High Capacity Rock Anchors at Marmet Locks and Dam, West Virginia: A Case History Providing Some Fundamental Observations on the Analysis of Stressing Data

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Abstract

Details of high capacity rock anchor design, construction and stressing are provided from a major recent project at Marmet Locks and Dam, West Virginia. Particular focus is provided on the analysis of stressing and testing data insofar as the calculation of apparent debonded lengths is concerned. Guidance is provided on the typical ranges of permanent movement and short-term creep and lock off efficiencies which can be anticipated. It is intended that the data will be of value to practitioners involved in the detailed analysis and assessment of rock anchor stressing data.

Introduction

It has been estimated that possibly 10 to 15 major rock anchor projects are conducted annually in North America for dams and dam related structures such as locks and plunge pools (Bruce, 2002). In recent years, there has been a reduction in the number of technical papers written on such projects and, as a result, many practitioners may not be aware of certain issues relating to the analysis and interpretation of stressing data. This paper describes insights gained from a recent major project, at Marmet Locks and Dam, West Virginia. Reflecting the units of load and movement used in this project, this paper is written using the Imperial System. A short conversion table is provided, for convenience, at the end of the paper.

General Background

Marmet Locks and Dam are located on the Kanawha River at Marmet, West Virginia, about 70 miles above its confluence with the Ohio River. The existing facility was completed in 1934 and consists of a non-navigable dam with twin locks (56 feet by 360 feet). Because of their relatively small size, the existing Marmet locks present a significant impediment to river commerce in the form of long delays associated with breaking down large barge tows in order to pass them through the lock. To improve the efficiency of the project, a new lock chamber (110 feet by 800 feet) is being built adjacent to the existing lock chambers on the right bank.

To permit construction of the new lock chamber and sill monoliths in the dry, the landwall of the existing lock is being used as the river wall of the excavation, together with cellular cofferdams which form the upstream and downstream

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boundaries of the excavation and a sheet pile wall on the landward side. A similar scheme had been successfully implemented in the early 1990's at Point Marion Locks and Dam, Pennsylvania (Bruce et al., 1994).

The bedrock is Pennsylvanian sediments of the Pottsville Group. The Kanawha Formation is prominent in the project area, comprising mainly thick cross bedded sandstones, alternating with thinner sandy shales, thin coal beds, the Kanawha Black Flint, and a few thin marine sediments. Due to the regional geological structure, the rock dips gently to both the northwest and southeast, and there is no extensive faulting apparent.

The sandstone member is the uppermost rock unit at the site and is about 23 to 43 feet thick. It contains thin carbonaceous stringers and beds. The sandstone has generally low permeability and is described as moderately hard (mean Unconfined Compressive Strength (about 8500 psi). It overlies 19 to 33 feet of shale which varies from soft to moderately hard (up to about 6700 psi). Laboratory grout-rock bond tests led the Owner to recommend working bond stresses of 100 psi and 55 psi for anchors founded in the sandstone and shale, respectively, with a nominal factor of safety of 2 against the ultimate bond value. The modulus of elasticity of these main lithological units is 1.05 and 0.63 x 10^6 psi, respectively.

There are known to be relatively weak continuous surfaces in the sandstone member, including micaceous bedding planes and thin zones of carbonaceous stringers.

Scope of the Anchor Works

Prestressed rock anchors were required to provide a satisfactory factor of safety against sliding for the monoliths of the existing lock wall and the new coffercells during the excavation. Monoliths 5 to 28 of the existing lock wall were each stabilized with 3 to 5 rows of anchors. Anchor inclinations varied from vertical for upper rows, to 45º for the lowermost anchors through the thrust block shown in Figure 1. Each monolith had from 9 to 21 anchors, with total drilling lengths for each 6-inch hole varying from 52 to 101 feet. The total of 377 anchors included 95 thrust block anchors. Bond lengths varied from 20 to 30 feet (for 9 strand tendons — the majority); 35 feet for 12 strand tendons and 40 to 45 feet for 15 strand tendons. For the coffer cells, a total of 87 thrust block anchors were installed: 9 strands per anchor, 7 or 8 anchors per cell, mainly 34-foot bond lengths, and total drilling lengths of 47 to 102 feet. There were in addition 73 anchors, each with 12 strands installed at 16º off vertical to stabilize the cells themselves. The tendon stress at Design Working Load was 60%, and at Test Load 80%, Guaranteed Ultimate Tensile Strength (GUTS). The multiple rows of ground anchors retaining the landside sheet pile retaining wall are not discussed in this paper.

