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THE CONSTRUCTION AND PERFORMANCE OF PRESTRESSED GROUND ANCHORS IN SOILS AND WEAK ROCKS:

A PERSONAL OVERVIEW

by

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THE CONSTRUCTION AND PERFORMANCE OF PRESTRESSED GROUND ANCHORS IN SOILS AND WEAK ROCKS: A PERSONAL OVERVIEW

Dr. Donald A. Bruce[1]

Summary

The paper reviews aspects of the construction and performance of prestressed ground anchors installed in soils and weak rocks. Attention is focused on good practice for drilling, tendon fabrication and handling, grouting, stressing and testing. Regional variations - not always logically justified - are described. The performance of anchors in a variety of geological conditions is illustrated with reference to recent full scale test programs.

1. Introduction

In the United States there has been steady growth in the use of prestressed anchors in materials other than competent or hard rock, in the years since their introduction in the late 1960's. This expansion has to a large extent been driven by specialty geotechnical contractors who both imported foreign developments and conducted their own basic researches, in locally prevailing conditions. With such data and experience, these engineers have had the confidence to promote "alternative solutions" for difficult earth support problems to receptive designers and owners. In turn, being engaged in the details of design and performance monitoring has given these parties the close exposure necessary for the real transfer and understanding of the technological concepts, and this knowledge has generated positive feedback for the contractors.

An excellent example of this was the forum provided by the ASCE Specialty Conference on Earth Retaining Structures at Cornell University in June, 1990, which was a worthy follow-up to the classic "Diaphragm Walls and Anchorages" conference organized by I.C.E. in London in 1974. There have been a myriad of other sessions and technical papers in the intervening years, as well as a number of textbooks (e.g., Hanna, 1982; Xanthakos, 1991), and the emergence of several national standards. Of these, the new British Code of Practice (BS8081) is by far the most comprehensive, while the forthcoming FIP "Recommendations for the Design and Construction of Prestressed Ground Anchorages", also coordinated by Prof. G.S. Littlejohn, should ensure a high degree of international standardization.

In the United States, most projects run with reference to the P.T.I. Recommendations (1986) while the Federal Highway Administration remains a focal point for ongoing research, (e.g., "Demonstration Projects Program", 1990).

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Against this generally positive background, it is clear from observing anchor practice across the country that there remain considerable variations in execution, and so in performance expectations, since mode of construction is a prime determinant of anchor capacity. Many of these differences reflect the logical response to local conditions - both geological and economic - but certain other variations now appear to be due to a relative inertia in research and development at one extreme and, regrettably, a lack of understanding of fundamentals at the other extreme. Such wide disparities are not so prevalent in the field of rock anchors, partly due to the availability of comprehensive case history type compendia such as Littlejohn and Bruce (1977), Barley (1988), and Bruce (1989a). In addition, the construction of rock anchors tends to be simpler, as virtually all are of the Type A category (Figure 1).

Comparable publications dealing with full scale anchors in soils and poor rocks are much less frequent, although the topic does lend itself to mathematical and laboratory modeling. In contrast to rock anchors, anchors in other media can be of Types A-D, each with its own construction implications. It is the purpose of this paper to firstly make some observations on aspects of construction, and to then illustrate the performances which can be expected using contemporary methods of installation. Hopefully, the paper will be of use to the owner or engineer faced with adjudicating the merits of contractors' proposals featuring substantially different construction procedures.

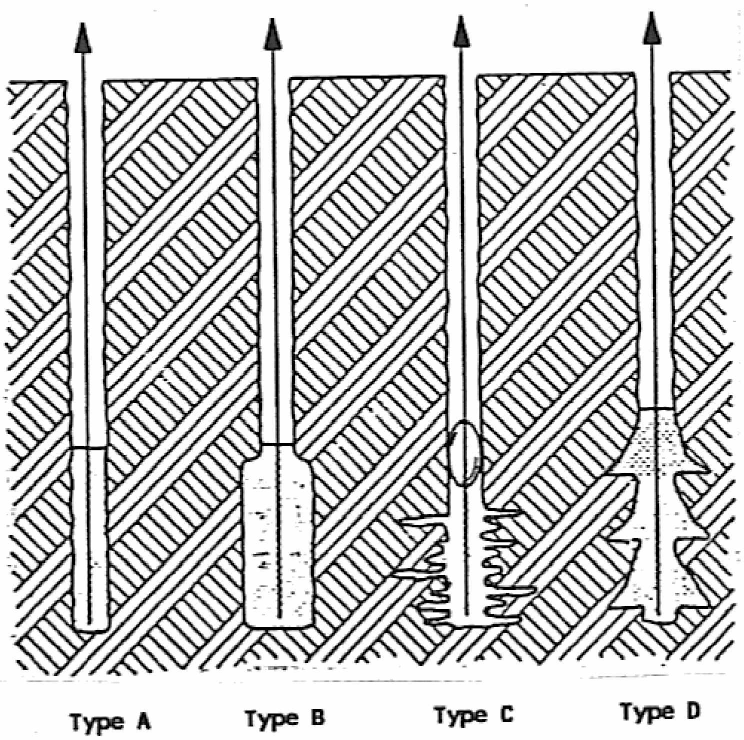

Figure 1. Main types of cement grouted anchors (Littlejohn, 1990)
Type A: straight shaft, gravity grouted
Type B: pressure grouted during installation
Type C: pressure grouted via a sleeved pipe after initial installation grout has set
Type D: underreamed, gravity grouted

2. General Aspects of Construction and Testing

2.1. Drilling

Perhaps more than any other aspect of construction, drilling techniques and methods seem to have greatest regional variation across the country. For example, eastern contractors tend to favor the use of some form of casing system, usually with water flush, while on the west coast, the use of larger diameter hollow stem augers has prevailed. Overburden drilling methods should ideally be related directly to the anticipated soil conditions, but appear to be most often dictated by the historical proclivities - and the equipment available - of the individual contractor.

Taken to extremes, this inertia can in fact rule out the possible use of anchors in a given project. For example, in a recent contract in Los Angeles, a relatively high degree of lateral rstraint had to be applied to a concrete diaphragm wall to satisfy local design rules. The use of the traditional large diameter hollow stem drilling method would have involved such a large number of low capacity anchors (due to the limited grout/soil bond capacity potential with this method) that the wall's structural integrity would have been threatened by the number and diameter of perforations through it. The use of a flushed casing method permitted the prestress to be distributed into a lower number of higher capacity anchors of much smaller diameter. Thus, the change of drilling method not only reduced the cost of the retention system, but in fact made anchoring a practical option in the first place.

