

## **PERMANENT ANCHORS IN WEAK ROCK FOR AN OFFICE BUILDING**

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### **SYNOPSIS**

Most of the case histories reported to date on the use of ground anchors are related to securing of slopes, caverns and excavation, anchoring of retaining walls and against hydrostatic uplift, ensuring anchorage against concentrated forces and stability against overturning and raising as well as strengthening of dams. This paper presents a case history for the use of permanent rock anchors founded in weathered rock in a five storeys prestressed concrete office building in Singapore which was constructed in 1981. Because of site constraints, rock anchors were installed to tie down ground beams and footings against uplift induced by the cantilevering of the beams used to support exterior columns. The installation, testing and evaluation of these rock anchors are discussed. The ultimate grout/rock bond stress is determined from a Southwell's plot of load and immediate permanent displacement of the test anchor even though the test anchor was not loaded to failure. An important acceptability criterion of the anchor is that differential settlements of the building, due to the reduction of extension of the anchor as a result of the reduction of live load, are within tolerable limits.

### **INTRODUCTION**

The use of ground anchors has increased in most parts of the world. Most of the case histories reported to date are related to temporary and permanent anchors for securing of slope, caverns and excavations, anchoring of retaining walls and against hydrostatic uplift, ensuring anchorage against concentrated forces and stability against overturning and raising as well as strengthening of dams (Anon, 1978). Anchors have been used in Singapore as temporary support for the construction of basements of various highrise buildings in the congested urban area and permanent anchors for various purposes. Some of these are reported in Ramaswamy (1975, 1979), Littlejohn & Macfarlane (1975), Kannan (1979, 1983), Kannan & Broms (1984) and Fishli (1983).

The purpose of this paper is to present a case history on the use of permanent rock anchors founded in weak rock for an office building.

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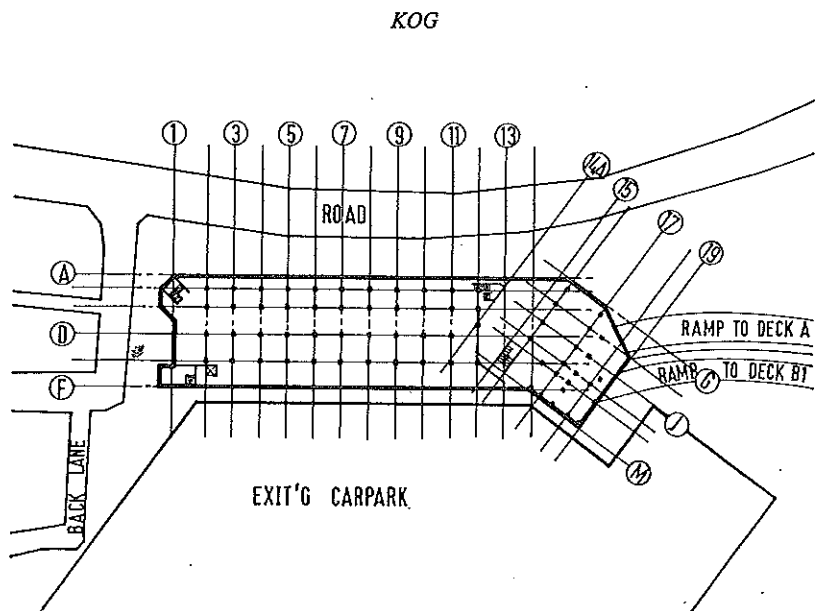


Fig. 1 Location Plan

### THE BUILDING

Permanent rock anchors were installed in a five storeys prestressed concrete building constructed in 1981 situated next to a three storeys carpark which was constructed in 1973 as shown in Fig. 1. Because of the good soil condition on site, all these buildings were founded on footings. It was not possible to construct footings to the exterior columns of the five storeys building because of the close proximity of the existing carpark foundations. To avoid this problem new footings were set back from which deep beams were cantilevered to support those exterior columns of the five storeys building as shown in Figs. 2 and 3. Six permanent rock anchors had to be installed to tie down these footings. Nominal working loads of these anchors varied from 400 kN to 1000 kN per anchor.

