $1 N/mm^2 \approx 145$ psi)

Pressure grouting also causes a recompaction or redensification of the soil around the borehold thus improving its frictional properties to the benefit of subsequent anchor load/displacement performance.

During initial grouting, pressures are limited, by design - to prevent upheaval or hydrofracture, and by operational phenomena - such as escape of grout up along the outside of the casing. "Though the head" pressures are typically seldom greater than 100 psi for casing systems, and much less for augers. The role of grouting pressure is paramount in many of the design methods used to estimate safe grout/soil bond values.

Hydrofracture grouting is therefore clearly not a factor to anticipate or be considered in primary grouting, although the benefits are being increasingly exploited in the technique of post-grouting (Type C anchor). A sleeved pipe (tube à manchette: Bruce, 1989b) can be incorporated into the bond length of the anchor. A few days after initial grouting, it is possible to reinject the anchorage zone through this pipe. In some way - possibly different in different soils - post grouting improves anchor capacity and so can be used, a priori, to safely minimize anchor dimensions, or it can be used, after initial stressing, to "repair" unsatisfactory anchors.

It is likely that the grout moves along the various interfaces of the anchorage, but especially the grout/soil interface where it both permeates and compacts. It is also possible that it causes simple enlargement of the bond zone by fracturing the initial grout and thrusting the fragments further against or into the soil mass. Equally some hydrofracture into the surrounding soil or permeation up into overlying strata may also occur, in both cases giving an improved soil condition benefiting subsequent anchor performance.

In any event, the true benefits of post grouting can only be realized systematically by conducting it in the correct fashion. This involves various features including

- the use of proper quality sleeved pipes;
- grouting through a double packer, from the bottom sleeve upwards
- placing the target regrout volume in discrete batches, to control and localize the effect.

On the contrary, it is common in some regions to simply connect the top of the regrout tube to the grout pump and inject the whole target volume in one shot, with the fervent hope that some grout will exit from each sleeve, or worse that "the grout will go where it's needed." Pressure grout is fundamentally lazy, and will exit at, for it, the easiest location. This is usually the uppermost sleeve, and so grout injected in this way has no guarantee of working on the entire bond length as anticipated. Post grouting conducted in this way leads to imprecise placing of the grout, which in turn gives rise to erratic and unpredictable results during stressing.

Grouts used in post grouting are typically of slightly higher water content than those used in the initial grouting, but still require mixing - to ensure high quality grout - in a colloidal, high speed mixer. The higher pressures needed largely to overcome line and sleeve back pressures can usually only be provided by piston pumps.

Examples of tests conducted in different soils are summarized by Bruce (1991).

As a final point on grouting, the need for strict on site quality control during mixing and injection must be stressed, especially when installing tendons in marginal soil conditions. Given the variabilities and uncertainties inherent in the soil medium alone, it is clearly logical to ensure that the materials placed in the borehole are of the highest and most uniform quality. Strength of the grout is clearly a key issue, but little attention is often paid to bleed potential. Bleed water trapped in the bond zone for geological or geometrical reasons can only do harm to anchor performance. As shown in Figure 7, bleed potential, as well as strength is controlled by the water/cement ratio. By monitoring this ratio during anchor grouting we can be satisfied that the grout is being prepared to specification and that changes are not being made deliberately, accidentally or systematically. An appropriate measuring instrument is the Baroid Mud Balance, while a Flow Cone will give a qualitative indication of grout production consistency.

In special conditions, involving grouts of low water content, long pumping distances or extreme heat, an additive may be considered to plasticize and/or retard the grout. Extreme caution must be used in the design of such grout mixes and in their preparation as typically "a little additive goes a long way." It is the author's opinion that no other type of additive, including expansive agents, should be used in anchor grouting, nor are they necessary. Any mix involving an additive should be thoroughly tested, on site, for both its fluid and set properties prior to routine use.

Corrosion Protection

With the increasing awareness of the problem of corrosion protection, more use is being made of a protective, corrugated sheath over the bond length to supplement the protection in the free length. In the United States, such an application (Figure 8) would be referred to as "double protection" to the steel, i.e., protection by both the interior grout and the surrounding corrugated sheath. By certain other standards (e.g., FIP 1986) this would only be regarded as single protection as the protective role of brittle cement grout is queried. This is a question our industry must address, especially as permanent soil anchors are being increasingly installed in urban or marine environments where the threat of corrosion due to natural and manufactured chemicals is very real.