The choice and application of the most appropriate drilling method must also reflect the overall site conditions in general. For example, the use of air - an excellent "scavenger" and an aid to fast penetration - as a flushing medium in urban areas should normally be discouraged. There have been numerous examples of structural damage to adjacent buildings as a result of ground fracture or upheaval, or simply the consequence of massive air losses acting directly on base slabs. Alternatively, in other applications, air flushing is a possible remedy against the water softening of cohesive soils which can subsequently reduce bond potential.

In such conditions without obstructions, augering may indeed be the most apposite choice, especially if drill depths are not great and subsequent anchor capacity is moderate. However, in soils with low cohesion or very poor cementation, the uncontrolled use of augers may result in severe decompression or cavitation of the soils around the borehole, again leading to reduced bond potential and the risk of adjacent structural distress.

<u>Table 1</u> summarizes the most common contemporary overburden drilling methods, on a general basis (Bruce, 1989b). Drilling costs depend upon so many variables that no attempt has been made to quote "typical" or "market" rates for each category.

DRILLING METHOD	PRINCIPLE	COMMON DIAMETERS AND DEPTHS	NOTES
1. Single Tube Advancement			
a) Drive Drilling	Casing, with "lost point" percussed without flush.	2-4" TO 100'	Hates obstructions or very dense soils.
b) External Flush	Casing, with shoe, rotated with strong water flush.	4-8" to 150'	Very common for anchor installation. Needs high torque head and powerful flush pump.
2. Rotary Duplex	Simultaneous rotation and advancement of casing plus internal rod, carrying flush.	4-8" to 200'	Used only in very sensitive soil/site conditions. Needs positive flush return. Needs high torque.
3. Rotary Percussive Concentric Duplex	As 2, above, except casing and rods percussed as well as rotated.	3-1/2 -7" to 120'	Useful in obstructed/bouldery conditions. Needs powerful top rotary percussive hammer.
4. Rotary Percussive Eccentric Duplex	As 2, except eccentric bit on rod cuts oversized hole to ease casing advance.	3-1/2 -8" to 200'	Obsolescent, expensive and difficult system for difficult overburden. Largely restricted to water wells.
5. "Double Head" Duplex	As 2 or 3, except casing and rods rotate in opposite senses.	4-6" to 200'	Powerful, newer system for fast, straight drilling in worst soils. Needs large hydraulic power.
6. Hollow Stem Auger	Auger rotated to depth to permit subsequent introduction of tendon through stem.	6-15" to 100'	Hates obstructions, needs care in cohesionless soils. Prevents application of higher grout pressures.

Table 1. Summary of Overburden drilling methods for anchors (after Bruce, 1989b)

Within the drilling industry overall, the trend continues towards the use of higher powered diesel or electro-hydraulic self propelled rigs. These have advantages in maneuverability, torque, thrust and pullup and have operational and environmental efficiency unmatched by older models including airtracks. Modern rigs often have long masts to minimize or eliminate rod or casing changes, and mechanical foot clamp/breakout wrench attachments to aid rod or casing handling by making it quicker, safer and less labor intensive. It is reasonable to assume that such rigs promote straight and accurate drilling in the hands of skilled operators - and in this regard the typical industry standard for maximum hole deviation is about 1 in 30 to 50 depending on a myriad of factors.

For cased, or duplex drilling operations, high pressure, high volume water flush is most common in appropriate geologies. Not only does it permit fast penetration but it tends to remove fines from the adjacent soil, thus rendering the subsequent anchor pressure grouting more effective, and so enhancing bond development potential, as with Type B anchors.

Air flushing alone, as noted above, is potentially damaging and is therefore quite rare in soils. However, recent developments with foaming additives could well see a resurgence in low volume air based flushing in certain geological or logistical circumstances. Similarly, there is a growing acceptance of polymer slurries in overburden drilling, often to the exclusion of simultaneous casing advancement. Care should be exercised, of course, to verify that any such flushing materials in no way will compromise subsequent anchor performance. For such reasons, the use of bentonite slurry alone as a flush must be avoided, although the advantages of certain "self hardening drilling muds" - cementitious mixes, with bentonite as an integral bulking component - have been demonstrated in voided or cavitated conditions.

The technique of underreaming 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.

2.2. Tendon Fabrication and Handling

There are numerous guides to good site practice, as referenced above, and all those guidelines still apply.

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 2) 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.

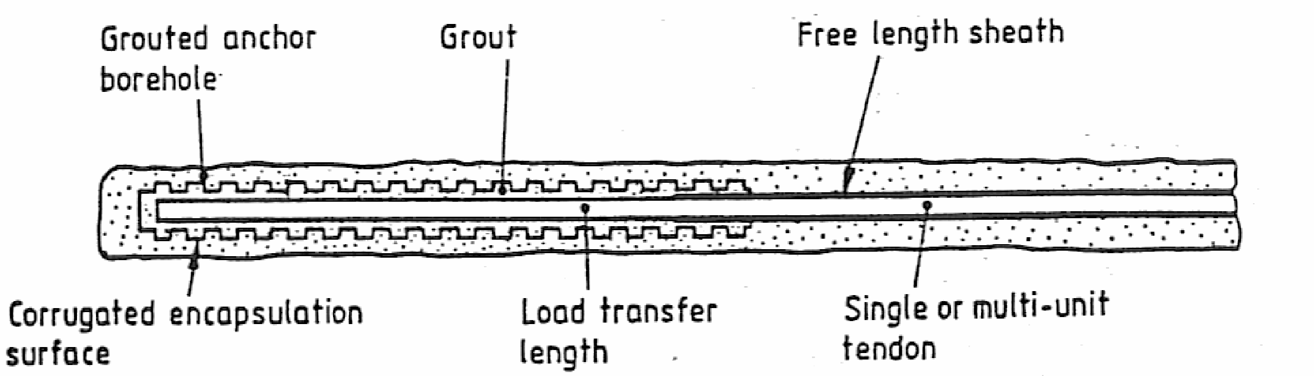

Figure 2. Encapsulation of bond length with corrugated protection

Interior grout, applied and cured prior to tendon intallation is liable 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 tubes.

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 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.

As a final point, 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. Equally, the converse is also valid.

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 unimiportant 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.

2.3. Grouting

There are fundamentally four types of pressure grouting for soil (<u>Figure 3</u>), if the simple target of <u>void filling</u> 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. <u>Jet grouting</u>, with the one exception of the field test run in England (Anon, 1988) is not currently a viable grouting method or concept, applicable for anchoring in the United States.