### SUBSOIL CONDITIONS

The site is located within the Jurong Formation according to the Geology of the Republic of Singapore (Anon, 1976). Generally, very hard decomposed

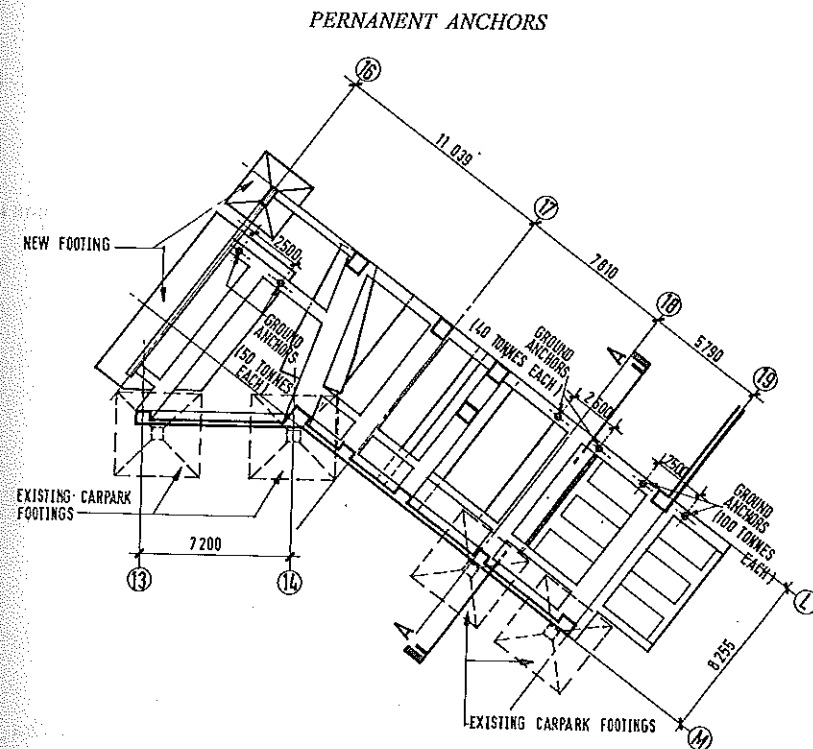


Fig. 2 Part Layout Plan of Footings

shales and sandstones were encountered at relatively shallow depth. The top 2-6 m consisted of loose sand or medium sandy silt, followed by very hard decomposed shales and sandstones at depth of 5 metres and below. Visual inspection revealed that the strata of shale and sandstone at the site are steeply dipping. Borehole 1 as shown in Fig. 4 is closest to the anchors. For depths greater than 3.45 metres, rotary diamond drilling was used. Drilling was in runs of 1.5 metre except where poor recovery was obtained or when the cores were blocked. The sandstone and shale of the Jurong Formation fall within the weak to moderately weak range i.e. between 1.25 and 12.5 MPa according to the unconfined compressive strength and the weathering grade II to IV (slightly weathered to highly weathered). No information was available regarding the soil aggressiveness and ground water, however this is not considered to be problematic. The RQD (Rock Quality Designation) ranges from 0 to 50% which indicates that the rock mass quality is very poor to poor. The point load index varied from 0.35 to 5.6 MPa.

