Review

Rock engineering design of post-tensioned anchors for dams – A review

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ABSTRACT

High-capacity, post-tensioned anchors have found wide-spread use, originally in initial dam design and construction, and more recently in the strengthening and rehabilitation of concrete dams to meet modern design and safety standards. Despite the advances that have been made in rock mechanics and rock engineering during the last 80 years in which post-tensioned anchors have been used in dam engineering, some aspects of the rock engineering design of high-capacity rock anchors for dams have changed relatively little over the last 30 or 40 years. This applies, in particular, to the calculations usually carried out to establish the grouted embedment lengths required for deep, post-tensioned anchors. These calculations usually make simplified assumptions about the distribution and values of rock–grout interface shear strengths, the shape of the volume of rock likely to be involved in uplift failure under the influence of a system of post-tensioned anchors, and the mechanism of that failure. The resulting designs are generally conservative. It is concluded that these aspects of the rock engineering design of large, post-tensioned rock anchors for dams can be significantly improved by making greater use of modern, comprehensive, numerical analyses in conjunction with three-dimensional (3D) models of the rock mass structure, realistic rock and rock mass properties, and the results of prototype anchor tests in the rock mass concerned.

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1. Introduction

Ground or rock anchors (sometimes referred to as anchorages) of a number of types have long been used for a range of purposes in rock engineering and in geotechnical engineering more generally (Hobst and Zajíc, 1977; Hanna, 1982; Habib, 1989; Xanthakos, 1991; Littlejohn, 1993, 1997). Rock anchors are capable of transmitting an applied tensile load to the rock mass and may be used, for example, to reinforce rock slopes and underground excavations; stabilise sheet pile, diaphragm and retaining walls; anchor building, bridge, power transmission line and other tower or mast structure foundations; anchor marine structures; stabilise large excavations such as dry or graving docks potentially subject to uplift; and for a number of purposes in dam engineering to be discussed in Section 2.

Depending on their purpose, rock anchors may be temporary or permanent, and may be passive or post-tensioned (sometimes referred to as pre-stressed). They may also vary greatly in orientation with respect to vertical, in size (including in both length and diameter), in the nature and size of the tendon (bar, wire or strand, either singly or in groups) providing the essential tensile capacity, and in the ultimate load-carrying capacity of the anchor. In this paper, emphasis will be placed on large-scale, high-capacity, permanent rock anchors of the type illustrated schematically in Fig. 1. Features of this class of rock anchor include the provision of a fixed anchor or tendon bond length, a free tendon length, a stressful (and preferably re-stressable) anchor head, and double corrosion protection. Obviously, in terms of their scale, purposes, rock engineering design and detailed design and installation, these large rock anchors can differ significantly from the rock bolts, cable bolts and rock socketed piles used widely in surface and underground construction and mining. However, these elements all have some features in common that allow data, understandings and elements of design approaches to be transferred from one application to another.

The purpose of this paper is to review international practice in the rock mechanics and rock engineering design of rock anchors which, as will be shown, has remained largely unchanged since the 1970s. The paper will deal mainly, but not exclusively, with the design of large anchors of the type shown in Fig. 1 and with their application in dam engineering, particularly for concrete gravity, buttress and arch dams. Some emphasis will be placed on Australian practice which is the source of the author’s practical experience.
in this area. It is recognised that some of the terminology used may differ from country to country.

2. Post-tensioned anchors in dam engineering

Post-tensioned, and in some cases, passive (Hobst and Zajíc, 1977; Bretas et al., 2010) rock anchors have been used for a range of applications in dam engineering over the last 80 years (Bruce, 1988). These applications include providing:

- resistance to overturning (Banks, 1957; Khaoua et al., 1969);
- restraint against downstream sliding (Maddox et al., 1967; Gosschalk and Taylor, 1970);
- reinforcement of excavated abutment and other slopes (Liang, 1995; She, 2004);
- stabilisation of thrust blocks (Dickson and Loar, 2011);
- additional seismic resistance (Singhal and Nuss, 1991; Bianchi and Bruce, 1993; Lawrence and Martin, 2007);
- tying down of spillway, training wall and stilling basin structures (Hobst and Zajíc, 1977; Cavill, 1997; Topham et al., 2013);
- stabilisation of plunge pools and other erosion features (Lowe et al., 1979; La Villa and Golser, 1982); and
- raising and/or strengthening older dams that no longer meet safety or capacity requirements (ANCOLD, 1992; Xu and Bennmokrane, 1996; Cavill, 1997; Snape, 2002).

The first recognised use of post-tensioned anchors in dam engineering was for the raising and strengthening of the 30 m high Cherfas gravity dam in Algeria in 1936 (Hobst and Zajíc, 1977; Khaoua et al., 1969). For some time following their introduction into dam engineering, post-tensioned anchors were used in the initial design and construction of concrete dams and their appurtenant structures (e.g. Banks, 1957; Wilkins and Fidler, 1959; Maddox et al., 1967; Gosschalk and Taylor, 1970; Hobst and Zajíc, 1977). More recently, the major use of post-tensioned anchors in dam engineering has been for the raising or strengthening of existing dams.

Xu and Bennmokrane (1996) carried out a major review of the then state-of-the-art of the strengthening of existing concrete dams using post-tensioned anchors. They listed the main reasons for strengthening existing dams as being to:

- meet changes in safety standards;
- overcome deficiencies in design and construction;
- recover loss of strength due to deterioration; and
- raise the heights of dams.

