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Full scale field tests of high capacity rock anchors

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ABSTRACT: High capacity multistrand rock anchors have been installed through the main arch and one thrust block of Stewart Mountain Dam, Arizona, to ensure the stability of the dam and foundation during extreme seismic events. This paper describes a full-scale test program performed beforehand to verify anchor design and performance assumptions.

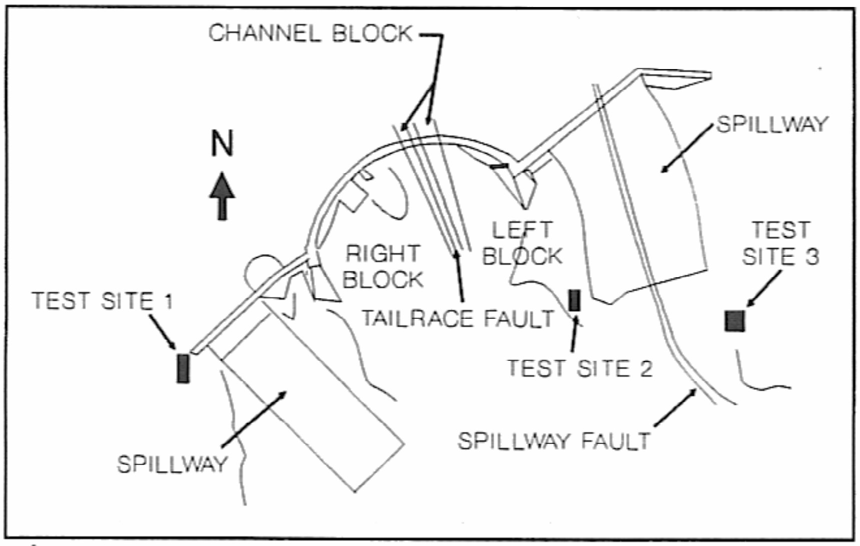

Figure 1. Plan of Stewart Mountain Dam

1 INTRODUCTION

Stewart Mountain Dam is a 65-m-high arch structure, with gravity thrust blocks and abutment sections (Figure 1). The dam was completed in 1930 on the Salt River approximately 40 miles northeast of Phoenix, Arizona. As part of dam safety modifications, 84 high capacity rock anchors were installed through the main arch and left thrust block to ensure stability of the dam and foundation during extreme seismic events. Concrete blockouts for the anchors had been constructed as part of a separate contract for other modifications, so there was no chance to relocate anchor positions in the event of initial anchor failure. Therefore, the contract specifications required a full-scale test program to verify that the design of the bond length would provide an adequate margin of safety in resisting the tension applied to the anchors. Additional testing and monitoring, described in section 3, were performed to maximize information gained from the tests, and to aid in better understanding the anchor behavior and performance. This paper describes the anchor test program and its results.

2 SITE GEOLOGY

The dominant rock types at the site are Precambrian quartz diorite and irregular intrusive granite dikes which have cut the diorite. Small dikes of fine grained diabase and silicics cut through both the diorite and granite. Underneath the left abutment spillway (Figure 1) and to the east, the rock is a fine grained granite mylonite. Although the rock under the main arch structure is generally fresh to slightly weathered, there is increasingly more fracturing and weathering from west to east. In addition, two inactive faults underlie the structure, striking to the northwest. The rock adjacent to each fault is intensely to very intensely fractured, and contains numerous shears. Based on these geologic conditions, the dam foundation was divided into three blocks: the channel block affected by the Tailrace Fault, the left block to the east of the channel block, and the right block to the west of the channel block. Test sites were selected to represent each of these blocks. Additional characterization of the test sites, including assessment of compressive strength, permeability, and rock mass modulus, was performed as discussed in the following section.

3 TEST SITES

Test site 1 was located adjacent to the upper west abutment. The rock is diorite and represents the right block. The rock is generally hard, moderately fractured, and slightly weathered with moderate weathering locally along shear zones. It is considered to be the best quality rock at the damsite.

Test site 2, located adjacent to the right side of the left abutment spillway, is representative of the left block and consists of diorite with some granite. The rock at this site is of slightly less quality than at site 1 due to increased weathering and fracturing.

