

## AN OVERVIEW OF CURRENT U.S. PRACTICE IN DAM STABILIZATION USING PRESTRESSED ROCK ANCHORS

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Abstract. Rock anchors have been exploited in many applications around the world since their inception at Cheurfas Dam, Algeria, in 1934. Their arrival in the United States is a more recent phenomenon, but during the last 20 years or so, there has been a growing application in the field of dam repair, in line with the general theme of infrastructure rehabilitation and upgrading. The paper highlights aspects of anchor design which are of most common concern to U.S. engineers, and reviews typical construction methods. Performance assessment and monitoring are also discussed in the light of the absence of a true native standard. Details from some recent major contracts are discussed throughout by way of illustration.

### Introduction

No concrete dam has ever failed in the United States. However, largely as a result of the Federal Energy Regulatory Commission's revision of its flood load criteria, and the fact that many concrete hydroelectric dams are up for renewal of their 50 year licenses, there is rapidly increasing activity in fields related to dam safety enforcement.<sup>1</sup>

To indicate the scale of the problem, it may be noted that State governments regulate more than 80,000 dams, and the Federal government owns 3,000 more. According to a Corps of Engineers' inventory, nearly one quarter of all U.S. dams are now at least 50 years old, compared to 5% in 1980, and a projected 33% in 2000.<sup>2</sup> In 1981 the Corps completed inspecting about 8800 non-federal dams classified as high hazard, based on threats to habitations downstream. It found one third unsafe by current criteria.

In this highly charged atmosphere, major steps are being made in the dam safety research effort. For example, the Corps launched a six-year, \$35 million research program in 1984, while the Bureau of Reclamation's \$650 million evaluation and repair program covering its 350 dams is expected to peak in 1992.<sup>1</sup> Universities, and bodies such as the Association of State Dam Safety Officials (ASDSO) and the U.S. Commission on Large Dams (USCOLD) are also contributing increasingly useful pools of information.

The burden of actually repairing the threatened structures has largely passed to outside contractors. Specialist geotechnical engineering and structural repair companies have responded vigorously to the challenge. For example, ingenious methods have been evolved for installing deep cut-offs through and under existing embankment dams,<sup>3</sup> for bonding fissured concrete structures under full hydraulic head,<sup>4</sup> and for combatting foundation liquefaction potential<sup>5, 6</sup> and seepage<sup>7</sup> by grouting methods. At the same time, levels of activity and expertise in the longer established technique of rock anchoring have reached new heights.

The use of prestressed rock anchors in dam engineering is as old as the technique itself: the first recorded use of anchors was to stabilize Cheurfas Dam, Algeria, in 1934. Since then, anchoring has gained worldwide recognition and application not only in connection with dams but for a multitude of other purposes.<sup>8</sup>

The classic applications for dams are to provide resistance to overturning (Figure 1) and restraint to sliding (Figure 2). However, there are countless examples of anchors being used as tie-downs for spillway construction (Figure 3) and for straightforward rock slope stability around tunnel portals or abutment excavations. Again, another famous example was in the Service Spillway Plunge Pool of Tarbela Dam, Pakistan (Figure 4) where almost

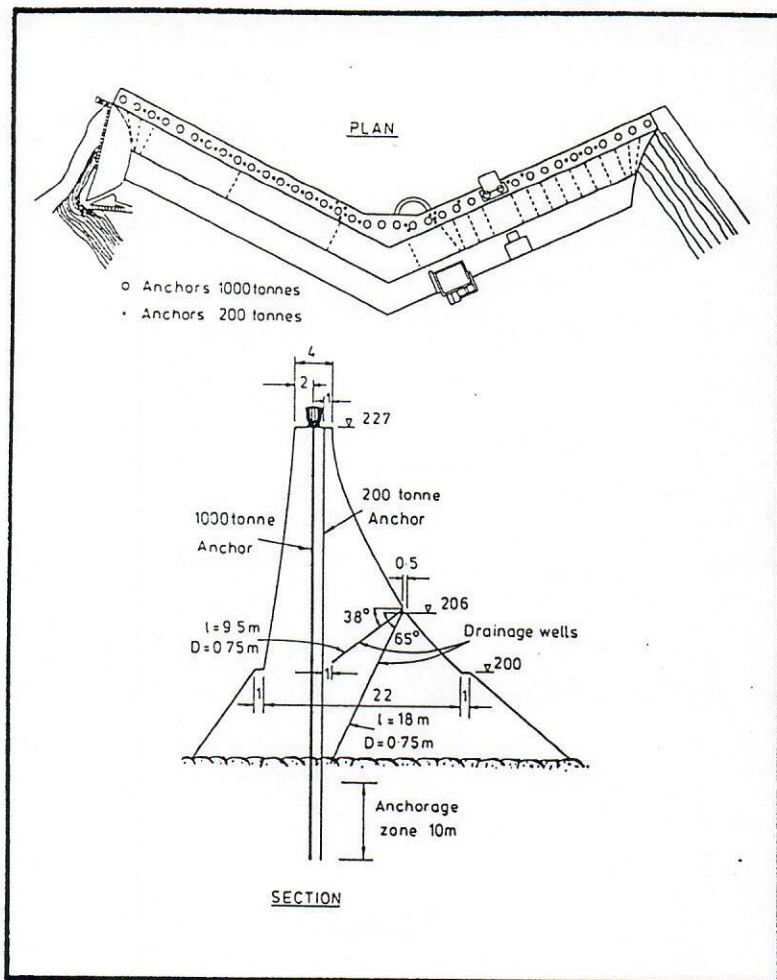


Figure 1. Layout of anchors to resist overturning, Cheurfas Dam, Algeria<sup>9</sup>.

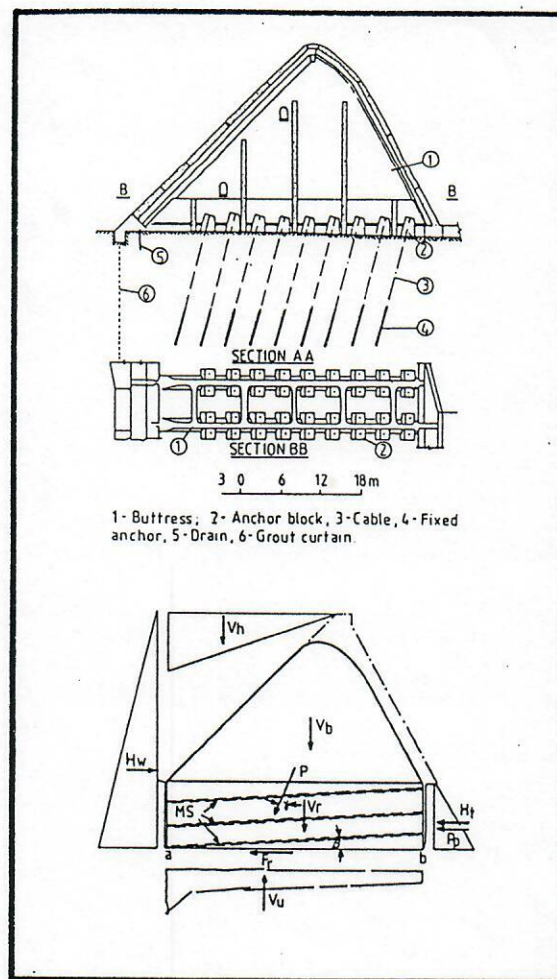


Figure 2. Anchors to resist sliding, Muda Dam, Malaysia<sup>10</sup>.

2000 anchors of 430 ton ultimate tendon capacity were installed to provide, in effect, a zone of "compressed rock" to further resist the tremendous dynamic forces exerted during operation of the Spillway/Flipbucket structure. A newer concept is foreseen for the repair of Stewart Mountain Dam, AZ: not only will the anchors provide against a general overturning failure of the concrete structure as a unit, they will also act to compress contiguous blocks (separated by horizontal construction joints) to prevent their relative movements within the structure in the event of high seismic activity.<sup>12</sup>

As noted below, there is a wealth of literature available to the engineer who wishes to design anchor systems. Equally there is generally a high degree of experience and expertise to exploit from within the ranks of the specialist contractors. This paper does not attempt to summarize these data, and is not a fundamental guide to the various steps of design, construction and stressing. Rather, it highlights major points in modern rock anchor technology which have been found to be controversial, little appreciated or novel. It therefore demands of the reader a certain knowledge of the basic theory and practice. Its major purpose is to help reduce conflict and debate between Owner and Contractor so that the technical and

economic advantages of the technique can be exploited to mutual benefit.