In many countries, the former dam safety standards and design criteria based on historical flood levels have been replaced by criteria based on probable maximum flood (PMF) or probable maximum precipitation design flood (PMP DF) water flows and levels. The predicted design flood levels based on annual exceedance probabilities (AEPs) can be higher than the historical levels used in the original designs, so that the design spillway capacities of many existing dams are inadequate to pass the new design floods safely. Furthermore, the dams may potentially fail by overturning and/or sliding. This is particularly the case for dams designed before about 1946 (in Australia, and at other times elsewhere) when uplift forces acting on the base of the dam were not allowed for adequately in design calculations (ANCOLD, 1992; Cavill, 1997; Kline et al., 2007; Duffaut, 2013). As a result, many existing dams have been in need of upgrading, including raising, to meet the new design flood and related stability criteria. Another change in safety standards has been the application of maximum credible or design earthquake (MCE or MDE) criteria (Xu and Bennmokrane, 1996; Bruce, 1997). Detailed discussion of these issues is beyond the scope of this paper. However, it is important to note that safety standards and terminologies may differ between countries and jurisdictions. For example, in the author’s home State of Queensland, Australia, the acceptable flood capacity (AFC) for a High A Hazard Category dam is the PMP DF (DEWS, 2013).

Xu and Bennmokrane (1996) collected and analysed data on 60 concrete dams that had been strengthened using post-tensioned anchors, mainly in the USA, Canada, Australia and Europe. They found that most dams had been strengthened for one or more of the following purposes:

- increasing spillway capacity and stability relating to the PMF;
- upgrading to the MCE;
- upgrading dam stability relating to other conditions such as ageing and deterioration;
- raising dam height related to the PMF;
- raising dam height for greater storage;
- remediating deficiencies in design (e.g. not allowing adequately for uplift);
- stabilising concrete cracks;
- strengthening dam abutments;
- stabilising dam foundations; and
- reinforcing lock walls or building locks into cofferdams.

Fig. 2 illustrates schematically the classical uses of post-tensioned rock anchors in stabilising concrete gravity dams against (a) overturning, and (b) downstream sliding. Fig. 2a shows the main forces used in calculating the anchor tension, , required to maintain moment equilibrium about the downstream toe of the dam. Fig. 2b shows the forces involved in a limiting equilibrium analysis of one of the several possible forms of downstream sliding (USACE, 1995; Fell et al., 2005). It must be recognised that, in practice, concrete dams located in valleys with sloped rock foundation abutments will behave as three-dimensional (3D) structures and so will require 3D stability analyses (Lombardi, 2007; Bretas et al., 2012). Clearly, different analytical models from those shown in Fig. 2 would have to be used for the design of post-tensioned anchors for stabilising other components of the overall
Aschenbroich, 2007; Cavill, 2000; Harman et al., 2010). Even larger anchors than this have been used in some cases.

3. Overview of post-tensioned anchor design

3.1. General principles

This sub-section will discuss the basic mechanics and general principles on which post-tensioned rock anchor design is based. The rock mechanics and rock engineering aspects of anchor design will be considered in more detail in Sections 4 and 5. Although strictly they lie somewhat outside the rock mechanics and rock engineering emphasis of this paper, for completeness and because they impact on the effectiveness of the rock engineering design, a number of important investigation, design and construction issues, including anchor stressing and testing, will be referred to briefly in Section 6.

Traditionally, the dimensioning of rock anchor systems for dams has been based on two-dimensional (2D) static limiting equilibrium analyses of problem configurations such as those illustrated schematically in Fig. 2. From these analyses, the anchor forces required to maintain stability per metre run, or at a particular location, can be established for particular load cases (often called normal or usual, unusual and extreme) and the associated factors of safety required by the Standards, Codes of Practice or Guidelines under which the design is being carried out (ANCOLD, 1992, 2013; BC Hydro, 1995; USACE, 1995, 2005; CDSA, 1999). It is important to note that different load factors or factors of safety may be required by different authorities for similar load cases. Informed comparisons between a number of these approaches are given by Ebeling et al. (2000) and Fell et al. (2005). More recently, linear and nonlinear numerical stress analysis approaches have been used to calculate forces, stresses and displacements in the concrete of the dam, the foundation and abutment rock, and at interfaces, and when assessing the dynamic stability of dams under earthquake loading (Alonso et al., 1996; Yu et al., 2005; Bureau et al., 2007; Scott and Mills-Bria, 2008; Lemos, 2011, 2014; Chopra, 2012). Some Standards, Codes of Practice and Guidelines now require the use of limit state approaches or of partial, rather than overall, factors of safety (Merrifield et al., 1997; Herweynen, 1998; ANCOLD, 2013; Farrell, 2013). However, it has been recognised recently that the partial factor, limit state design approach used in Eurocode 7 (CEN, 2004), for example, may not be applicable to a range of rock engineering problems in which some of the uncertainty in parameter values is epistemic rather than fully aleatory as is assumed in the Eurocode approach (Bedi and Orr, 2014; Harrison, 2014; Lamas et al., 2014). Increasingly, reliability, risk-based, probabilistic and fragility methods of dam analysis and assessment are being used in rock anchor design for dams and other applications (Ellingwood and Tekie, 2001; Phoon et al., 2003; Barker, 2011; Westberg Wilde and Johansson, 2013). Further discussion of these approaches is outside the purpose and scope of this paper.

3.2. Principal modes of failure of grouted, post-tensioned anchors

As well as providing the resistance or stabilising forces required for the purposes referred to at the beginning of Section 2, grouted post-tensioned anchors themselves must be able to resist the four principal modes of applied tension-induced failure illustrated in Fig. 3:

- Mode A – steel tendon tensile failure.
- Mode B – grout—tendon bond or interface failure.

3.3. Design against tendon tensile failure (Mode A)

In Mode A failure, the applied axial tensile load exceeds the yield and, ultimately, the ultimate tensile strength of the steel in the tendon. If \( A_t \) is the cross-sectional area of steel in the tendon and \( \sigma_{tu} \) is the ultimate tensile strength of the steel, then the ultimate tensile capacity is given by

\[
Q_{tu} = \sigma_{tu} A_t
\]

This value is sometimes referred to as the minimum breaking load (MBL) (ANCOLD, 1992). In practice, it is usual to define the design working load (WL) as a proportion of the MBL, often taken as 60%–65% in Australia. During installation of the anchor, it is common practice to test load the anchor and fittings beyond the normal WL (for example, up to 78%–80% MBL) and to “lock-off” the load with sufficient allowance above the WL (for example, to 68%–72% MBL) to allow for subsequent load losses caused by creep, relaxation and draw-in (ANCOLD, 1992). This is one of the areas in which different terminologies may be used in different countries and by different authorities.