Test site 3 was located in a fault-disturbed zone adjacent to the left side of the left abutment spillway. This site was chosen as a worst-case representation of the Tailrace Fault, and the rock at this site is considered to be the poorest in the vicinity of Stewart Mountain Dam. It is shattered diorite and granite mylonite with slickensided montmorillonite clay-coated fracture surfaces.

Two vertical test anchors were installed about 3.7 m apart at each site, one with a bond length of about 3 m designated "A" (i.e. 1A, 2A, 3A), and the other with approximately a 6-m bond length designated "B" (i.e. 1B, 2B, 3B). The unbonded (free) length was about 9 m in all cases. Core holes (76-mm-diameter) were drilled and logged at each test anchor location, and then enlarged to 254 mm diameter using rotary percussive (down-the-hole hammer) equipment for anchor installation. Characterization of the test locations included the following:

1. Point load tests to estimate the strength of core from the bond zones.
2. Dilatometer tests to measure rock mass modulus in the bond zone of the core holes, at pressures up to 20 MPa.
3. Empirical assessment of the bond zone rock mass modulus.
4. Evaluation of jack settlement (rock mass modulus) during tensioning.
5. Staged cyclic permeability (pump-in) tests in both the 76- and 254-mm holes, at pressures up to 310 kPa.

Table I. Rock mass modulus and core strength estimates

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

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

The results of this characterization are summarized in Tables I and II. Notable observations include: (1) the rock mass modulus values determined from the dilatometer tests are considerably lower than those determined from empirical relationships or jack settlement calculations, and (2) the lower Lugeon (water test) values recorded after redrilling indicate that the down-the-hole air flush method of enlarging the holes probably plugged joints and fractures rather than opening them up, providing better conditions for anchor grouting.

4 ANCHOR MATERIALS AND INSTALLATION

Full scale production tendons were used in the test program. Each was comprised of 28 epoxy-coated seven-wire strands, 15.2 mm diameter, of 1.862-GPa steel. The guaranteed ultimate tensile strength (GUTS) was 7.295 MN, and the design load (DL) was 4.381 MN for each anchor. Testing was permitted to 80 percent of the GUTS. Spacers and centralizers were included in the tendon makeup for the bond length.

Single point extensometers were attached to the strands in the bond zone of each anchor. The extensometers consisted of a 254-mm-long deformed bar anchorage connected to a 6-mm-diameter fiberglass rod, which passed through a polyethylene sheath to the collar of the anchor hole.

A program of grout testing was conducted to aid in the selection of an appropriate anchor grout mix. A stable but pumpable mix with a water/cement ratio of 0.45 by weight, and a superplasticizer additive of 0.5 percent by weight of cement was selected. Average properties determined by testing of 76- by 152-mm cured cylinders of the grout mix are summarized in Table III.

Table II. Water test results

Hole	76-mm-diameter		254-mm-diameter	
	Depth (m)	Lugeons[1]	Depth (m)	Lugeons[1]
1A	3.0-6.1	<1	3.0-9.1	0
	6.1-9.1	<1		
1B	3.0-6.1	<1	3.0-12.2	<1
	6.1-9.1	0		
	9.1-12.2	<1		
2A	3.0-6.1	<1	3.0-6.1	10
	6.1-9.1	>100	6.1-9.1	<1
2B	3.0-6.1	0	3.0-12.2	0
	6.1-9.1	6		
	9.1-12.2	<1		
3A	7.6-9.1	<1	n/a[2]	n/a[2]

[1]Based on cyclic procedure proposed by Houlsby (1976)
[2]Grouting performed at site 3 based on water loss during falling head and constant head water tests

The tendons were installed in their respective holes using a crane (Figure 2) and suspending them by steel frames bearing on the underside of the temporary anchorage head. Grouting was then conducted through a centrally placed 25-mm-diameter tube exiting below the distal end, after water had been flushed through the system for 5-10 minutes. The grout was mixed in a colloidal mixer, where the additive was placed in the mixer after the cement and water had been preblended. The grout mix was temporarily transferred to a screw-agitated storage tank from which it was pumped to the holes by a progressive cavity pump. The amount of grout necessary to give the desired bond length was calculated theoretically and measured into each hole. Due to problems inherent with achieving a hole of exactly the diameter of the drill, particularly for anchor 3B, the actual bond lengths were slightly different than

Table III. Grout test results

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

those planned. The actual bond lengths, determined by sounding the completed installations, are summarized in Table IV.