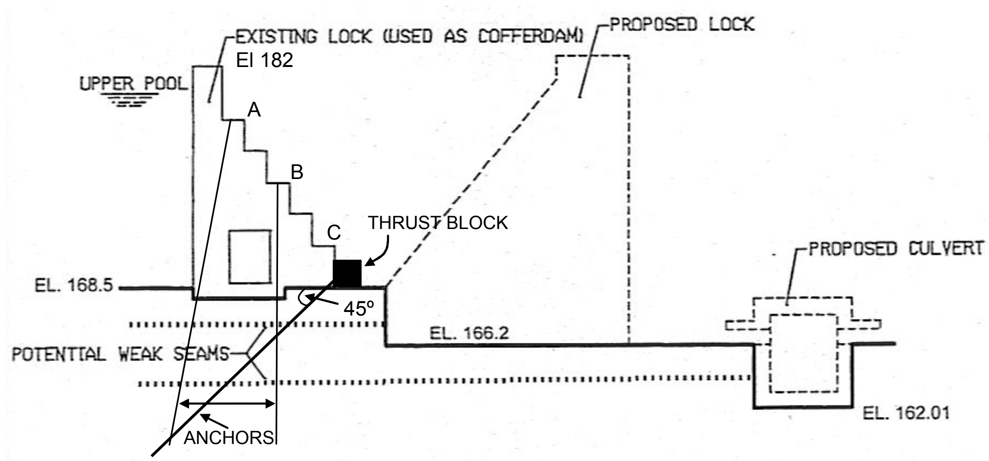

Figure 1. Schematic cross section showing "Daylighted" Weak Rock Seams and the typical positions of anchors, Marmet Locks and Dam, WV.

Construction practices were in accordance with the U.S. Army Corps of Engineers' own anchor project specifications. Each hole was water pressure tested (to a 10 gallons in 10 minutes at 5 psi criterion (four times more lenient than the PTI criterion) and pregrouted and redrilled if necessary (in about 55% of cases). Tendon grouting was conducted in two stages, the second (along the free length) after stressing. Due to the high spacial intensity of anchoring, drilling accuracy was crucial to maintain. Holes were drilled with down-the-hole hammers fitted with hammer and rod centralizers, and acceptable tolerance was verified with a multi-shot borehole survey tool.

Given the presence of argillaceous rocks (potentially creep susceptible) in the foundation, and the consequences of failure of the excavation support system during construction, the Corps specified an unusually large number of Performance Tests (about 14% of total anchors), and Extended Creep Tests (about 7% of the total). This intensity compares with the relevant PTI (1996, and 2004) Recommendation (Section 8.3.2): "the number of Performance Tests may be increased, especially when the anchors are being used for permanent applications, when creep susceptibility is suspected, or when varying ground conditions are encountered, but normally will not exceed 5% of the total number of anchors." Regarding Extended Creep Testing, PTI (1996, and 2004) (Section 8.3.4) states: "Extended creep tests normally are not performed on rock anchors since they do not exhibit time dependent movements. However, anchors installed in very decomposed or argillaceous rocks may exhibit significant creep behavior."

Each monolith or cell was to be subjected to each type of test in a well defined order. In addition, hollow center 300-ton vibrating wire load cells were incorporated on anchors throughout the cofferdam. Two load cells were placed under Proof Tested anchors in each of the 24 existing river wall monoliths, one on a vertical anchor, and one on an inclined anchor. A load cell was also placed on an inclined Proof Tested anchor in each of the coffercell thrust blocks. These load cells, as well as inclinometers and tilt meters, were closely monitored to demonstrate the safe performance of the old monolith/coffercell retention system during subsequent excavation for the new monoliths.

At the time of writing, the anchor project was nearing completion and a total of 24 anchors had been subjected first to a Performance Test, and second to an Extended Creep Test. The data from these tests form the basis of this paper.

