ASSOCIATION OF STATE DAM SAFETY OFFICIALS

SIXTH ANNUAL CONFERENCE

OCTOBER 1-5, 1989 ALBUQUERQUE, NM

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The Stabilization of Shepaug Dam, CT Using High Capacity Prestressed Rock Anchors

by

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Abstract. The use of prestressed rock anchors to stabilize dams in North America has become a routine remedial technique over the last twenty years or so. However, there is a trend towards designs incorporating longer anchors of higher capacities, which in turn puts additional pressures on the specialist contractor who often has to carry out the construction under the difficult and restricted access conditions found on most dam sites. At the same time, closer attention is being paid to the analysis of the stressing and testing information, towards improving understanding of anchor performance. Current practice in dam anchor technology is illustrated with reference to the work, recently conducted at Shepaug Dam, CT, and especially notable for the extraordinary size and capacity of the tendons employed and the novel method of transferring load to the structure.

1. BACKGROUND

Shepaug Dam is a 34-year-old concrete buttress/gravity dam on the Housatonic River near Sandy Hook, CT. The purpose of the 130' high, 1412' long structure is flood control and power generation. The stability of the dam was reassessed, given that the PMF was found equivalent to 20' above the present crest. High capacity prestressed rock anchors were selected as the preferred method of safeguarding against overturning. Calculations showed that given the particular anchor capacities and spacings, each anchor per 45' long block of the structure is equivalent to the addition of an additional 5' height of concrete, at the existing crest width (a minimum of 10').

2. DESIGN

2.1 Overall Stability

The overall stability requirements were calculated with reference to the standard inverted cone method, and using only the weight of rock engaged in the potential failure volume. The bond length, 27-49' long (Table 1), commenced 5' below the dam/rock interface. The free length was dictated by the thickness of the concrete drilled (ie., 29'-145'). Individual anchor design working loads of 983 to 1860 kips were calculated. The design featured 83 anchors installed from the crest and ranging from vertical to 2.3^0 upstream, and a further 14 anchors (inclined 49^0 down) from an elevation 80' below on the downstream face of the spillway.

Block	Number of Anchors	Depth to Top of Bond Length (ft)	Bond Length (ft)	Number of Strands	Design Load (kips)	Secondary Bond Length (ft)
1	2	24-28	48	51	1790	Top Anchorages
2	2	57-65	48	51	1790	39.5 - 40.0
3	4	84-112	48	51	1790	36.6 - 39.5
4	5	116-131	49	52	1825	36.0 - 38.0
5	5	126-130	49	53	1860	36.5 - 38.5
6	3 1*	128-129 65	49 40	52 42	1825 1474	36.5 - 39.0 Top Anchorages
7	1 2*	129 60-62	40 40	50 42	1755 1730	42.5 Top Anchorages
8	1 2*	129 63	47 40	50 42	1755 1474	38.3 Top Anchorages
9	1 2*	126 63-65	47 40	50 42	1755 1474	41.0 Top Anchorages
10	1 2*	136 70	47 49	50 52	1755 1825	40.5 Top Anchorages
11	1 2*	133.5 70	47 49	50 52	1755 1825	37.0 Top Anchorages
12	1 2*	132 68-72	47 44	50 47	1755 1650	36.3 Top Anchorages
13	3 1*	134-135 68	49 40	52 42	1825 1474	38.8 - 40.5 Top Anchorages
15	6	138-144	49	52	1825	39.5 - 43.0
16	6	143-144	49	52	1825	41.5 - 45.0
17	6	139-145	49	52	1825	41.5 - 45.0
18	6	129-139	49	52	1825	43.0 - 45.5
19	5	116-127	49	52	1825	41.5 - 44.0
20	4	103-116	44	47	1650	44.5 - 52.0
21	4	85-105	48	51	1790	44.0 - 46.5
22	3	77-86	44	47	1650	45.0 - 47.0
23	2	72	48	51	1790	44.3 - 46.0
24	2	69-71	48	51	1790	42.8 - 46.0
25	2	59-63	48	51	1790	41.3 - 44.0
26	2	40-46	27	28	938	Top Anchorages
27	2	31-39	27	28	938	Top Anchorages
28	2	29-31	27	28	938	Top Anchorages
TOTALS	83 14*	29-145 60-72	27-49 40-49	28-53 42-52	938-1825 1474-1825	75 Secondary bond 8 Top Anchorages 14 Top Anchorages

Table 1. Summary of anchor lengths and capacities

* Refers to inclined spillway anchors

The crest anchors were located at centers of 6-14', while the spillway anchors ranged from 21' 6" to 23' 6" apart.