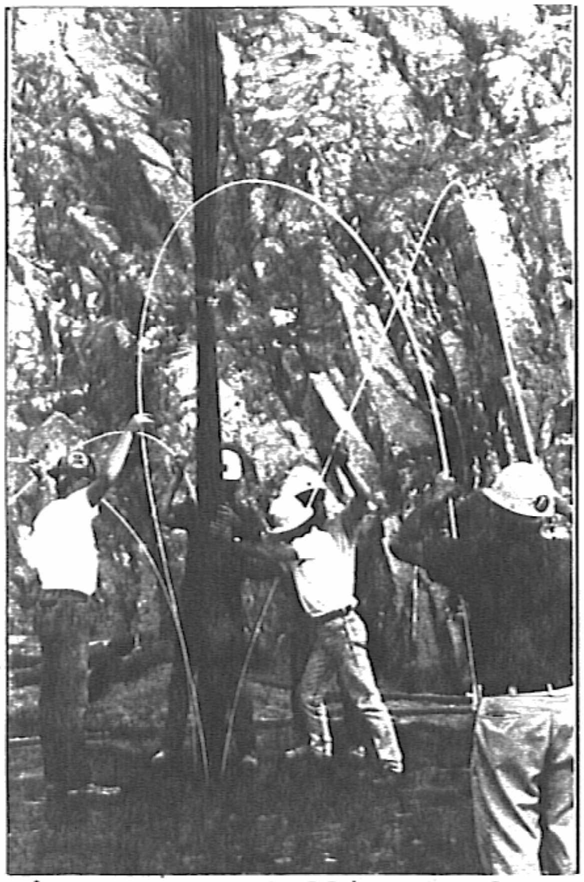

Figure 2. Installing tendon and extensometers

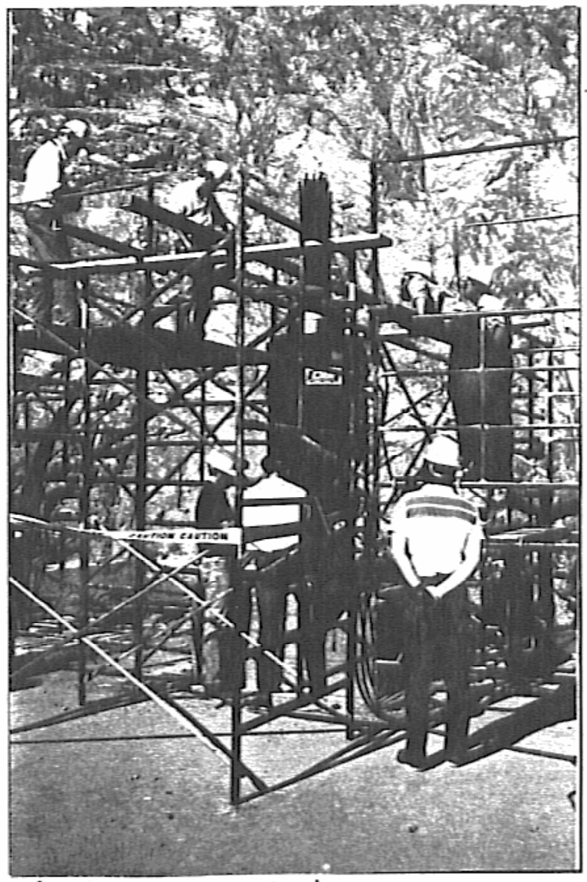

Figure 3. Test in progress

5 STRESSING

The stressing setup used in the test program is shown in Figures 3 and 4. Reinforced concrete bearing blocks, 1.5 by 1.5 by 0.9 m, were fabricated onsite and placed on leveling pads of high strength nonshrink grout cast around the collar of each anchor. A composite load cell, consisting of eight 1.779-MN strain-gauge load cells sandwiched within two specially machined bearing plates, was read by use of a portable computer. The pressure for the 12.454-MN hollow ram multijack was displayed by both large face dial gauge and an electronic digital display. Dial gauges were used to monitor displacements including: (1) 2 gauges on the stressing head to measure total tendon extension, (2) a gauge to measure settlement of the system during stressing, and (3) a gauge on each extensometer rod. Each of these gauges was attached to an independent reference frame. In addition, a gauge was used to measure pull-in of the wedges relative to the stressing head.

