GROUND ENGINEERING

Long-term performance of high capacity rock anchors at Devonport

by G. S. LITTLEJOHN, BSc(Eng), PhD, CEng, MICE, MIStructE, FGS
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Introduction

WITH THE STEADY development of ground anchor technology over the years, there has been an increasing awareness of the need to obtain and provide information concerning long-term performance of post tensioned anchors.

The present authors (1977) have written of the benefits to be gained from such data; the engineer being able to "feed back" performance observations into future designs and thereby optimise such parameters as overload allowances and safety factors; the prospective client being accurately and confidently informed of how anchors installed at his expense will perform after installation. Furthermore, such data collection permits all parties to judge at the earliest possible stage whether anchors being monitored are, in fact, acting satisfactorily. On a more general front, this form of monitoring may permit correlation of anchor load and structural movement, which will lead to a better understanding of anchor/ground/structure interaction.

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At the planning and instrumentation stage of this study, both authors were members of the Geotechnics Research Group, Department of Engineering, University of Aberdeen

Site description

The first stage in the construction of the Submarine Refit Complex at HM Dockyard, Devonport, featured one of the largest and most interesting anchoring contracts undertaken in the UK.

Twin dry docks were constructed in an existing basin approximately 140m square, originally formed as part of the major dockyard expansion between 1896 and 1907, and surrounded on three sides by mass concrete retaining walls founded directly on bedrock. The minimum depth of the walls is 18m but around the north-west corner a depth of 30m is reached due to the areal dip of the rockhead.

Initially, the project featured the production of a dredged and dewatered basin some 18m deep necessitating the construction of a cellular steel sheet pile cofferdam across the south of the basin, and the stabilisation of the existing basin walls against overturning (Fig. 1). In addition part of the dock floor was prestressed, by installing and post-tensioning anchors beneath 15m of water using specially trained divers†.

†The horizontal thrust slab illustrated in Fig. 2 was designed to give additional support to the basin walls at the north west corner. The thrust slab does form a foundation for one section of dock floor but the anchoring of the dock floors was carried out under the main civil engineering contract

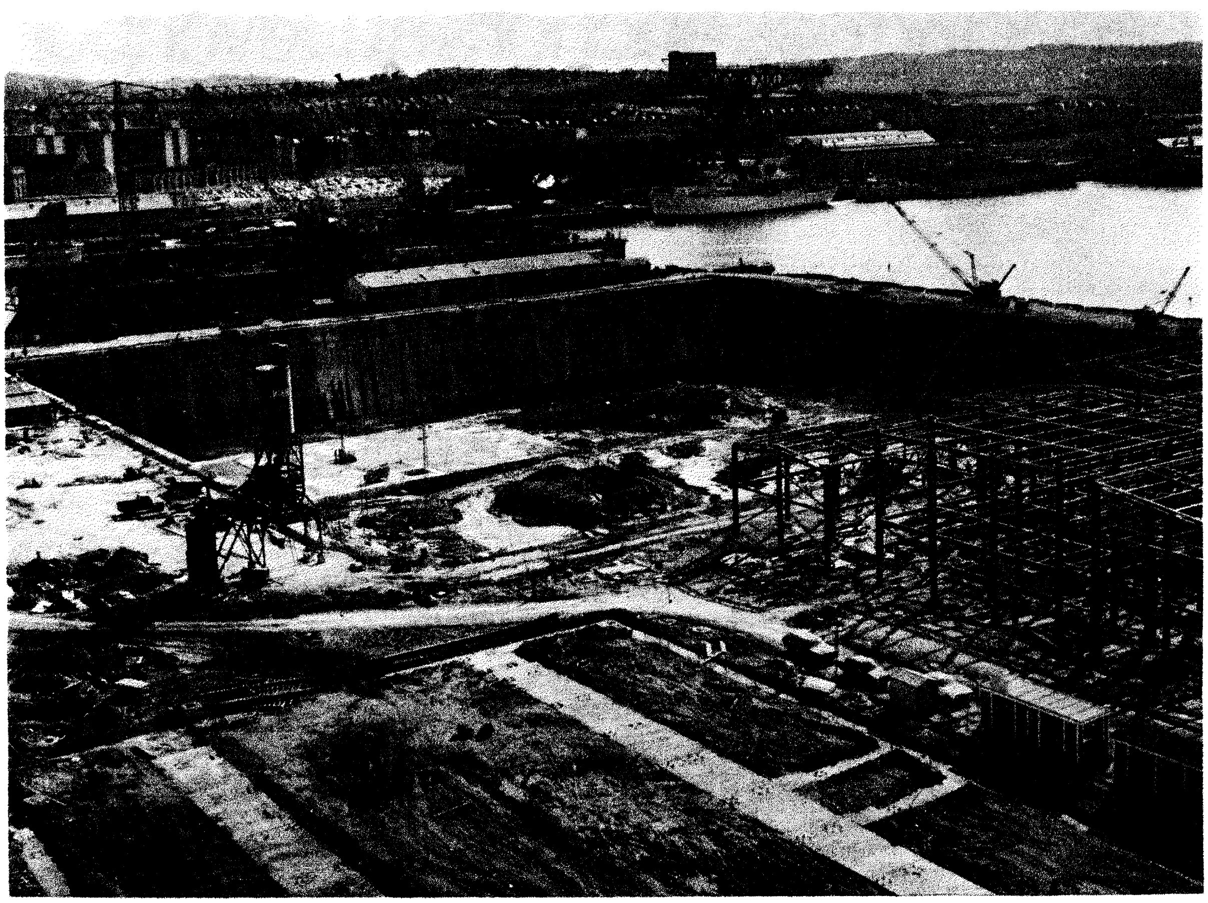

Fig. 1. General view of dewatered basin prior of construction of dry docks at HM Dockyard, Davenport