### Sources of Data

The stabilization of dams by anchors is always a process involving a high engineering content, and frequently provides unique or exceptional facets such as those arising from difficult access restrictions, extreme construction tolerances or tendons of great weight or length. For example, anchors of 1400 ton ultimate tendon capacity up to 380' long were installed from the 12' wide crest of Lalla Takerkoust Dam, Morocco,<sup>13</sup> while in the U.S., tendons weighing over 5 tons have been installed under equally restrictive conditions at Shepaug Dam, CT. For these same reasons, good case histories abound in the engineering press, usually authored by the design engineer or specialist contractor involved.<sup>14, 15</sup> Frequently, however, such papers can have a strongly commercial motive, and although they contribute to the general pool of knowledge, often do not approach the subject in a generic manner. In contrast, major literature survey reviews have proved very useful to the anchor community in that they provide a perspective on all aspects of the technology from design

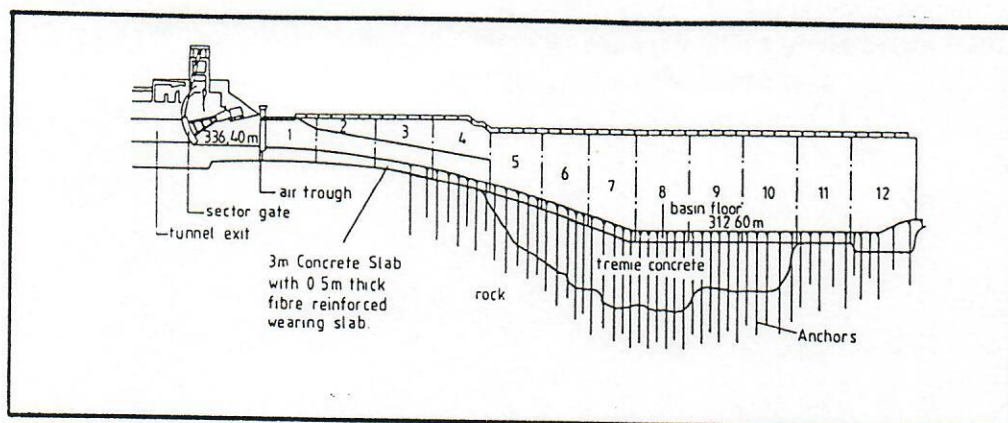


Figure 3. Anchors to repair stilling basin, Tarbela Dam, Pakistan<sup>11</sup>.

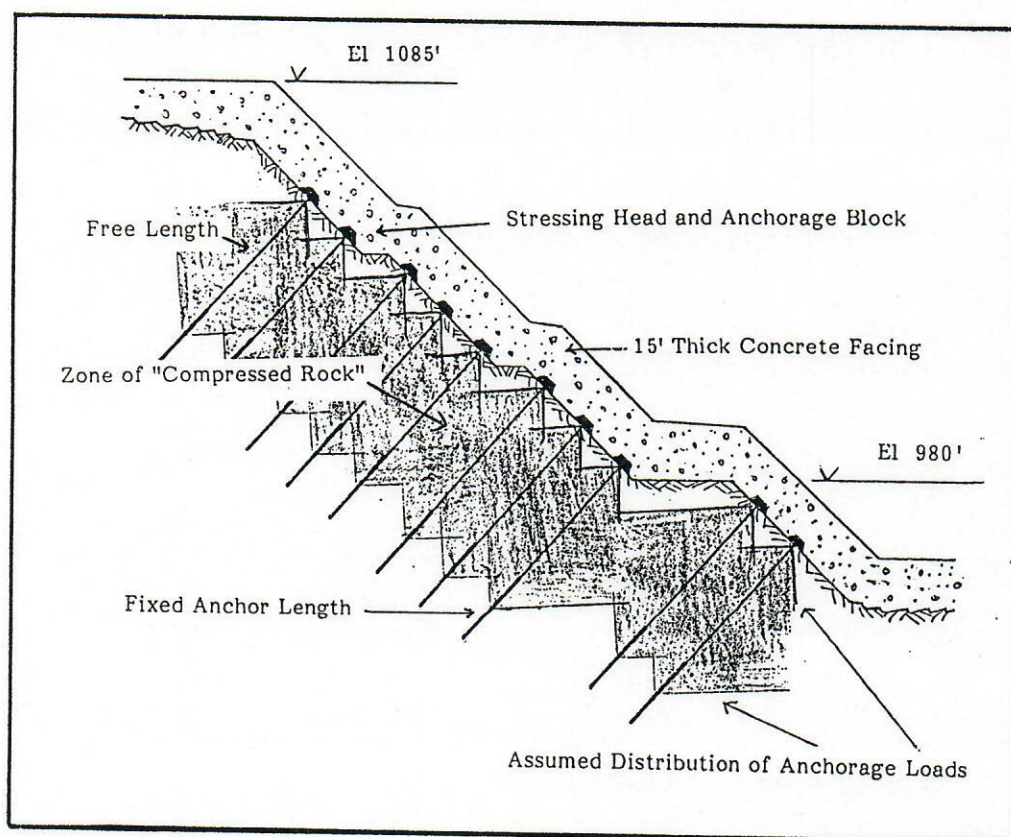


Figure 4. Anchors for slope stability and rock reinforcement, Service Spillway Plunge Pool, Tarbela Dam, Pakistan (idealized section).

and construction, through corrosion protection to stressing, testing and long-term performance.<sup>16, 17, 18</sup>

With respect to national construction standards, there is, for example, currently no ASTM Code for rock anchors, earlier attempts having been disrupted by conflicting proprietary viewpoints. However, the Post Tensioning Institute Recommendations do constitute an acceptable and well regarded substitute, having been prepared by a representative cross section of anchor specialists.<sup>19</sup>

Probably the most comprehensive and up-to-date work on the world scene is the new British Code of Practice BS 8081<sup>20</sup> which supersedes the Draft for Development DD81<sup>21</sup> for six years tried, tested, and modified by the industry. These documents complement the excellent FIP\* documents on Design and Construction<sup>22</sup> and Corrosion and Corrosion Protection.<sup>23</sup>

\*FIP are the familiar initials of the Fédération Internationale de la Précontrainte.

Table 1. Proposed significance of various site investigation parameters in rock anchorage technology. (Note: excludes descriptions of lithology and petrography and core sampling, which are always necessary.)

|                                 | ROCK PARAMETERS                     |     |               |                   |                     |            |                   |               |            |  | HYDROGEOLOGIC PARAMETERS  |                  |   |
|---------------------------------|-------------------------------------|-----|---------------|-------------------|---------------------|------------|-------------------|---------------|------------|--|---------------------------|------------------|---|
|                                 | Structure/<br>Bedding/<br>Fractures | UCS | Shear<br>Str. | Abrasive-<br>ness | Tensile<br>Strength | Weathering | Permea-<br>bility | Unit<br>Wght. | E<br>Value |  | Flow/<br>Movement/<br>GWL | Agress-<br>ivity |   |
| <b>DESIGN</b>                   |                                     |     |               |                   |                     |            |                   |               |            |  |                           |                  |   |
| Overall Stability               | 4                                   | 3   | 3             | 0                 | 3                   | 3          | 2                 | 4             | 2          |  | 4                         | 4                | 0 |
| Rock Grout Bond                 | 2                                   | 4   | 4             | 0                 | 3                   | 4          | 3                 | 1             | 3          |  | 1                         | 1                | 1 |
| Grout-Steel Bond                | 1                                   | 3   | 1             | 0                 | 1                   | 1          | 2                 | 1             | 3          |  | 0                         | 0                | 0 |
| <b>CORROSION PROTECTION</b>     |                                     |     |               |                   |                     |            |                   |               |            |  |                           |                  |   |
| Design of Tendon                | 2                                   | 0   | 0             | 0                 | 1                   | 2          | 4                 | 0             | 1          |  | 4                         | 4                | 4 |
| <b>CONSTRUCTION</b>             |                                     |     |               |                   |                     |            |                   |               |            |  |                           |                  |   |
| Drilling                        | 3                                   | 3   | 2             | 4                 | 2                   | 3          | 1                 | 2             | 3          |  | 3                         | 1                | 1 |
| Installation                    | 3                                   | 0   | 0             | 0                 | 0                   | 2          | 0                 | 0             | 0          |  | 1                         | 4                | 0 |
| Water Test/Pre-grout/Redrilling | 4                                   | 1   | 1             | 0                 | 0                   | 2          | 4                 | 0             | 0          |  | 4                         | 4                | 2 |
| Grouting                        | 3                                   | 0   | 0             | 0                 | 0                   | 2          | 4                 | 0             | 0          |  | 3                         | 3                | 4 |
| <b>PERFORMANCE</b>              |                                     |     |               |                   |                     |            |                   |               |            |  |                           |                  |   |
| Short Term                      | 1                                   | 2   | 2             | 0                 | 1                   | 3          | 1                 | 0             | 3          |  | 0                         | 0                | 0 |
| Long Term                       | 3                                   | 2   | 2             | 0                 | 1                   | 3          | 3                 | 0             | 1          |  | 3                         | 3                | 4 |