When grouting anchors in soil, the aims are typically to <u>permeate</u> for some finite distance around the drill hole, to enhance the "effective bulb" diameter, and to cause some <u>compaction</u> of the ground if 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

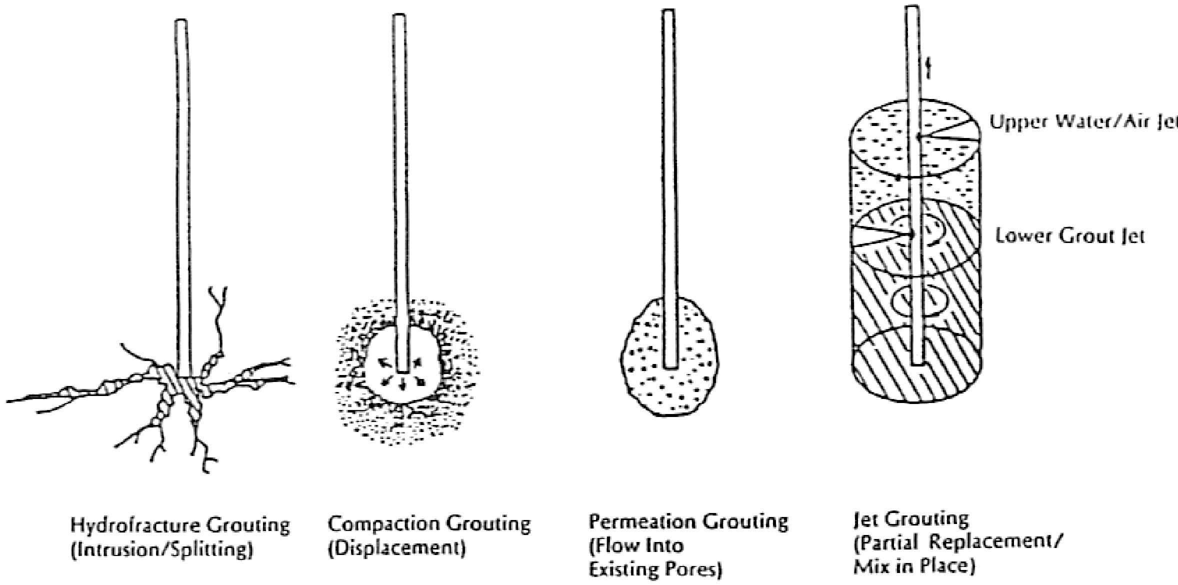

Figure 3. Basic categories of soil grouting methods

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 4: 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.

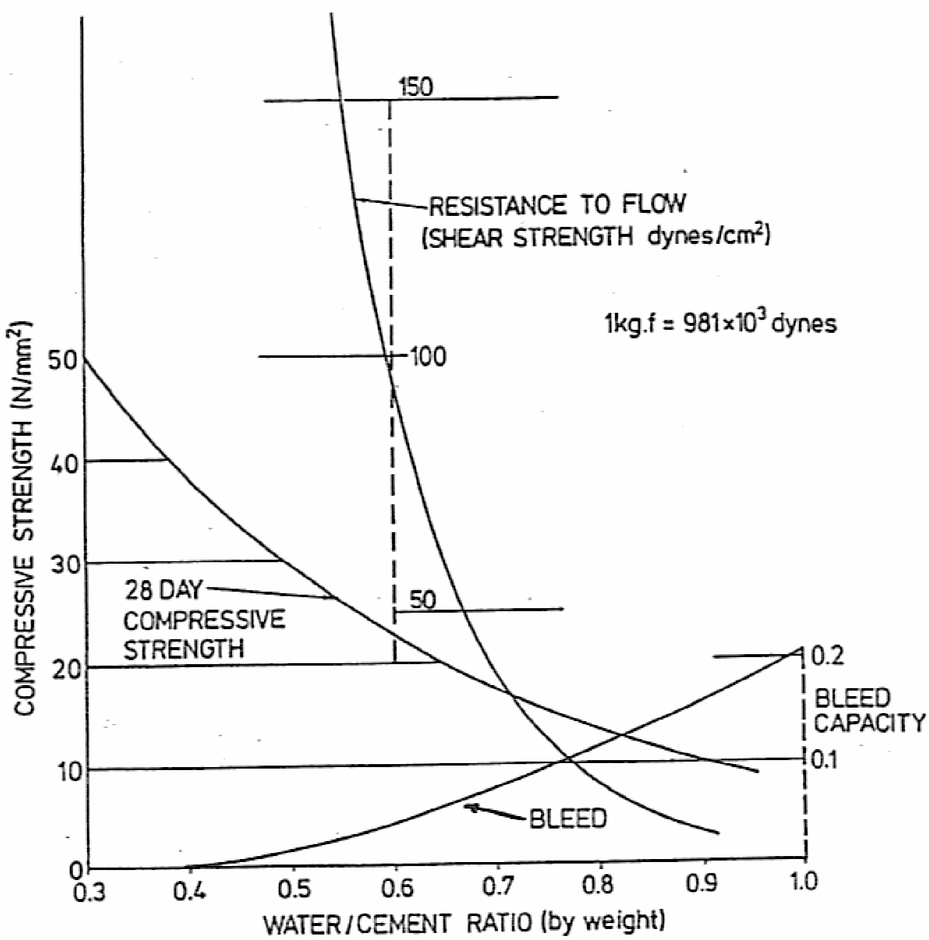

Figure 4. Effect of water content on grout properties
Note: 1 N/mm^2 $\approx$ 145 psi

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

During initial anchor 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. "Through the head" pressures are typically seldom greater than 100 psi for casing systems, and much less for augers. The role of grouting pressure is clear 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 1989c) 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 anchor, 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 in 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.

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 4, 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.

2.4. Stressing and Testing

There is still a degree of confusion in some quarters regarding measured extensions and their limits of acceptability, despite the efforts of PTI (1986).

In routine Proof Testing (Figure 5) total tendon extensions are being recorded. Such extensions comprise primarily elastic extension of the tendon, plus any permanent movement of the anchor zone through the soil mass. (There are other contributory movements but they are normally relatively small and unimportant.) However, these data are judged for acceptability against control lines based on purely elastic criteria, i.e., the elastic extensions of two theoretical free lengths.

Especially in the case of anchors in soils and weak rocks, the rigid application of these control lines can lead to the rejection of anchors on account of their having "gone outside" the control envelope. This may be the case even though they have performed - elastically - perfectly well, but have owed their

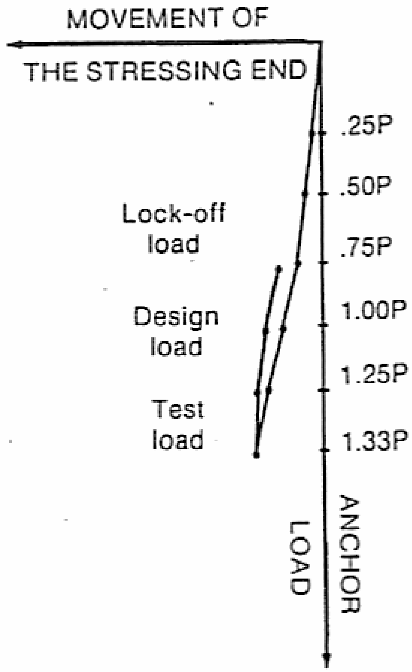

Figure 5. Sequence for proof testing (PTI, 1986)

overlong total extensions to permanent components, which may tend to increase with load. In such cases, it is logical to permit the anchor to be returned to Alignment Load and restressed to the target load. If the extension is now within the control lines, the anchor is clearly performing adequately. If not, but the performance is consistent with that of the first cycle, then this degree of repeatability may well argue for acceptance of the anchor anyway, at the design, or slightly reduced, load. This is wholly within the spirit of the new international codes of practice, and is an eminently sensible way of bridging the gap of applying elastic acceptance criteria to total extension data.