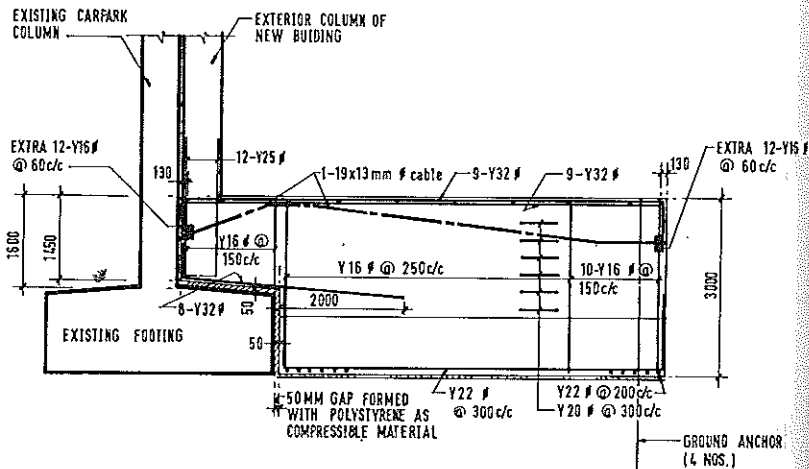


Fig. 3 Details of Footing

TEST ANCHOR

The borehole log indicated a fairly complex geological structure for such a small area. Shale and fine grained silty sandstone exposures are found at the site. Site investigation suggested that both weak weathered shale and sandstone could be met within the depth of fixed anchor lengths. A test anchor was installed to establish suitable design parameters in relation to empirical design rules for the design of the permanent rock anchors and identify unforeseen construction difficulties. The test anchor design working load  $T_w$  was 1000 kN. Its geometry and the in-situ soil profile is shown in Fig. 5 where the top soil layer was excavated. Stressing was carried out twenty eight days after grouting. The seating load  $T_0$  was  $0.1 T_w$  and load increments of  $0.1 T_w$  to  $0.25 T_w$  were applied until the strand reached 82% GUTS ( $2.56 T_w$ ). After the load reached  $0.5 T_w$  and thereafter at each successive higher load increment, the strand was unloaded to  $T_0$ . The top anchorage displacements occurring at loads below  $T_0$  were not measured. The load extension curve is shown in Fig. 6. Strand extensions during the first loading cycles comprised fixed anchor displacement, strand extension, wedge pull in, bearing plate and structural movement. The interpretation and analysis of the load extension data are restricted to those obtained in second and subsequent load cycles. The reproducibility of load-extension characteristic of

Depth (m)	Core Recovered (m)	SPT/ % Recovery	Depth of SPT / Length of Run (m)	Legend	Description
1.00	-	31 blows (0.30 m)	1.65 to 1.95		Fine sandy silt
	-	114 blows (0.30 m)	3.15 to 3.45		Hard shale / Decomposed shale
0.85	100	4.30			
1.50	100	5.80			
1.20	100	7.00			
1.50	100	8.50			
1.50	100	10.00			
11.30	1.30	100	11.30		Hard sandstone
12.40	1.10	100	12.40		
	0.90	100	13.30		Hard Decomposed shale
	0.80	100	14.10		
	1.20	100	15.30		
16.60	1.00	100	16.30		Hard sandstone / Weathered Decomposed sandstone
	1.35	90	17.80		
	0.50	35	19.30		
	1.40	95	20.80		
	1.40	95	22.30		Alternate Hard Decomposed shale / sandstone layers of 0.3 m - 0.9 m thick each
23.90	0.70	100	23.00		
	0.70	100	23.70		
	0.70	100	24.40		
	1.10	100	25.50		
	0.40	100	25.90		
	0.30	100	26.20		
	0.30	100	26.70		
	0.40	100	27.10		
	0.90	100	28.00		
	0.60	100	28.60		
	0.70	90	29.40		
	0.50	100	30.00		