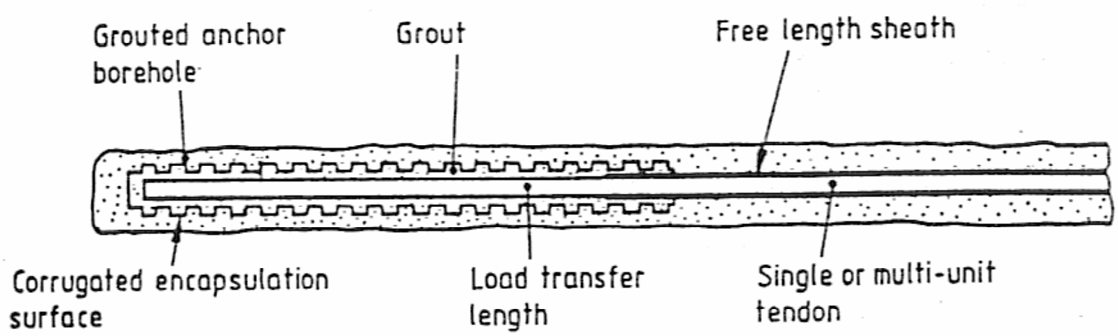


Figure 8. Encapsulation of bond length with corrugated protection

Interior grout, applied and cured prior to tendon installation is susceptible to cracking during handling unless a long rigid installation frame is used to support the pregrouted bond length. Such an option is often not practical bearing in mind the hole entry restrictions common to most tieback applications, and general logistical considerations.

The alternative is to grout simultaneously inside and outside the sheath after tendon installation. Such operations place a premium on the grouting skills on the contractor, and on the details of tendon assembly, especially the layout of the tremie tube or tube.

Although the use of spacers with intermediate clamps along the bond length has traditionally been proposed, and adopted, consideration should be given to the actual details of assembly. Without due care the strands may in fact not be correctly or efficiently separated within the corrugated tube, and may inhibit grout-steel bond development by lying in contact with the corrugated for most of their lengths, especially in shallow inclined anchors. Such problems are intensified if the corrugated diameter is imprudently minimized to reduce fabrication or overall installation costs. Close liaison between the designer, the tendon fabricator and the contractor is essential to avoid such problems, the results of which are not typically evident until stressing begins. If these cannot be resolved, there is always the option to use the epoxy coated strand as used at Stewart Mountain. Cost, however, remains a major drawback to this option.

Attention should never be relaxed on the subject of the nature of the strand surface. "Foreign substances" can be present on the tendon prior to installation. While it is conventional to think that this is a problem caused by, and therefore to be rectified by, the contractor on site during his handling and installation activities. There remains the potential for substances to be present as a result of processes used in strand manufacture, tendon assembly and/or tendon shipment activities. This is clearly the responsibility of the tendon fabricator to guard against. In general, however, the development of a uniform non-flakey rusting on the steel, prior to the installation remains a good field indicator of acceptable strand conditions. Otherwise, foreign substances have the potential, by both physical or chemical means, to reduce grout/steel bond potential. This may lead, per se, to premature tendon pull out. Alternatively and more insidiously, this reduction may push the tendon into a marginal condition where very slight and otherwise unimportant variations in anchor construction techniques may be sufficient to cause failure. Such marginal conditions, while easy to conceptualize, are very difficult to locate or prove, as the evidence may often be confusing, non-conclusive or apparently self contradictory, even though on a time related, or statistical basis the truth is patently obvious.

Finally, the most authoritative discussions of corrosion and corrosion protection have been provided by Littlejohn (1990a, b, 1992) drawn partly from his work for FIP in 1986. He also recommends that one of the "duties" of the designer is the "definition of anchorage life (permanent/temporary) and requirement for corrosion protection".

PINPILES AS IN SITU EARTH REINFORCEMENT

Background

In the last decade or so in the United States, there has been increasing use made of small diameter cast-in-place bored inclusions. Most have been designed to act as conventional load bearing piles, commonly known as pinpiles (Bruce, 1988a; 1989c, 1992a). However, these elements (4-10 inches in diameter) are finding growing popularity in the field of slope stabilization, where, installed in densely spaced patterns (Figure 9), they act as in-situ reinforcement (Bruce and Jewell, 1986; 1987). The concept of their performance is that they form a composite structure with the included soil: this structure then constitutes an in-situ barrier to arrest actual or potential slope movements.

General Features

Early applications of conventional, axially loaded pinpiles indicated, surprisingly, a positive "group effect", thought to be due to beneficial soil-structure interaction (Bruce, 1988b). This advantage was then exploited in slope stability applications in Western Europe and later - but infrequently -