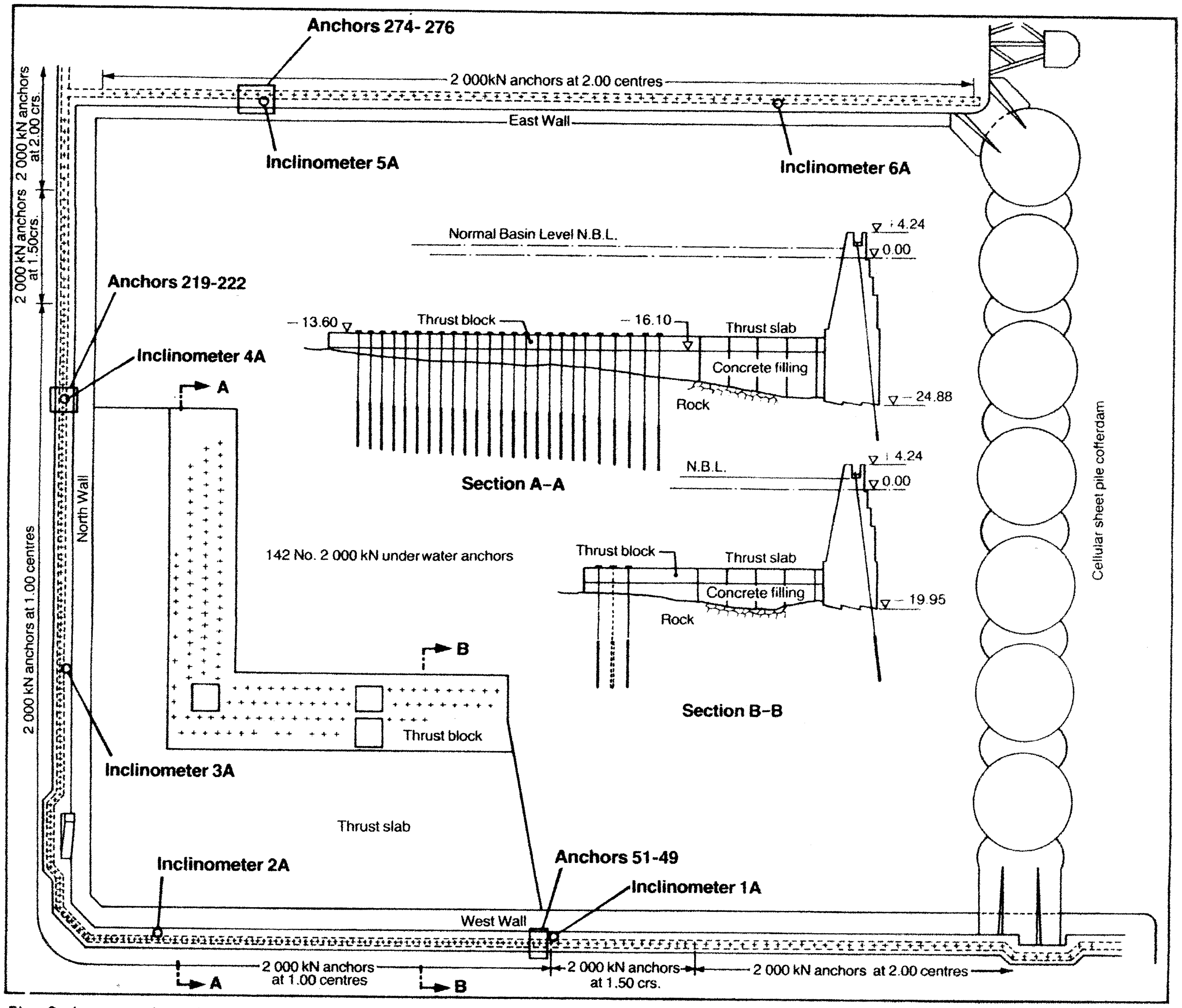

Fig. 2. Layout of anchors for the Devonport Submarine Refit Complex, Stage 1, highlighting the position of the anchors and inclinometers monitored during service

The method of ensuring wall stability was to install 330 No. 2 000kN anchors in holes angled as near to the heel of the wall as possible, and founded in the underlying bedrock (Fig. 2 — Sections *AA* and *BB*). The design required that these anchors, inclined 7-15° from the vertical, were not more than 2.5m apart, and in places, as close as 1.0m. An existing redundant services trench, 1.2m wide and 2.0m deep, running around the top of the walls provided the location for a heavily reinforced anchor beam, into which the load distribution plates and guide tubes were cast prior to drilling the holes, and installing the anchors, (Fig. 3). The design, construction and stressing of the anchors have been described by Littlejohn & Truman-Davies (1974).