Fig. 4 Bore-hole Details

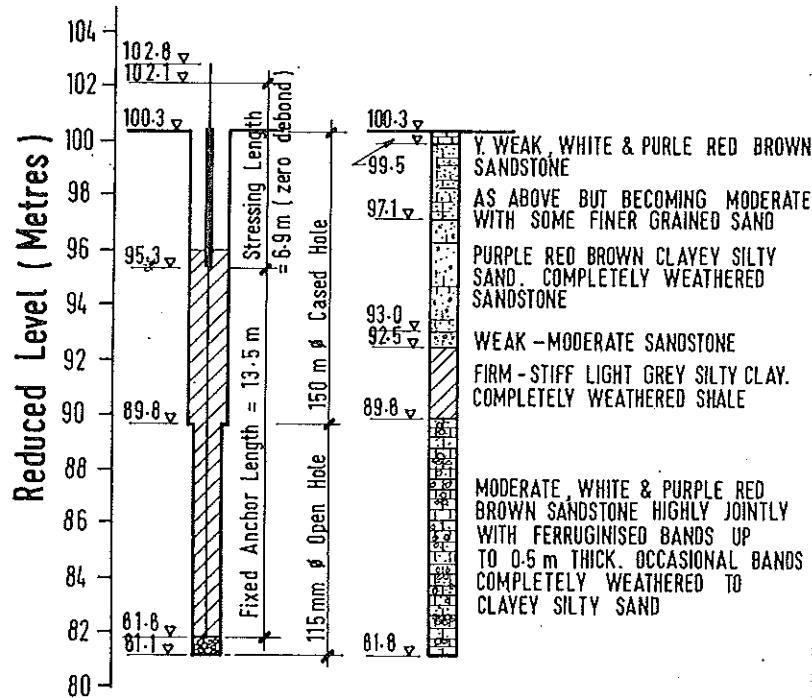


Fig. 5 Geometry and Insitu Soil Profile of Test Anchor

the anchor is good up to the fifth cycle which was loaded to  $T_w$ . No marked deviation from linearity is observed which implied that no debonding in the fixed anchor at the grout tendon interface and no fixed anchor movement have taken place. As shown in Fig. 7, debonding is not significant up to  $1.5 T_w$ , involving only 4% of the fixed anchor length. However it becomes significant at  $2.5 T_w$  where the value increases to 23%. It is also noted that the debonded length of the anchor cannot exceed 30% of the fixed anchor length. Because the load transfer of the anchor is by bond, the failure cone for the overall stability analysis as shown in Fig. 8 is deemed more appropriate despite the oversimplification. The test anchor is considered acceptable if the elastic extensions fall between the two boundary lines (a) and (b) as shown in Fig. 9. The upper boundary line (a) corresponds to tendon extension equivalent to the free length plus 50% of the fixed

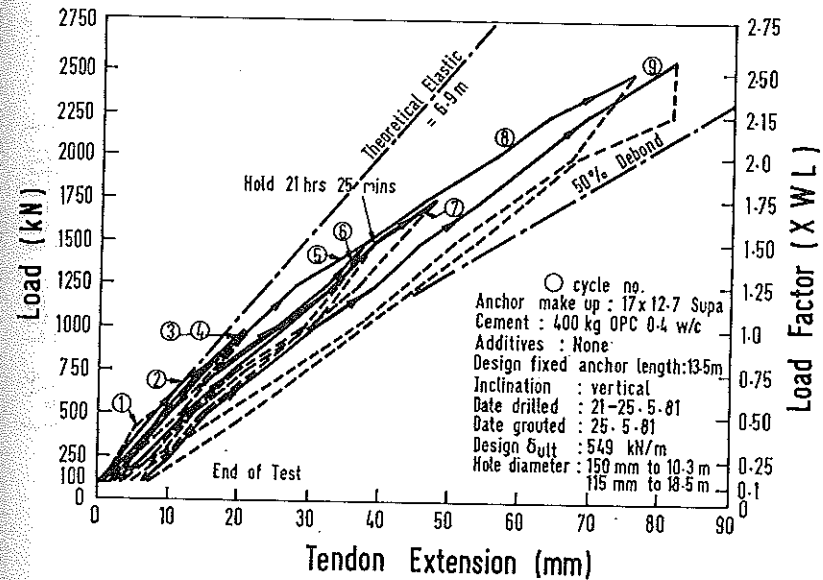


Fig. 6 Load Extension Curve of Test Anchor

anchor length. The lower boundary line (b) corresponds to 80% of the free length of the tendon. Creep at  $T_w$  was 0.03 mm/m of free anchor length after maintaining  $T_w$  for ten minutes which is less than the permissible 0.135 mm/m (Littlejohn & Bruce 1975). At  $1.5 T_w$ , creep of the anchor after 22 hrs was 0.5 mm which is less than the permissible 1-2 mm (Ostermayer, 1974).