The anchors were stressed when the grout had reached an age of about 7 days. Initially, a single-strand jack was used to apply an alignment load (AL) to the tendon of approximately 10 percent of the design load (DL) by stressing each strand individually. This ensured uniform seating of the wedges and equal load sharing between strands during multistrand stressing. Cyclic multistrand testing was conducted to permit

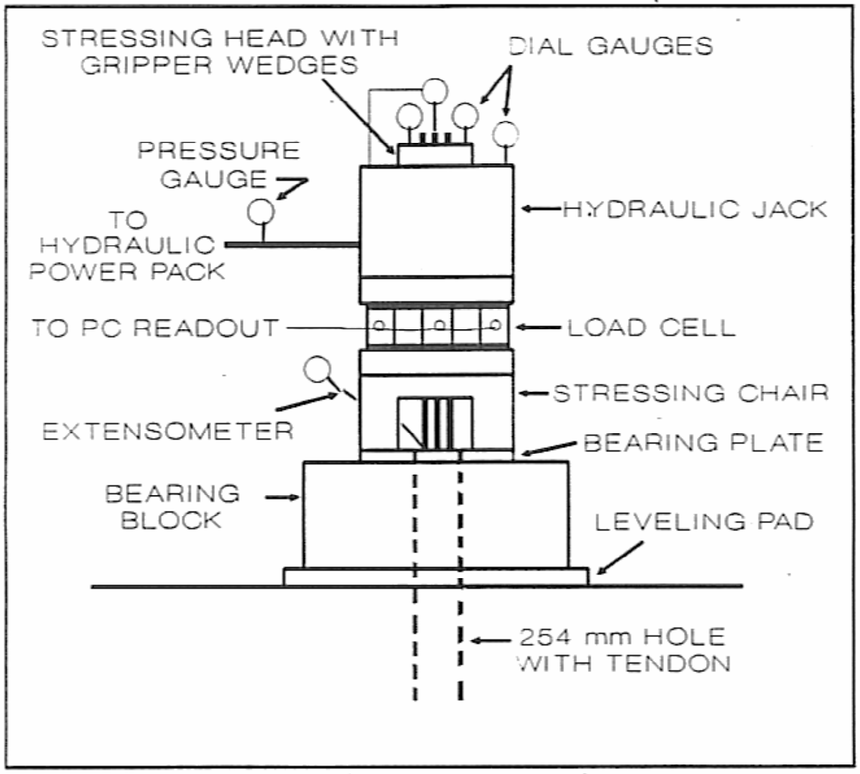

Figure 4. Schematic of stressing setup

analysis of elastic and permanent displacements, in the following load increments (and cycle maxima): 25% DL (1.094 MN), 50% DL (2.193 MN), 75% DL (3.287 MN), 100% DL (4.381 MN), 120% DL (5.258 MN), 127.5% DL (5.587 MN), and 133% DL (5.827 MN). Short term (approximately 10-minute) creep displacements were recorded at cycle maxima, and permanent displacements were recorded upon unloading to AL. Long term creep measurements were not performed due to time and equipment limitations.

6 ANCHOR PERFORMANCE

All anchors held 133 percent of the design load except anchor 3A with the shorter bond length in the worst rock type. This anchor sustained 97 percent of the DL (4.250 MN) before failure occurred. The load/extension plots shown in Figures 5 and 6 represent the range of response recorded during the testing. The anchors at sites 1 and 2 behaved very elastically, whereas the anchors at site 3 demonstrated more inelastic response.

Short term creep data recorded for the anchors at 100 percent of design load are shown in Figure 7. At this load, the creep was small and stabilized rapidly. Permanent (nonrecoverable) displacements are shown in Figure 8, corrected for wedge pull-in. As expected, site 1 anchors generally showed the least creep and permanent displacement, whereas site 3 anchors showed the most. The second anchor tensioned at each site

Table IV. Bond lengths and apparent debonding

Hole	Target bond length (m)	Actual bond length (m)	Apparent debonded length (m)
1A	3.0	3.11	0.55
1B	6.1	6.07	0.64
2A	3.0	3.57	1.00
2B	6.1	6.49	1.13
3A	3.0	3.60	3.60
3B	6.1	3.96	1.86