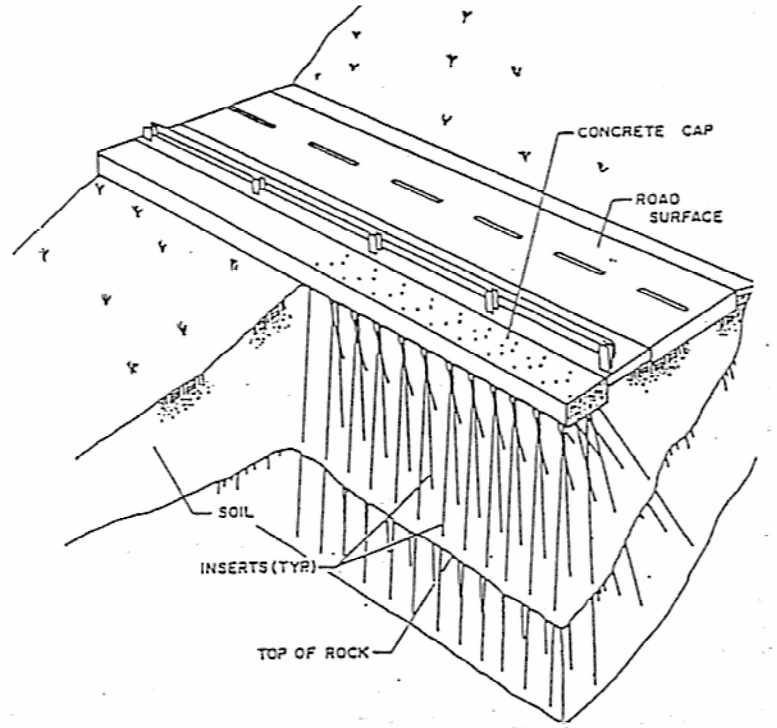


Figure 9. Typical arrangement of pinpiles as INSERTS

in the United States. In urban environments similar groups of pinpiles (or "INSERTS" in this context) can be used in cut and cover, as well as bored tunnel, construction. There the concept is to create protective structures in the ground to separate the foundation soil of the building from the zones that are potentially subject to disturbance (Figure 10). All these INSERT structures rely for their effectiveness on soil/pile interaction. This composite structure - referred to as a "Type A Wall" because of its distinctive cross sectional appearance - is intended to stop loss of soil from behind it, and to prevent sliding along potential failure planes passing through it.

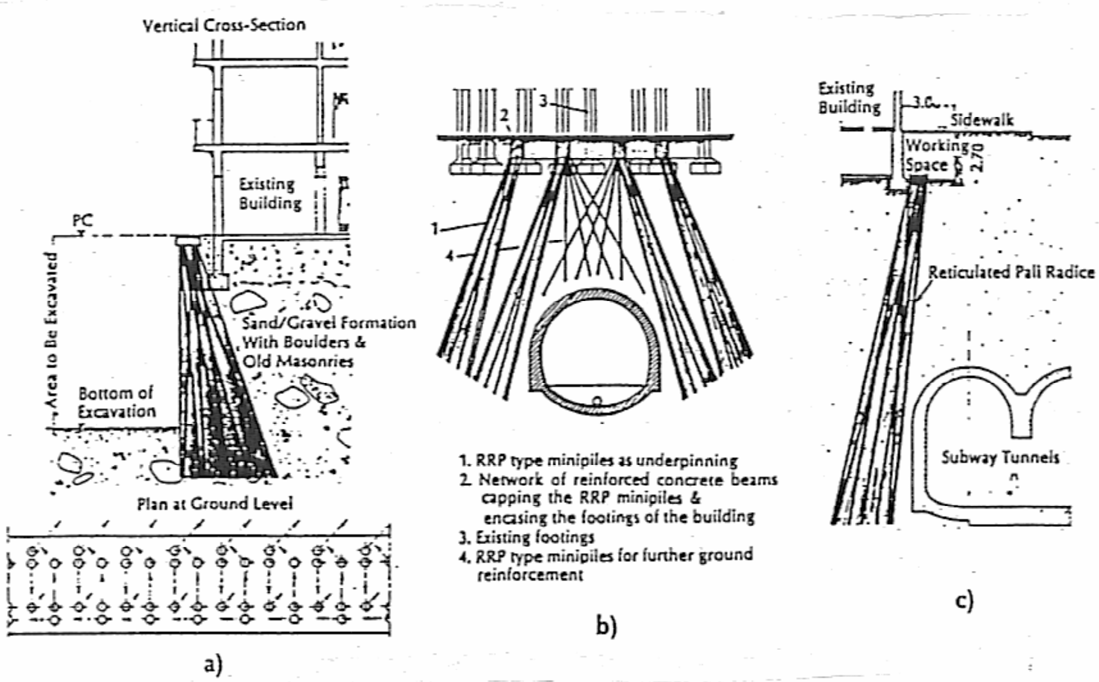


Figure 10. Applications of INSERTS: a) for cut and cover and b) and c) around bored tunnels

Design approaches continue to lag behind other aspects of the technology, but several instrumented field programs have confirmed that reinforcement stresses and overall wall movements in service are minimal, and that most probably designs have been highly conservative. Even their original proponent - Fernando Lizzi - confirmed in 1982 that "it is not yet possible to have at our disposal an exhaustive means of calculation ready to be applied with safety and completeness." In addition, an ASCE Committee (1987) also alluded to the great reliance placed on soil/pile interaction, the safe exploitation of "which is still subject to experience and intuition".

The typical approach to design is, of course, relatively simple, and involves standard basic steps:

- estimating loads (active and passive) on the wall;
- conducting a stability analysis to determine the shear force needed to maintain a required factor of safety;
- determining the number of INSERTS needed to provide the required shear resistance;
- calculations (similar to those for a conventional gravity wall) to check stability against overturning, sliding and bearing failure at the base of the wall.

Usually the INSERTS are extended into bedrock where economically possible, but, in any event, always below the potential failure plane. INSERT Walls can be constructed in close proximity to existing buildings in relatively tight access locations without the need to excavate or underpin, and without causing any decompression of the foundation soil. Given their mode of construction, as detailed below, they can be installed in any type of ground, including through boulders, old foundations or other obstructions with no constructional limitation on hole inclination or orientation.