It can be concluded therefore, that creep of the anchor founded in this highly weathered shale/sandstone is not significant. Visual inspection of the superstructure made seven years after the completion of the building confirms this conclusion. As for prestress loss, a maximum of 5% in 24 hrs may be considered acceptable (Hanna, 1980).

The anchor load  $P$ , is related to the fixed anchor length  $L$  by the empirical equations.

$$P = \pi d L C_a$$

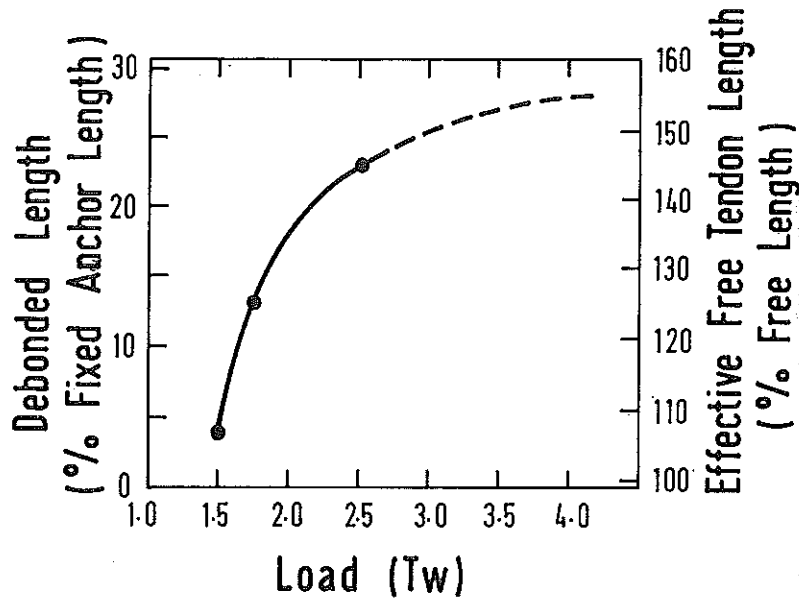


Fig. 7 Debonded Length of Test Anchor

where  $d$  = effective anchor diameter;  $C_a$  = grout/rock bond stress. Coates & Yu (1970) found that when the ratio of the elastic moduli of the anchor material and the rock ( $E_a/E_r$ ) is larger than 10, it is reasonable to assume that the bond is evenly distributed along the anchor. According to Littlejohn & Bruce (1975), the magnitude of  $E_a$  is  $(10\sim 21) \times 10^3$  MPa. Therefore, in this case  $E_a/E_r$  is greater than 50 and equation 1 is satisfactory.

Because the stress-strain relationship of rock in direct tension or in shear is hyperbolic, the relationship between the immediate permanent displacement  $\Delta$  of the anchor and the load  $P$  will be approximately hyperbolic, viz:-

$$\frac{\Delta}{P} = m \Delta + C$$

where  $m$  and  $C$  are constants. The immediate permanent displacement of the anchor is obtained by subtracting permanent displacement due to creep, from