Monitoring programme

At an early stage in the anchoring contract, permission was granted for the authors to monitor the time-related performance of selected production anchors. The study had two principal aims:

(*1*) to investigate the actual anchor loads during the crucial basin dewatering and subsequent construction stages and thereby judge the performance of the anchor/wall system, and,

(*2*) to provide a case history of the long-term behaviour of permanent high capacity rock anchors.

The authors trust that this report will illustrate the relative ease and simplicity with which such a programme may be inaugurated, and the value of the results: it is hoped that it will thus act as a spur to the conduct and publicising of similar projects.

Site geology

The site is underlain by a series of geosynclinal Upper Devonian sediments, mainly in the form of hard grey, purple, and dark blue slates, known locally as "shillet". Numerous thin quartzitic greywacke beds, and less frequent igneous intrusions are found in nearby exposures, but none appear to have been intersected by any boreholes drilled in the vicinity of the anchors.

The rock surface dips at an average of 3.5° from north-east to south-west across the site, and the uppermost 1.5m or so is commonly recorded as very weathered and fissile, with frequent softer shale or clay bands. Generally the rock is tightly and strongly folded, due to its participation in the Amorican orogeny, and the cleavage dip varies from 60-80°.

Hand specimens show frequent quartz and calcite veins both along and across the fissility, whilst iron staining is also common along virtually every planar surface.

Very little geotechnical data were actually made available upon which to base design — core recoveries of 80-100%, and a submerged density of 1.28Mg/m^3. Some core samples were later obtained which enabled diametral point load tests to be conducted. The actual specimens were not of ideal shape, due to the small angle between core axis and rock cleavage, and the very close separation of the cleavage planes. However, twelve tests gave values of I_s in the range 0.45 – 0.97N/mm^2, and an average of 0.67N/mm^2 (moderately weak to moderately strong). According to Walker (1975) this average value would relate to estimates of uniaxial compressive strength, elastic modulus and uniaxial tensile strength of 12.0, 3.1 × 10^3, and 1.0N/mm^2 respectively.

The anistropy index ranged from 8 to 18 with a mean of 11.

Anchor and instrumentation details

General

The salient features of the anchors monitored may be summarised as follows:

(*i*) The fixed anchor length was 8.0m, with a nominal diameter of 140mm as drilled by DTH hammers, giving an average rock-grout bond at service load of approximately 0.6N/mm². A factor of safety in excess of 3 against failure of the rock-grout bond was verified by one test anchor.

(*ii*) The tendons consisted mostly of twelve Dyform 15.2mm strands, with a working stress of 55% fpu and a steel section/borehole area ratio of 14.2%. Over the free length the strands were individually protected from corrosion, and debonded from the surcharge grout, by 1.5mm wall thickness plastic sheath with grease infilling.

(*iii*) Special spacer-centraliser units were located at 2m centres in the fixed length and the tendons were noded at intermediate distances.

(*iv*) The tendons were homed mechanically into the holes, and then fully tremie grouted in one operation, with neat 0.45 w/c Rapid Hardening Portland cement grout.

Anchors under observation

Ten anchors were selected for monitoring as detailed in Table I, and their location in plan is shown in Fig. 2.

All those anchors except Nos. 49 and 51 had been previously stressed by multistrand jack, and therefore were destressed prior to installation of the instrumentation.

Each group of anchors also straddled an inclinometer station (Fig. 2) so that any wall movement could be analysed and correlated with anchor performance and vice versa.

Installation of load cells

Vibrating wire load cells, as supplied by Cementation Research Ltd., were chosen in the belief that cells of this type were most suited to the demands of long-term monitoring programmes.

Following removal of the original anchor head plate where necessary, the surface of the load distribution plate was thoroughly cleaned with a wire brush.

A carefully machined bright steel bearing ring (tolerance = 0.1mm) was then fixed to the plate with Devcon, a rapid hardening "liquid steel". The annular load cell was located in the recess formed in this bearing ring, and finally a new top anchor block, with a specially machined locating recess on the underside, was fitted. The use of Devcon and the closely matched surfaces of the bearing ring, load cell, and anchor block were designed to promote axial loading conditions for the cell.

After stressing, each anchor head assembly, including projecting strands, was enclosed by a protective sheet metal cylinder. The wires from the cell led to a socket fitted to this unit and into which the recording unit could be connected.

Each load cell contained three sensing elements arranged at 120° intervals and sampled by means of a portable battery powered meter, manufactured by Gage Technique Ltd. The average of the three vibratory frequency readings was related to a frequency-load correlation chart.

Anchor stressing

Due to the increased height of the anchor head above the thrust pad once the load cell had been installed, a multistrand jack could not be easily employed. A Titan 30 monojack was therefore used throughout, and stressing proceeded in four equal increments per strand, in a sequence designed to ensure uniform loading of the cell. A final fifth operation was applied on occasions to mobilise the appropriate total anchor load, or to attain an equal load distribution between strands. Typical load-extension data, in this case for Anchor 221, are shown in Fig. 4: extensions were measured by stiff rule to an accuracy of 1mm, after each stressing stage.