For the large, high-capacity, post-tensioned anchors now being used for concrete dams, it is unusual for Mode A failure to be a controlling feature of the design. However, as will be discussed briefly in Section 6, several other factors which are beyond the scope of this paper have to be taken into account in the detailed design, fabrication, installation, stressing and monitoring of the anchors.

3.4. Design against grout–tendon bond or interface failure (Mode B)

The shear resistance developed at the grout–tendon interface arises from adhesion, friction and mechanical interlock between the grout and the tendon. On the initial application of the axial tensile load to the anchor, shear failure at the grout–tendon interface is resisted by adhesion and interlocking. Then as displacement increases, these components of shear resistance may be overcome at points along the bond length and friction then makes the major contribution to shear resistance (Kim and Cho, 2012). Because of the difficulty of distinguishing between and evaluating these components of shear resistance, it is common practice to calculate the grout–tendon shear resistance as

In addition to these four major modes of failure under applied tension, for large-scale post-tensioned rock anchors of the type illustrated in Fig. 1, it is necessary to consider the possibility of shear failure at the internal and external interfaces between the grout and the corrugated sheathing providing a layer of corrosion protection. In the usual case, the depth of the corrugations in the sheathing provides an adequate degree of interlock.

In design practice, it is a common procedure to carry out design calculations to select the length of anchor embedment required to produce pre-determined factors of safety against failure Modes B, C and D. The design length of embedment is then selected as the largest of these values (USACE, 2005). In most practical cases it is found that Mode C is more critical than Mode B.

The essential purpose of the remainder of this paper will be to review the rock mechanics and rock engineering aspects of design against failure Modes C and D. However, for completeness, summary discussions of design against failure Modes A and B will be given. In the discussions of failure Modes A, B, C and D to follow, a single post-tensioned anchor will be generally considered. It must be recognised, however, that in dam engineering practice, anchors are usually used in relatively closely-spaced rows and sometimes in multiple rows. The interactions between adjacent anchors, particularly in the Mode D case, must be taken into account in design calculations.

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**Fig. 3.** Principal modes of failure of grouted rock anchors under applied axial tension. (a) Mode A – steel tendon tensile failure; (b) Mode B – grout–tendon bond or interface failure; (c) Mode C – rock–grout bond or interface failure; and (d) Mode D – rock mass uplift failure (Pease and Kulhawy, 1984).

- Mode C – rock–grout bond or interface failure.
- Mode D – shear or uplift failure within the surrounding rock mass.

Following Kim and Cho (2012), if the ultimate tensile load capacities of an anchor for failure Modes A, B, C and D are \( Q_{tu} \), \( Q_{gtu} \), \( Q_{gu} \) and \( Q_{tu} \), respectively, then the ultimate tensile capacity of the anchor, \( Q_{tu} \), will be the minimum of these four values, i.e.

\[
Q_{tu} = \min\{Q_{tu}, Q_{gtu}, Q_{gu}, Q_{tu}\}
\]  

(1)

In addition to these four major modes of failure under applied tension, for large-scale post-tensioned rock anchors of the type illustrated in Fig. 1, it is necessary to consider the possibility of shear failure at the internal and external interfaces between the grout and the corrugated sheathing providing a layer of corrosion protection. In the usual case, the depth of the corrugations in the sheathing provides an adequate degree of interlock.

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failure is not necessary because consideration of the rock–grout bond failure (Mode C) alone will produce a more than adequate tendon embedment length (Littlejohn and Bruce, 1977).

A number of detailed investigations have demonstrated that the simplifying assumptions made using the approach outlined in the preceding paragraphs are rarely justified. For example, several authors have shown that the grout–tendon interface shear strength depends on the normal confining pressure and stiffness, the roughness of the tendon, whether or not the cement in the grout is expansive, and the grouted bond length (Benmokrane et al., 1995; Barley, 1997; Jarred and Haberfield, 1997).

The recorded influence of the grouted bond length probably reflects the now widely accepted finding that the distribution of shear stress along the grouted length is far from uniform, tending to decay exponentially from the free-length value at the end of the grouted length or proximal end, towards zero at the bottom of the grouted length or distal end (see Fig. 4), depending on the assumptions made in the analysis (Coates and Yu, 1970; Farmer, 1975; Benmokrane et al., 1995; Ivanović and Neilson, 2009; Liu et al., 2013). The ratio of the elastic modulus of the steel tendon, $E_{st}$, to that of the grout, $E_{g}$ (if there is a thick annulus of grout), or alternatively of the rock, has been found to have a particularly important influence on the distribution of elastic shear stress along the embedment length. As shown in Fig. 4 for the case of a fully embedded, shallow anchor, for low modulus ratios of between 0.1 and 1.0 for stiffer grouts and rocks, the stress distribution is markedly non-uniform, but as this ratio increases in softer grouts and rocks, the stress distribution becomes more closely uniform (Coates and Yu, 1970; Liu et al., 2013).

In practice, the possibility must be considered that debonding will occur at the more highly stressed proximal end of the bonded length (the top in Fig. 3b), most probably during prestressing (Benmokrane et al., 1995), resulting in a modification to the elastic stress distribution as illustrated schematically in Fig. 4 for the $E_{st}/E_{g} = 0.1$ case. Distributions of shear stress of the types illustrated in Fig. 4 have been obtained analytically and numerically, and measured in a range of instrumented laboratory and field tests on rock bolts, cable bolts and rock anchors (Kaiser et al., 1992; Benmokrane et al., 1995; Hyett et al., 1995; Woods and Barkhordari, 1997; Ivanović and Neilson, 2009). The stress distribution has also been found to vary with imperfect bonding and when a discontinuity crosses the tensioned tendon or bolt (Zou, 2004; Zhao and Yang, 2011). A number of authors have shown how the non-uniform distribution of shear stress along the anchor length, including the effects of de-bonding, may be modelled by piece-wise linear shear stress distributions (Jarred and Haberfield, 1997; Woods and Barkhordari, 1997; Ivanović and Neilson, 2009; Ren et al., 2010).