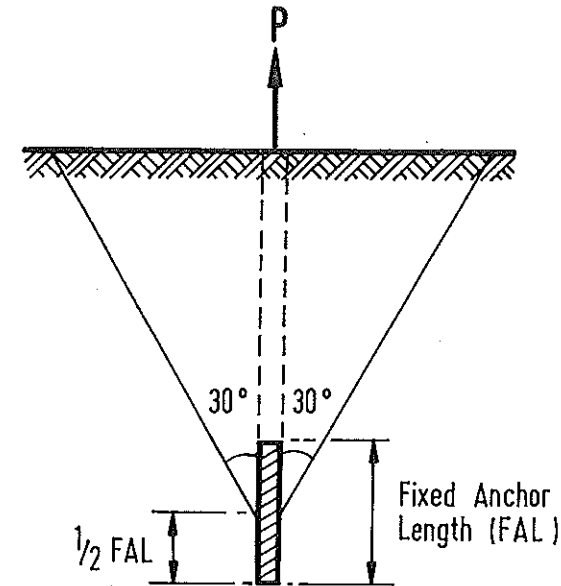


Fig. 8 Failure Cone for the Overall Stability Analysis

the permanent displacement. A Southwell's plot of  $\Delta/P$ , against an abscissa of  $\Delta$ , will produce a fairly linear relationship (Southwell, 1969). The ultimate load of the anchor is given by the inverse of the straight line obtained from the Southwell's plot, even though the test anchor is not loaded to failure. The validity of this approach for determining the ultimate capacities of piles under compression have been amply demonstrated by Chin (1970). Since the load displacement relationships between friction piles under compression and anchors under tension are substantially similar, (ie both have little or no end bearing), the Southwell's plot of  $\Delta/P$  and  $\Delta$ , as shown in Fig. 10, should yield a good approximation of the ultimate load of the anchor. The ultimate load of the test anchor as determined from Fig. 10 is thus 7050 kN. For the purpose of the design of permanent rock anchors founded in weathered shale/sandstone, the grout/rock bond stress can be taken as 640 kPa for a safety factor of two, as recommended by Anon (1982). This value is within the range of the recommended design values for shale and sandstone reported by Littlejohn & Bruce (1975).

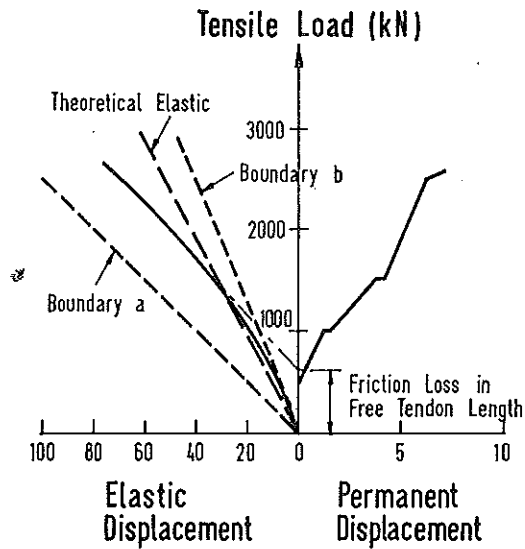
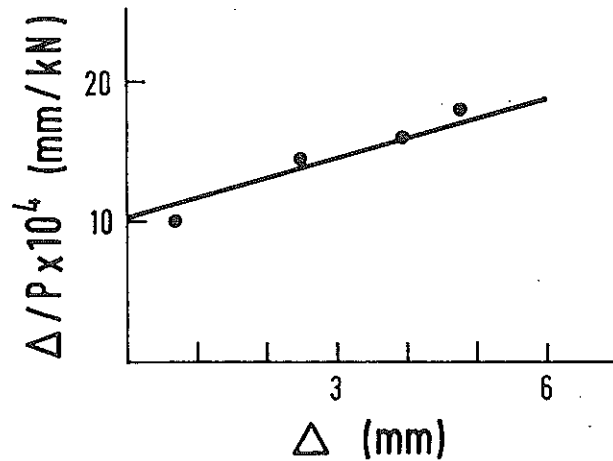


Fig. 9 Elastic and Permanent Displacements of Test Anchor

Fig. 10 Southwell's Plot of  $\Delta/P$  and  $\Delta$  for the Test Anchor

## PERMANENT ROCK ANCHORS

Based on the design parameters established by the test anchor, two permanent rock anchors each were designed, installed and load tested for working loads of 400 kN, 500 kN and 1000 kN respectively. The geometries and load-extension curve of these anchors are shown in Fig. 11. The spacings between these anchors were at least 2.5 metres which is more than one metre (Littlejohn & Bruce, 1975) or five anchors diameters (Hanna, 1980), where a check on interaction of these anchors is necessary. Because of the laminated nature of the shale/sandstone on site, the founding levels of the anchors were staggered to reduce the risk of laminar failure along a horizontal bedding plane. The dipping condition of the shale and sandstone layers should be accounted for here. The founding levels also depend on the overall stability analysis, which required the consideration of the interaction of inverted cones. Each anchor was stressed to  $0.1 T_w$  after which