In general the load-extension curves of individual strands were parallel, as should be expected, although at lock-off the total extensions deviated by up to 15% from the mean. As the strands for each tendon were cut from the same reel, and therefore had approximately the same *E* value, it is most likely that these deviations were caused by frictional and lock-off losses in the main.

The stressing records for Anchors 220 and 276 indicated that one strand from each tendon had experienced grout-steel failure. It is noteworthy that both "failed" strands were located on that part of the tendon circumference which, due to the inclination of the boreholes, would have most intimate contact with the borehole wall during homing, and therefore the highest chance of contamination.

A simple analysis and comparison of the original multijack and subsequent monojack stressing records appear in Fig. 5. Both records show that the recorded extensions for the complete tendon were less than those calculated theoretically, although field results were generally within the limiting boundaries *A* and *B*, first proposed in DIN 4125 (1972). The authors consider that these limits offer realistic acceptance criteria in practice pro-

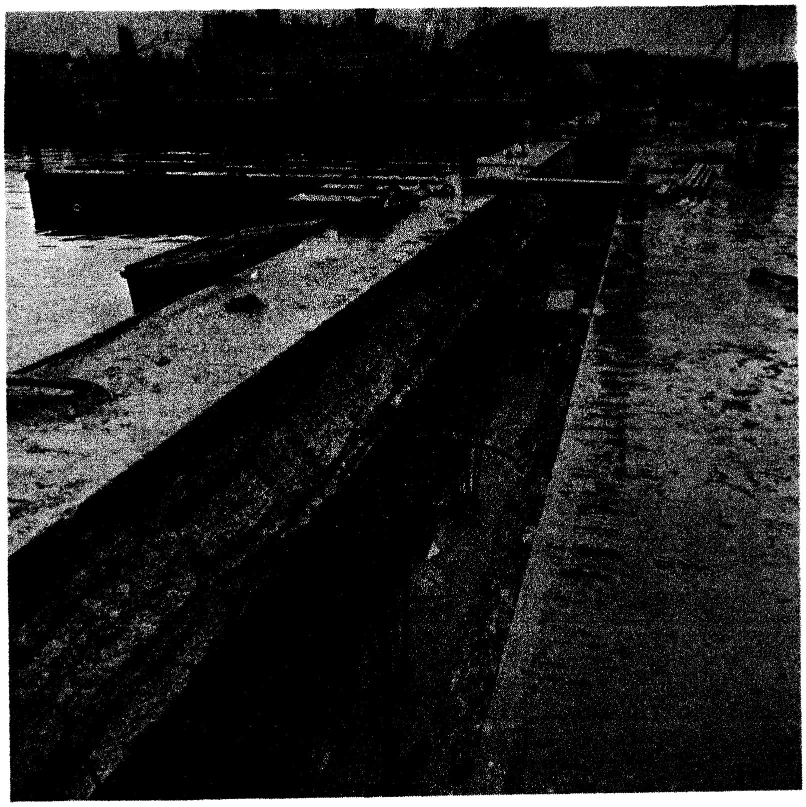

Fig. 3. Anchor heads located in redundant services trench

TABLE I. REFERENCE NUMBER, POSITION AND LENGTH OF MONITORED ANCHORS

Anchor No.	Load cell No.	Wall	Anchor head spacing, m	Anchor free length, m
49	1	West	1.2	26.33
50	2	West		29.16
51	3	West	1.0	27.16
219	4	North	1.0	27.21
220	5	North	1.0	29.21
221	6	North	1.0	27.21
222	7	North		29.21
274	8	East	2.0	22.69
275	9	East	2.0	22.69
276	10	East		22.69