Analysis of Data

The stressing data confirmed that every anchor was found to be contractually satisfactory in every regard, in that each met the specified elastic extension, creep and lift off criteria. However, the detailed analysis of the stressing data has allowed several interesting additional observations to be made, which may be of interest to practitioners.

Elastic Extension. The specifications require that the elastic extension at Test Load (80% GUTS) be proportional to no less than 80% of the theoretical free length (plus jack length) and to no more than 100% of the theoretical free length (plus jack length) plus 50% of the design bond length. As shown in Table 1, 23 of 24 Performance Test results indicated calculated apparent debonding lengths of less than or equal to 10 feet (average 2.7 feet). This compares to values of half the bond lengths varying from 8.5 to 22.5 feet. The exception was anchor M13-D1 (17.1 feet of apparent debonding, but still contractually acceptable since half the bond length is 22.5 feet.). Close examination of this particular anchor's load/movement data indicated absolutely minimal debonding during stressing (the plot of the anchor was essentially a straight line) and so it would seem that its bond zone grout level was actually lower than foreseen from the start of the load test. This is an important clue when analyzing the behavior — especially atypical behavior — of two stage grouted anchors. Indeed, the elastic extension curve of all anchors was extremely linear, indicative of minimal debonding during cyclic testing to Test Load.

In general, the low average amount of debonding is characteristic of well bonded anchors in moderately strong rock wherein high stress concentrations at the top of the bond zone are created and the more distal parts of the bond zone see little or no load (Littlejohn and Bruce, 1977).

As illustrated in PTI (2004) by Figure 2, the calculation of the apparent debonded length relies on the simplistic assumptions that (i) load transfer in the bond length is uniform, and (ii) that the distance (calculated by elastic theory) into the bond zone that the tendon acts as if completely debonded is in fact an accurate indicator of the in situ reality of complex load transfer mechanisms. This universal convention has, however, been found to be an excellent and reliable diagnostic of anchor acceptability and has general approbation.

The elastic performance of each anchor was closely replicated during the subsequent (cyclic) Extended Creep Testing (average debonded length estimated for the 23 anchors was 2.4 feet). This proves that further progressive debonding was not induced by the rigors of the Extended Creep Testing process.

Permanent Movement. There is no acceptance criterion for permanent movement in the PTI recommendations or in the Corps' specification. The maximum permanent movement can only be measured when the load is reduced back to Alignment Load following the total extension reading made at Test Load. Therefore, it is typically measured only during Performance or Extended Creep Tests since many

Table 1. Details of anchors subjected to Performance and Extended Creep Testing, Marmet Locks and Dam.

