Stabilising an excavation in soft rock by prestressed anchors: A case study

Stabilisation par tirants précontraints d'une paroi de fouille en roche tendre: Etude de cas

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SYNOPSIS The design of a high standing hotel complex on a hillside nearshore in the Piraeus area, Greece, necessitated the building to be partially located inside a man-made excavation on the existing slope, so that the vertical face of the cutting should have a total depth of 24 m. The ground was composed primarily of sandy marls interbedded with loose sandstones, chalk and fractured limestone deposits. The presence of local shattered zones, weak bands and a great number of slickensided planes inclined unfavorably to the face was thought to give rise to instability : it was thus decided to stabilise the excavation with a rc diaphragm wall supported by permanent prestressed anchors and to monitor the wall displacements and the anchor load fluctuations during and after construction. The aim of this paper is to present and discuss the results of the observations made.

INTRODUCTION

The last decades the employement of permanent rock anchors in excavation engineering has become world wide, as these devices are an engineer's tool either in handling the economics of design, either because in certain cases they provide the only solution in stabilising potentially unstable opencast pit slopes. However, after nearly 40 years of experience, rock anchoring - especially in soft sedimentary rocks - is still regarded more as an art than as a science. Because relatively little field research has been conducted into this area of geotechnical construction, there is a growing need of field evaluating the design criteria and of quality control. It is the authors' belief that only detailed monitoring of real tieback systems can improve knowledge relating to the short and long term behaviour of rock anchors in service, as well as to the interaction between the different components of the anchored slope, as excavation proceeds.

Based on this philosophy, it was decided to implement an instrumentation programme to observe wall movements and anchor load fluctuations during and after construction of a 24 x 26 m vertical soft rock face on a hillside nearshore in the Piraeus area, Greece. It was for the purpose of erecting a high-standing hotel complex, where the building ought to be partially located inside a man-made excavation on an existing smooth slope overlain by a nearly 10 m high rc retaining wall; the bottom of the excavation was located at sea level. Various types of retaining systems were considered in the pre-design stage taking account of the seismicity of the area, and the solution finally adopted consisted of a rc diaphragm wall, 0.40 m thick, supported by nine rows of permanent DYWIDAG single threadbar rock anchors, located on the nods of a regular 1.5 x 1.5 m grid; the excavation was opened in successive horizontal zones of 1.5 m of depth with progressive support proceeding from the top downwards, after having installed two rows of temporary prestressed anchors at the base of the existing rc retaining wall.

This paper presents and discusses the results of the observations made during various phases of the tieback construction, as well as of the long term behaviour of the permanent anchors in a time period of nearly 10000 hours.

GROUND CONDITIONS

Geologic setting

The wider area of the project forms one of the main hills of the Piraeus penninsula (Kastella) consisting of lower Pleiocene marine and littoral sediments. An engineering geology study over the narrow project area had shown that the site extended over a nearly continuous, subhorizontal sequence of undulating neogene formations composed primarily of easily weatherable and friable sandy marls interbedded with loose sandstones, chalk and fractured marly limestone deposits. Two main groups of subvertical neogene faults, perpendicular to each other, have been mapped, while a number of individual bands of fully softened mudstones and shales of high clay content, located at nearly 4 m intervals and dipping 10° to the horizontal had also been observed in the surface outcrops, fig 1. The rock material appeared to be heavily overconsolidated and cemented; however, due to intense deformation of both subhorizontal and subvertical character, the rock mass had been locally shattered and it was considered to be in a presheared condition.