2.2 Rock-Grout Bond

The bedrock was a micaceous granitic gneiss with a material U.C.S. of about 10,000 psi. Average rock-grout bond values at design working load ranged up to 100 psi. For the 10" diameter holes used, the bond lengths thus varied from 27 to 49'.

2.3 Grout-Tendon Bond

Typically it is assumed (Littlejohn and Bruce, 1977) that a bond length selected with respect to rock-grout bond considerations will be adequate to provide safe grout-steel embedment, and this was the case here. For example, at the maximum test load of 2477 kips, the average working grout/steel bond over 53 strands, with 49' embedment, was only 42 psi. This compares with the figure of about 300 psi allowed as the ultimate strand/grout bond in typical international codes (eg. BS8081).

2.4 Grout

Type 2 was specified, with a w/c ratio (by weight) of W=0.45. This was targetted to provide a minimum crushing strength of over 3500 psi at about 7 days. As shown in Figure 1, this grout would also be adequately fluid, without the need for any additives (typically discouraged in anchor practice anyway - Bruce, 1989), and would have minimal bleed capacity.

2.5 Tendon

Each anchor tendon consisted of groups of 0.6" diameter low relaxation 7-wire steel strand of 58.6 kips GUTS. At design working load each strand would be operating at 60% GUTS, with temporary test load stresses of 80% GUTS being reached before lock-off at 70% Tendons varied from 68' to 206' long including 6-1/2' of "tail" and comprised from 28 to 53 strands. Maximum tendon weight was therefore about 5 tons.

In the bond length the strands were noded and centralized at 10' centers, to promote grout penetration around the strands and ease installation. However, it was in the free length that a major innovation was made, principally in order to effect a major cost saving to the owner. In all 14 spillway anchors, and in 8 crest anchors where the depth to top of bond zone was less than 50', the conventional lock-off system was used: the prestress is maintained in the free length after testing, by using a top anchorage assembly, with wedges gripping the strands, and bearing on the concrete of the dam crest (Figure 2). The free length of each strand is protected by an individual full length greased sheath, and surrounded after lock-off by the secondary grout.

For all the other anchors, however, the load was maintained by bond between the bared upper part of the free length (about 36-55' long) and the secondary grout, as sketched in Figure 3 and described below. This scheme saved the considerable cost of coring large diameter recesses in the dam crest to accommodate the conventional top anchorage hardwear. The 75 anchors of this type were referenced to as Secondary Bond Anchors.

3. CONSTRUCTION

The general anchor installation procedure was as follows, with S referring to Secondary Bond Anchors, and P referring to the 22 anchors with Permanent Top Anchorages of the conventional type:

1. Drill 10-inch diameter hole (S and P)
2. Cut recess (P only)
3. Clean hole with potable water (S and P)
4. Water pressure test hole (S and P)
5. Angle survey of hole (S and P)
6. Insert tendon (S and P)
7. Flush hole with potable water (S and P)
8. Primary Grout Anchor Bond Zone (S and P) to minimum of 10' above top bond zone
9. Stress - Performance and/or Proof test as required (S and P)
10. Secondary Grout (S and P) to 5' below top of hole
11. Load transfer (S)
12. Remove anchor head and bearing plate (S)
13. Cut strand tails (S and P) (2" below concrete surface - S; 2" above anchor head - P)
14. Fill recess with non-shrink concrete/grout (P)
15. Fill remaining portion of borehole (S) with non-shrink grout.