Key: No significance - 0  
V. Little - 1  
Some - 2

A Lot - 3  
Essential - 4

### Requirements from Site Investigation Programs

The site investigation is obviously the starting point in the serious evaluation of a rock anchor design. Frequently, however, there is uncertainty about what parameters are most relevant, or should have priority, in an investigation, leading to omissions or irrelevances in the final report. As a guide, Table 1 is a matrix linking the various geotechnical parameters with the different aspects of anchoring. An assessment of this type should be made on a site specific basis by the planner. The following points are provided for clarification and explanation.

#### Overall Stability

Conventionally the overall stability (or ultimate uplift capacity) is calculated on the assumption that a single anchor engages a cone of rock, of a certain geometry and location (Figure 5). The uplift capacity is then estimated as the weight of rock in that cone (based on submerged weight, where appropriate). Investigations show clearly that this is normally an extremely conservative approach as it does not take into consideration the contribution of "rock strength" acting over the surface of the theoretical failure volume.<sup>16, 24</sup> An appreciation of the rock mass structure will give a more accurate indication of a likely "failure volume," and also of the main rock strength parameters likely to contribute most strongly within and around it. Analyses of this type will therefore permit anchor embedment lengths to be minimized rationally, and so reduce overall costs.

### Rock Grout Bond

In the absence of direct test data on ultimate bond values or shear strength parameters, one reliable rule of thumb<sup>25</sup> is to estimate the ultimate bond to be 10% of the U.C.S. (unconfined compressive strength) of massive rock (to a maximum value of U.C.S. of 6000 psi). This assumes also that the grout has sufficient shear strength in situ. The ultimate value is then factored to give an estimated safe working bond value. The safety factor should reflect the degree of weathering and the uniformity and nature of the rock structure, as well as the quality of the investigation data.

#### Significance of Knowing E Value of Rock Mass

Theoretical analyses<sup>26</sup> and field data<sup>27</sup> confirm that the distribution of bond stress in and along the fixed anchor length is dependent on the ratio of the elastic moduli of the anchor material ( $E_a$ ) and the rock mass ( $E_r$ ). The smaller this ratio (i.e., the higher the rock modulus) the greater is the bond stress concentration at the top of the fixed anchor (Figure 6). Only in very soft rocks is it reasonable to assume, therefore, that the bond is evenly distributed and that the design may be based accurately and directly on the shear strength of the weaker medium. The probable stress concentration may also be used as a guide to the selection of the position of the apex of the "cone of rock" approach, used in overall stability design (above). This also affects the assessment of anchor

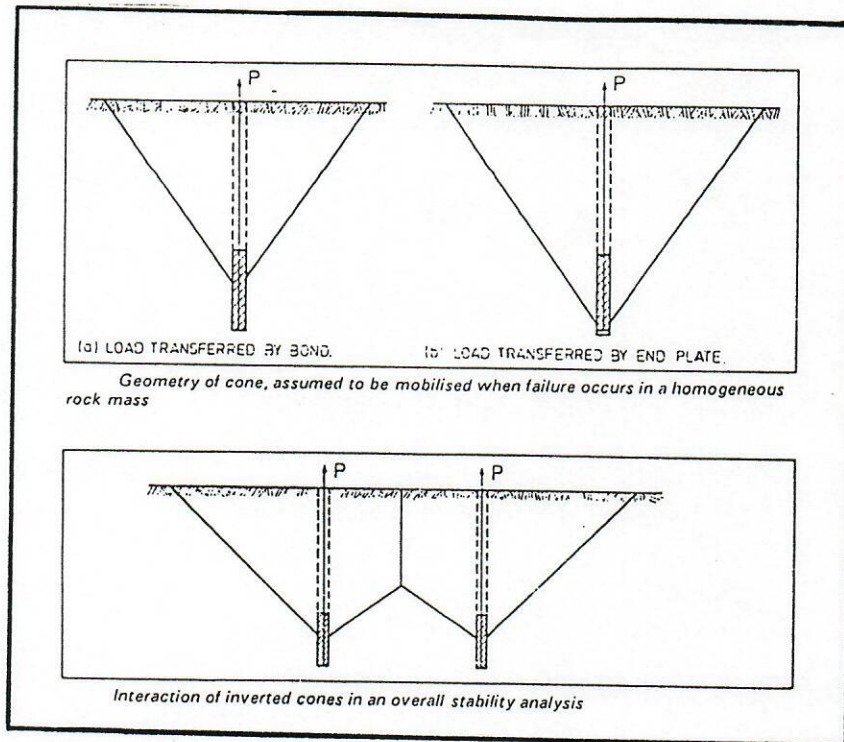


Figure 5. Conventional basis of designing for ultimate resistance to pull out in homogeneous rock masses<sup>16</sup>.

stressing information, especially during Performance Testing (see below) where careful analysis will indicate the amount of apparent debonding in the fixed anchor length and so influence judgements on anchor acceptability.

#### Corrosion Protection

Although this is discussed below it is worth reiterating that this must be regarded as an integral and vital part of anchor design. The degree of corrosion protection to be provided must be based not only on the chemical and dynamic properties of the groundwater, but also on the permeability of the rock mass—before and after any phase of pregrouting and redrilling. It should be noted that rock mass permeability testing, if conducted according to Housby's method, will provide information on fissure geometry as well as water acceptance characteristics.<sup>28</sup>

#### Construction

Throughout every phase, it is clear that the more detailed the site investigation data provided, the less uncertainty there is about the suitability of the techniques foreseen. This in turn will lead to optimized technical and program performance and reduced costs.

#### Construction

#### Drilling

There is no reasonable argument against the normal use of a down-the-hole hammer to drill unreinforced concrete and competent rock. It is a rotary percussive method routinely employing delivered compressed air pressures of up to 300 psi (actual air pressures acting on

the borehole wall are, of course, much less). It provides holes of exceptional straightness as the percussion is applied at the bottom of the hole, just above the bit: the drill string is thus pulled as opposed to pushed. Typical tests show deviations of less than 1 in 100 to be routinely attainable when proper drilling practices are observed. For the same reason, rates of penetration are more or less constant with depth, as no percussive energy is lost between drill head and drill bit. Penetration rates are high (up to 60 ft./hr. is not exceptional). Presently, mechanical practicalities restrict conventional hole sizes to 4-12" in

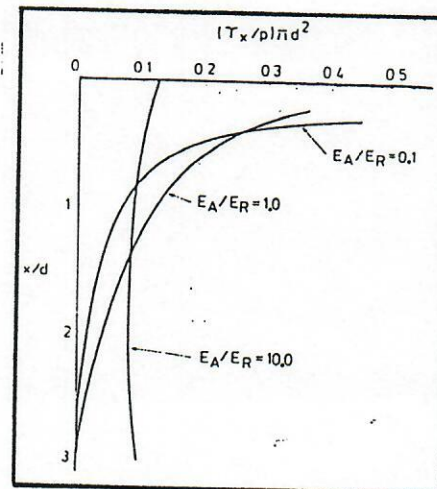


Figure 6. Variation of shear stress with depth along the rock/grout interface of the bond length.  $E_R$  and  $E_A$  are the moduli of elasticity of the rock and