4. Design against rock–grout interface failure (Mode C)

4.1. The standard approach

In an analogous manner to the grout–tendon interface, the shear resistance at the rock–grout interface is developed by adhesion, friction and mechanical interlock with friction playing the major role after the other components have been overcome through initial relative axial displacement. As with the grout–tendon interface, the ultimate rock–grout axial shear resistance is commonly evaluated as

\[ Q_{rgu} = 2\pi r_{g} \int_{0}^{l_{rgu}} \tau_{rgu}(z) dz \]  

(4)

where $r_{g}$ is the outside radius of the grouted annulus around the tendon, $l_{rgu}$ is the length of the rock–grout bond, and $\tau_{rgu}(z)$ is the shear resistance generated at distance $z$ along the rock–grout interface.

Again, in common with design against grout–bond tendon failure, it is generally assumed that the shear resistance is distributed uniformly over the rock–grout bond length, $l_{rgu}$, so that the ultimate shear resistance may be calculated as

\[ Q_{rgu} = 2\pi r_{g} l_{rgu} \tau_{rgu} \]  

(5)

where a constant value of $\tau_{rgu}$ is assumed. Many Standards and Codes of Practice indicate that it is required, or at least preferable, that the value of $\tau_{rgu}$ used in design be based on the results of pull-out tests of trial anchors installed at the site (see Section 6 below). However, it is still often selected on the basis of the rock type and condition using tables of presumptive values generally derived from those first published by Littlejohn and Bruce (1975–76, 1977).

Table 1 is a condensed version of the table given by Littlejohn (1992, 1993) in which the sources of Littlejohn’s entries have been omitted. As shown in Table 1, working bond values are lower than ultimate values. Commonly, the working rock–grout shear strength is assumed to be approximately 10% of the uniaxial compressive strength (UCS) of the rock up to a maximum of 4.2 MPa. In some cases, the value is based on the grout compressive strength. Barley (1988) also estimated rock–grout bond strengths from the installation and testing of 10,000 anchors in a wide range of rock conditions in the United Kingdom.

This standard approach which has been used in several countries for several decades suffers from a number of deficiencies to be outlined in the following sub-section.

4.2. Deficiencies in the standard approach

Deficiencies in the standard simplified approach to design against rock–grout interface failure have been found to include:

- a uniform shear stress along the rock–grout interface is assumed as discussed in Section 3.4 for the tendon-grout bond. As noted by Bruce (1997) in a review of the stabilisation of concrete dams by post-tensioned anchors in the U.S.A., this
Table 1  
Some rock—grout bond values recommended for use in design (after Littlejohn and Bruce (1975–76, 1977); Littlejohn (1992, 1993)).

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Working bond (MPa)</th>
<th>Ultimate bond (MPa)</th>
<th>Factor of safety</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Igneous</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium hard basalt</td>
<td>5.73</td>
<td>3–4</td>
<td></td>
</tr>
<tr>
<td>Weathered granite</td>
<td>1.5–2.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Basalt</td>
<td>1.21–1.38</td>
<td>2.8–3.2</td>
<td></td>
</tr>
<tr>
<td>Granite</td>
<td>1.38–1.55</td>
<td>3.1–3.5</td>
<td></td>
</tr>
<tr>
<td>Serpentinite</td>
<td>0.45–0.59</td>
<td>2.6–3.5</td>
<td></td>
</tr>
<tr>
<td>Granite and basalt</td>
<td>1.72–3.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Metamorphic</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Manhattan schist</td>
<td>0.7</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>Slate and hard shale</td>
<td>0.83–1.38</td>
<td>1.5–2.5</td>
<td></td>
</tr>
<tr>
<td>** Arenaceous sediments**</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Limestone</td>
<td>1</td>
<td>2.83</td>
<td>2.8</td>
</tr>
<tr>
<td>Chalk – Grades I–III (N = SPT in blows/0.3 m)</td>
<td>0.005N</td>
<td>0.22–1.07</td>
<td>2 (Temporary)</td>
</tr>
<tr>
<td>Tertiary limestone</td>
<td>0.83–0.97</td>
<td>2.76</td>
<td>2.9–3.3</td>
</tr>
<tr>
<td>Chalk limestone</td>
<td>0.86–1</td>
<td>2.76</td>
<td>2.8–3.2</td>
</tr>
<tr>
<td>Soft limestone</td>
<td>1.03–1.52</td>
<td>1.5–2.5</td>
<td></td>
</tr>
<tr>
<td>Dolomitic limestone</td>
<td>1.38–2.07</td>
<td>1.5–2.5</td>
<td></td>
</tr>
<tr>
<td><strong>Calcereous sediments</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hard coarse-grained sandstone</td>
<td>2.45</td>
<td>1.75</td>
<td></td>
</tr>
<tr>
<td>Well-cemented mudstone</td>
<td>0.69–0.85</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Bunter sandstone</td>
<td>0.6</td>
<td>2</td>
<td>2.25</td>
</tr>
<tr>
<td>Bunter sandstone (UCS &gt; 2.0 MPa)</td>
<td>0.4</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>Hard fine sandstone</td>
<td>0.69–0.83</td>
<td>2.24</td>
<td>2.7–3.3</td>
</tr>
<tr>
<td>Sandstone</td>
<td>0.83–1.73</td>
<td>1.5–2.5</td>
<td></td>
</tr>
<tr>
<td><strong>Argillaceous sediments</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Keuper marl</td>
<td>0.17–0.25</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>(0.45c_u)</td>
<td>0.35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weak shale</td>
<td>0.1–0.14</td>
<td>0.37</td>
<td>2.7–3.7</td>
</tr>
<tr>
<td>Soft shale</td>
<td>0.21–0.83</td>
<td>1.5–2.5</td>
<td></td>
</tr>
<tr>
<td><strong>General</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Competent rock (where UCS &gt; 20 MPa)</td>
<td>UCS – 30</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>(up to a maximum value of 14 MPa)</td>
<td>UCS – 10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Weak rock</td>
<td>0.35–0.7</td>
<td>4.2</td>
<td></td>
</tr>
<tr>
<td>Medium rock</td>
<td>0.7–1.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strong rock</td>
<td>1.05–1.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wide variety of igneous and metamorphic rocks</td>
<td>1.05</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Wide variety of rocks</td>
<td>0.98</td>
<td>2</td>
<td>2–2.5 (Temporary)</td>
</tr>
<tr>
<td>0.5</td>
<td>2</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>0.7</td>
<td>1.2–2.5</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>0.7</td>
<td>1.2–2.5</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>Concrete</td>
<td>1.38–2.76</td>
<td>1.5–2.5</td>
<td></td>
</tr>
</tbody>
</table>