Construction Aspects

The successive steps involved in the construction of a Type A Wall are illustrated in Figure 11. The capping beam may be installed before or after the INSERTS are formed, although field evidence suggests that the latter option allows for an earlier benefit from the reinforcement. The drilling method is chosen to ensure minimal disturbance or upheaval to the soil. Of the seven generic methods of overburden drilling (Bruce, 1989d), the most common method is rotary drilling with water flush, either via a single casing or by the duplex method, depending on ground conditions. Once the casing has been advanced to target depth it is filled with a stable, high strength cementitious grout, and the permanent reinforcement is placed. This may be a solid high strength steel bar, typically 1-2 inches in diameter, or a steel pipe of suitable dimensions, as dictated by the structural design requirements. The drill casing is then withdrawn from the hole as grout continues to be injected under pressure. The effect of the pressure grouting is three-fold in most conditions:

- it ensures all voids or drilling related disturbances to the soil are filled;
- it permeates a little into sands and gravels;
- it compacts somewhat soils around the pile that is too fine to be permeated.

Individual piles are oriented in different directions in each plane to promote the most effective soil/pile network. After installation of the INSERTS, the capping beam is simply graded over, or it can form the base of a guard rail or similar: the whole wall is thus wholly out of sight and maintenance free.

Case History: Road in Armstrong County, PA

Portions of State Route 4023 north of Kittanning, Pennsylvania, were constructed on a slope adjacent to the Allegheny River. A 240 ft. long section of the two-lane road, and the railroad tracks located upslope, experienced damage caused by slope movements toward the river. In June, 1988, and January and February, 1989, the owner conducted a subsurface exploration program and installed inclinometers to monitor the slope movements. The inclinometers indicated that a slip-plane was located approximately 26-36 ft.

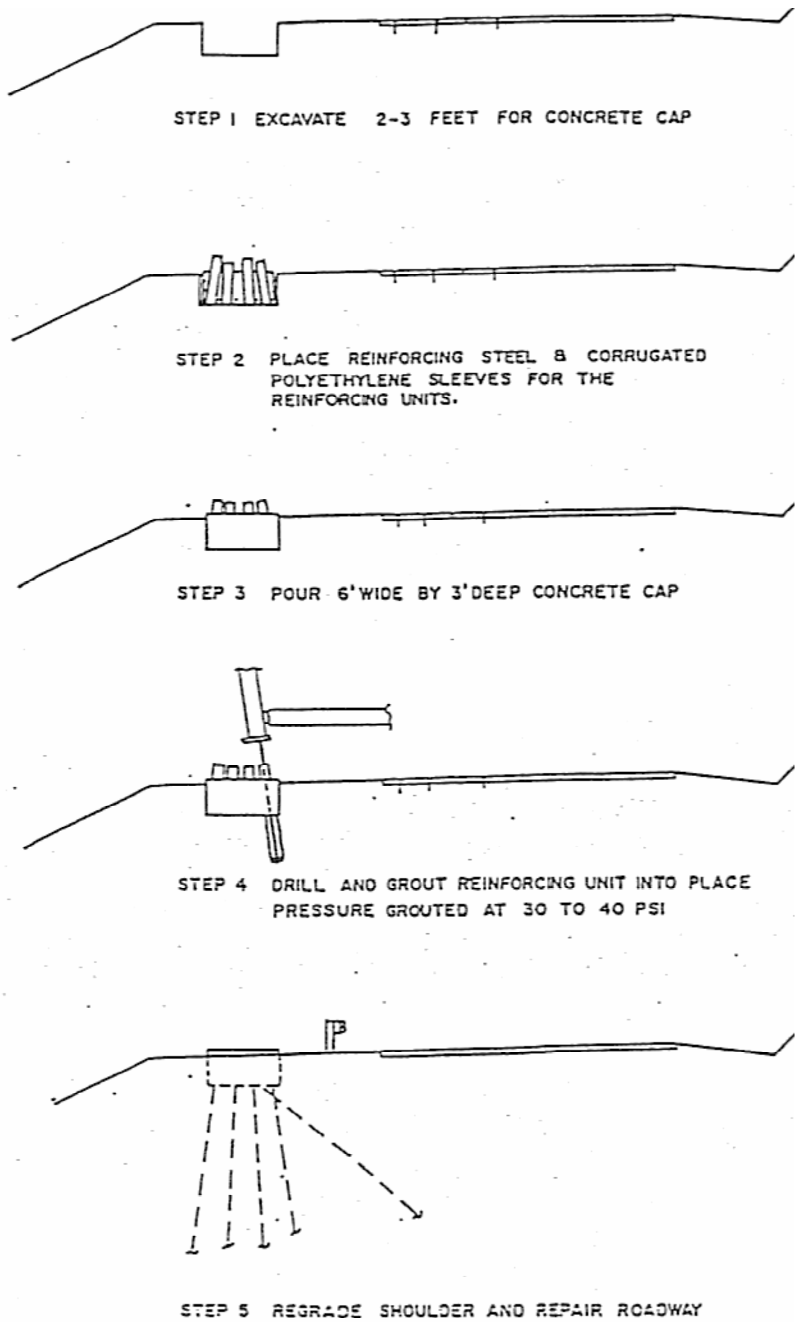


Figure 11. Typical steps in INSERT Wall construction

below the roadway and that the slope was moving at a rate of up to 0.75 inches per month downwards toward the river.