Geotechnical Investigation

A detailed site investigation programme was carried out involving initially ten vertical boreholes 15 to 28 m deep including sampling and SPT testing, four trial pits and a trial trench, while a thorough laboratory programme had permitted the relevant soft rock properties to be displayed; falling head and pumping tests had also been conducted. In a second phase, three boreholes, 17 m deep and inclined at 10° to the horizontal were drilled at the base of the retaining wall;

they included continuous sampling for the main purpose of monitoring the discontinuity pattern inside the rock mass to be supported : the investigation revealed a number of slickensided planes inclined unfavorably to the excavation face, at a mean angle of 60° to the horizontal (fig 1), spacing only 2 to 3 m. In a third and final phase, five boreholes 20 to 28 m deep including sampling and laboratory testing have been executed in the lower part of the excavated slope at an angle of 20° to the horizontal, for the purpose of checking the strength parameters of the rock mass deduced at previous stages, and which were later used in a trial back analysis.

Geotechnical parameters

Laboratory tests had shown that the sandstones were fairly marly, with fine material (No 200 sieve) ranging from 35% to 46% and furnishing values of the UCS between 0.7 and 11 N/mm2; RQD indices varied between 20% and 40% and they were slightly lower than those estimated in the marly limestones, while SPT tests indicated penetration rates as high as 80 mm per 50 blows.

Marly layers were fairly sandy, with fine material ranging between 60% and 80%, while in the fully softened bands fine material was as high as 99%. In the former case, Atterberg limits varied between 27% and 43% (LL) and between 16% and 23% (PL), while in the latter case the range was 57% to 79% and 23% to 28% respectively; water content was very near to the plastic limit in both cases and the degree of saturation was nearly 80%. Void ratio varied between 0.32 and 0.80, with usual values between 0.50 and 0.65. In direct shear U tests, intact speciments revealed a strain softening behaviour with strength parameters ranging considerably, as can be seen in Table I below. In remolded state, the cohesion has dropped as much as 66%, while Φ presented an insignificant loss.

TABLE I : Strength Parameters of the Soft Rock

Test		D.Shear	Triax (UU)	Triax (CU)
Φ (°)	pic	23 - 44	27 - 60	30 - 38
c (N/mm2)		0.03 - 0.15	0.05 - 0.40	0.04 - 0.09
Φ (°)	res	16 - 23	23 - 60	28 - 31.5
c (N/mm2)		0.05 - 0.10	0.08 - 0.20	0.006 - 0.0

The intact geological material was fairly watertight, so that the rock mass permeability was due to secondary fracturing; the in-situ permeability tests had furnished a wide range of values, between 1×10^{-2} and 5×10^{-5} cm/sec. The GWL in the boreholes were fairly close and slightly above the mean sea level.

DESIGN, CONSTRUCTION AND INSTRUMENTATION ASPECTS

Some three hundred total stress stability analyses with reduced values in the measured parameters were conducted to examine the sensitivity of the overall factor of safety against local failure or deep seated movement (Hoek & Bray, 1977) to changes of the strength parameters due to local variations in rock quality along the anchorage zone or due to constructional errors or inefficiencies, which could lead to softening as a result of the rapid increase of the water content occuring by the penetration of the water into the opened fissures. The minimum value of the safety factors considered in the design were : 1.6 for failure of the steel tendon, 1.25 and 1.50 for failure of the rock mass along any potential surface (with and without earthquake effects, respectively) and 3.5 for failure of the rock - grout bond. The minimum required anchor service load was then assessed as high as 500 KN for the configuration adopted, fig 2. Provision of draining the rock mass was made by boring a number of subhorizontal drains on the nods of a regular grid of 4.5 m of aperture, through which the water was allowed to drain in the excavation.

Bearing in mind that the anchor spacing was relatively small (1.50 m), it was considered essential to monitor drill penetration rates, grout consumptions and check pull-out tests during construction in order to highlight changing rock conditions over short distances. Trial drilling tests with a Wagon Drill torsional-percussive machine had shown that the overall cost might only be competitive, if the drilling rates were high enough and this could easily be achieved by water flushing; however, a slippery film might then devellop. Instead, production anchor boreholes were finally drilled using soap foam, then rinced with water and blown out by compressed air, while tremie grouting was made as soon as possible after drilling, with neat 0.45 W/C Portland ciment including some Tricosal 181 and Interplast additives. In order to overcome the eventual loss in shaft resistance by friction due to remolding, anchor shaft length was fixed to 11 m, with a nominal diameter of 100 mm, giving an average rock-grout bond under service conditions not eceeding 0.20 KN/mm2 approximately. All production anchors have been subjected to one loading cycle up to 1.25 the minimum required service load prior to lock off at an overload of 10%, as an allowance for relaxation and creep.