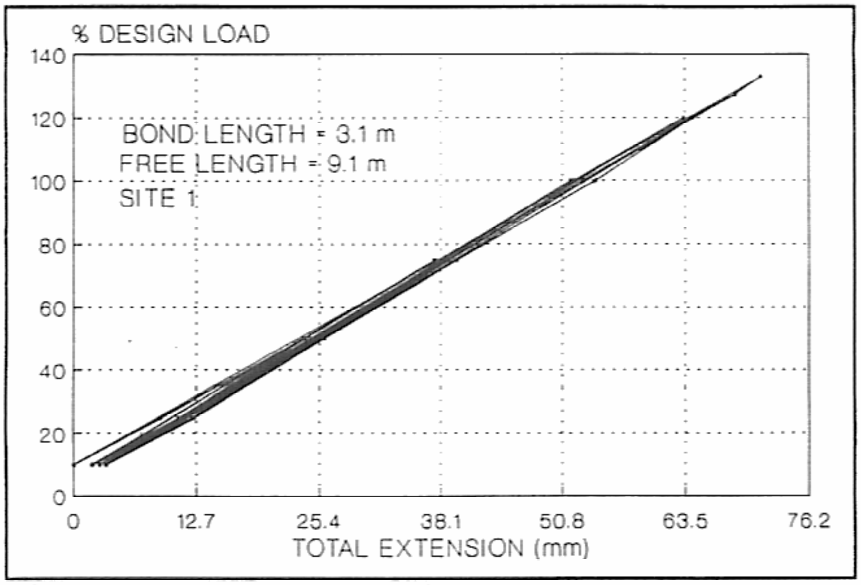

Figure 5. Load-extension response for anchor 1A

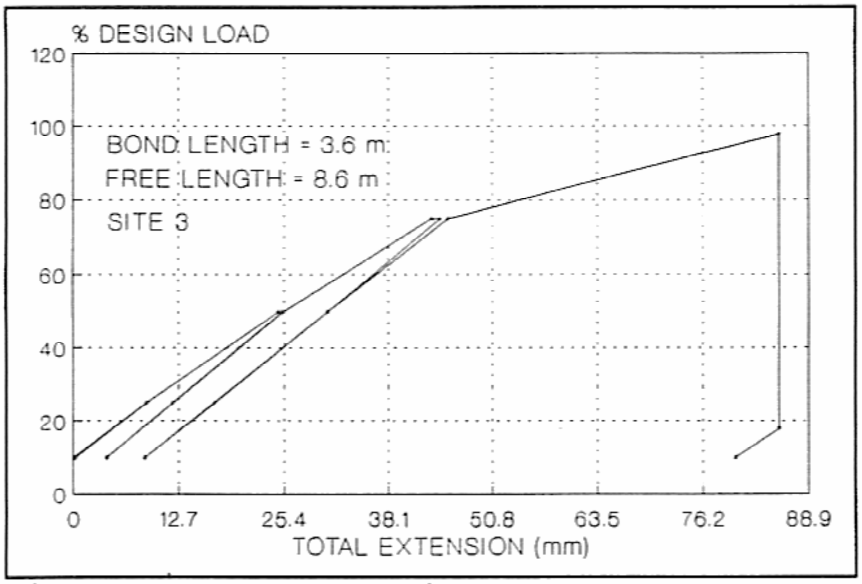

Figure 6. load-extension response for anchor 3A

(1A, 2A, and 3B) exhibited smaller creep and permanent deformations than the first. Interestingly, the largest creep deformations did not occur at the largest loads, which is counter to the normal assumption (Post Tensioning Institute, 1986). The largest amounts generally occurred at 75-100 percent of DL, and were less at higher loads (Bruce et al, 1991).

The apparent length of grout debonding can be calculated from the elastic elongation recorded during stressing, as $L_d = L_a - L_i$, where L_i is the initial (measured) free length, L_a is the apparent free length estimated from the stressing as eAE/P, e is the measured elongation, A is the area of steel, E is the Young's Modulus of the steel based on testing of the strand, and P is the applied load. The maximum apparent debonded lengths based on these calculations are summarized in Table IV.

Due to head rotation, and slight differential settlement of the jack and its associated infrastructure, many of the extensometers were pinched between strands in the hole. In addition, the upper two measurement points for anchor 3B subsequently proved to be located above the bond zone. Thus, only five extensometers are considered to have given valid readings. A summary of the response of these five extensometers is given in Table V. The response is consistent with the apparent debonding calculations.