Site investigation drilling showed that a significant amount (20-30 ft.) of fill had been placed at the site apparently during the construction of the roadway and/or railroad tracks. The fill consisted of intermixed loose to medium dense rock fragments and medium stiff silty clay. Underlying the fill was a 5 to 10 ft. thick layer of stiff colluvial clay with rock fragments, in turn overlying a 3-20 ft. layer of weathered claystone. Competent rock was encountered at about 50 ft. below the roadway, and generally consisted of medium hard siltstones and sandstones.

The owner designed a repair of the failed section using an anchored caisson wall extending into competent rock. The earth pressures used for the design were based on the results of stability analyses, for which the soil along the slip-plane was assigned a residual friction angle of 17^{o}. This design provided a minimum factor of safety with regard to the overall slope stability equal to 1.5 and 1.2 for the normal and rapid drawdown conditions, respectively. A row of 3 ft. diameter caissons were foreseen at a center-to-center spacing of 4.5 ft. and located immediately downhill of the roadway. The caissons were to be connected at the top by a cast-in-place reinforced concrete cap which was to have 90 ft. long prestressed rock anchors extending underneath the roadway at 7 ft. lateral intervals.

In 1989, the owner accepted the alternative contractor design employing an INSERT Type "A" Wall. The wall consisted of four rows of pinpiles extending across the slip-plane and into competent bedrock. It comprised two equal length sections designated as Wall A and Wall B. Wall A contained a higher density of piles than Wall B, because the top of weathered rock dipped to a lower elevation in the area of Wall A which resulted in a larger volume of soil to be stabilized in this area. In general, Wall A contained 4 piles per lineal meter, and Wall B contained 3 piles per lineal meter. Besides providing a significant cost savings over the original design, the selection of the INSERT Wall allowed for one lane of roadway to remain open during construction (February to May, 1989). The wall was constructed as described above, with the cap poured after pinpile installation for practical reasons.

To monitor the INSERT Wall performance, two sections of the wall were instrumented with strain gauges, inclinometers, telltales, and survey pins. The inclinometers yielded the most useful information regarding the performance of the wall. The data for inclinometers located relatively close to and within the wall indicated that up to 1.5 inches of horizontal movement occurred during the 75-day construction period, but that a maximum of 0.3 inches of movement occurred in the 7-month period following the completion of the wall.

Overall, the inclinometer data indicated that the wall performed as expected, and had effectively stopped the slope movements at the site. These data also confirmed that some deflection of the relatively flexible INSERTS was required to mobilize their lateral resistance.

CEMENT GROUTING

Rock grouting has been conducted in the United States for almost a century and soil grouting is well into its fifth decade (Bruce, 1992b). Various aspects of grouting are discussed elsewhere in this seminar by Welsh, and others, while this country has recently hosted the major international grouting conference of the period (ASCE, New Orleans, 1992). In this section, attention is focused on certain novel developments in the practice of cement based grouting for fissure injection and soil permeation.

Rock Grouting Methodology

In general, U.S. grouting practice as described comprehensively by Houlsby (1990) and Weaver (1991) may be regarded as "conservative" in comparison with that of other countries. This traditionalism is reflected in specifications relating to drilling type (rotary), permeability testing (simple), grout mix design ("thin"), and pump type (Moyno). However, there are two areas where major change is occurring: automatic parameter recording, and staging philosophies

- Parameter recording by electronic means has become standard practice on all federal jobs and on most others also. This may range from a simple "in the field" chart recorder, to the telemetric system, devised by the Bureau of Reclamation at their massive New Waddell Dam project in Arizona (Aberle, et al., 1990). There, electronic pressure transducers, magnetic flow meters and density meters in the field constantly relay data via a Remote Telemetry Unit to a Central Telemetry Unit, where all the grouting parameters are displayed in real time. Graphical data consist of flow rate, pressure, bag rate, and water-cement ratio. Numerical data include hole and stage number, target pressure, volume, density, w/c ratio, take rate, depth, cumulative take, date and time. Numerical data from six stages can be monitored instantaneously. The field inspector is in constant communication via radio with the CTU office to exchange information and instructions. Data are stored for future technical analyses and reports, and also for payment purposes. Aberle et al. concluded that these systems are extremely valuable and greatly help to direct and optimize the grouting.
- Regarding staging practices, the competent rock available and selected for past sites was ideally suited to ascending stage operations, and this method has become the traditional standard. Descending stage grouting is becoming more com- mon, reflecting the challenges posed by more difficult site conditions in the remedial and hazardous waste markets. The work described by Weaver et al. (1992) related to the sealing of dolomites under an old industrial site at Niagara Falls, NY, represents a statement of the best of American practice.