Particular points of note on the construction were as follows:

3.1 Drilling

All holes from the crest were drilled 10" diameter with down the hole hammers, mounted on diesel hydraulic drill rigs while the drilling for the spillway anchors was similarly conducted from a special platform fixed to the face. This rotary percussive drilling method has proved itself to be the quickest, cheapest, and straightest way to drill holes from 4-12" in diameter to considerable depths through unreinforced concrete and competent rock formations (Bruce, 1988). Although delivered air pressures of over 300 psi are used, there are no records of the system having caused structural damage to dams, even when holes are drilled as close as 2' to free concrete edges. In this case, holes approached within 5' of a free edge (ie. in the inspection gallery, or the upstream face, Figure 4).

Spiralled centralizers placed behind the hammers, and 20' long "barrelled" rods (to 9-5/8" dia.) were both used to improve hole straightness and linearity. Each hole was measured at three depths (10' from top, middle, and bottom) by an Eastman Whipstock R instrument. Meticulous records were maintained, indicating a maximum deviation (downstream) of less than 1^0 in the crest anchors. Each hole was overdrilled by about 3' to form a 'sump' for debris not evacuated from the hole during flushing.

3.2 Water Testing

Water pressure testing of the bond length of each hole was conducted according to the criterion of 0.1 gallon/inch diameter/foot of hole, at 5 psi excess pressure over 10 minutes, No holes were recorded as having failed this test, with the great majority having flows less than one half of the target. No separate stage of pre (or "consolidation") grouting, followed by redrilling and retesting was therefore necessary.

3.3 Tendon Placing

Especially when there is no outer, group sheath on the fre length, it is advisable to control the installation of tendons with a mechanical device. In this particular case, where the tendons were exceptionally long and heavy, with restricted space at their point of entry, such a device was essential. Under similar conditions, other contractors have used cranes or helicopters, but in this case a less exotic uncoiler was used. After installation, a steel frame was placed at the top of the hole, from which the tendon was suspended off the bottom of the hole during the subsequent primary grouting operations.

3.4 Grouting

Grouts were mixed in a high speed colloidal mixer and pumped via a 1" diameter tremie tube. A carefully measured volume of grout was placed in each hole, to bring the primary grout level to at least 10 feet above the tendon bond length. This grout was allowed to set for at least five days prior to tendon stressing. After stressing, the secondary grout (also w/c = 0.45) was placed above the top of primary grout through a 3/4" bore steel pipe and allowed to set for at least five days. Thereafter the jack load was released and the load maintained in the tendon either by standard permanent top anchorage (22 anchors) or by bond between the secondary grout and the bare steel of the upper part of the tendon (75 anchors). Excess strand lengths were then cropped off, and in the case of the secondary bond anchors, the remaining 5' of hole was backfilled with non-shrink concrete.

4. STRESSING AND TESTING

The extraordinarily high anchor capacities (maximum test load 2430 kips) necessitated very large scale and sophisticated stressing equipment. Thus, for example, the 3000 kip capacity hollow ram jacks used each weighed over 2 tons and had to be transported and set up by crane or forklift.

A total of three Performance tests were conducted on normal production anchors. As described in PTI (1986), such cyclic tests allow the performance of the anchor to be examined with respect to its elastic behavior (reflecting on the amount of debonding in the fixed length), and permanent displacement (Figure 5). Overall, it was found that at the maximum test load (ie, up to 80% GUTS = 1.33 WL) apparent debonding to about 6-8' down the fixed length had occurred, whereas the total permanent displacement (from numerous sources in the anchor and in the structure) was less than 0.5" (out of total extensions of up to 14").

Creep performance was also excellent. Lift off checks after 7 days indicated minimal strand relaxation losses after locking off.

An important practical comment on the stressing merits attention. It is not uncommon for the strands in long tendons to become disordered in the hole in the free length during installation. So, when a multistrand jack is applied to simultaneously stress the strands, the load might not be uniformly applied to each strand. There is, therefore, the problem of unusual extensions and, if the overall tendon stress level is high, then the scale of the maldistribution may be such that certain strands will rupture, while adjacent strands have loads substantially below the notional average.

On this project, given the size of the tendons and the fact that they were being stressed to an overall average of 80% GUTS, it was particularly necessary to make individual strand loads uniform from the outset. This was achieved by applying the alignment load (AL = 5% testload) by monostrand jack. This operation revealed that some strands needed as much as 6" of ram extension to get them straight and holding AL. Although time consuming, this operation was proved of worth as the project was concluded without rupturing a single strand.