The approach using a uniform shear stress and “average” bond values can lead to “extraordinarily and wastefully long bond zones”.
- the wide-spread use of decades old, empirically-based, presumptive interface shear strengths based on rock type and condition and, more rarely, grout compressive strength (see Table 1);
- as with the grout—tendon interface, no allowance is made for the influence of the normal stress acting on the grout—rock interface before and during shearing, its variation over the embedment length (Barley, 1997), or the influence of grouting methods and pressures on the normal stresses generated (Park et al., 2013);
- no recognition is taken of the progressive nature of the rock—
  grout bond failure process in which at least three distinct stages may be identified — pseudo-elastic behaviour at small deformations, failure or de-bonding development at the rock—
  grout interface, and residual behaviour after larger displacements (Pease and Kulhawy, 1984; Park et al., 2013);
- no allowance is made for the effects of structural features such as faults, shear zones and jointing in the rock mass or of other local variations in rock strength (Dados, 1984; Zou, 2004);
- the influence of the borehole roughness and diameter, and therefore the thickness of the grout annulus, are not considered; and
- the type of cement used in the grout can have a significant influence on the shear resistance generated (Haberfield and Baycan, 1997).

In dam engineering practice, the preferred way of overcoming some of these deficiencies is by the use of a necessarily average interface shear strength value established through full-scale pull-out tests on trial anchors of the anticipated design, carried out in the rock mass concerned before the design is finalised (Scott and Bruce, 1992; Feddersen, 1997; Cavill, 2000). In tests carried out for this purpose, it is important that the test configuration is selected to ensure that the failure occurs at the rock–grout interface.

5. Design against rock mass failure (Mode D)

5.1. The standard approach

Although it is generally found to be conservative, from a rock mechanics or rock engineering perspective, the commonly-used or standard approach to the design of post-tensioned anchors against rock mass failure by uplift (Mode D), is probably the least satisfactory of the design approaches discussed in this paper.

As illustrated in Fig. 5 for the case of a single anchor, the simplest standard approach assumes that the uplift force generated by the post-tensioned anchor is resisted by the weight of a cone of homogeneous rock. It generally ignores the fact that for uplift to occur, not only the weight of the rock cone, but also the shear and/or tensile strength of the rock mass must be overcome. Depending on the nature of the anchorage and the assumed properties of the rock mass, the cone angle and the location of the apex of the cone may vary. Fig. 5 illustrates the commonly assumed cases in which the cone angle is either 60° or 90°, and the apex of the cone is located at the mid-point or at the distal end of the grouted embedment length. Some designers may also locate the apex of the cone at the

Fig. 5. Cone geometries used in uplift capacity calculations for single tensioned anchors (after Littlejohn and Bruce (1975–76, 1977), Hubst and Zajic (1977) and Littlejohn (1993)).
For a general cone angle of \( \theta_c \) (\( \theta_c = 60^\circ \) or \( 90^\circ \) in Fig. 5) and the apex of the cone located at the mid-point of the grouted embedment length, the uplift force required to overcome the weight of the cone of rock, \( W_c \), may be calculated as

\[
Q_{u} = \frac{1}{3} \pi \gamma \tan \frac{\theta_c}{2} (D + \frac{L}{2})^{3}
\]  

(6)

where \( \gamma \) is the unit weight of the rock, \( D \) is the depth from the surface to the top or proximal end of the embedment length, and \( L \) is the grouted embedment or bond length. Although it is more conservative to use the full unit weight of the rock in this calculation, in some applications in dam engineering the buoyant or submerged unit weight should be used. Commonly in design, the required bond length, \( L \), is calculated by equating the rock–grout bond resistance given by Eq. (5) to the uplift force required to overcome the weight of the cone of rock given by Eq. (6) and applying a factor of safety, often taken to be 2.

In the author’s experience, this simple, conservative approach is generally used in dam engineering in the dimensioning of large post-tensioned anchors against failure of the rock mass by uplift. However, two modifications may be made to allow more realistically for the circumstances and conditions met in practice.

Firstly, for major applications such as the anchoring of concrete dams in the manner illustrated in Fig. 2a and the anchoring of dam toes, spillways and training walls, anchors are generally installed in rows at relatively close spacings of a few metres. In these cases, cones such as those illustrated in Fig. 5 will overlap or interact as shown conceptually in two dimensions in Fig. 6a. In this case, the calculation of the weight of the cone given by Eq. (6) may be replaced by a simple approximate 2D calculation assuming a triangular cross-section to give the uplift resistance available per unit length along the row of anchors as

\[
Q_{u} = \frac{1}{2} \gamma (D + \frac{L}{2})^{2} \tan \frac{\theta_c}{2}
\]  

(7)

where \( s \) is the spacing of anchors along the row.