Based on the results of the extensometer measurements and the apparent debonding calculations, it is clear that load was transferred to the bottom of the grout zone and grout rock separation occurred for both anchors at site 3. Anchor 3A totally debonded whereas anchor 3B did not. It is inferred that bond separation occurred between the grout and strand for anchor 2B, since no permanent displacement was recorded by the extensometer 0.40 m down, but apparent debonding occurred to 0.64 m below the top of the bond zone.

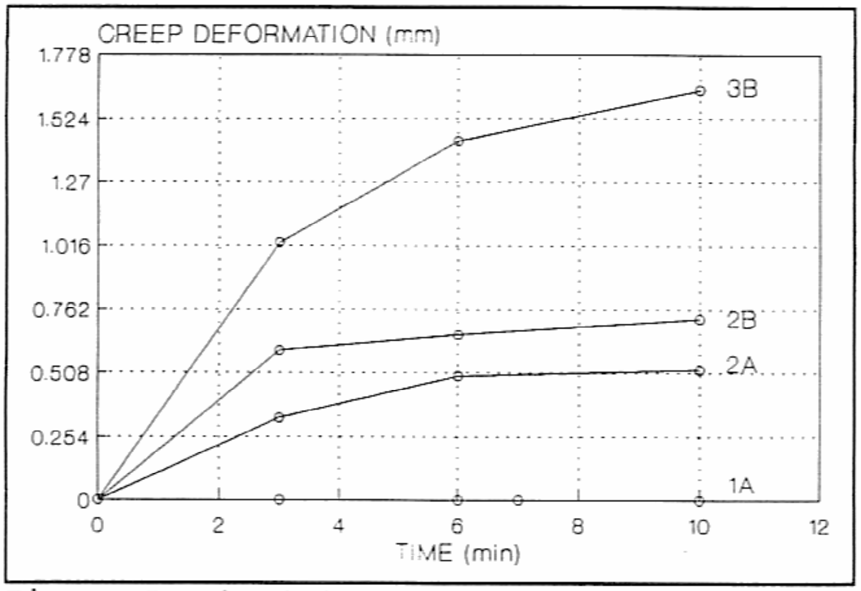

Figure 7. Short term creep response at 100% DL (Note: 1B data not valid, 3A did not reach 100% DL)

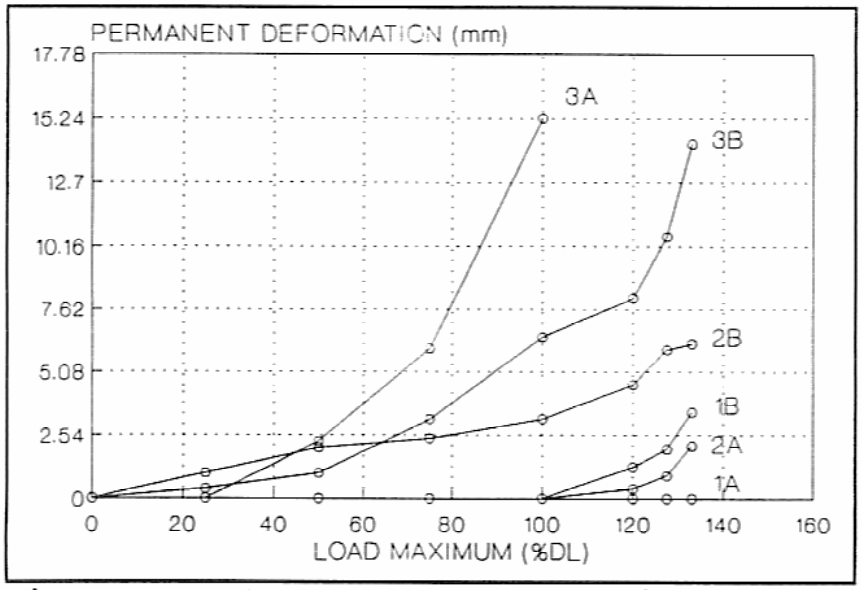

Figure 8. Net permanent deformations

7 DISCUSSION AND CONCLUSIONS

Predictably, the anchors in the best rock (site 1) performed the best, and the anchors in the worst rock (site 3) performed the poorest in terms of load-extension, creep, and permanent deformation characteristics. It is interesting to note that the second anchor tensioned at each site showed less creep, permanent deformation, and apparent debonding than the first, presumably due to some "rock mass improvement" during tensioning of its neighbor. Contrary to theory, the maximum creep did not occur at the maximum test load. This may also indicate "rock mass tightening" at lower stresses.