Table 2. Summary details of some dam anchoring projects undertaken by the Nicholson Construction Company. (Note: DTH = Down-the-Hole Hammer).

| CONTRACT              | LOCATION              | YEAR OF COMPLETION | CONTENT                                                                 | BEDROCK                              | DRILLING METHOD             | TOLERANCE (WORST)                      | OWNER                                |
|-----------------------|-----------------------|--------------------|-------------------------------------------------------------------------|--------------------------------------|-----------------------------|----------------------------------------|--------------------------------------|
| Cannelton Lock & Dam  | Tell City, ID         | 1972               | 27 inclined anchors, 9" dia. av. 165' deep                              | Sandstone, sandy shale               | Rotary                      | 1 in 120                               | U.S. Army Corps, Louisville District |
| Newburg Dam           | Newburg, ID           | 1973               | 18 inclined anchors, 9" dia. av. 135' deep                              | Soft shale                           | Rotary                      | 1 in 120                               | U.S. Army Corps, Louisville District |
| Hildebrand Lock & Dam | Morgantown, WV        | 1974               | 70 inclined anchors, 6" dia. av. 110' deep                              | Siltstone & sandstone w/ shale seams | Rotary & DTH                | None: no inter-section of holes        | U.S. Army Corps, Pittsburgh District |
| Lake Lynn Dam         | Lake Lynn, PA         | 1974               |                                                                         | Shale, sandy shale                   | DTH                         | About 1 in 100                         | Allegheny Power Services Corp.       |
| Lock & Dam 3          | Monongahela River, PA | 1977               |                                                                         | Med. hard silty shale                | DTH                         | None                                   | U.S. Army Corps, Pittsburgh District |
| Pickwick Lock & Dam   | Savannah, TN          | 1978 & 1981        | 129' vertical & inclined anchors, 9" dia. to 145' deep                  | Shale, limestone                     | DTH                         | Holes within 2' of 80' high conc. face | TVA Chattanooga                      |
| Edgar M. Hoopes       | Wilmington, DE        | 1979               | 10 vertical anchors, 9" dia. 140 - 220' deep                            | Mica schist                          | DTH                         | 1 in 120                               | City of Wilmington                   |
| Wallen-paupack Dam    | Wallen-paupack, PA    | 1979               | 12 inclined anchors, 6" dia. 100' deep                                  | Sandstone and quartzite              | Rotary (DTH not allowed)    | 18" radius at 100' i.e. 1 in. 67       | PA Power & Light Allentown           |
| Lock & Dam 1          | Minneapolis, MN       | 1979 & 1981        | 57 inclined anchors, 6" dia., 70' long plus 57 anchors 5" dia. 50' deep | Limestone, shale                     | DTH in conc. Rotary in rock | 1 in 60                                | U.S. Army Corps, St. Paul District   |

diameter—a range which encompasses by far the larger portion of anchor requirements. Lengths in excess of 300' can be reached. The down-the-hole method, not relying on very high torques or down pressures for its efficiency, can be operated from relatively small drilling rigs, platforms, or crane suspended masts provided they constitute a stable, rigid frame. This is a considerable advantage for operations on existing dams in that contractors can routinely install anchors in places and with orientations judged technically optimal by the designer. In addition, the fact that the percussion is activated in the hole renders the method relatively quiet, and provided the rock debris blown out of the hole during drilling are properly dealt with, the whole

system is very sympathetic to the environment, especially as there are no drilling fluids to dispose of. There have been no published instances of the method causing damage to the existing concrete structure, even when holes have been drilled as close as 2' to free faces, e.g. at Pickwick Lock and Dam, Tennessee. In short, diamond drilling may be necessary for only the reinforced portion if substantial steel reinforcement of the concrete is foreseen, and rotary (tricone) drilling in the softer sedimentary rocks may be apposite, but overall, down-the-hole drilling is the most logical and economic choice in most instances. Examples of projects conducted by the author's company alone are summarized in Table 2.

CONTINUED

|                       |                      |              |                                                     |                                           |                             |                                            |                                           |
|-----------------------|----------------------|--------------|-----------------------------------------------------|-------------------------------------------|-----------------------------|--------------------------------------------|-------------------------------------------|
| Bagnell Dam           | Lake of Ozarks, MO   | 1982         | 277 vertical anchors, 9" dia., max. 300' deep       | Limestone                                 | DTH                         | 1 in 200, verified on each                 | Union Electric Co., St. Louis             |
| Bath Co. P.S.         | Warm Springs, VA     | 1982         | 24 inclined anchors, 8" dia. max. 75' deep          | Vertically bedded shale (Drilling strike) | DTH                         | 1 in 576                                   | Virginia E & P Co. Warm Springs, Virginia |
| Wilson Shoals,        | Muscle AL            | 1982         | 8 vertical anchors, 9" dia. max. 188' deep          | Limestone                                 | DTH                         | 1 in 120                                   | TVA Chattanooga                           |
| Barker Dam            | Denver, CO           | 1984         | 59 vertical plus 35 anchors, 9" dia. 70 - 225' deep | Granitic Gneiss                           | DTH                         | 1 in 100                                   | Public Service Co. Colorado               |
| Montgomery Lock & Dam | Beaver, PA           | 1985 to 1986 | 379 anchors, 4-1/2" - 6" dia. 110' deep             | Shale                                     | DTH in conc. & rock         | 1 in 120                                   | U.S. Army Corps Huntingdon District       |
| Elkhart Dam           | Elkhart, IN          | 1986         | 31 inclined anchors av. 210' long (9 - 6")          | Overburden & shale                        | DTH in conc. rotary in rock | 1 in 60 overall, 1 in 240 for straightness | Indiana & Michigan Electric Co            |
| Wallen-paupack Dam 2  | Wallen-paupack, PA   | 1986         | 33 vertical anchors 8" dia. av. depth 100'          | Sandstone and Quartzite                   | Rotary (DTH not allowed)    | 1 in 120 to 1 in 240 in places             | PA Power to Light Co. Allentown           |
| Lock & Dam 8          | Point Marion, PA     | 1988         | 63 inclined anchors av. 80', 6" dia.                | Sandy shale                               | DTH                         | 1 in 100                                   | U.S. Army Corps, Pittsburgh District      |
| Shepaug Dam           | Near Danbury, CT     | 1988         | 97 inclined anchors from 75-210' long 10" dia.      | Schist                                    | DTH                         | Max. deviation of 1/2°                     | N.E. Utilities Connecticut                |
| Great Falls Dam       | Near McMinnville, TN | 1989         | 114 inclined anchors from 127-146' long, 8 5/8"     | Limestone                                 | DTH                         | Could not be measured                      | TVA Chattanooga                           |

Regarding borehole deviation, specific project requirements should always be addressed. A good drilling contractor should routinely provide holes with an overall deviation not greater than 1 in 50, although deviations of 1 in 100 or less can be provided, if really required, by the careful use of certain drilling accessories, e.g., drill string centralizers, and using the best drilling practice. To specify tolerances finer than this is practically unreasonable, and in any event may not be possible to verify given the in-hole instrumentation currently available to measure them. As a general guide, PTI recommends that "drill holes normally can be started within an angle tolerance of 1 to 3 degrees from their planned orientation. A deviation of 1-2" in 10' can be maintained with normal drilling methods." Bearing

in mind the length and proximity of most anchors on dams, these recommendations should be regarded as a minimum standard. It is interesting that recent international standards allow deviations of up to 1 in 30 for routine anchor installations. Drill deviations are typically greater in inclined holes than in vertical holes.

As a final word on drilling, it is worth reiterating that it remains good practice to over drill every hole by 2-4', thus providing a sump which will accommodate any drilling and other debris and so permit the subsequent introduction of the tendon to the full, designed depth. This extra length should be measured and paid.

## Permeability Testing

Excluding the case of exceptionally porous sedimentary deposits, the permeability of rock masses is a reflection of the geometry and characteristics of the discontinuities, principally their size and frequency. Therefore, it is fundamentally incorrect to express rock mass permeabilities in terms of soil units such as cm/sec. When water-testing rock anchor boreholes, we are more interested in what can escape from the hole, as opposed to what the rock mass can accept (which is the grouter's viewpoint).