In some cases of extremely weathered and/or collapsing bedrock, even descending stage methods can prove impractical, and two recent projects illustrate innovative trends. Firstly, at Lake Jocassee Dam, SC, a remedial grouting project was conducted (Bruce, et al., 1992) to reduce major seepages through the Left Abutment of the dam. *Given the scope of operating within innovative contracting procedures*, the contractor was able to vary his methods in response to the extremely variable ground conditions actually encountered. Some holes permitted ascending stages, others needed descending stages, while the least stable holes had to be grouted through the rods during their slow withdrawal.

A second example is the grouting of poorly cemented hard rock backfill 2700 ft. below ground level in a copper mine in Northern Ontario, Canada (Bruce and Kord, 1991). This medium proved so difficult to drill that none of the conventional grouting methods could be made to work. Instead, the first North American application of the MPSP system, devised by Rodio, in Italy, was called for. The Multiple Packer Sleeved Pipe System is similar to the sleeved tube (tube à manchette) principle in common use for grouting soils and the softest rocks (Bruce, 1982). The sleeve grout in the conventional system is replaced by concentric polypropylene fabric collars, slipped around sleeve ports at specific points along the tube. After placing the tube in the hole, the collars are inflated with cement grout, via a double packer, and so the grout pipe is centered in the hole, and divides the hole into stages (Figure 12). Each stage can then be grouted with whatever material is judged appropriate, through the intermediate sleeved ports. Considerable potential is foreseen in loose, incompetent, or voided rock masses, especially karstic limestones (Bruce and Gallavresi, 1988).

As a final note, there remains considerable activity in bulk infill, principally associated with older, shallower mining operations in the

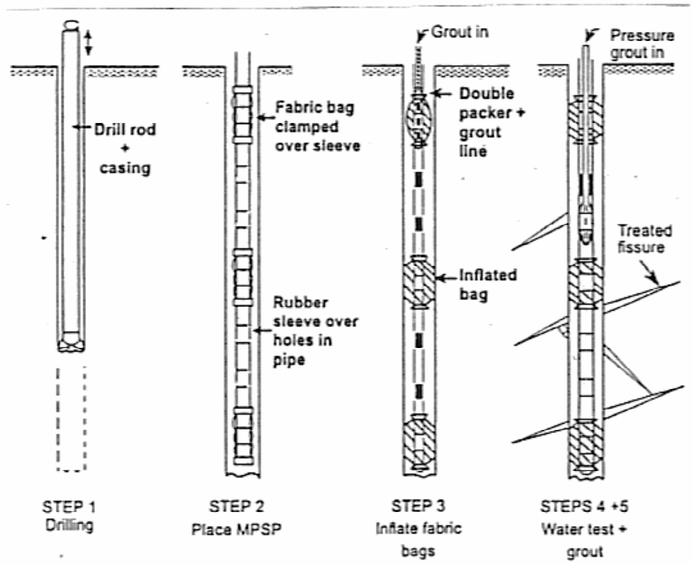


Figure 12. Typical steps in MPSP installation and grouting

Appalachians, and in Wyoming. Rotary and rotary percussive drills, often of water well drilling type, are common, with the void filling (either partial or total) being executed with cementitious grouts or concrete prepared in large scale site batching plants. Innovations are restricted to improved automated parametric recording and the development of special foamed grouts intended to extinguish mine fires.

Materials

Microfine cement grouts were introduced into the United States in 1984. Manufactured in Japan, the earliest example (MC500) is a mixture of finely ground Portland cement and slag in the ratio of about 4:1 (Karol, 1990). It can be used like a conventional cement grout with 4-5 hour setting time, or with sodium silicate to accelerate set to 1-3 minutes. It has been used on many relatively small projects in North America.

Clarke, et al., (1992) describe the use of two new products, MC300 (an ultrafine Portland of Greek origin) and MC100 (ultrafine slag) which can be mixed in varying amounts with dispersant to give a range of hardening times. Both are finer ground than MC500, and so have enhanced penetration potential. Other foreign manufactured materials are also available, including the aptly named "Stealth" grout. All these prebagged materials, however, despite their technical attractions, do share certain problems associated with availability, handling, preparation and cost, and much favorable attention has recently been focused on an alternative principle.

The Cemill[R] technology (DePaoli, et al., 1992a) permits microfine grouts to be produced, on site, from normal cement grouts, in a wet regrinding process. Excellent grain size characteristics are produced (Figure 13), resulting in enhanced penetrability characteristics (Figure 14). Yet to be exploited in the U.S., this method is proving highly successful - technically and economically, in Italy.