As a final point, it should be noted that the lift off checks were conducted by lifting the whole anchorage head - as a unit, and not by regrabbing the strands and trying to free them from their wedges in the head. This latter method - though common in practice - may lead to damage of both strand and wedge, leading therefore to inefficient locking of the prestress when the test is concluded.

5. FINAL COMMENTS

The use of prestressed rock anchors to stabilize existing concrete dams has reached the status in this country of a reliable and reputable construction technique. This increasing sense familiarity brings with it ever more ambitious designs and increasingly sophisticated construction techniques. Typically, the anchor community has responded with vigor to these challenges, but it is still valid to recall the remarks of Rutledge, of the FIP Commission on Anchors (1982):

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

ACKNOWLEDGEMENTS

On site, the work was supervised by Mr. Paul Gregory on behalf of the owner, North East Utilities Service Company, and Mr. Rodney Wang of Harza Engineering Co. (Chicago office). Their cooperation is gratefully acknowledged.

REFERENCES

British Standards Institution (1989), "Recommendations for Ground Anchorages." Prepared for issue in 1989 by Technical Committee CSB/22, BSI, London, as BS 8081.

Bruce, D.A. (1988), "Practical Aspects of Rock Anchorages for Dams." Association of State Dam Safety Officials Fifth Annual Conference. Manchester, NH, September 25-28, 26pp.

Bruce, D.A. (1989), "An Overview of Current U.S. Practice in Dam Stabilization Using Prestressed Rock Anchors." 20th Ohio River Valley Soils Seminar, Louisville, KY, October 27, 15 pp.

Littlejohn, G.S. and Bruce, D.A. (1977), "Rock Anchors: State of the Art", Foundation Publications, Ltd., Brentwood, Essex, U.K. 50pp.

PTI (1986) "Recommendations for Prestressed Rock and Soil Anchors." Post Tensioning Manual, Fourth Edition, pp. 236-276. Published by PTI, 301 W. Osborn, Suite 3500, Phoenix, AZ 85013.

Rutledge J.C. (1982) Chief Geotechnical Control Office, Hong King, and Member of Ground Anchor Working Committee, Federation Internationale de la Precontrainte.

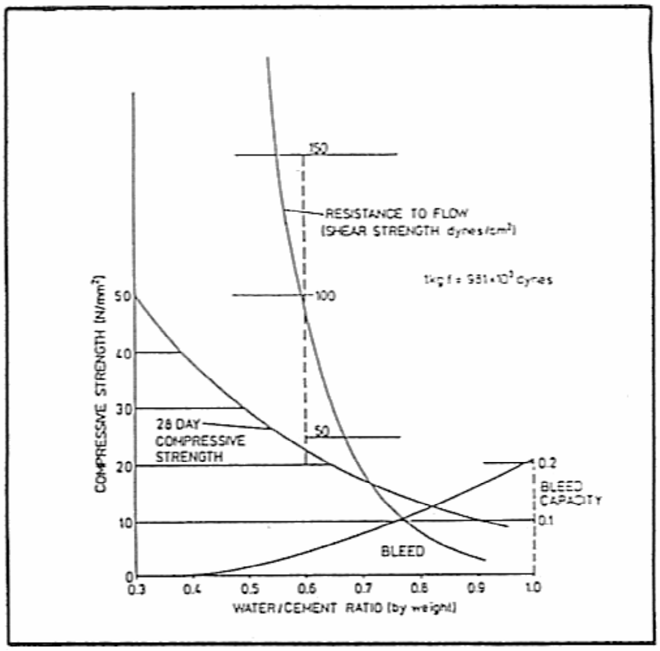

Figure 1

Graph of strength, fluidity and bleed capacity for neat cement grouts (Littlejohn and Bruce), 1977