A fluid cement grout is particulate: what geologically defines its penetrability (and so potential escape from an anchor borehole) is the number and aperture of the fissures in the surrounding rock mass that the hole intersects. It has intrinsically nothing to do with the diameter of the hole. Therefore, permeability test analyses—which dictate whether pregrouting is necessary, and also impact the choice of corrosion protection—should be conducted with understanding and care.

Experimental studies suggest that a fissure tighter than  $160\mu$  will not accept Type I cement grout particles. A fissure of this width will permit a flow of about 0.4 gal/min at an excess head of 1 atmosphere (15 psi). Therefore, if the total water loss from a rock stage is less than this value, the possibility of appreciable anchor grout loss may be discounted, and no phase of pregrouting and redrilling will be required. It is noteworthy that with the smaller cement particles common in finely ground cements the limiting fissure width reduces to  $100\mu$ , through which water flow would be 0.15 gal./min./atmosphere.

It is clear that this loss of grout potential should not be related to borehole length: in a worst case, a single fissure over  $160\mu$  wide may exist in a 10' borehole or in a 100' borehole. Therefore, it is strongly suggested that the limiting value for determining the need for pregrouting should be specified simply in terms of gal./min./atmosphere without reference to hole diameter or length. This contradicts the current PTI Recommendations<sup>19</sup> but is in line with FIP<sup>22</sup> and BSF<sup>20</sup>, which set a realistic criterion of about 1 gal./min. at an excess pressure of 1 atmosphere. The position of the local water table must of course be measured in order to calculate the excess pump pressure. In addition, FIP confirms that any hole showing artesian water gain should be pregrouted irrespective of the magnitude of inflow.<sup>22</sup>

## Tendon Handling and Installation

Reviews of good practice<sup>16</sup>, and recent codes and recommendations, all confirm that the presence of rust on strands per se is no detriment to bond development with grout, but rather quite the contrary. Only if the rust is loose or flaky should remediation be demanded. All practical steps should, of course, be taken to protect tendons during transportation, storage, and handling.

There is a trend towards longer and heavier anchor tendons as projects become more ambitious, and the 220' long Shepaug Dam tendons weighing up to 5 tons have

already been noted. To avoid damage to the tendons and their corrosion protection—as well as for safety and practicality—it is essential to specify (1) the necessity for some form of mechanical installation device (for example, cranes and helicopters have been used) and (2) proper, strong and frequent spacer/centralizer units at regular intervals along both free and fixed lengths. Bar tendons are typically limited to anchors of short length (less than 50') or low capacity (50 tons), and where access to the drill hole entry point is easy. They, therefore, have restricted application in major dam repairs. As a final word, research<sup>30</sup> has shown that within the fixed anchor length, not more than about 15% of the borehole volume should be occupied by steel (as opposed to grout). This limit should ensure that there will be sufficient interstitial space to permit the grout to penetrate uniformly between the strands thus allowing proper load transfer and efficient corrosion protection.

## Grouting - Mixes and Mixing

Poor or inefficient grouting procedure represents the most common cause of anchor failure when construction—as opposed to design or corrosion protection—is to blame, and within this field improper mixing and grout mix design are the major culprits.

High speed grout mixers<sup>31</sup> must always be used to ensure uniform and intimate mixing of the cement particles and the water. This high wetting efficiency permits the preferred lower water cement ratios (say w/c = 0.4-0.45 by weight) to be used, leading directly, therefore, to higher and earlier strengths, and greatly reduced bleed (Figure 7) without the need for additives. Bleed potential is essential to monitor, as bleed water in long ducts such as in the case of anchors may not necessarily migrate to the top of

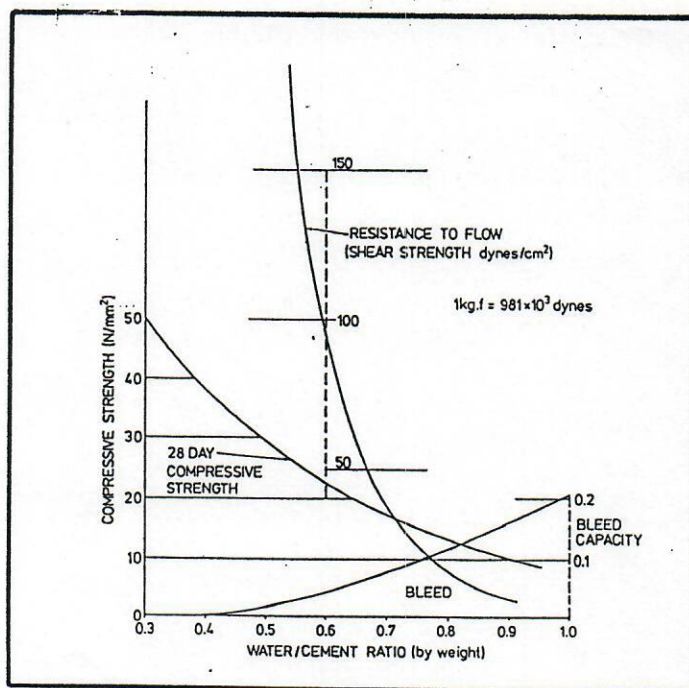


Figure 7. Influence of water cement ratio on grout strength, bleed and fluidity<sup>16</sup>.

the grout column. It typically forms long "ribbons" or lenses which may be in contact with the strands. Upon setting of the grout, therefore, the absence of uniform grout/steel contact will reduce the efficiency of bond development and compromise the security of the corrosion protection. BS 8081<sup>20</sup> states that for relatively impermeable ground conditions, anchor borehole grout bleed should not exceed 2% after 3 hours, or 3% ultimately (at 68°C). Where a w/c ratio lower than 0.40 is truly required, for example to reach very high early strength, then the use of a plasticising additive will be necessary. Generally this is not common in America.

It was recently confirmed that selected plasticiser additives were necessary to mix Type I grouts of  $w \leq 0.4$ .<sup>32</sup> It was also confirmed that gas producing expanding additives (e.g., aluminum powder) should never be used, for many reasons but especially the tendency for large gas bubbles to agglomerate locally. Equally, gelling or thixotropic additives for primary anchor grout should be strictly avoided due partly to the extreme sensitivity of the grout properties to the amount of agents added, and partly as their presence severely compromises bond development in situ.

It is strongly recommended, therefore, that, provided the grout is to be mixed in a high-speed mixer and the w/c ratio is 0.4-0.45, there is no need for the use of additives under normal conditions. Indeed, it has been demonstrated that certain additives, even when used consistently in the correct dosages, can have a markedly deleterious effect on set grout properties. However, in very hot conditions, and/or for long pumping distances, a preblended cement, incorporating a minor amount of pozzolan, may be used at equivalent water contents to improve pumpability. Such proprietary preblends should offer set grout properties at least as good as comparative pure cement grouts with respect to stability, durability, and ultimate strength. Alternatively, under such extreme conditions, the use of a carefully selected retarding additive may be considered, but with strict batching control.

Generally cement grout type should be chosen to reflect the aggressiveness of the ground. FIP lists<sup>22</sup> the following criteria indicative of water aggressive to concrete or hardened cement paste:

pH values < 6.5 (later<sup>23</sup> revised to 5.5)  
 hardness < 3°d  
 CO<sub>2</sub> content > 15 ppm  
 NH<sub>4</sub><sup>+</sup> content > 15 ppm  
 Mg<sub>2</sub><sup>+</sup> content > 100 ppm  
 SO<sub>4</sub><sup>2-</sup> content > 200 ppm

#### Grouting - Quality Control and Assurance

Conventionally, grout quality control has revolved around using bagged cement and a water measuring tank or meter to guarantee the target water-cement ratio. Quality assurance has been provided retrospectively by cube crushing results, the conventional 28-day target being a minimum of 4000 psi. On larger projects there is a trend towards using bulk handling and mixing facilities. These

offer a number of practical and economic advantages, principally, electronic precision weigh batching, reduction in waste, less labor involvement, and lower material costs. However, it must be borne in mind that such systems are notoriously sensitive to atmospheric moisture and dams are usually built in areas of appreciable rainfall. It is good practice, therefore, to conduct periodic checks on the resultant fluid grout prior to pumping. For example, the Baroid mud balance gives a direct reading of specific gravity (and hence the w/c ratio - Table 3), while a flow cone or Colcrete Flowmeter quickly demonstrates pumpability and fluidity. These very simple instruments can be calibrated to provide absolute readings of w/c ratio, or can just as easily be used to confirm repeatability and consistency of batching.