Precise surveying (Costopoulos, 1983) was used to determine displacements, both horizontal and vertical, fig 2; a grid of movement points was installed progressively ahead of the excavation face and the plan coordinates of the points were established relative to a number of six control points installed in the area around, while three movement points on the wall crest were recording settlements. On the other hand, two inclinometer tubes installed in the immediate viscinity of the excavation face had permitted the horizontal wall movements to be displayed at 1.5 m intervals over the whole depth of the excavation; an MK III Soil Instruments torpedo with a resolution of $\pm$ 0.1 mm was used. Anchor load fluctuations on ten selected production anchors (fig 6) were monitored using flat Dywidag hydraulic cells, with a nominal precision of 10 KN; however, air temperatures were recorded througout, as there would appear to be a temperature effect on the apparent load due to susceptibility of the load cells to temperature variations.

RESULTS AND EVALUATION OF MEASUREMENTS

As the excavation was opened section by section and the inclined anchors loaded in sequence to their theoretical design values, wall crest horizontal displacements - as measured by the incli-

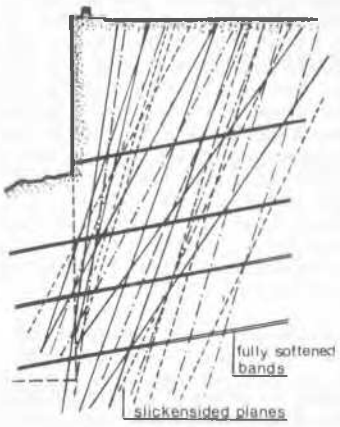

Figure 1. Panorama of the discontinuity pattern behind the cutting

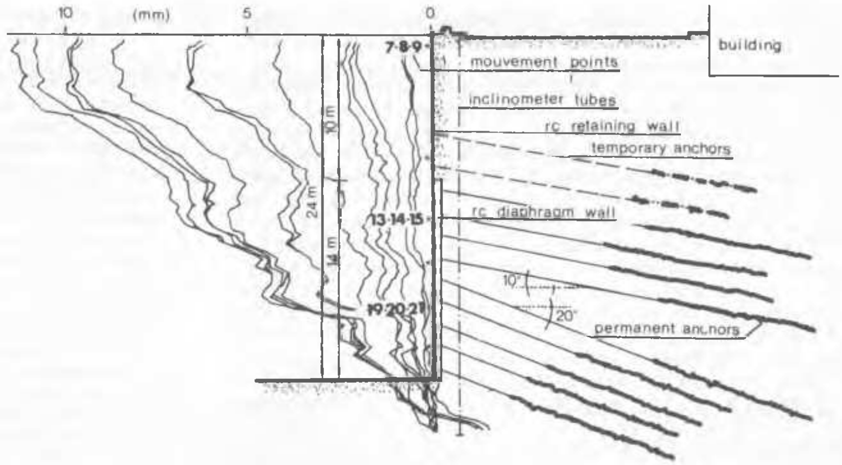

Figure 2. Geometry of the anchored rc diaphragm wall showing inclinometer tubes and survey measurement points. Displacement pattern displayed by the inclinometer T1, direction 1-1