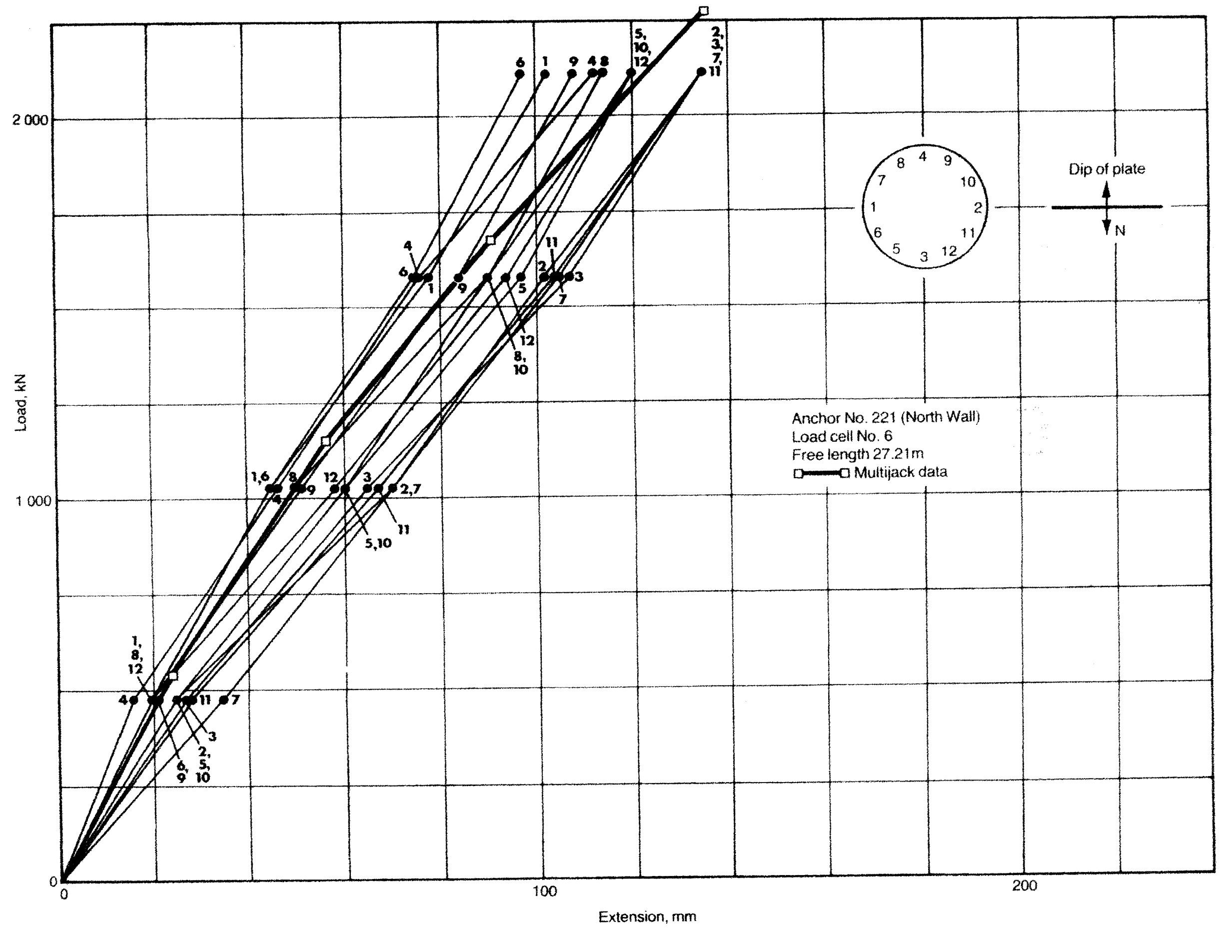

Fig. 4. Typical load-extension data

vided that they are applied wisely in that accuracy of monitoring and probable variations in E value are taken into account during interpretation of the results.

Table II provides a summary of the initial loads for each anchor. The period of installation, instrumentation, and stressing of the ten anchors was four days.

Service behaviour of monitored anchors

(*i*) Up to 24 hours

The load on each anchor was recorded at approximately hourly intervals in the first day after proof stressing. Seven of the anchors gave records similar to that shown by Anchor 50 (Fig. 6).

Rather anomalous patterns were noted in the case of Anchors 49, 51 and 274, where the load actually increased by 160kN (7.3%), 14kN (0.6%) and 67kN (2.9%) respectively.

(*ii*) Up to 4 500 hours

This period extended for approximately 27 weeks after final stressing and included the crucial basin dewatering operation.

Fig. 7 shows typical load-time records in this case for Anchors 50 (previously stressed multijack) and 51. The beneficial influence of prestressing history (2 000kN for 3 100 hrs) on the subsequent performance of Anchor 50 is clearly illustrated. Air temperatures were recorded throughout as tests had shown that apparent load fluctuations of up to 4kN might reasonably be ascribed to temperature variations affecting the load monitoring equipment. Fig. 7 also illustrates the frequency of readings, which was tailored to provide maximum information at certain stages: daily for the first week, weekly for the next 6 months, but twice daily during dewatering.

As noted in the earlier section on "Anchors under observation", wall movements were measured by inclinometers, and during dewatering, when the support

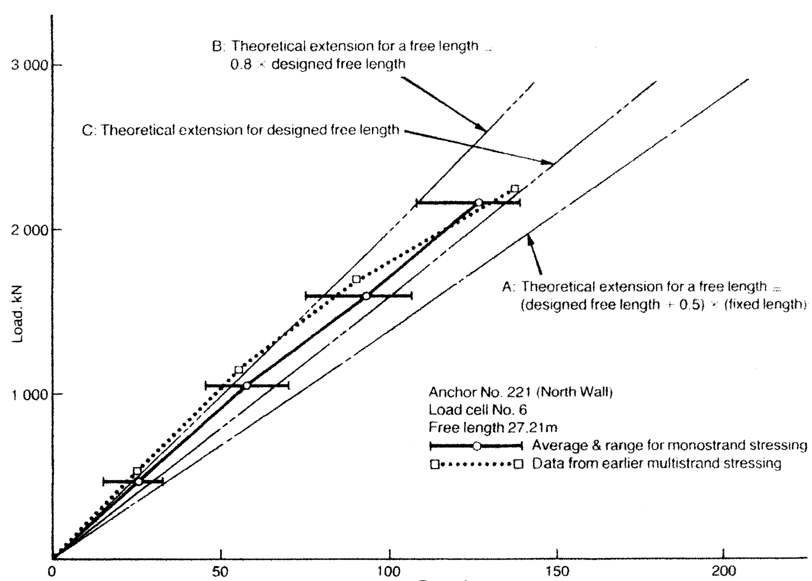

Fig. 5. Comparison of original multijack and subsequent monojack stressing data

of 13m of water was removed, regular monitoring over a period of one month indicated that the walls inclined inwards from the base to give maximum lateral displacements in the range 5 to 50mm at crest level over wall depths of 18 to 30m.