This problem of partitioning total extensions into elastic and permanent components is of course resolved when progressive cyclic testing is conducted (Figure 6), as in the Performance Test. By returning the load to AL (Alignment Load) after each cycle maximum, the permanent movement accrued at that maximum can be demonstrated. By deducting this value from the total extension at that maximum, the actual elastic extension can be calculated. Analysis of this elastic extension then provides a calculation of the effective tendon free length, and hence the amount of apparent debonding - important in determining the correct functioning of the anchor with respect to load transfer mechanisms and efficiency. Cyclic loading in this fashion should always be conducted on preliminary test or production anchors, as it provides the maximum reasonable anchor performance data short of the use of expensive and intricate strain gauges or extensometers. Data on permanent movements at load cycle maxima can then be applied - with caution - to the results from the (noncyclic) Proof Tests to estimate true elastic performance at each incremental load step.

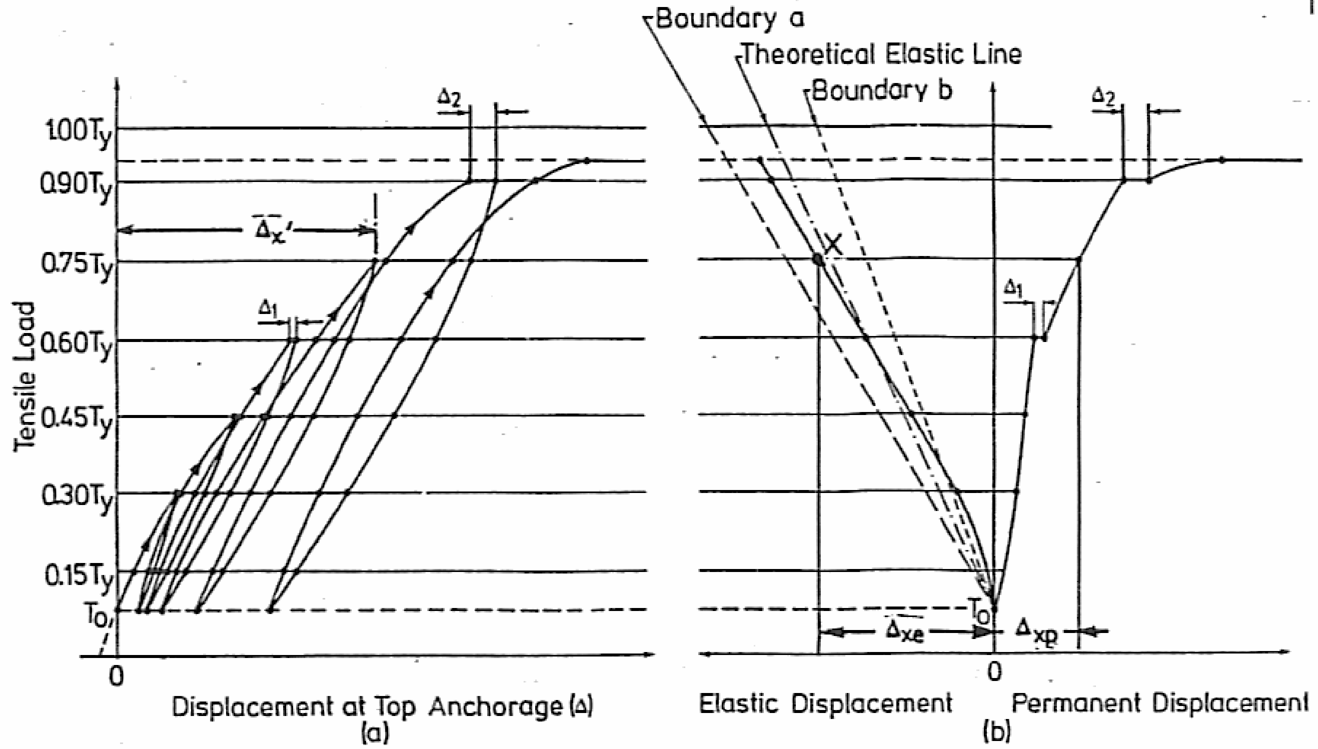

Figure 6. a) Sequence of stressing for performance testing, and b) Partitioning of total extensions into elastic and permanent components (after German DIN 4125-1972)

Regarding creep criteria, the provisions of PTI (1986) are generally followed throughout the industry. Again, however, some original thought and interpretation should be applied, especially in the case of anchors in weak, heavily fissured rock masses. In cohesive soils creep amounts and rates typically increase with increasing anchor loads. Simply we suspect that interfacial phenomena around the bond zone result in relative movements sufficient to cause local remolding of the soil and so irrevocably reduce its shear strength capacity. Therefore, increasing the load simply acerbates the problem and so accelerates the creep, leading to progressive and accelerating failure.

On the other hand, in weak, fissured rock masses movement of the bond zone under load may occur not by grout/rock bond breakdown, but by simply crushing and compressing the strata local to the anchor. Once this amount of compression is locked in, there may be little tendency for creep to continue unless the loading is significantly increased. In such cases therefore, if an anchor has failed the creep test, it would be reasonable to destress and then restress it - to have the same load as before - to verify if the anchor will then have less tendency to creep, following this "precompression." This approach is particularly useful where tendons are long, and the option of replacement impractical or prohibitively expensive. An example is given below.

3. Examples of Performance

Thousands of anchors, both temporary and permanent are installed each year around the United States in all types of rock and soil.

Designers and contractors alike are knowledgeable about the practicalities and economics of the technique in each region. The following examples have been selected from the many recent full scale test programs run by Nicholson Construction across the country. When considering the results, it is worth to bear in mind the reminder of Littlejohn (1990): "Anchorage construction technique and quality of workmanship greatly influence the pullout capacity, and the latter, in particular, limits the designers' ability to predict accruately, solely on the basis of empirical roles ... In anchorage technology, practical knowledge is just as essential to a good design as the ability to make calculations."