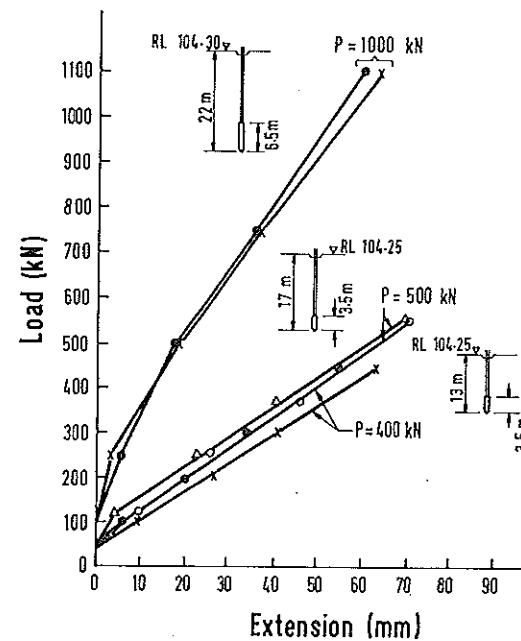


Fig. 11 Load Extension Curves of Rock Anchors

it is stressed in one operation to  $1.1 T_w$ . The overstress of 10% was to compensate for the expected losses at lock off. One important consideration in the design of permanent anchors in a situation as described herein, is that the differential settlements of the exterior columns due to the variation in the loading of these columns is within the allowable limits as recommended by various authors (Grant et al, 1974; Burland & Wroth, 1974, Wahls, 1981). Since live load is estimated to be about 30% of dead load for these exterior columns, the working load for each rock anchor could be reduced by 23%. Cyclic loading was not considered to be a problem. The estimated maximum differential settlements of the exterior columns C1, C2 and C3 as shown in Fig. 2 were 6.5, 5.8 and 3.4 mm respectively, which were well within the allowable limits.

### CONSTRUCTION OF TEST AND PERMANENT ANCHORS

A tungsten tipped finger bit is suitable for moderate to hard formations such as hard shale, siltstone and soft decomposed sandstone found in this site whilst a fishtail bit is more appropriate for soft shale and stiff/hard clay. At this site, rotary roller and drag bit with a Kobe 904B mounted mast is the most suitable and efficient drilling method. The drilling rate of tungsten carbide-tipped bits is chiefly affected by the air pressure at the rock drill, diameter of the borehole, the shape of the cutting edge of the bit and the power of the rock drill. The drilling of the 18.5 m test anchor was completed in a few hours. For the 150 mm hole, a drilling rate of 2-3 m/hr in hard rock, 3-8 m/hr in medium rock and 8 to 15 m/hr in soft rock can be achieved. For the test anchor, the hole was cased to 89.8 m R.L. with a 150 mm casing, open holed to 81.8 m R.L. with 115 mm drill tools, roller and drag bits. Air is the commonest fluid used with drag-bit and roller-bit rotary drilling. Thorough air flushing of the hole after drilling to full depth was carried out prior to the homing of the anchor tendon. When the spacing of anchors is larger than 2 m, as in this instance, alignment of the hole is usually not important. Nevertheless, the deviation of the anchors from the vertical was within the permissible tolerance of 1 in 50.