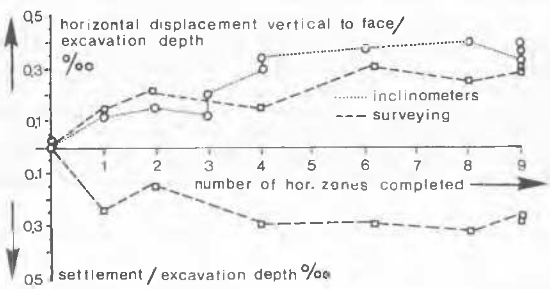

Figure 3. Development of movements on the wall crest (points 7,8,9) with the advance of the excavation opening and support

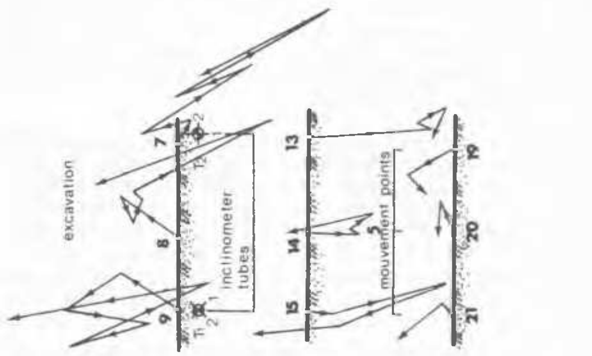

Figure 4. Displacement vectors of several mouvement points on the excavation face displayed by precise surveying

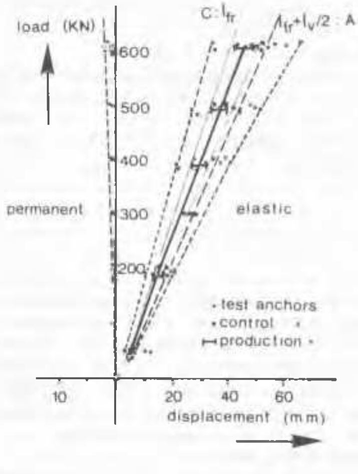

Figure 5. Behaviour of permanent anchors during stressing

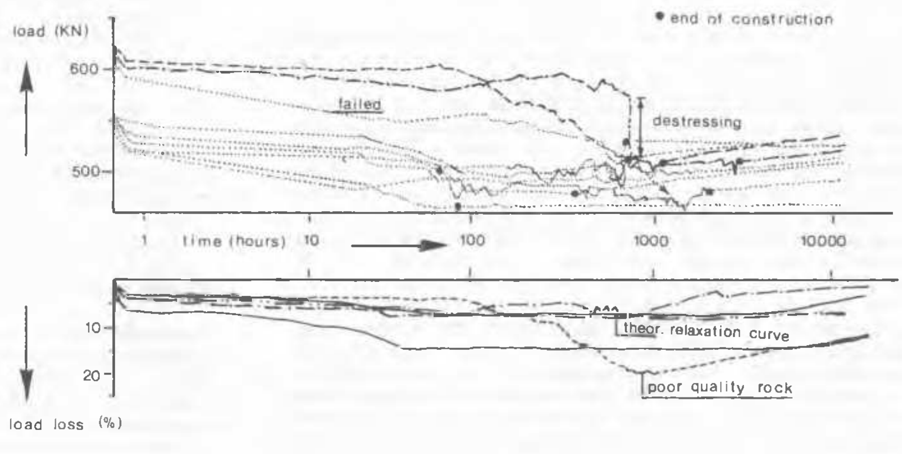

Figure 6. Long term behaviour of permanent anchors as given by the control anchors

nometers - were seen to increase progressively towards the excavation from the top downwards; maximum values recorded were only 0.4‰ of the total excavation depth. However, survey measurements have revealed the occurence of inward movements followed by relatively important settlements immediately after locking off the anchors in each zone. After the first excavation stage, a sudden increase of the vertical movements had been recorded probably due to the downward pull of the temporary anchors, while in subsequent stages settlements were increasing accordingly with horizontal displacements at a more or less constant ratio of the order of 0.78, fig 3. The drilling operation and the injection procedure for the permanent anchors had altered but very little this displacement pattern. On the contrary, delays in the support of the open face were seen to contribute a lot in the rate increase of the horizontal displacements. It is interesting to note that the retaining wall was moving more or less as a rigid body, tilting about a theoretical horizontal axis, the position of which was fairly close and slightly below the bottom of the excavation, whereas each section of the diaphragm wall had a net tendency to move independently of the others. One year after the end of construction, movements have practically ceased, their major part - nearly 60% - having occured by the time the last excavated zone was complete, fig 2,3,4.