An important practical case arises when the rock mass contains sub-horizontal bedding or structural features including jointing. In this case, the anchorage horizons within a row of anchors may be staggered vertically such that the tendency to uplift a volume of rock defined by the one sub-horizontal plane is avoided (Xu and Benmokrane, 1996). In the writer’s experience, this practice has been used when the dam is founded on igneous rock in which sheeting jointing has developed in an incised river valley.

Secondly, in the calculations of uplift resistance in a range of applications, some authorities and authors seek to allow for the tensile or shear strength of the rock mass, generally through the use of assumed values of rock mass tensile strength, shear strength or cohesion (Coates, 1970; Hobst and Zajíc, 1977; Anon, 1996; Kim and Cho, 2012). The estimated values of tensile or shear resistance may be used either instead of, or in addition to, the resistance to uplift provided by the weight of the cone of rock. In presenting approaches of this type, some authors use idealised 2D representations with a triangular section as shown in Fig. 6b rather than the 3D cone. For example, Hobst and Zajíc (1977) gave a number of solutions in terms of the rock mass shear strength. Following Coates and Yu (1971), Kim and Cho (2012) suggested that the tensile resistance on the failure surface of the rock cone in Fig. 5, \( f_t \), is given by

\[
f_t = \frac{\sigma_u \pi D_c^2 \tan \frac{\theta_c}{2}}{\cos \frac{\theta_c}{2}}
\]  

(8)

where \( \sigma_u \) is the tensile strength of the rock mass and \( D_c \) is the depth of the rock cone, equal to \( D + L/2 \) in Fig. 5.

Of course, this approach begs the question of the exact physical meaning of the assumed parameter, \( \sigma_u \), and how it might best be estimated. In an alternative approach, Weerasinghe and Adams (1997) proposed the use of a rock mass shear strength, rather than a tensile strength, back-calculated from the results of pull-out tests, assuming a theoretical failure cone of the type being discussed here. Clearly, the use of this approach to estimate rock mass shear strength would be impracticable for the very high-capacity, large-scale anchors such as those described by Cavill (1997, 2000) now being used in dam engineering, not least because it is difficult to understand how the assumed failure mechanism could develop in most practical circumstances. This and related issues will be discussed in more detail in Section 5.2.2 to follow.

5.2. Deficiencies of the standard approach

As with the approaches used to design against grout–tendon (Mode B) and rock–grout (Mode C) bond failures discussed previously, from a rock mechanics and rock engineering perspective, the commonly-used approach to design against rock mass uplift (Mode D) failure assuming a conical uplift failure mechanism, suffers from a number of deficiencies. The major rock mechanics and rock engineering deficiencies will be discussed briefly below in terms of the implicitly assumed stress distribution imposed within the rock mass, the rock mass uplift failure mechanism, the influence of the rock mass structure, and the assumed uniform values of rock mass tensile and/or shear strengths.

5.2.1. Stress distribution induced in the overlying rock mass

If failure of, or large deformations in, a rock mass influenced by a single anchor or a system of post-tensioned anchors is to be assumed, knowledge is required of the distribution of stresses induced in the rock mass by the anchor and other means. In many cases, concrete dams are constructed in or across incised valleys or gorges. In these cases, the major principal in situ stress near the surface is likely to be oriented parallel to the valley surface. In the cases illustrated in Figs. 5 and 6, the weight of the rock and the transfer of stress from the tensioned anchor to the rock mass must also be considered in calculating the distribution of stresses within...
the rock mass around and above the anchor(s). In many cases, the rock mass can be expected to have anisotropic elastic properties. In the case of a deep, post-tensioned anchor loaded through a surface bearing plate, a bulb of compressive vertical stress will be induced in the rock mass around the anchor head, and the stress distribution around the anchor embedment length at some depth can be expected to be quite complex and non-uniform. Furthermore, it must be expected that the final induced stress distribution will be influenced by weathering profiles, changes in rock type and any major structural features transecting the section of the rock mass influenced by the anchor-related stresses. Because of this range of influences, it is not easy to conceptualise a general pattern of stress induced in the rock mass by a single, deep, post-tensioned anchor or by a system of deep, post-tensioned anchors that is likely to lead to a conical failure surface.

The assumption of a cone of failure discussed in Section 5.1 implies that either the tensile or shear strengths of the rock mass, or both, must be overcome over the surface of the cone before uplift can occur. If, as is usually the case, it is assumed that the shear and tensile strengths of the rock mass are constant over any given depth, this implies, in turn, that the shear and/or tensile stresses induced in the rock mass must reach their maxima along some cone-like surface (in the case of a single anchor). While, as will be discussed in Section 5.2.2 below, it can be readily envisaged that this will occur in the case of a short anchor that is grouted to, or close to, the surface, it is more difficult to understand why this should be the case when the anchor is grouted only over an embedment length, $L$, which is often significantly less than $D$, the depth below the ground surface of the proximal end of the grouted length. In the first instance, the distribution of induced elastic stresses could be investigated through a series of relatively simple numerical stress analyses of sample problem configurations.

5.2.2. Rock mass uplift failure mechanism

Most direct observations of the rock mass uplift failure mechanism of which the author is aware, were associated with field pull-out tests in which, almost of necessity, the anchor was short in comparison to the lengths of the large post-tensioned anchors used in dam construction and rehabilitation projects, and the anchor was grouted to, or close to, the ground surface (Dados, 1984; Benmokrane et al., 1995; Carter, 1995; Weerasinghe and Littlejohn, 1997; Serrano and Olalla, 1999; Thomas-Lepine, 2012). Furthermore, in some of these cases, the test anchor was installed in near-surface, weak rock.