The detail of the rock anchors is shown in Fig. 12. The main feature of the system was that the load transfer length of the strand was embedded within a grout filled corrugated plastic capsule. The capsule was sealed at the bottom by epoxy. Should cracks develop within the first stage grout column, the strand is sealed from any corrosion agent by the capsule. The free stressing length is itself protected by a grease filled high density polyethylene sheathing continuous to the under side of the load distribution plate at the top anchorage end. A grease filled

### PERMANENT ANCHORS

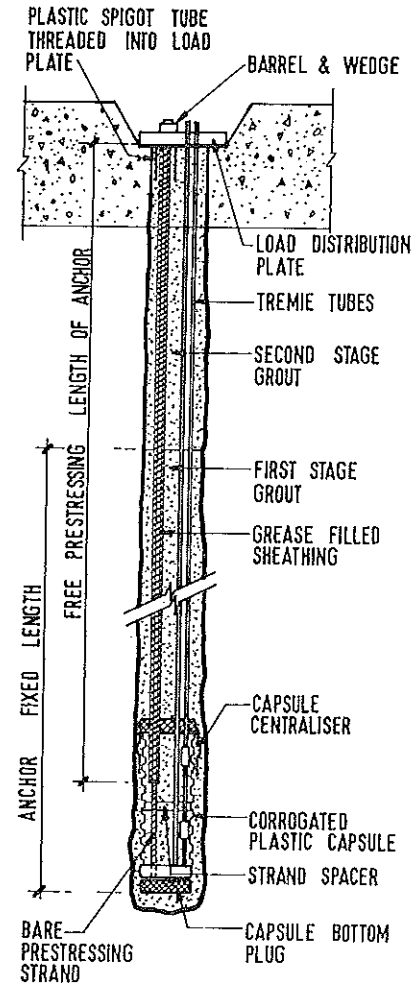


Fig. 12 Details of Rock Anchor

plastic spigot tube threaded into the load distribution plate overlaps the sheathing to ensure a protection seal at that point. The annulus between the plastic sheath and the borehole is filled with second stage grout. After anchor stressing and acceptance, the top anchorage was concreted into the ground beam. With this arrangement, the fixed and free length of the anchor overlapped as illustrated in Fig. 12. The anchors consisted of 12.7 mm diameter prestressing strands, the number depended on the working loads of each anchor. Four, five and ten 7-wire super strands were employed for working loads of 400 kN, 500 kN and 1000 kN respectively. Only seventeen out of the nineteen strands of the test anchor were stressed because the other two strands were damaged. Spacers positioned over the load transfer length ensured 5 mm grout cover to each strand. Centralisers were fixed to the capsule to ensure that it is set centrally in the borehole. Grouting was done through two tremie tubes attached to the tendon, one internal and one external to the capsule. As grouting proceeded, the tremie tubes and the casing were steadily withdrawn. The first stage grout of the fixed anchor length was done on the same day as drilling. The capsule and hole annulus were filled with predetermined quantities of cement grout of water cement ratio 0.4. During the installation of the anchors, the plug was grouted first, then the capsule and the fixed length outside the capsule. Usually, the anchor was then stressed after the grout tube strength reached at least 28 MPa. In this instance, the test anchor was stressed when the grout's cube strength was 40 MPa. However, for the permanent anchors, the grout's cube strength at the time of stressing was only 21 MPa. No water test was undertaken since the grouting pressure could be held for the test anchor. The level of the top of the grout was monitored and no drop in this level was observed.

### CONCLUSIONS

The case history on the use of permanent rock anchors founded in weak rock in an office building in Singapore shows that permanent rock anchors can be used in multi-storey building located within a congested urban area despite the onerous requirements of the architect. This is confirmed by the satisfactory performance of the office building in service to date since 1981. In addition to others, an important acceptability criterion of the anchor is that differential settlements of the building (due to the reduction of extension of the anchor as a result of the reduction of live load) is within tolerable limits. Creep of anchors founded in highly weathered shale/sandstone is not significant. Even though the test anchor is not loaded to failure, the ultimate grout/rock bond stress can be determined from a Southwell's plot of load and immediate permanent displacement of the

test anchor. The grout/rock bond stress of an anchor founded in slightly to highly weathered shale/sandstone of the Jurong Formation in Singapore can be taken as 640 kPa for a safety factor of two.

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