Thrust or Monolith	Anchor	Date of Stressing (all in 2004)	Date of Report (and number)	Anchor Geometry				Performance Test Analysis				Extended Creep Test Analysis				Lift Off Load (% of Lock Off)
				Bond Length (feet)	Free Length (feet)	Jack Length (feet)	Inclination (degrees)	Calculated Free Length (feet)	So, Apparent Debonding at Test Load (feet)	Permanent Movement at Test Load (inches)	Creep in 1 to 10 Minutes (inches)	Calculated Free Length (feet)	So, Apparent Debonding (Est.) at Test Load (feet)	Permanent Movement (Est.) at Test Load (inches)	Creep Analysis at each Log Cycle at each Load (inches)	
Th	M25-D1	09/08	9/17 (#28)	35	41.4	5.21	45	48.6	2.0	0.314	0.002	48.8	2.2	0.065	All except one (0.006) ≤ 0.005	101
Th	M21-E1	08/31	9/13 (#27)	30	45	5.21	45	49.9	zero	0.357	0.004	50.8	0.6	0.044	All values ≤ 0.005	101
Th	M23-E1	09/01	9/6 (#26)	30	45	5.21	45	57.0	6.8	0.410	0.005	57.4	7.2	0.065	All values ≤ 0.006	99
Th	M19-E1	08/24	8/31 (#25)	30	45	5.21	45	53.2	3.0	0.423	0.004	52.8	2.6	0.060	Most values ≤ 0.004, three up to 0.006	101
Th	M17-E1	08/23	08/31 (#24)	25	43	5.21	45	50.7	0.5	0.402	0.002	50.7	0.5	0.003	All values ≤ 0.005	101
Th	M15-C1	08/06	08/06 (#23)	30	42	5.21	45	48.6	1.6	0.451	0.001	47.9	0.7	0.100	All values ≤ 0.004	102
Th	4-5/B6	06/29	08/06 (#22)	34	65	6.21	45	71.8	0.6	0.669	0.002	72.0	0.8	0.400	All values ≤ 0.004	99
Th	M5-1/B3	08/02	08/04 (#21)	34	65	6.21	45	72.5	1.3	0.528	0.007	72.0	0.8	0.155	Mainly ≤ 0.006 two 0.008	100
M	M23-B2	07/28	08/02 (#19)	25	42	5.21	Vert	56.1	8.9	0.242	0.003	55.1	7.9	0.150	All values ≤ 0.003	99
Th	M13-D1	07/27	08/02 (#18)	45	30	5.21	45	52.3	17.1	0.324	0.002	55.0	19.8	0.080	All values ≤ 0.003 (several redrills)	101
Th	2-3/B6	06/30	07/26 (#17)	34	65	6.21	45	84.8	13.6	0.619	0.002	Not Conducted.				101
Th	M11-D1	07/20	07/26 (#16)	25	42.9	5.21	45	48.6	0.5	0.392	0.002	48.1	zero	0.100	All values ≤ 0.003	100
Th	M9-D1	07/20	07/26 (#15)	25	43	5.21	45	49.6	1.4	0.451	0.001	49.9	1.7	0.040	All values ≤ 0.003	101
Th	M7/D1	07/16	07/19 (#14)	20	47.9	5.21	45	54.0	0.9	0.442	0.002	54.7	1.6	0.100	All values ≤ 0.002	101
Th	M5-D3	07/14	07/16 (#13)	25	44.3	5.21	45	52.3	2.8	0.426	0.001	52.6	3.1	0.109	All values ≤ 0.004	103
M	M23-C1	05/26	07/16 (#12)	25	42	5.21	20	57.2	10.0	0.506	0.009	56.2	9.0	0.215	Max value 0.015, typically ≤ 0.010	99
M	M25-C4	07/12	07/15 (#11)	25	51	5.21	35	56.7	0.5	0.415	0.002	56.2	zero	0.180	Max value 0.004, typically 0.001 to 0.003	100

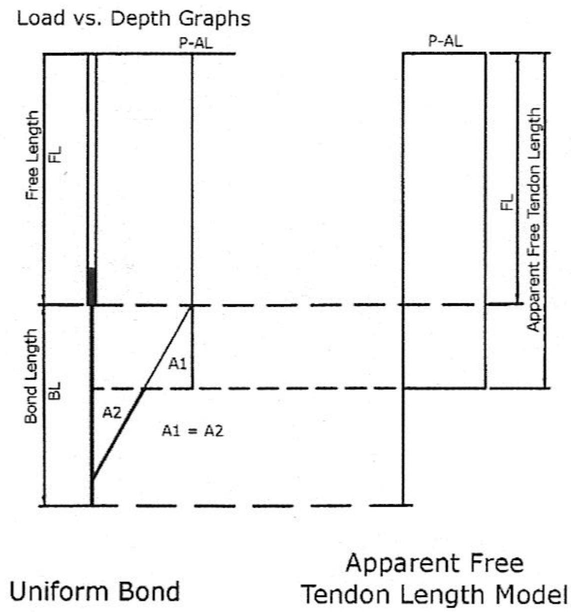

Figure 2. Apparent Free Tendon Length (PTI, 2004 Figure 8.2)

practitioners do not utilize the option given in the PTI recommendations to do so during Proof Testing, due to the extra time and effort required.