Site 1. Medium-Coarse Sand/Gravel

In advance of a major 700 tieback contract on the West Coast, a special test program was conducted featuring six anchors. They were constructed in exactly the same fashion as foreseen for the subsequent production anchors, and were founded in dense alluvial interbedded sands and gravels, lightly cemented. Extra tensile capacity was incorporated into the tendons to permit safe stressing to potential loads of up to the 583 kips specified. Construction details are summarized in Table 2. Drilling was conducted with 5-1/2 or 7" casing rotated into the sands with vigorous water flush. Primary pressure grouting was conducted through the drill head during pregressive withdrawal of the casing. Of the four multistrand tendons equipped with a regrout tube, two (TA4 and 5) were postgrouted before initial stressing.

ANCHOR	BOND DIA.	FREE LENGTH	# OF STRANDS	POSTGROUTING	RESULT
TA1	5-1/2"	41'	13	--	Reached 545 kips OK (225% WL)
TA2	5-1/2"	39.5'	13	--	Reached 545 kips OK (225% WL)
TA3	7"	40'	13	--	Reached 575 kips OK (250% WL)
TA4	7"	40'	13	Before first stressing	Reached 575 kips OK (250% WL)
TA5	7"	37.5'	17	Before first stressing	Reached 430 kips OK (250% WL)
TA6	7"	40'	17	After first stressing	Pulled at 540 kips, then postgrouted and reached 583 kips OK (225% WL)

Table 2. Details of Test Anchors, Site 1 (All inclined down at 20^{o}, with nominal 20' bond, all initially pressure grouted through the drill head, i.e., Type B)

The results from these cyclic stressing operation are summarized in Table 2. Only TA6 appeared to pull, and so was postgrouted and retested four days later to a successful test load of 583 kips. The principal conclusions drawn from this test program were as follows.

1. With the prevalent geological and construction conditions, loads as high as 583 kips could be safely resisted with nominal 20' bond lengths, although the occurrence of one failure at 540 kips does suggest that the upper safe limit was being approached.

2. Postgrouting did not appear to influence significantly the amount and rate of tendon debonding, but did seem to make these phenomena very consistent.
3. Postgrouting did, however, reduce the amount of permanent zone displacement to about 60 or 70% of the typical, and made it very linear with increasing load (Figure 7).

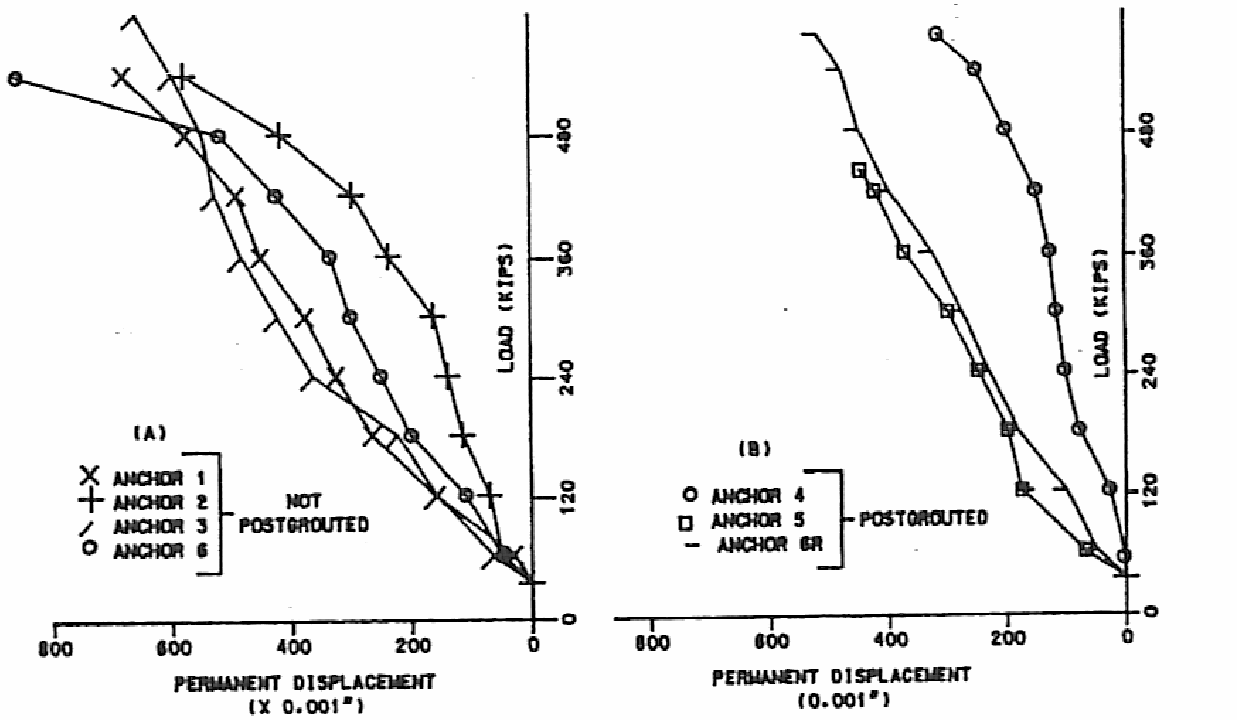

Figure 7. Comparison of permanent displacements obtained from normal Type B anchors (left), and Type C anchors (right), Test Site 1

4. Postgrouting was demonstrably effective in "repairing" the pulled anchor, TA6. It appeared to reduce a little the amount of tendon debonding, but by a larger factor the amount of permanent displacement, interpreted as the non-recoverable movement of the bond zone towards the stressing head. It also minimized creep at the test load.

   During the subsequent production work, excellent anchor performance was uniformly achieved.

Site 2. Glacial Tills

The lateral support for a 79' deep structural diaphragm wall in the Northeast featured four rows of prestressed anchors, as in Table 3. Each row had 56 anchors.

Row	# of 0.6" Strands	Design Load (kips)	Free Length (ft)	Bond Length (ft)
1	4 - 6	134-192	30	20
2	6 - 9	210-311	20-25	25-35
3	6 - 10	200-344	15	35-40
4	7 - 11	242-349	15	35-40

Table 3. Summary of Anchors in Till, Test Site 2

To verify design assumptions involved, six preproduction test anchors were installed vertically, into two types of glacial till (Table 4). The Upper Till was clayey with silts and had a fine sand matrix comprising 30-40% of its volume. The gravel was subangular to rounded. It was dense to very dense and plastic (N = 30-60). The Lower Till was similarly composed but even denser (N > 100). Both tills had low permeabilities (about 10^{-5} cm/sec). The ground water was about 38' below the surface, i.e., 2' below the interface between the tills.