Equally attractive to the U.S. market is the concept of improving the penetrability of cementitious grouts by fundamentally examining their rheological and internal stability characteristics. The Mistra[R] series of grouts (DePaoli, et al., 1992b) has already been successfully exploited in Europe and provides extremely stable mixes with greatly reduced cohesion (Figure 15). Both these features generate major technological and economical benefits, and the concept is attracting favorable interest in the U.S.

	grain size (μmm)					
	D 95	D 85	D 60	D 50	D 15	D 10
CEMILL® 6	15.0	9.0	6.0	5.0	1.3	0.9
CEMILL® 9	9.0	5.5	3.5	2.5	0.6	0.4
CEMILL® 12	6.0	4.0	3.0	2.2	0.4	0.3
ONODA MC-500	8.0	60.0	4.5	4.0	2.5	2.0
Portland 525	40.0	22.0	11.0	8.0	2.5	2.0
bentonite	60.0	40.0	15.0	10.0	1.7	1.2

a b c sands for injection tests
a $\gamma = \gamma_{max} = 1.713$ g/cm^3
b $\gamma = \gamma_{max} = 1.701$ g/cm^3
c $\gamma = \gamma_{max} = 1.690$ g/cm^3

d bentonite
e Portland 525 cement
f ONODA MC-500 cement
g h i CEMILL® mixes

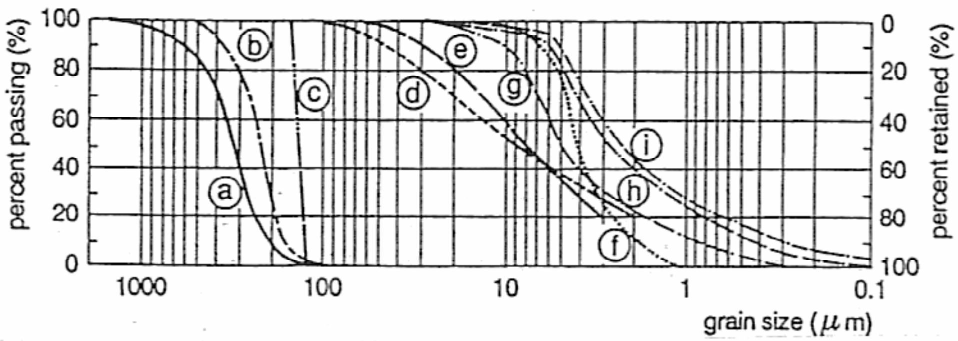
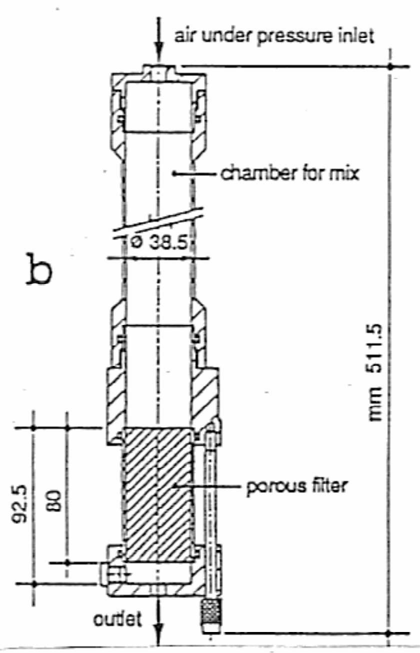


Figure 13. Grain size distribution curves for sands, dry materials and grouts (DePaoli et al., 1992a)

filter no.	permeability (m/s)		grain size (μm)					porosimetry (μm)							specific surface cm^2/g	retaining capacity (μm)
	theoretical Hazen (C = 1.45)	experim. (permeam.)	D 95	D 60	D 15	D 10	U	theoretical (Kozeny) D 80	D 50	D 30	experim.(Hg porosimetry) D 95	D 85	D 15	D10		
07	$5.9\cdot10^{-3}$	$3.8\cdot10^{-3}$	1500	900	700	640	1.41	300	240	150	380	300	170	160	28	70
06	$2.3\cdot10^{-3}$	$8.3\cdot10^{-4}$	750	620	450	400	1.55	160	133	90	360	260	130	124	37	60
04	$7.7\cdot10^{-4}$	$4.5\cdot10^{-4}$	700	480	250	230	2.09	110	90	60	300	140	70	64	56	40
01	$2.8\cdot10^{-4}$	$1.6\cdot10^{-4}$	400	230	160	140	1.64	58	49	32	120	64	46	44	111	10
005	$1.4\cdot10^{-4}$	$9.5\cdot10^{-5}$	180	120	110	100	1.20	35	25	18	90	46	32	30	125	5

a

b



c

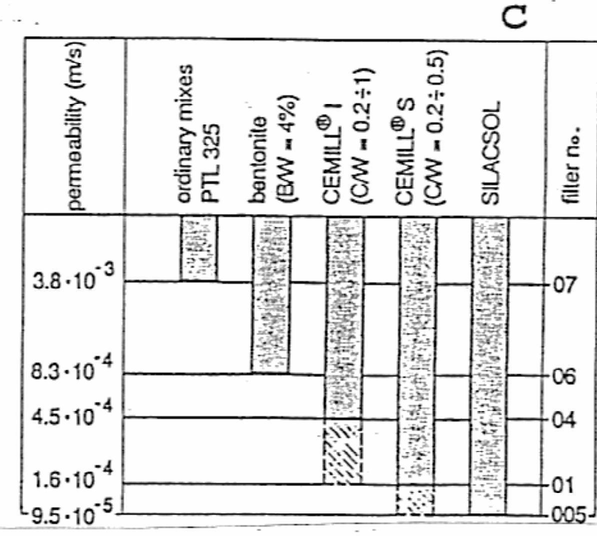


Figure 14. Injection test details: a) porous stone filter characteristic b) apparatus, and c) penetrability limit of different mixes into filters (DePaoli et al., 1992a)