It is therefore useful to note in Tables 1 and 2 the magnitude of the permanent movement measured during the Performance Testing: a range of 0.140 to 0.697 inches (average 0.444 inches). In hard rock conditions such as here it is inconceivable that such movement is caused by the whole bond zone pulling through the rock mass. Equally, there was no possibility of friction gripping the strands of the free length (since there was no grout in place there at the time of testing). Therefore it must simply be concluded that the permanent movement was mainly caused by "bedding in" of the top anchorage and its components (i.e., non-recoverable movements of the hardware and structure immediately associated with the top anchorage). In addition, it is logical to assume that the "pent up load" or negative friction phenomenon may have contributed to the larger permanent movements recorded during the first, Performance Test, stressing phase. This phenomenon is described by Richards elsewhere in this conference. This view is supported by a comparative examination of the permanent movements recorded during the subsequent Extended Creep Testing. In every case in Table 1, the subsequent permanent movements were much smaller — at each load and overall — usually by several times. Table 2 provides further information on three anchors as examples. The average permanent movement at Test Load in the later Extended Creep Tests was 0.127 inches — about 30% of the value recorded during the earlier Performance Test.

Creep. During a Performance Test, a 10-minute creep test is run at Test Load only. Not surprisingly, since the anchors were transferring their load in a predominantly strong sandstone horizon, creep was minimal in these tests: zero to 0.013 inches in the period 1 to 10 minutes (average 0.004 inches) into the load hold. PTI allows 0.040 inches as the acceptance criterion in this period.

Table 2. Comparison of Permanent Movements Measured after Test Load in Each Type of Test for Three Representative Anchors, Marmet Locks and Dam, West Virginia.

LOAD STEP (% DESIGN WORKING LOAD)	PERMANENT MOVEMENT (INCHES)					
	ANCHOR M15-C1		ANCHOR M5-1/B3		ANCHOR 3-4/B4	
	PERFORMANCE TEST	EXTENDED CREEP TEST	PERFORMANCE TEST	EXTENDED CREEP TEST	PERFORMANCE TEST	EXTENDED CREEP TEST
25	0.080	0.020	0.141	0.005	0.100	0.024
50	0.135	0.028	0.214	0.013	0.272	0.053
75	0.170	0.041	0.318	0.037	0.423	0.081
100	0.225	0.055	0.367	0.045	0.458	0.092
120	0.301	0.060	0.424	0.054	0.640	0.099
133	0.451	0.100	0.528	0.100	0.697	0.110

Notes:

(1) In each case, permanent movement after Test Load (133%) in the Extended Creep Test had to be extrapolated.

(2) For each anchor, the shape and linearity of the elastic extension curves obtained during Performance and Creep Testing were practically identical.

The same small creep values were noted in all log cycles during the Extended Creep Testing (to 300 minutes). All anchors also showed a diminishing rate of creep with time. The Corps' criterion is 0.080 inches per log cycle during the final log cycle of the test, regardless of load. The PTI criterion is also 0.080 inches in any log cycle.

Lift Off Test. The load transferred at Lock Off is measured by a Lift Off Test immediately after Lock Off. A 5% tolerance is permitted. The range as shown in Table 1 was -2 to +3%, indicating that, by using good practice, accurate results can be routinely achieved.

Final Remarks

In this project, an exceptionally intense level of anchor testing has been conducted. Close study of the data has permitted insights to be gained into aspects of anchor performance that are not commonly revealed in routine practice. The data confirm typical ranges for (apparent) permanent movements, but there is strong evidence that the great proportion of such movements is not related to movement of the bond zone through the rock mass. Apparent debonding lengths (i.e., reflecting phenomena at the tendon-grout interface) are confirmed as being minimal in medium/hard rocks where high quality anchor construction has been assured. Likewise, data are provided on short-term creep performance, and Lift Off Test variations which can be anticipated under similar geological and construction circumstances.

It is the hope of the authors that data of a similar scope can be presented by rock anchor specialists in future publications. Although the PTI Recommendations, and those of the U.S. Army Corps of Engineers are in themselves excellent guidelines and specifications, published data will lend support and amplification to these documents. Such support will help resolve apparent inconsistencies or confusions on a project-specific basis.

Acknowledgements

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CONVERSION TABLE
1 mile is equivalent to 1.609 km
1 foot is equivalent to 305 mm
1 inch is equivalent to 25.4 mm
1 kip is equivalent to 4.448 kn
1 psi is equivalent to 6.895 kPa