#### Grouting Operations

Most rock anchors do not require pressure grouting to develop adequate rock-grout bond resistances, and so grout pumps need not have very high pressure capacities. Nevertheless, pumping distances in dam anchor work are frequently long, and holes may be deep. High line pressures may, therefore, be developed, especially during the final stages of tremie grouting if the injection line has not been extracted during grouting. Pumps—either progressive cavity or piston—must therefore provide a certain guaranteed minimum rating—and 150 psi may be regarded as that minimum.

Regarding grouting in cold weather, BS 8081<sup>20</sup> states that special precautions be taken to prevent the temperature of grout falling to 32°F during the early stages of hardening. Grouting in air temperatures below 36°F should only be undertaken if the following precautions are taken:

- a) grout temperature during injection is at least 41°F,
- b) mix constituents are free from snow, ice or frost,
- c) tendon and any surface which will be in contact with the fresh grout must be free of snow, ice, or frost, and preferably should be at a temperature which will not chill the grout.

Pregrouting (or consolidation grouting) of the anchor borehole is conducted when the hole fails the water test, or when collapsing conditions prevent the hole being drilled stably to full depth. Typically, any major instabilities and water losses (or gains, if the hole connects with fissures passing upstream) occur at and just below the base of the

**Table 3.** Calculated Specific Gravity vs. Water-Cement Ratio<sup>16</sup>.

| Specific gravity | Water/cement ratio |
|------------------|--------------------|
| 2.10             | 0.3                |
| 1.95             | 0.4                |
| 1.84             | 0.5                |
| 1.74             | 0.6                |
| 1.67             | 0.7                |
| 1.61             | 0.8                |
| 1.56             | 0.9                |

dam. Under these conditions, it is simplest and safest to carry the hole down to the top of the intended bond zone, and pregrout this free length via a packer placed at the top of the hole in the dam. Afterwards the hole can be continued to full depth and the critical bond length tested via a down-the-hole packer, with the security that the water is not simply escaping above the packer into very open zones, and that the bond length itself is being properly investigated. This procedure is operationally, technically, and economically preferable to trying to insert a steel liner through the suspect zone, to the top of the bond length to prevent collapse. If the bond length fails the subsequent water test then it can be regouted, as before, and redrilled.

Corrosion protection of the fixed anchor length by applying an outer corrugated plastic sheath is becoming increasingly more common. Except in the case of very short monobar tendons, it is impractical to specify pregrouting of the tendon inside the corrugation, and letting it set, prior to later installation. Such a system vastly complicates handling and insertion—especially in dam environments—and in any case there is a high probability that the grouted encapsulation will be internally cracked and damaged during installation. It is much more practical, and safer, to grout the inside and the outside of the protected tendon after installation, even if this may dictate the use of two grouting tubes.

Assuming the free length is greased and sheathed, anchor hole grouting is most easily accomplished in one operation. Of course, where the newer "Stewart Mountain" type approach is to be used, then a Primary/Secondary sequence is necessary—the latter being done only after stressing has been completed. In general, however, two-stage grouting (1) does create a "construction joint" (above the Primary grout) which may be a weak point in a corrosion protection system, and (2) does introduce another separate operation into the overall construction program.

As a general rule, tendon installation and grouting should be conducted as soon as is practical after final preparation of the borehole. The actual amount of time a borehole may remain open prior to tendon installation will vary from site to site and will be dictated by the composition and structure of the bedrock. Under U.S. conditions, there is typically no real prospect of borehole wall deterioration within the normal contract timeframe.

Equally, any restrictions on work sequencing should be site-specific. For example, in especially laminated rock, it may be necessary to stipulate that no drilling will be conducted, say, within 30' of any anchor grouted for less than 24 hours. In very massive, tight rock, it may be that no interconnections between even adjacent hole positions occur, and so the spacial separation can be relaxed.

### Stressing and Testing

The PTI Recommendations<sup>19</sup> form a good basis for conducting and analyzing stressing programs. Guidance is provided on both short term testing and long term

performance. The following points are made, with special reference to dam anchoring.

### Setting AL

Tendons are usually long and may comprise many tens of strands. Stressing is therefore conducted with a multijack which loads the strands simultaneously. However, individual strand breakages, and anomalous extensions have frequently been recorded at stress levels from 70 to 80% of the overall tendon guaranteed ultimate tensile stress. In the great majority of cases, this has resulted from unequal setting of AL, the Alignment Load, typically 2-10% (average 5%) of the design load. This can be done accurately and uniformly if the time and care are taken to apply AL to individual strands with a monojack, prior to routine stressing with the multijack. As an example, for tendons over 200' long and comprising 54 strands 0.6" in diameter installed at Shepaug Dam, CT., a monojack was used to apply AL, and the apparent extensions of individual strands recorded to reach this load varied by about 6". This merely reflects the fact that different strands lie in different paths in the hole and need, therefore, to be straightened out by different amounts. Multijack stressing was conducted thereafter to 80% theoretical tendon GUTS with no records of individual strand rupture.

### Phenomenon of Wedge Pull-In

During stressing, the gripping wedges, holding each individual strand in the head plate, are progressively "sucked in" as the load is increased. For smooth profile 0.6" diameter (Dyform) strand (GUTS 33 tons), the measured amount of pull-in is shown in Figure 8. The magnitudes shown are in close comparison with data recorded for normal 0.6" strands at other sites. As a preliminary step, therefore, in analyzing stressing data—especially for the important Performance Tests—this amount of relative movement must be subtracted from gross anchor head (or jack ram) extension. In addition, any change of elevation of the stressing head during stressing—i.e., through bedding in or by structural movement—must also be measured and subtracted from gross ram extensions prior to detailed analysis. Care should be taken during the initial seating of the gripping wedges. A "power seater" provides consistently uniform setting of individual wedges and their component parts, and allows this to be done without danger to hands and fingers.

### Performance Tests

Such tests (also known as "on-site suitability tests" by FIP<sup>22</sup> are conducted on the first group of anchors installed. As defined by PTI<sup>19</sup> they determine whether the anchor has sufficient load carrying capacity, that the free length has actually been established, and indicate the magnitude of the residual (or permanent) anchor movement.

Typically, testing is conducted to 1.33 or, better, 1.5 times Design Working Load, while ensuring the tendon stress at Test Load does not closely approach 80% GUTS. The cyclic nature of the stressing allows the elastic and permanent components of total extension to be separated