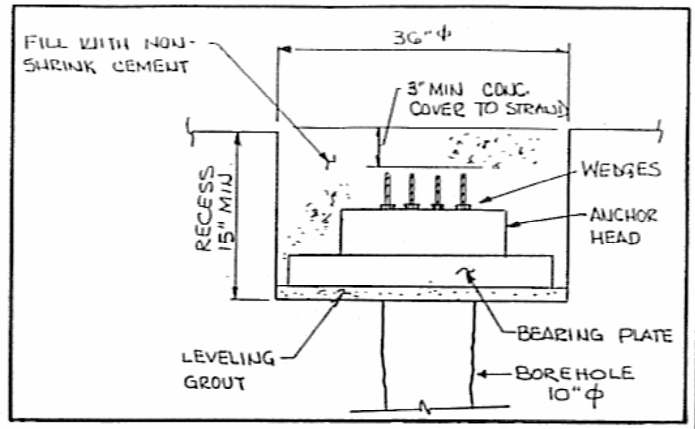

Figure 2

Sketch of conventional load retention arrangement with permanent top anchorage and bearing plate on concrete

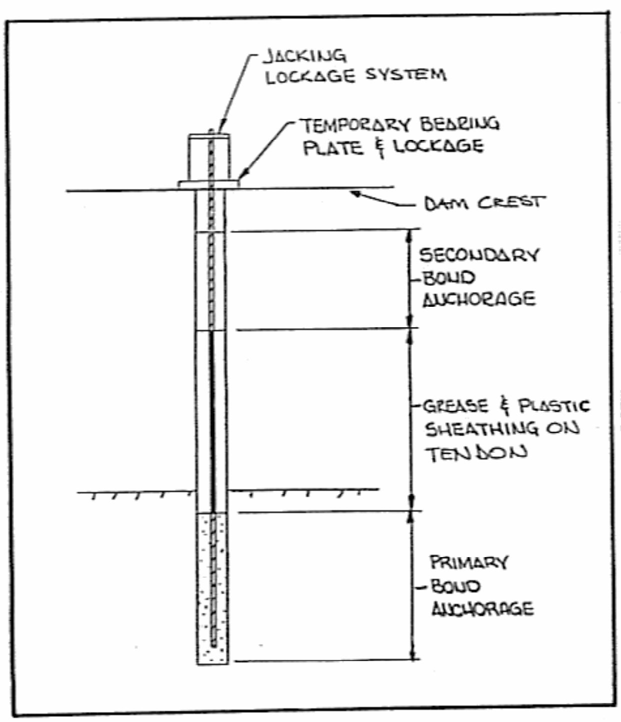

Figure 3

Alternate method of load retention by transfer of load by bond from upper (bare) of tendon via Secondary grout to concrete of borehole wall.

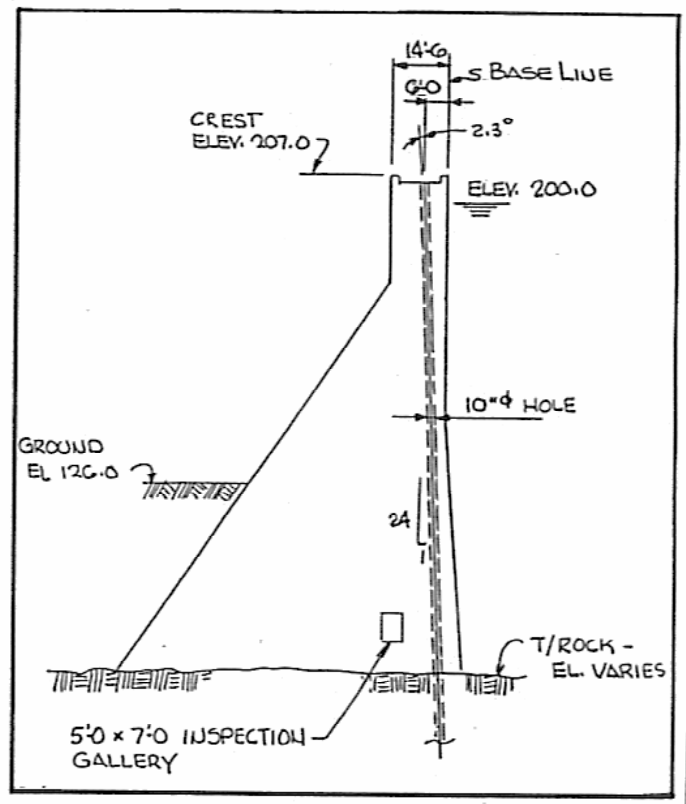

Figure 4

Section through Block N20, showing proximity of anchor holes to upstream face, and inspection gallery.