ANCHOR	BOND LENGTH	FOUNDING TILL	FREE LENGTH	# OF STRANDS	RESULT
TA1	25'	Upper	15'	6	Pulled at 211 kips (60% GUTS)
TA2	25'	Upper	15'	6	Pulled at 253 kips (72% GUTS)
TA3	25'	Upper	15'	6	Pulled at 260 kips (74% GUTS)
TA4	35'	Lower	40'	10	Held 493 kips (84% GUTS)
TA5	35'	Lower	40'	10	Held 469 kips (80% GUTS)
TA6	35'	Lower	40'	10	Held 493 kips (84% GUTS)

Table 4. Details of test anchors, Site 2 (all vertical, all 6-1/2" diameter, all tremied with grout, i.e., Type A)

Rotary drilling was used to provide open holes of 6-1/2" nominal diameter. The grout was preplaced prior to tendon insertion.

Cyclic testing provided the results of Table 4 and Figures 8 and 9. For the shorter anchors in the Upper Till, average grout-soil ultimate bonds of 8-10 kips/ft were recorded. The longer anchors in the Lower Till displayed a remarkable degree of uniformity, reaching the 80% GUTS tendon limit without pulling (13-14 kips/ft). The anchors in the Upper Till had apparent tendon debonding 2 to 3 times greater than that recorded in the Lower Till anchors. This was in inverse relation to the ratio of the deformation modulus of each soil type and supports theoretical studies (Littlejohn and Bruce, 1977) on load transfer mechanisms.

In general, creep increased in amount and rate with increasing load. This was especially so in the Upper Till anchors, which nevertheless established stabilization trends after about 5 minutes of observation, except at loads approaching failure. For the deeper anchors, creep at loads above 350 kips did not increase with load, but remained uniform, or decreased. This is indicative of a compressive "tightening up" of the soil around and/or above the bond zone.

This very significant test clearly confirmed the validity of the bond length design, and the production work proceeded accordingly.