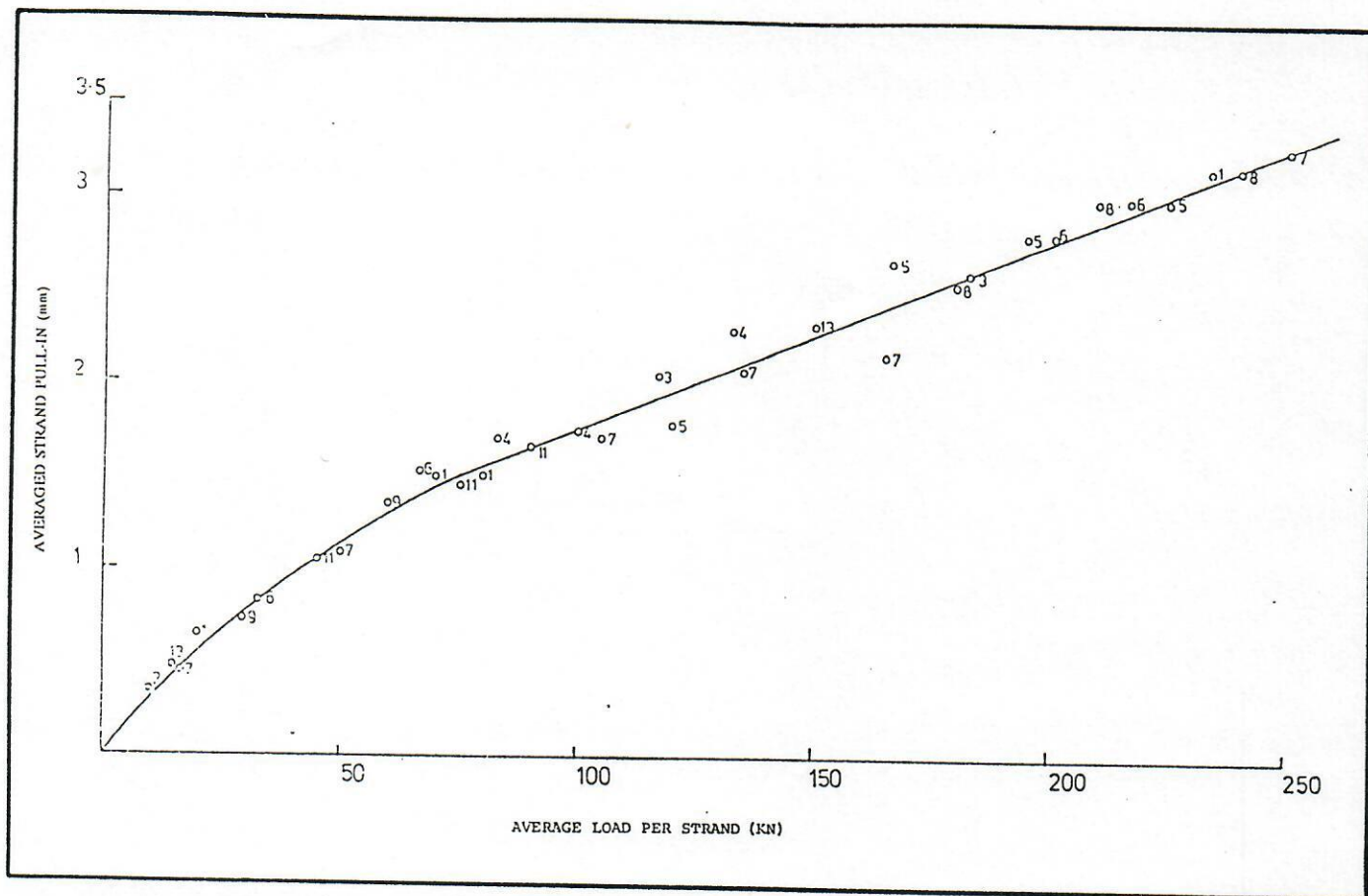


Figure 8. Amount of strand wedge pull-in measured on individual strands during testing of 23 full scale 10 strand anchors. Numbers refer to number of readings at each point ( $8.9\text{KN}=1\text{ ton}$ )<sup>30</sup>.

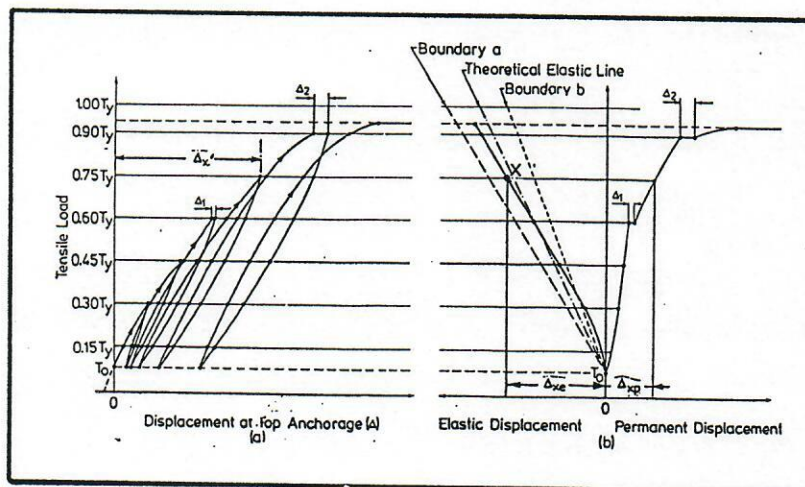


Figure 9. Resolution of total tendon extension (a) into Elastic and Permanent (Residual) Components, (b) for analysis of Performance Tests<sup>33</sup>.

out for each successive cycle maximum (Figure 9). Examination of the former permits conclusions to be drawn about the efficiency of load transfer down the free length, and the apparent extent of effective debonding in the fixed anchor. Reflecting the principles of world practice, PTI recommends acceptance if the elastic movement at Test Load exceeds that movement corresponding to 80% of the designed free length (say, Line A) but is less than that movement corresponding to 100% design free length plus

50% of designed fixed length (say, Line B). These criteria may be regarded as being generous in dam anchoring, especially if the data corrections listed above are applied in advance. For example, an anchor performing near the 80% line implies that load is being dissipated above the fixed anchor length. On long tendons, this could possibly be within the dam, affecting therefore overall stability concepts. Equally, a tendon apparently debonding 24.9' in a 50' fixed anchor zone would theoretically be acceptable,

Table 4. Typical relaxation of stress (as percentage of initial stress), from an initial stress of 70% GUTS at a temperature of 68°F for various tendon types<sup>20</sup>.

| Tendon                                                                            | Class of relaxation | Typical relaxation of stress |       |      |       |       |        |
|-----------------------------------------------------------------------------------|---------------------|------------------------------|-------|------|-------|-------|--------|
|                                                                                   |                     | Elapsed time                 |       |      |       |       |        |
|                                                                                   |                     | 0.1 h                        | 1.0 h | 10 h | 100 h | 500 h | 1000 h |
| Prestraightened wire                                                              | Normal              | 0.25                         | 0.80  | 1.90 | 2.90  | 4.00  | 4.50   |
|                                                                                   | Low                 | 0.09                         | 0.20  | 0.40 | 0.65  | 0.95  | 1.10   |
| 7-wire strand                                                                     | Normal              | 0.35                         | 1.15  | 2.10 | 3.40  | 4.70  | 5.50   |
|                                                                                   | Low                 | 0.07                         | 0.33  | 0.60 | 0.84  | 0.98  | 1.10   |
| Low alloy bar                                                                     | Normal              | 0.90                         | 1.80  | 2.45 | 2.75  | 2.90  | 3.05   |
| Stainless steel wire and bar*                                                     |                     | 1.0                          | 3.0   | 4.5  | 5.1   | 5.9   | 6.3    |
| * Relaxation figures for are wire only and are based on very limited information. |                     |                              |       |      |       |       |        |

although it is well known that in hard rock conditions, little stress should be transmitted beyond the upper few feet of bond zone.<sup>16</sup>

BS 8081<sup>20</sup> sets the boundary lines as being 90% of a free length for Line A, and either free plus 50% fixed or 110% free, for Line B. Where the inferred free tendon length falls outside these boundary Lines A and B, a further two cycles should be carried out to Test Load to gauge reproducibility of the load-extension data. If the anchor behaves consistently in an elastic manner, it may be accepted.

PTI<sup>19</sup> provides no guidance as to limits of acceptability for residual movements, presumably leaving the choice to engineering judgement, backed by reference to the elastic and creep performances. FIP<sup>22</sup> confirms that the matter should be "agreed by consultant and contractor jointly," as a basis for analyzing the subsequent Proof Tests. Regarding creep performance, PTI<sup>19</sup> does outline the procedure and the analysis but notes that "in all but the most decomposed rock formations (it) seldom yields any useful information."

#### Proof Tests

This is a fast economical test, (known by FIP as "Routine Acceptance Test") which, used in conjunction with Performance Test data, verifies the acceptability of the installation. It does not feature cyclic loading, and so examination of performance in terms of elastic and residual contributions is not directly possible. However, if the Performance Tests can provide a reliable and consistent data base of the scale and range of residual movements to be expected at different tendon stress levels, then, with care and understanding, a more detailed analysis of the Proof Test data can be undertaken for investigatory reasons, in appropriate conditions.

#### Long-Term Performance

Assuming that there is no structural movement and that the corrosion protection is adequate to prevent attack to the tendon, the significant sources of long term load loss after lock off for anchors in existing dams\* are creep in the rock or grout or relaxation of the strand. The former is not considered a major potential problem in anchors in most rock conditions, and should be quantifiable from the creep testing process. Conversely, all steel tendons relax, the amount and rate reflecting many factors including the type of the strand, (Table 4) the temperature, its lock off stress level, and its loading history<sup>16</sup>. However, these amounts are quantifiable and predictable, and long term performance should be judged extraordinary only if the characteristics conflict with these tendon performance parameters.