A number of such investigations have found that while discernible surface uplift extends for some distance laterally from the anchor borehole, the limit of “significant” uplift can be represented reasonably well by a $90^\circ$ cone. For example, Fig. 7 shows the failure mechanism observed and inferred by Weerasinghe and Littlejohn (1997) for a straight shafted anchor grouted to the ground surface in a weak mudstone (Weerasinghe and Littlejohn, 1997) for a straight shafted anchor grouted to the ground surface in a weak mudstone. (Weerasinghe and Littlejohn, 1997) for a straight shafted anchor grouted to the ground surface in a weak mudstone. Weerasinghe and Littlejohn (1997) argued that, for fully grouted anchors, as anchorage depth increases, the resistance of the rock mass cone will become smaller compared to the shaft (grout–rock) resistance. Numerical experiments using discrete element codes provide perhaps the most promising means of exploring potential failure mechanisms of deep anchors in rock masses for which rock mass structure models or discrete fracture network (DFN) simulations are available (Panton et al., 2014). Ideally, numerical analyses of this type should be 3D rather than 2D.

As was noted at the end of Section 5.1, in dam engineering, the potential influence of a continuous or near-continuous horizontal or sub-horizontal structural feature such as a bedding plane or sheeting joint can be reduced by staggering the elevations of the anchor lengths of adjacent anchors in a row. In either the shallow or

![Fig. 7. Failure by uplift of a fully grouted, straight-shafted, shallow, tensioned anchor in weak mudstone (Weerasinghe and Littlejohn, 1997).](image)

![Fig. 8. Influence of rock mass structure on the uplift failure mechanism of a shallow, fully grouted, tensioned anchor (after Wyllie (1999)).](image)
deep anchor case, or in the case of a row of anchors as illustrated in Fig. 6, it must be recognised that, as shown by a range of pull-out tests and by preliminary numerical simulations carried out by Panton et al. (2014), the rock mass uplift failure mechanism will be progressive.

5.2.4. Estimation of rock mass shear and/or tensile strengths

Existing design approaches using the 90° or 60° uplift cone hypotheses, variously assume shear, cohesive and/or tensile strengths of the rock mass which may be the presumptive values or be back-calculated from the results of pull-out tests using a “theoretical” failure cone (Hobst and Zajíc, 1977; USACE, 1995; Weerasinghe and Adams, 1997). Clearly, these simple approaches do not take into account the variability of the governing strength parameters with depth and with varying local rock mass structure, or the progressive and complex nature of the rock mass failure mechanisms involved. Nor do they always allow for the influence of the total distribution of stresses within the rock mass. It is suggested that these simplistic approaches can, and should, be improved.

5.3. Suggested alternative approaches

At the end of Section 3.1, it was noted that linear and nonlinear static and dynamic numerical stress analyses (Alonso et al., 1996; Yu et al., 2005; Bureau et al., 2007; Scott and Mills-Bria, 2008; Lemos, 2011, 2014; Chopra, 2012), limit state (Merrifield et al., 1997; Herweynen, 1998; ANCOLD, 2013; Farrell, 2013), reliability, risk-based, probabilistic and fragility methods (Ellingwood and Tekie, 2001; Phoon et al., 2003; Barker, 2011; Westberg Wilde and Johansson, 2013) are now being used for dam design analysis and assessment. These approaches may be used when analysing the overall stability of dams stabilised by post-tensioned anchors in the manner illustrated schematically in Fig. 2. As well as the traditional limiting equilibrium analyses, numerical stress analysis methods may use continuum, equivalent continuum or discontinuum methods. As was indicated in Section 5.2.3 and has been demonstrated by Panton et al. (2014), numerical methods, particularly discontinuum methods used in conjunction with a structural or DFN model of the rock mass concerned, potentially provide a powerful means of investigating the likely responses of rock masses to the uplift imposed by post-tensioned anchors. Provided adequate input data are available (see Section 6.1 below), these methods can also allow the influence of the in situ stresses and the stresses induced in the rock mass to be evaluated more rationally and more fully than does the standard design approach discussed in Section 5.1. However, in the author's experience, detailed design analyses of this proposed type are rarely carried out in practice.

Although the previous discussion concerned Mode D or rock mass uplift failure, it should also be noted that, for any given case, comprehensive numerical analyses can be used to improve the methods generally used to estimate the distribution of normal and shear stresses along tendon–grout and rock–grout interfaces and to evaluate the onset and progression of interface bond failure and slip. Although this may appear to be relatively straightforward, for its complete numerical analysis, this problem requires the evaluation and input of a number of rock and grout properties, including stiffnesses. As noted in Section 3.4, historically a significant number of analyses of this type have been carried out for rock and cable bolts and for shallow or near-surface anchors, but to the best of the author’s knowledge rarely, if at all, for the large post-tensioned anchor currently being used for dam stabilisation.

As discussed in Section 5.2, there are a number of problem configurations for which the 60° or 90° uplift cone assumptions do not provide realistic models for use in design analysis. In a range of other geotechnical engineering applications, including some involving anchored structures, the upper and lower bound theorems of limit state theory have been used to identify likely failure mechanisms and the associated limit loads (Sloan, 1989, 2013; Regenass and Soubra, 1997; Soubra and Regenass, 1997). In these geotechnical engineering applications, numerical methods, notably finite element methods, including large deformation finite element methods, have been used in conjunction with limit state analysis to obtain solutions to otherwise intractable problems (Sloan, 1989, 2013; Wang et al., 2013). It is suggested that such an approach could be used to good effect to identify potential rock mass uplift conditions associated with the use of large post-tensioned anchors to stabilise or strengthen dams.