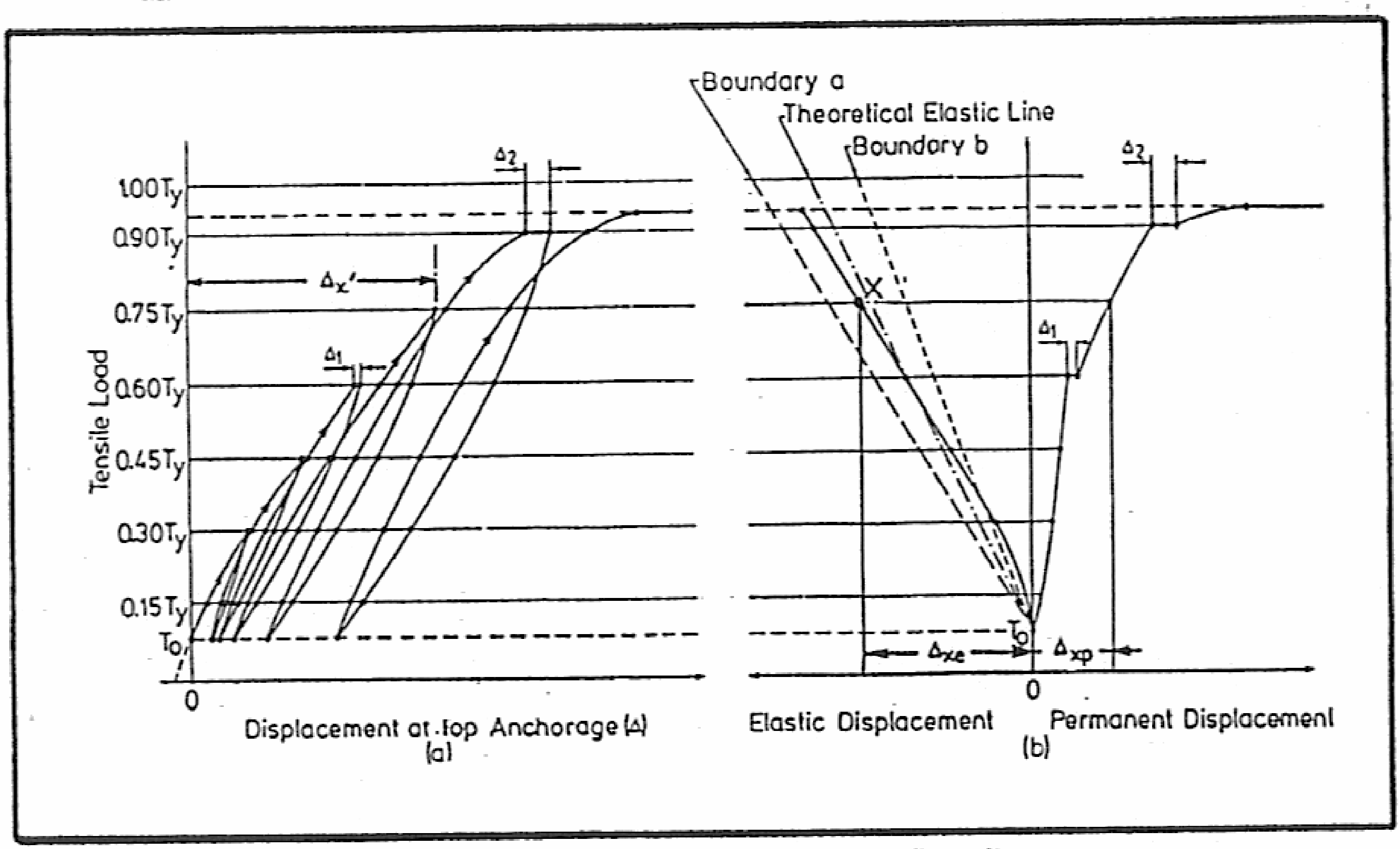

Figure 5

Resolution of total tendon extensions (a) with elastic and permanent (residual) components, (b) for analysis of performance tests.