(iii) **Up to 33 000 hours**

Due to the contractor's site activities, it proved both inconvenient and impractical to take regular readings throughout the period. Following dewatering, each cell was read at monthly intervals until 10 000 hours after stressing. Thereafter, readings were taken at approximately four monthly intervals up to about 18 000 hours, followed by a final set at 33 000 hours (196 weeks).

The records obtained after 10 000 hours were further influenced by the progress of the works as a whole. For example the North Wall anchors were destressed after 13 600 hours (Anchor 219) or 14 600 hours (Anchors 220-222), and other cells were damaged during construction activities, as summarised in Table III.

The long-term records for each group of anchors are shown in Figs. 8-10.

Discussion of results

(i) **Behaviour to 24 hours**

Seven of the ten anchors illustrated patterns which are both predictable and widely recorded — a relatively rapid initial load loss, quickly and progressively reducing in rate. Such load losses are ascribable partly to the relaxation characteristics of the tendons themselves, but also to other discrete sources such as movements associated with the bedding-in of the anchor head assembly. The maximum loss recorded on site was 44kN (2%) in Anchor 50, 90% of this occurring in the first four hours. However, the immediate performance of the other three anchors was both surprising and anomalous in that load increases were recorded, after final lock-off, and the complete removal of the jack.

The interpretation of this phenomenon, which has also been observed although not officially recorded on a number of occasions by field operatives known to the authors, is outside the scope of this study, and the particular fields of knowledge of the authors. What is clear is that some external source of energy acted upon the three anchors in question after final lock-off; what is not clear, is the source of this energy.

By way of speculation, the reader is referred to the results of laboratory and field experiments conducted by Nichols & Abel (1975). They highlighted that residual energy locked into igneous and metamorphic rock masses may be released by engineering activities. This energy release is usually manifested by rock bursts, small scale deformations, and larger scale deformations along geological discontinuities and equivalent to earthquakes up to Richter magnitude 3.

An excellent review by Lee, Nichols & Savage (1976) also concluded that "appreciable recoverable internal energy exists in most rocks at shallow depths" and cite a multiplicity of recorded examples.

It seems possible, therefore, that the "engineering activities" of drilling a borehole and subsequently exerting over 2 000kN of prestress on the rock mass may just have triggered off such a release of captive energy, which in three cases resolved to effect an increase in tendon prestress.