It has been shown that the shear stress distribution along the grout/rock interface is related to the ratio between the grout and rock modulus (Littlejohn and Bruce, 1977). When the grout is stiff relative to the rock, there tends to be a relatively uniform distribution of shear stress. When this is not the case, the shear stress tends to concentrate near the proximal end of the bond zone. This is quantitatively shown in Figure 9, based on the results of finite element studies. The anchors at sites 1 and 2 behaved similarly to the latter case, based on the small values of apparent debonding, and the limited extensometer response at these sites. The anchors at site 3 behaved as though the grout was stiff relative to the rock, as confirmed by the apparent debonding and extensometer data. Thus, all anchors appear to have behaved consistently with theory.

The bond lengths for the production anchors in the dam foundation were originally designed at 11-12 m. The test program indicated that this would provide a high degree of safety against pull out for rock similar to that at sites 1 and 2. It was felt that shortening this length for marginal commercial savings was not justified. In rock similar to site 3, it was felt that the bond length should actually be increased. Due to anticipated difficulty in making a field determination by individual hole, the bond length was arbitrarily increased by 3 m in the few holes penetrating the channel block rock.

Table V. Extensometer response

Anchor	Location	Evaluation
1A	3.11 m below top of bond zone (bottom of zone)	Zero readings - no load experienced at this depth
2B	0.40 m below top of bond zone	Very slight consistent recoverable response
3A	2.07 m below top of bond zone	Significant response - anchor movement
3B	0.91 m below top of bond zone	Significant response - bond separation
3B	3.96 m below top of bond zone (bottom of zone)	Slight response - some not recovered

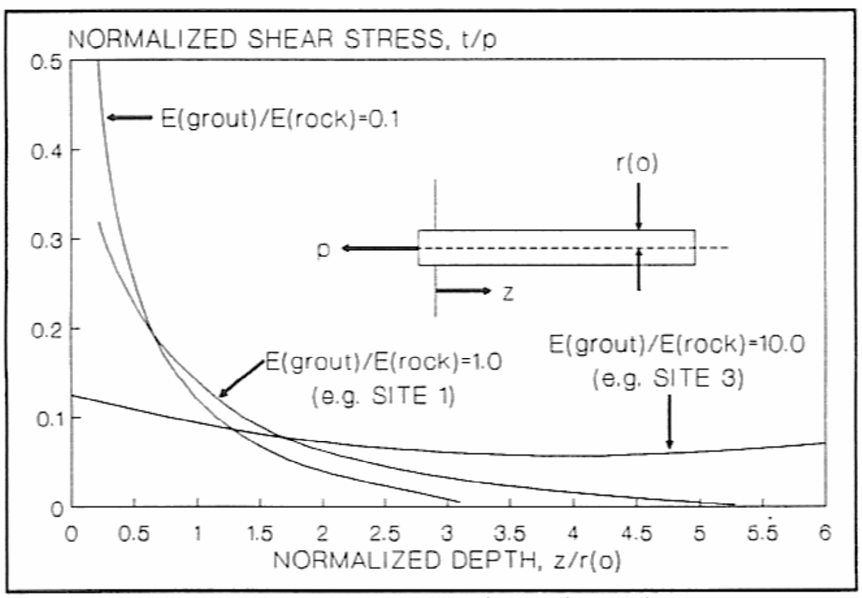

Figure 9. Shear stress distribution, after Coates and Yu (1970)

The test program described in this paper proved to be very valuable in verifying the anchor design, and understanding the anchor performance. Full scale field tests, involving all the foreseen construction steps, are strongly recommended for any large anchor job. Load-extension testing, especially an elastic analysis, should be performed, and creep behavior (both short term and long term, if possible) should be monitored. Wedge pull-in is a significant component of "permanent displacement" for anchors in stiff rock, and should also be monitored. Improved extensometer design should be considered to help answer the question as to whether debonding occurs between the grout and rock or grout and tendon.

The rock mass modulus is an important consideration in understanding anchor performance. All data from this program indicate that the dilatometer tests underestimated the rock mass modulus for this site. Dilatometer tests in a material of known modulus, and comparison to jack settlement or tests using a borehole jack should be considered in the future to help resolve this apparent discrepancy.

8 ACKNOWLEDGEMENTS

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