6. Other investigation, design and construction issues

6.1. Overview

The major rock mechanics and rock engineering issues associated with the design of post-tensioned rock anchors for concrete dams have been discussed in the preceding sections of this paper. However, in order to ensure acceptable performance of the anchors and of the structures that they are intended to stabilise or reinforce, a range of other site investigation, design and construction issues associated with the anchors have to be planned and executed satisfactorily. To discuss all of them in detail would extend the scope of this paper beyond reasonable limits. However, for completeness, they will be referred to briefly here. These issues include:

- site investigation and characterisation;
- the installation and testing of trial anchors;
- grout design and grouting trials;
- the fabrication, installation, tensioning and possibly re-tensioning of the anchors;
- the provision of corrosion protection to the anchors and anchor heads; and
- the in service monitoring and maintenance of anchors and their fittings.

Many of these issues are covered by Standards, Codes of Practice and Guidelines. Some of them will be discussed briefly below.

6.2. Site investigation and characterisation

As the author has argued elsewhere (Brown, 2011), the design and construction of large dams played a significant role in the development of rock mechanics and rock engineering knowledge and techniques that took place in the middle decades of the 20th century. This applied particularly to site investigation methods and to the assessment of the mechanical properties of rocks and rock masses, including discontinuities. In modern dam engineering, adequate site investigation and the resulting site characterisation, and geological, structural and geotechnical model development remain of paramount importance. In terms of the various types of design analyses discussed in this paper, particular site characterisation requirements include:

- estimating the in situ stresses at the dam site, particularly under incised valleys;
- mapping discontinuities in the foundation rock mass and developing complete 3D structural models for the geotechnical domains identified, preferably using a DFN approach;
- estimating rock and rock mass strengths and deformabilities;

- measuring or estimating the basic friction angles, roughnesses and normal and shear stiffnesses of persistent discontinuities likely to be involved in the sliding mode of failure illustrated in Fig. 2b;
- establishing design values of the shear and tensile strengths of old and new foundation rock—mass concrete interfaces (Lo et al., 1991; EPRI, 1992); and
- measuring or estimating rock mass permeabilities and identifying significant water-bearing discontinuities in the foundation (Farinha et al., 2011).

Examples of some of the modern approaches and techniques used to obtain data of these types are given by Brown and Marley (2008), Powell et al. (2008), Shaffner et al. (2009), Friz et al. (2011), Hencher et al. (2011) and Agharazi et al. (2012).

6.3. Anchor and grouting trials

It is essential that before the detailed design of a system of large, post-tensioned anchors is completed, trial anchors of prototype size should be installed in the rock mass concerned and load tested. This overall process should include tests of drilling accuracy and drill hole stability in the candidate rock mass; tests of proposed grout mixes and grouting tests to enable suitable grouting pressures to be determined and the extent of the likely loss of grout into the surrounding rock mass to be assessed; and, perhaps most importantly, load testing of a generally small number of trial anchors for a range of possible purposes, including the identification of potential failure mechanisms and, with suitable test design, the estimation of mean rock—grout interface strengths. As will be discussed in Section 6.4 to follow, it is also common or required practice to test load each prototype anchor on installation and subsequently during the anchor’s service life. Examples of anchor testing procedures and results are given by Habib (1989), Scott and Bruce (1992), Littlejohn (1993), Bruce (1997), Feddersen (1997) and Cavill (2000).

6.4. Anchor stressing and in service monitoring

The initial anchor stressing and proof-testing, and the inspection, monitoring, maintenance and re-stressing of anchors during their service lives, are critically important in ensuring the satisfactory in service performance of post-stressed anchor systems. As Littlejohn (1993) noted, “stressing is required to fulfil two functions. (i) To tension the tendon and to anchor it at its secure load. (ii) To ascertain and record the behaviour of the anchorage so that it can be compared with the behaviour of control anchorage systems, subjected to on-site suitability tests.” The structural design of the anchor stressing system, including the load cell, anchor head and bearing plate, is an important element of this undertaking (Littlejohn, 1993). As was noted in Section 3.3, during installation of the anchor, it is common practice in Australia to test load the anchor and fittings beyond the normal WL (for example, up to 78%-80% MBL) and to “lock-off” the load with sufficient allowance above the WL (for example, to 68%-72% MBL) to allow for subsequent load losses caused by creep, relaxation and draw-in (ANCOLD, 1992). Following this initial stressing, it is necessary to protect the anchor head against corrosion while ensuring that it remains accessible for subsequent inspection and re-stressing (Littlejohn and Mothersille, 2007).

The volume edited by Littlejohn (2007) contains a wide range of papers on the in service inspection, maintenance and monitoring of ground anchors and anchored structures, including dams (Aschenbroich, 2007; Wolfhope et al., 2007; Zicko et al., 2007).

7. Conclusions

High-capacity, post-tensioned anchors have found wide-spread use, originally in initial dam design and construction, and more recently in the strengthening and rehabilitation of concrete dams to meet modern design and safety standards. Despite the advances that have been made in rock mechanics and rock engineering during the almost 80 years in which post-tensioned anchors have been used in dam engineering, the author’s reading of the literature, and his own practical experience in Australia, suggest that some aspects of the rock mechanics and rock engineering design of rock anchors for dams have changed relatively little over the last 30 or 40 years. This applies, in particular, to the calculations usually carried out to establish the grouted embedment lengths required for deep, post-tensioned anchors. These calculations usually make simplified assumptions about the distribution and values of rock—grout interface shear strengths, the shape of the volume of rock likely to be involved in uplift failure under the influence of a system of post-tensioned anchors, and the mechanism of that failure. The resulting designs are generally conservative. While this is understandable for important structures like dams for which public safety is a major consideration, in some cases, the degree of conservatism is considered to be excessive and to represent poor engineering practice.

It is concluded that the aspects of rock mechanics and rock engineering design of large, post-tensioned rock anchors for dams of major concern here, the rock—grout bond and the uplift failure mechanism, can be significantly improved by making greater use of modern, comprehensive, numerical analyses in conjunction with 3D models of the rock mass structure, realistic rock and rock mass properties, and the results of prototype anchor tests in the rock mass concerned.

Conflict of interest

The author has no known conflict of interest associated with this review paper.

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