During the loading cycle, nearly all production anchors have experienced a similar response fullfilling the acceptance criterion according to DIN 4125 (1972) : the load - extension curves were almost linear lying within the limiting boundaries A and C (fig 5), with a net tendency towards the mean curve expressing the theoretical extension for a free length equal to the design value (l_{fr}) plus half of the shaft length (l_v). On the other hand, permanent displacements recorded at the end of the loading cycle ranged around a mean value of 5 mm and they were in good agreement with the corresponding values displayed by the six test anchors examined earlier; these latter anchors had revealed an almost linear variation of the permanent displacements with load.

After locking off and within the first hour, all control anchors have exhibited a relatively rapid initial load loss ranging from 2% to 6% of the theoretical prestress load. However, this loss had shown to be progressively reducing in rate, at least until the end of the construction period, where the maximum loss was recorded : the range for the ten anchors being monitored was between 3% and 19%, with a mean value around 10%. It is worthy to note that the two extreme values previously mentioned have been recorded on two control anchors which have been locked off at an overload of 25% the minimum required service load, instead of 10% used in all other anchors : the maximum loss (19%) appeared in the case of the poorest quality rock necessitating an overgrouting as high as five times the normal quantity; the minimum value (3%) was associated with a destress down to the service load after a period of 1000 hours, giving an idea of the beneficial effect such a loading programme might have on anchor performance. However, total losses in every tested anchor should be attributed to the relaxation of the tendon, to the creep of the soft rock formation and, primarily, to structural movements during construction, fig 6. In effect, the theoretical steel relaxation is shown to be as low as 2% in in the first hour, increasing linearly with time up to value of nearly 6% at 10000 hours; on the other hand, readjustments in anchor loads ranging between 2% and 8% have been recorded following each excavation stage, thus giving good indications that load loss due to rock creep should be as low as 4% the initial prestress load. It can be seen that average values of load loss at 1000 hours was of the same order of magnitude as the one occuring within the first 100 hours and about double of that recorded at 10 hours; these results are in good agreement with the indications given by Littlejohn and Bruce (1977).

In an attempt to evaluate the mobilisation of shear strength inside the rock mass, a number of trial back analyses were conducted using a simplified elastic model of soil - structure interaction after Birkenmaier (1953) and used earlier in praxis by the senior author (Costopoulos, 1983). Because of the anchor behaviour discussed earlier in this paragraph, it seemed likely that the basic hypothese of the model were consistent. In effect, the permanent anchors had proved an almost linear elastic behaviour during stressing; on the other hand, because of the soft nature of the rock it is reasonable to assume that the bond is evenly distributed along half of the fixed anchored zone, where low partial debonding should have occured. The analyses have revealed a total thrust on the diaphragm, which - according to the limit equilibrium stability analysis model used in the design- was corresponding to a partial mobilisation of the angle of shearing resistance on the potential failure surfaces, with values ranging between the pic and the residual. This conclusion is consistent

CONCLUSIONS

Interpretation of the monitoring data lead the authors to draw the following conclusions :

1 - The performance of the tieback during and after construction is considered satisfactory given the very low movements occured, which have reached a maximum value of 0.4% one year after the end of construction. This might be attributed to the high prestress load of the anchors installed and which was equal to 100% of the design service load.

2 - Important losses in the permanent control anchors have been monitored being general around a mean value of 10% one year after the end of construction, their major part having occured during construction.

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