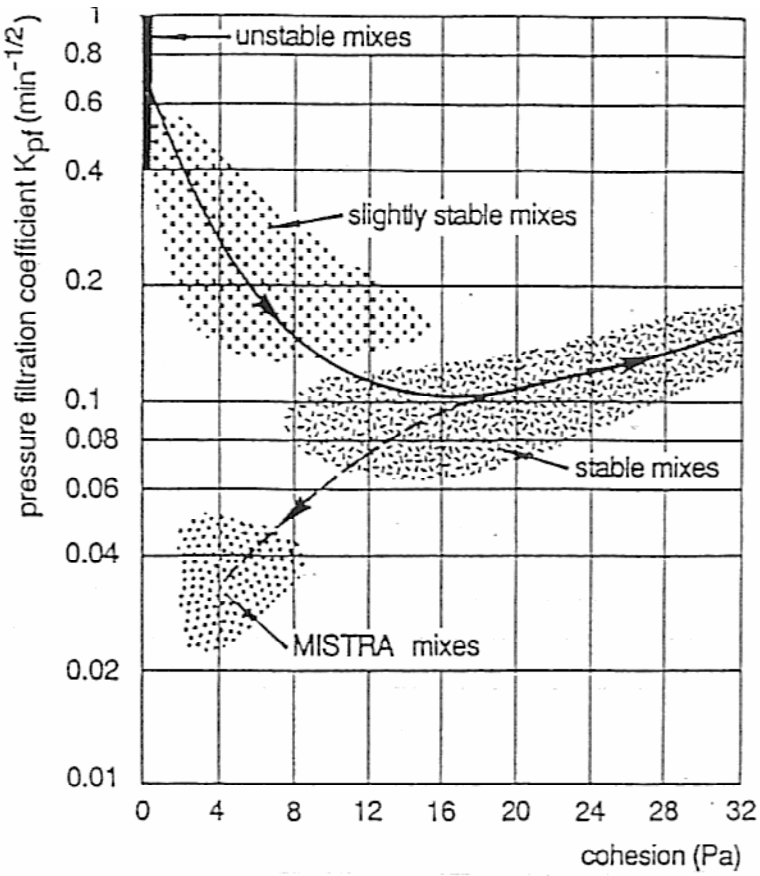


Figure 15. Relationship between stability under pressure and cohesion for the different types of mixes (DePaoli, et al., 1992b)

Contracting Practices

Weaver (1991) lists the elements necessary for a successful grouting project as follows:

- a design accommodating the site geological conditions;
- specifications that allow or facilitate modifications to the grouting program as the site conditions are revealed;
- an "experienced, competent, cooperative and honest" contractor;
- appropriate materials, equipment and techniques;
- knowledgeable inspection staff, and
- an effective quality assurance program.

While reviewing the history of grouting in the U.S., however, it is clear that rarely have these elements been simultaneously in place. The author believes that there are two fundamental reasons: inflexible specifications, and "low bid" procurement systems.

Regarding specifications, these must "be tailored to the project in hand and to the objectives to be accomplished" (Weaver, 1991). Instead, successive generations of specifications have been cobbled together from sections lifted from previous documents, and often contain "boiler plate" sections which may be contradictory and always perpetuate the use of outmoded procedures and/or inappropriate materials. Specifications of this nature have dissuaded domestic contractors from innovating and have discouraged foreign specialists from competing.

The procurement system has proved equally stifling: the low bidder on a tightly specified job invariably wins the award, although he then operates as little more than a broker of labor, equipment and materials. However, in recent years there have been encouraging signs that a more enlightened approach is surfacing (Nicholson and Bruce, 1992).

As a first step, stronger prequalification criteria are being applied to prospective bidders and their personnel. Specifications are being changed to "performance" types, so encouraging bidders to be creative and innovative, and most significantly, awards are being made not just on the basis of a low bid (Nicholson, 1990). In addition, many owners, including federal agencies, are promoting the concept of having "partnering" agreements between all the involved parties. This concept is a recognition that every contract includes an implied covenant of good faith. The process attempts to establish working relationships through a mutually developed formal strategy of commitment and communication. It tries to create an environment where trust and teamwork prevent disputes, improve quality, promote safety and continue to facilitate the execution of a successful project. Significantly, it is wholly endorsed by the Associated General Contractors of America, a group which has not always favored the more innovative procurement procedures in the past.

FINAL REMARKS

In each of the three major technologies addressed, there are major advances being made. In the case of ground anchorages these are largely in construction technique, and in the improved understanding of anchor-ground-structure interaction. For pinpiles used as Type A Walls, the developments are in design approaches and the benefits will accrue in the form of more cost effective schemes. Changes in grouting cover a wider range of facets including methodologies and materials. Perhaps the most significant changes, however, may arise from the alternative bidding practices, and other contractual changes which are being promoted. If allowed to flourish, they will encourage and reward innovation and imagination, while at the same time they will foster the spirit and reputation of geotechnical specialists in this country: their horizons have been too often lowered in the past by the pressures exerted by the legal profession and its heavy handed acolytes.

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