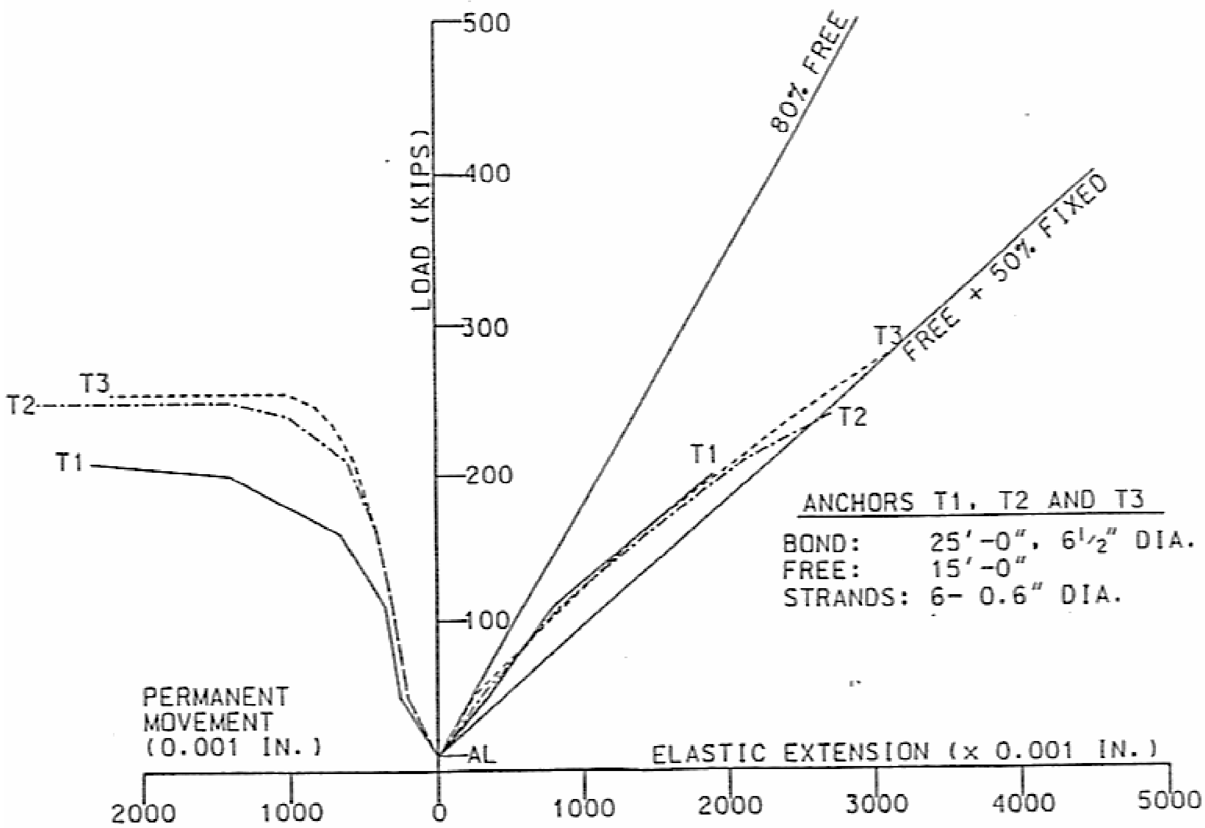

Figure 8. **Analyzed performance of Test Anchors 1-3, Site 2**

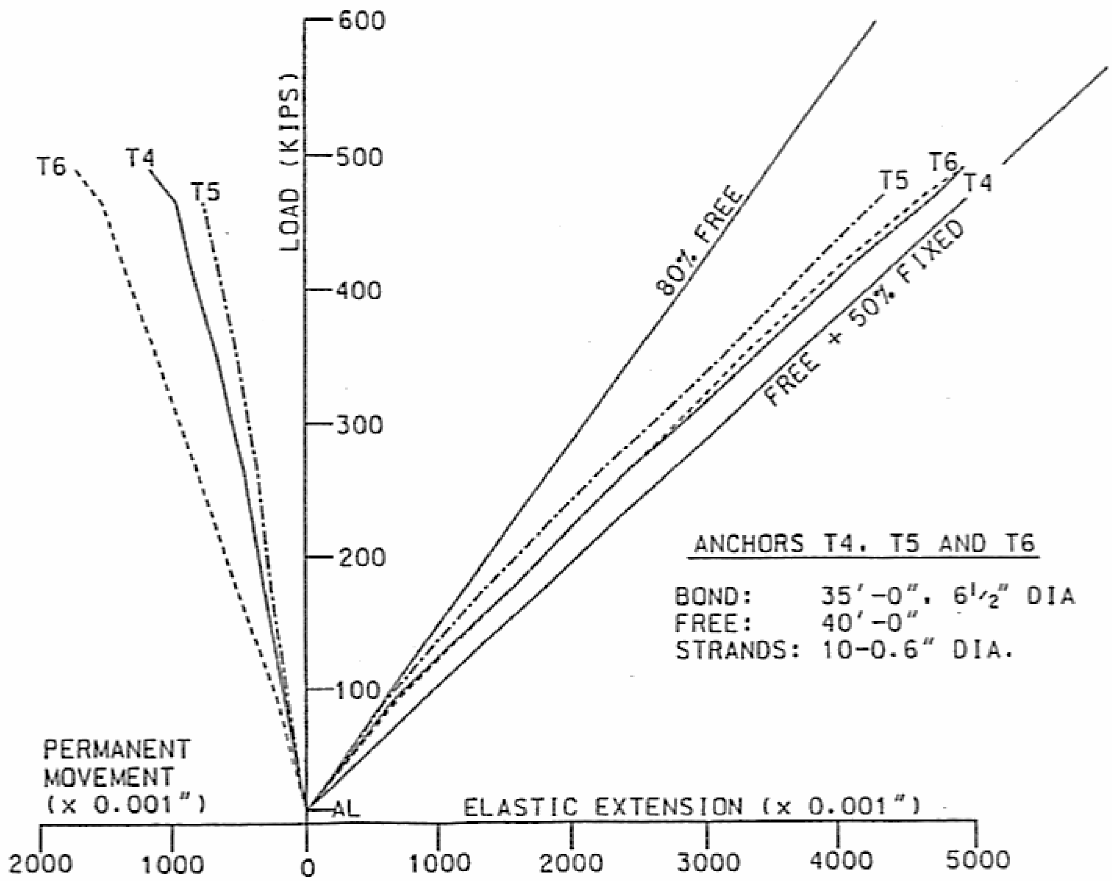

Figure 9. **Analyzed performance of Test Anchors 4-6, Site 2**

Site 3. Marine Clay

A special private research program was conducted in this important soil type during the currency of an existing contract on the East Coast. The prime variable on this occasion was the grouting method, used during installation of the six vertical test anchors (Table 5), each with five, 0,6" dia. strands. The recent borehole logs record the clay as being silty, and medium stiff to very stiff, although during anchor drilling it appeared not so competent. The founding stratum was well beneath the water table.

ANCHOR	TYPE OF GROUTING	RESULT
TA1	Tremie - Type A	Pulled at 93 kips (0.940" Perm.), postgrouted once, pulled at 78 kips (0.810" Perm.)
TA2	Tremie - Type A	Pulled at 93 kips (0.478" Perm.). Not retested.
TA4	Pressure - Type B	Pulled at 126 kips (2.370" Perm). Not retested.
TA5	Pressure and Postgrout - Type B/C	Postgrouted, pulled at 63 kips (2.668" Perm.), postgrouted and pulled at 117 kips (0.635" Perm.)
TA6	Pressure and Postgrout - Type B/C	Postgrouted, pulled at 156 kips (2.590" Perm.). Not retested.
TA7	Pressure and "Local" Postgrout - Type B/C	Pulled at 118 kips (1.266" Perm.). "Locally" postgrouted and pulled at 100 kips.

Table 5. Details of test anchors, Site 3 (all vertical, all 20' bond, all 5-1/2" diameter, TA3 abandoned due to construction problem.)

Standard Nicholson methods were used to drill and case the 5-1/2" diameter holes. Primary pressure grouting through the head reached 45-60 psi before surface breakout from around the casing. As in the other examples, the grout was mixed colloidally, had the minimum practical water content, and had reached at least 4000 psi strength before stressing commenced. Regrouting was rigorously conducted where appropriate, but the opportunity was also taken to investigate the possible effectiveness of a "home made" regrout system grouted in one pass, as a duplication of local practice.

Practical problems prevented TA3 being satisfactorily completed or regrouted and restressed after initial failure, while TA5 was regrouted twice.

Analyzed load-extension data are shown in Figure 10. Discounting the aberration of TA5 with its premature failure (drilling records indicate an old open pipe being encountered in the overlying fill) Table 6 summarizes the results.