Long-term monitoring demands access to the anchor head and this may not be practical in certain cases. However about 5-10% of anchors are typically equipped for monitoring on a world wide basis. This may be in the form of a load cell, giving a local or remote read out. Conversely, it may be practical to perform a "Lift off" check, wherein the load needed to pick up the head plate by a certain distance—usually the thickness of a dollar bill—is recorded and equated to residual tendon load. When conducting a lift off check, it is preferable to lift the head plate directly, and not the strands: separately lifting the strands out of their wedges may lead to later damage to that top anchorage system and subsequent loss of load as a result of strand/wedge slippage.

\*Eberhardt and Veltrop<sup>34</sup> concluded that for new dams concrete shrinkage and creep were significant sources of possible load loss, contributing up to a 6% reduction in lock-off load.

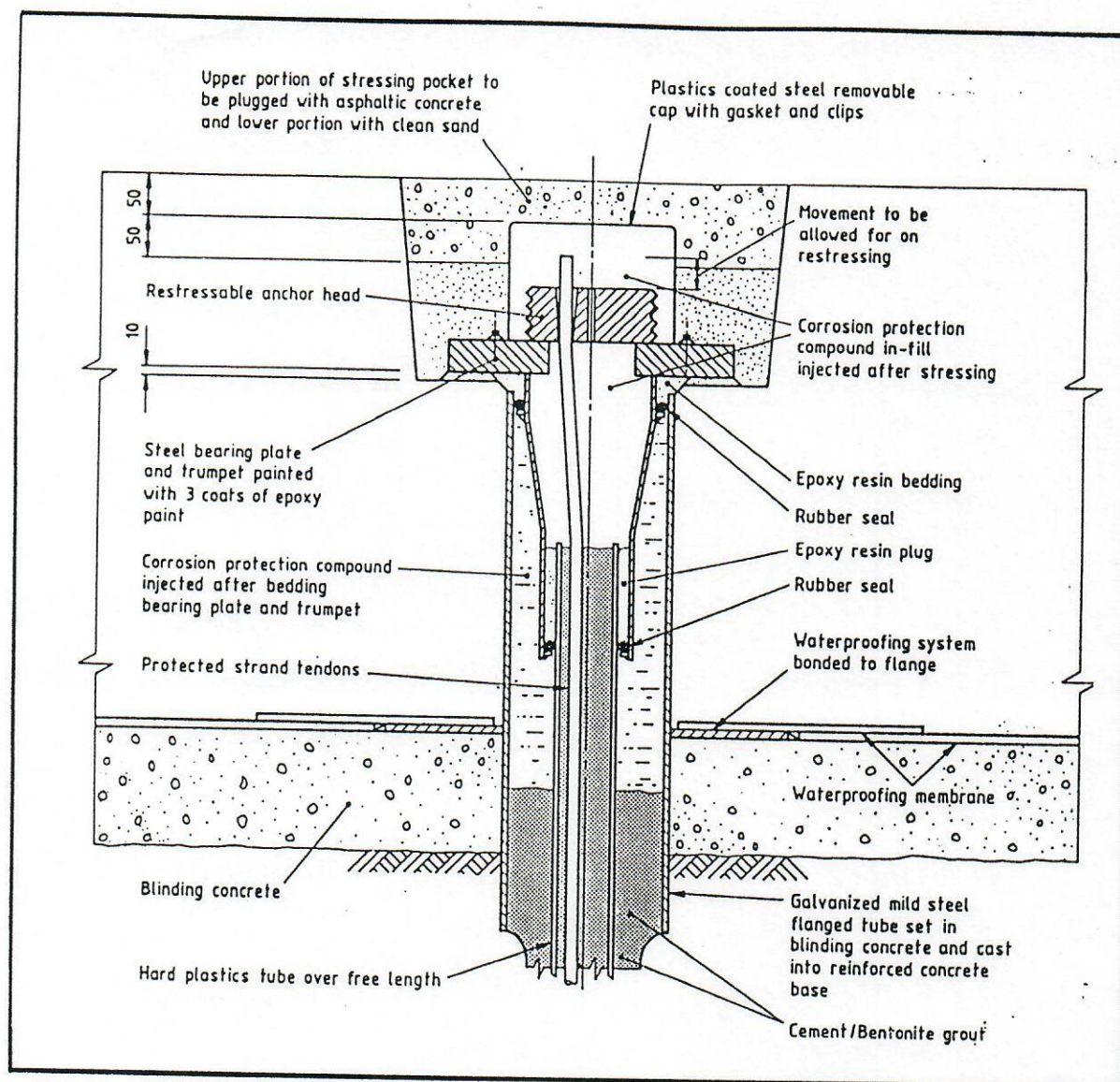


Figure 10. Typical anchor head detail for double corrosion protection of a restressable strand tendon in a water retaining structure<sup>20</sup>.

### Corrosion Protection

Anchors for dams are nearly always permanent installations. All international standards argue that permanent anchors must be properly protected against corrosion. As FIP<sup>23</sup> states, "the protection system against corrosion should be designed in such a way that the presence of aggressiveness is always assumed." Whereas uncracked cement grout generally provides an effective protection, cracks are almost inevitable, especially in the fixed anchor length. For example, the exhumation of a test anchor at the Tarbela project showed radial and longitudinal fissures up to 0.1" wide as far as halfway down the fixed anchor length. In addition, there was crushed grout at the top of that length.

However, it is relevant to consider the ground anchor market in those European countries where the standards were born. Most anchors are installed in older industrial, urban or seafont applications where at least the free length is usually in materials other than rock. The

environment is usually aggressive, whether from natural or artificial causes, and there are great pressures on speed of execution. It is therefore wholly understandable that an extremely high level of protection is demanded under such conditions. In contrast, anchors for U.S. dams are commonly installed into the very hard, impermeable bedrocks so assiduously sought by our original dam builders. Furthermore, dam anchors are typically and deliberately conservative in design, and are installed with an unusually high level of expertise and supervision. As a consequence, the traditional reliance on "grout only" protection for the tendon in the bond length should perhaps not automatically be dismissed as being old fashioned, or even necessarily dangerous.

Regarding the free length, plastic sheathing (polyethylene or polypropylene) has proven effectiveness and suppliers can tailor properties to ambient conditions and temperatures. Special attention must be paid to the "transition points"—especially between stressing head and free length. The proper protection of the stressing head

and just below it is absolutely critical, bearing in mind the exposure to atmospheric conditions, construction damage and so on. This problem has been addressed competently in recent years by reputable tendon supply companies, but the proper installation still requires great care and attention to detail during construction (Figure 10).

Overall the definitive work currently available is the FIP State of the Art Report on "Corrosion and Corrosion Protection of Prestressed Ground Anchorages".<sup>23</sup> It reviews and summarizes mechanisms and types of corrosion, ground aggressivity, case histories of failures and various types of systems and intensities of protection—which are "the responsibility of the designer." The question of protection specification philosophy is concisely expressed: "the problem is to balance the safety of people and property in the event of anchorage failure against the cost of providing protection. Since unprotected ground anchorages of steel will probably corrode in time, it is also necessary to decide whether the rate of corrosion merits the expense of protection. Corrosion rates vary enormously according to anchorage environment and working mode. Further, there is no certain way of identifying corrosive circumstances with sufficient precision to predict corrosion rates. The question of degree of corrosion protection should be determined by the Owner and the Designer jointly, as this will have a major impact on project cost. Although U.S. practice does not favor heavy protection of the fixed and free tendon lengths, and so differs from contemporary European standards, this might not necessarily be a source of possible long-term damage, given typical geological and construction conditions in U.S. dams.

### Concluding Remarks

Where the lack of safety in concrete or masonry structures is related to overturning, sliding, or certain other forms of structural distress, then anchoring is a tried, proven and reliable technique. Anchors are also used in new dam construction, but clearly this market is more limited today. Rock anchor practice in the U.S. in general compares favorably with practice anywhere in the world, and indeed has some unique aspects stemming from the scale and complexity of the individual projects. Only in the controversial subject of tendon corrosion protection may it be inferred that a reappraisal of our approach may be constructive. However, even in that case, it can be argued strongly that our approach is acceptable, given the local conditions.

Anchoring is an extremely valuable technique to the dam engineering community and the potential for its continuing development is tremendous. It is also, however, a complex technique, and the advice given by Rutledge<sup>36</sup> of FIP should be carefully heeded: "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 world wide 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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