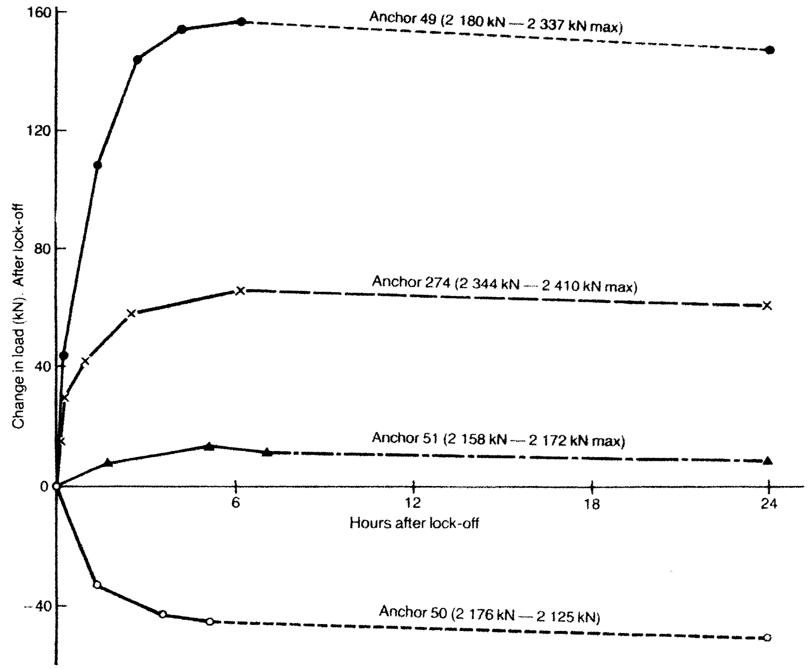

Fig. 6. Anchor performance up to 24 hours

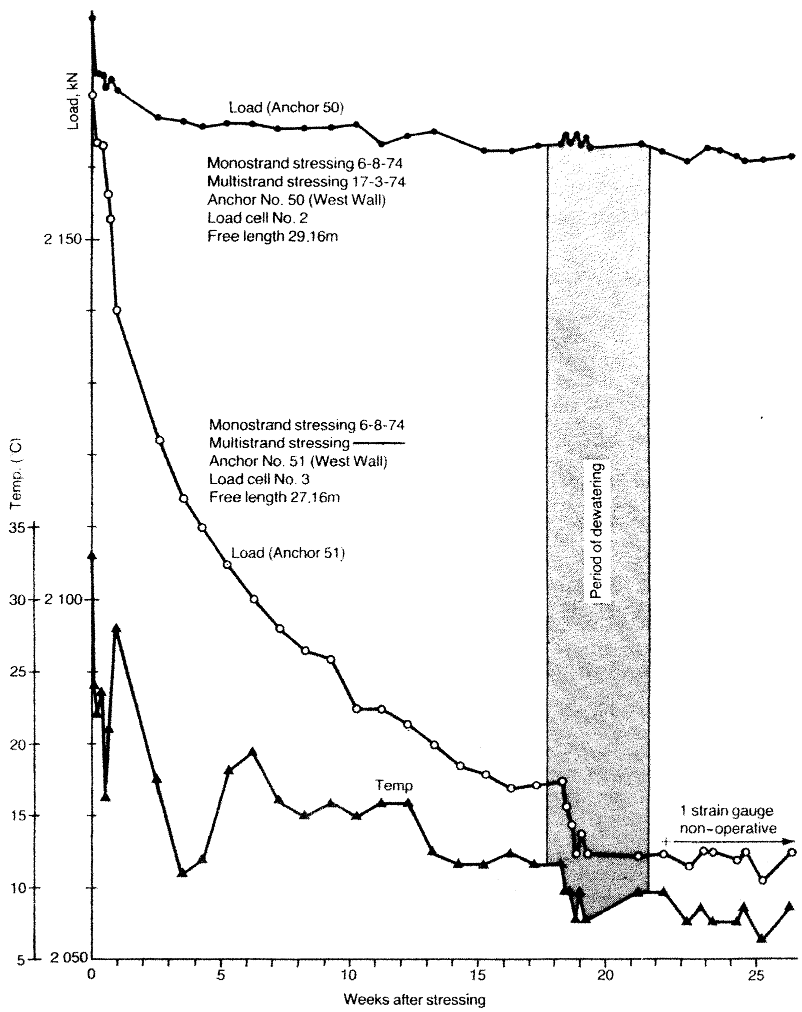

Fig. 7. Anchor performance up to 4 500 hours

(*ii*) **Behaviour to 4 500 hours**

With regard to the crucial dewatering phase, the records showed no major fluctuations in load; those recorded were scarcely above the order of variation to be expected from the influence of ambient air temperature on the load sensing equipment.

On the basis of the ten anchors monitored it is apparent that the major change in external loading in the basin upon dewatering achieved minimal load change in the wall anchors.

(*iii*) **Behaviour to 33 000 hours**

As explained in the earlier section "Service behaviour of monitored anchors": (*iii*) up to 33 000 hours", several occurrences restricted the number and accuracy of readings after 10 000 hours (60 weeks) although good records for the East and West Wall anchors were maintained beyond 17 500 hours (104 weeks).

It is evident from Figs. 8-10 that the long-term load time curve consists of two distinct phases — a rapid loss phase (I), followed by a slower and more uniform reduction in prestress (Phase II). This is illustrated in Table IV. In six cases the amount of load lost in this initial phase (of up to 18 weeks i.e. 3 000 hours duration) was in excess of 85% of that measured at around the two-year stage, and of the other four, the lowest figure was 57%.

Conclusions

(*i*) Monitoring of the service performance of ten anchors on this site indicate two distinct phases in terms of rate of prestress loss. Phase I is reflected by a stabilising, but fairly rapid loss with time, occurring within a period of 3 000 hours. Thereafter, a slower and more uniform rate of prestress loss is observed (Phase II). Based on these limited results, it is recommended that where service performance is being studied the duration of the study should be at least 5 000 hours. This period should cover completion of Phase I, and hopefully provide sufficient results at say monthly intervals to indicate a clear trend for Phase II and thereby permit an extrapolation of the results to cover the service life of the anchors.

(*ii*) The final set of readings which could be taken revealed residual loads in Anchors 49, 51 and 275 of 2 275kN, 2 020 kN, and 2 190kN respectively, after 33 000 hours. In all three cases, this confirms the very gradual rate of load loss of Phase II, the total losses being 62kN (2.7%), 152kN (7.0%) and 60kN (2.7%) respectively. The maximum prestress loss recorded was 4.7% at 3 000 hours when the rapid loss Phase I was complete and 7% after 33 000 hours. These values are reassuring bearing in mind the 10% overload allowance commonly stipulated in anchor practice.

(*iii*) It is known that restressing tendons after a certain period reduces the subsequent prestress losses due to relaxation (Littlejohn & Bruce, 1977). Bearing in mind that eight of the tendons in question had undergone two phases of stressing (multijack and monojack), the generally low prestress losses recorded are consistent with this view. This is particularly well illustrated by the West Wall anchors.

(*iv*) It may be generally concluded that the anchors have functioned satisfactorily in terms of load-holding capacity during a crucial construction phase, and for the monitoring period of almost four years after stressing.