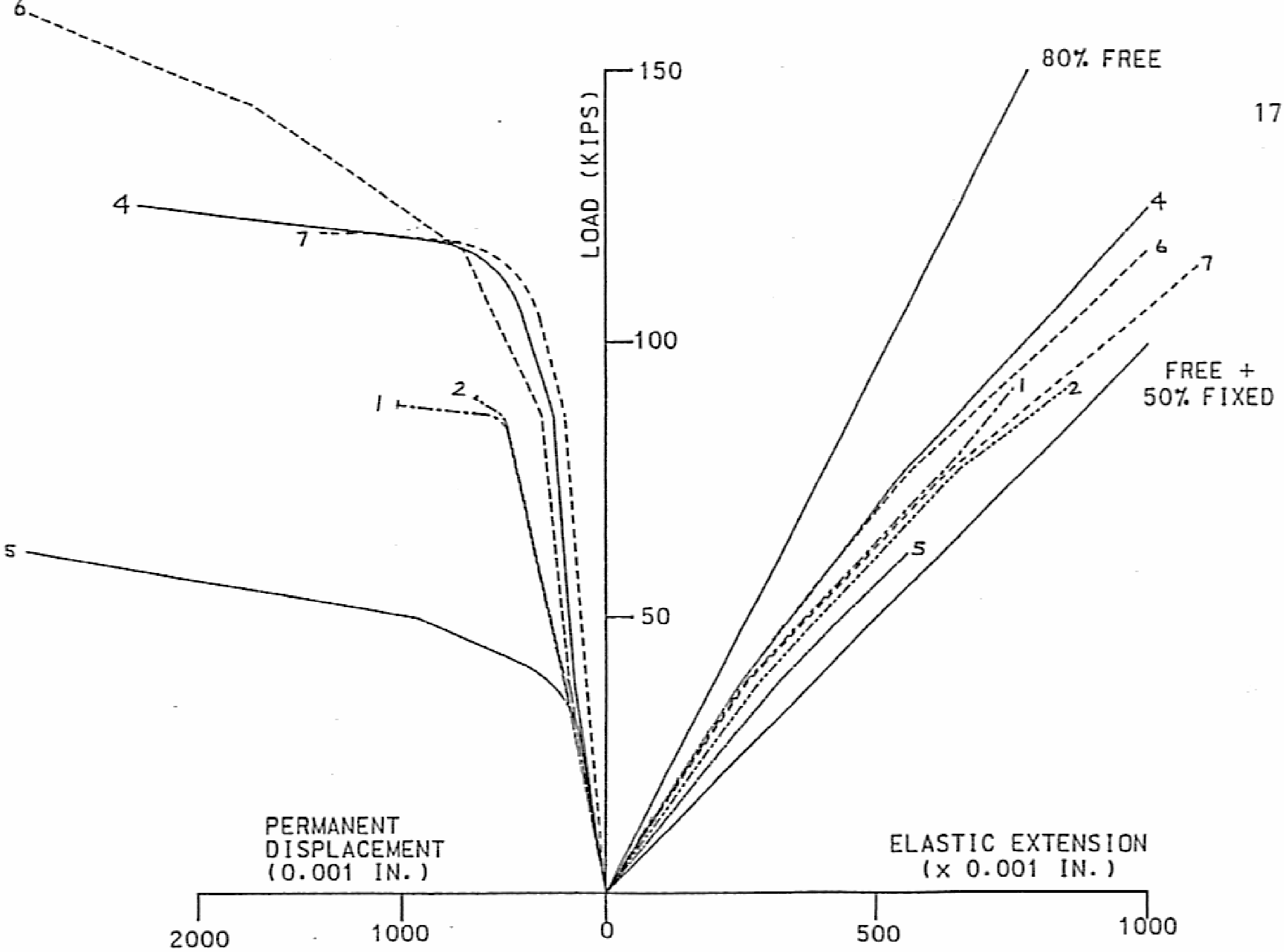

Figure 10. Analyzed performance of Test Anchors 1, 2, 4-7, Site 3 (first stressing)

Anchor	Mode of Grouting	Max. Good* Behavior Load (kips)	Max. Recorded Load (kips)
TA1, TA2	Tremie (Type A)	85	93
TA4, TA7	Pressure (Type B)	110	118-126
TA6	Pressure & Post Grout (Type C)	130	156

Table 6. Synthesis of Test Anchor Results, Test Site 3 (* means good load/extension data, and acceptable creep.

These results clearly demonstrate the influence of initial grouting method, assuming all other factors to be equal. Regarding the influence of post grouting <u>after</u> initial stressing, the "home made" system of TA7 had no beneficial effect, and indeed both elastic and permanent data from the restress were inferior. For the proper system used in TA1 and TA5, although no significant improvement was noted in the former, the capacity of the latter was almost doubled. These results would suggest that normal field procedure would require regrouting two or more times <u>before any stressing</u> to ensure benefit, as is done routinely in the incaceous silts and clays common in the southern states as a result of chemical weathering of igneous and metamorphic parents.

Regrouting in such soils after initial grout/clay bond failure may not be able to repair this interface as the clay will have passed peak strength.

Creep values and rates increased with load, and became large and unstable within 10-20% of the maximum recorded load. They followed classical patterns.

Site 4. Horizontally Bedded Claystone - Shale Sequence

A special preproduction test anchor was installed vertically into a typical Pennsylvanian fissured claystone stone/shale/coal sequence. The hole was 7-3/8" diameter, rotary percussive drilled, and 75.3 ft. deep, of which 9.7 ft. was the bond length for the 18 strand tendon.

The test was conducted not only to establish the adequacy of the design of the subsequent high capacity anchors - to be used for a high dam stabilization - but to investigate the time dependent performance under various loads. Tests on NX cores gave average UCS value for the rock material of 1000-1500 psi, while RQD figures averaged 74%. The modulus of deformation was estimatd as about 150 ksi.

Incremental cyclic loading to 674.2 kips, without failure, confirmed the adequacy of the interfacial bond values selected. However, the creep performance proved most interesting. As shown in Table 7, creep increased steadily through each load increment to Cycle 8 (485.4 kips), by which time the anchor would have failed the PTI criterion and ultimate failure was considered imminent. However, when Cycle 9 was run (to the repeat lower load maximum of 377.6 kips) the creep was very small (30% of that recorded at its earlier Cycle 7). Raising the load to 593.3 kips (Cycle 10) predictably increased the creep again, while the subsequent reading at 485.4 kips (Cycle 11) gave, again, a very small value compared to Cycle 8. The short Cycle 12 to the new maximum of 674.2 kips would have given large creep again, whereas subsequent holds at 593.3 (Cycle 13) and 650 kips (Cycle 14) again gave very low creep amounts.

CYCLE	MAXIMUM LOAD (kips)	CREEP IN 1440 MINS (x.001")	STABILIZING TREND?	CREEP (x0.001") DIVIDED BY LOAD (kips)
1	53.9	Zero	Yes	Zero
2	107.9	46	Yes	46
3	161.8	58	Yes	36
4	215.8	92	No	43
5	269.7	107	No	40
6	323.6	160	No	49
7	377.6	170	No	45
8	485.4	449	No	93
9	377.6R	53	Yes	14
10	593.3	527	No	89
11	485.4R	92	Yes	19
12	674.2	500 Est.	No	75
13	593.3R	105	Yes	18
14	650R	75 Est.	Yes	12

Table 7. Summary of test anchor creep performance, Test Site 4

By the end of the test, the bond zone had recorded a permanent movement of 2.37 inches, and yet the elastic and creep performances, especially at repeat maxima were excellent. This argued against the permanent movement having been caused by grout-rock interfacial failure - the conventional explanation. Instead, the whole test pointed towards a progressive tightening up of the rock mass as a result of closing of fissures and/or crushing of certain horizons (e.g., thin coal beds) above the bond zone.

These data were then borne in mind when the stressing progressed on the production anchors: in the event of an anchor having an unacceptably high amount of creep at the test load, it could be destressed and then restressed to verify both elastic, repeatable performance, but especially its superior behavior after the rock mass had been precompressed from its earlier loading sequence. None of the production anchors on this project were subsequently rejected or needed derating.

4. Final Remarks

There is considerable variability in the methods used to install anchors in the softer ground conditions nationwide. Since mode of construction has a major influence on subsequent behavior, performance expectations likewise vary throughout the country. However, by trying to match the drilling system sympathetically to the conditions anticipated, and by striving to understand the mechanisms and effectiveness of grouting, specialty contractors do have the potential to provide anchor capacities of considerable magnitude, and more predictable long term performance. This is equally to the benefit of the owner and the designer in that more cost effective solutions can be offered with confidence.

Ground anchoring, nevertheless, remains a relatively sophisticated construction technique, and the advice given by Rutledge (1982) is singularly appropriate: "The work of designing, fabricating, installing, grouting, stressing and monitoring ground anchors is of a highly specialist nature in which standards and methods are improving worldwide at a rapid rate. Technical specifications and directions cannot replace professional experience and conscientiousness of a contractor's staff at all levels. A valuable role of a specialist subcontractor, as compared with a main contractor, is as a specialist adviser to the main overall project designer during the pre-tender design process. Such specialist advice is rarely available from a main contractor. In general, it is my opinion that ground anchoring is best carried out by a specialist subcontractor rather than by a main contractor installing anchors made from material supplied by a post-tensioning firm."

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