TABLE II. INITIAL SERVICE LOADS AFTER PROOF STRESSING

Anchor No.	Initial load, kN	Initial tendon stress, % fpu	Remarks
49	2337	65	Not previously stressed (multijack)
50	2180	61	Previously stressed (multijack) to 2 000kN for 3 100 hrs.
51	2172	60	Not previously stressed (multijack)
219	2094	58	
220	2085	63	11 strands effective
221	2110	59	All North Wall anchors previously stressed (multijack) to 2 280kN for 1 150 hrs.
222	2384	65	
274	2410	73	11 strands effective
275	2250	63	All East Wall anchors previously stressed (multijack) to 2 280kN for 1 200 hrs.
276	2114	59	

TABLE III. LENGTH OF RECORDS, AND CAUSES OF TERMINATION

Anchor No.	Length of record (hours)	Notes
49	33 000	2 out of 3 channels stable and reliable.
50	10 000	Cell destroyed due to construction activity.
51	33 000	2 out of 3 channels stable and reliable.
219	10 000	Destressed after 13 600 hours.
220	14 000	Destressed after 14 600 hours.
221	14 000	Destressed after 14 600 hours.
222	14 000	Destressed after 14 600 hours.
274	18 500	Cell destroyed due to construction activity.
275	33 000	Only 1 out of 3 channels stable and reliable.
276	18 500	Cell destroyed due to construction activity.

TABLE IV. SUMMARY AND ANALYSIS OF LOAD LOSSES

Anchor	Max. initial load (kN)	Rapid loss Phase I (kN) (A)	(weeks)	Final load loss monitored (kN) (B)	(weeks)	(*A*)/(*B*) %	Remarks
49	2337	39	(16)	45	(110)	87	Not previously stressed
50	2180	13	(3)	20*	(110)	65	
51	2172	102	(18)	114	(110)	89	Not previously stressed
219	2094	31	(8)	34*	(81)	91	11 strands effective
220	2085	85	(5)	89*	(87)	96	
221	2110	52	(16)	56*	(87)	93	
222	2348	7	(2)	9*	(87)	78	
274	2410	20	(4)	35	(110)	57	11 strands effective
275	2250	32	(16)	42	(110)	76	
276	2114	13	(1)	15	(110)	87	

*Extrapolated

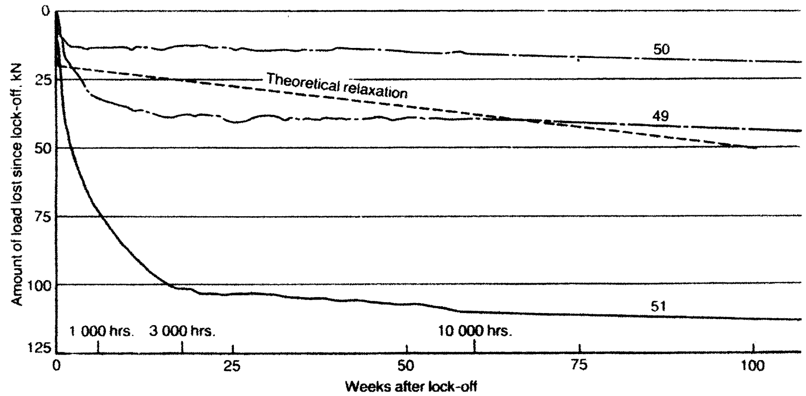

Fig. 8. Long-term performance of West Wall anchors up to 18 500 hours

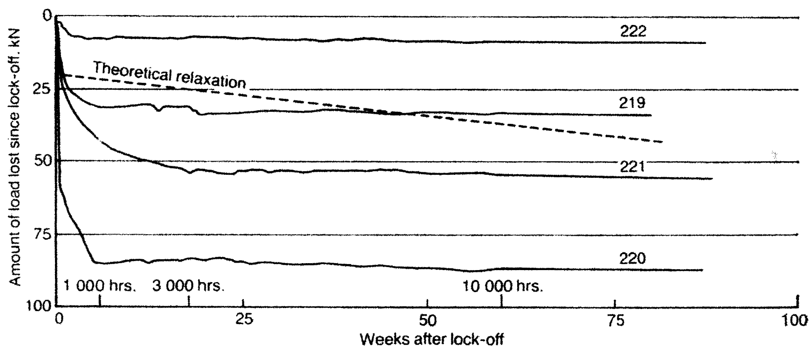

Fig. 9. Long-term performance of North Wall anchors up to destressing at 14 600 hours

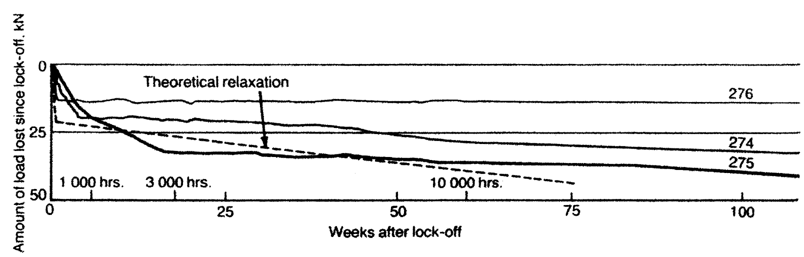

Fig. 10. Long-term performance of East Wall anchors up to 18 500 hours

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