Underpinning a multi storey building using micropiles
Les réprises en sous-oeuvre dans un édifice utilisant des micropieux

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SYNOPSIS : This paper describes the underpinning of a three storey office building using Ropress micropiles. The building had been founded on shallow spread footings located in soils residual from weathered granite. Subsequent leakage of water from a collector drain led to severe settlement of part of the building. The geotechnical conditions at the site are discussed and the causes of the settlement are presented. The design and construction of remedial piles socketed into rock are discussed. Typical pile test results are given and back-analyzed to estimate the load carried in side shear and end bearing.

1 INTRODUCTION

A large three storey office building was constructed on a site to the north of Johannesburg in South Africa. A reinforced concrete framed structure was provided comprising of columns and flat slabs. Brick and glass exterior panels and gypsum board and brick interior partition walls were fitted. The building comprises of three blocks, labelled A, B and C radiating outwards from a central lobby. Blocks A and C were constructed in areas of cut and fill respectively whilst Block B was constructed at ground level. Based on the results of a foundation investigation the designers provided conventional spread footing foundations located at average depths of 1,5m to 2,0m below ground level.

In Block B a 500mm diameter stormwater drain was routed under the centre of the structure. This drain discharged water collected from the roof of the building via downpipes located inside selected columns.

The building was completed in 1982 and occupied by the owner. Later that year it was noted that a number of cracks had formed in some of the interior brick partition walls on the upper floor of Block B. Minor cracks were noted in Blocks A and C. The designers were notified and level measurements were made. Widespread cracking occurred in the following two years and in July 1985 the owners appointed the first writer's firm to identify the causes of the movement, and to assist in the selection of remedial measures and in the supervision of their installation.

Regular monitoring of level stations in Block B was instituted in late 1984 and early 1985.

2 SETTLEMENT HISTORY

A floor plan of Block B is presented in Fig 1. This plan shows the location of the columns and the settlement profile of the upper floor as at September 1985. This profile was prepared assuming that the floor had been constructed level. The location of the collector drain is also shown on the figure.

Figure 2 shows plots of the settlement records with time for two of the level stations. These reveal that settlement occurred in a series of "steps" characterized by rapid initial settlement followed by a decreasing rate of movement. Analysis of rainfall records show that the "steps" may be correlated with periods of heavy rainfall.

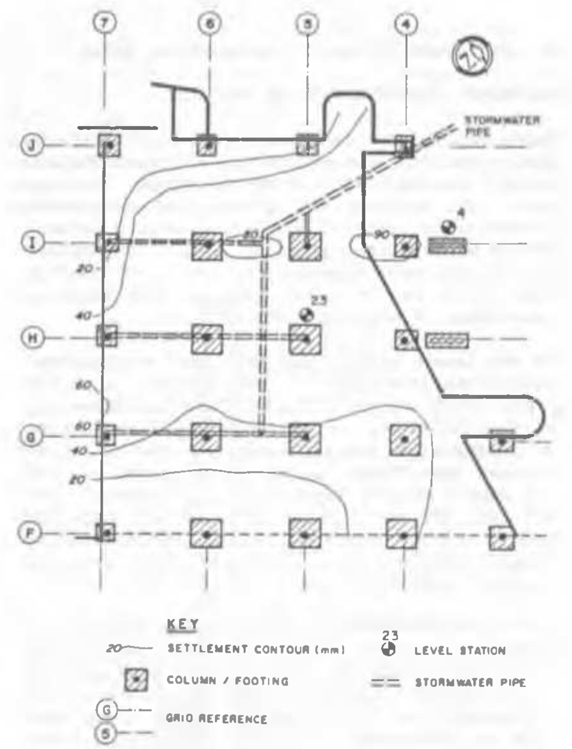

Figure 1: Floor Plan of Block B.

Structural analysis of Block B showed that the deformations had caused a pronounced redistribution of stresses in the structural frame. It was shown that additional large settlements would cause overstressing of certain elements.

Evaluation of the upper floor slab profiles in Block B showed angular distortions between the columns of between 1/1000 and 1/2000. At a few locations angular distortions of up to 1/200 were recorded. The settlement profile resulted in most of the slabs deforming in the hogging mode. This led to severe cracking of the brick walls supported on the slabs ie crack widths of between 5 to 15mm (Burland et al 1977).

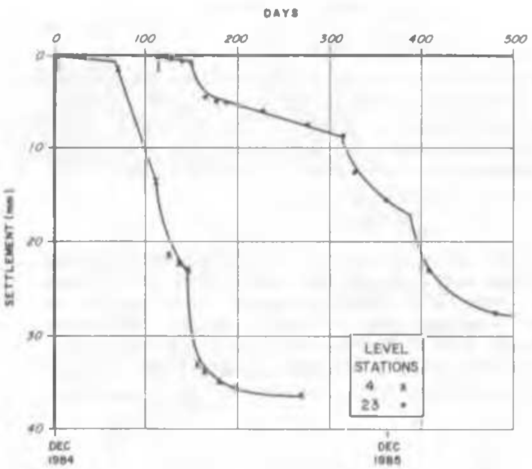

Figure 2 : Settlement Records - Upper Floor, Block B

3 GEOTECHNICAL INVESTIGATION AT THE SITE

The original foundation investigation conducted at the design stage involved the excavation of seven shallow test pits and the sinking of three boreholes for cone SPT tests. An opinion was given that reasonable founding conditions occurred at depths ranging between 2 to 3 metres below existing ground level. Conventional spread footings were subsequently specified founding at between 1,5m to 2m below surface and exerting bearing pressures of about 250 kPa.

After the settlement problem had developed a supplementary geotechnical investigation was carried out at the site by the first writer's firm. This investigation involved the drilling of eleven augered trialholes 760mm in diameter at the perimeter of the building. The trialholes were taken to practical refusal of the machine at depths ranging from 7,8 to 14,5 metres. All trialholes were dry and stable, and each was inspected and logged in situ. Disturbed bag samples and undisturbed block samples were recovered from selected horizons within each trialhole.

Continuous Standard Penetration Tests were carried out in six boreholes. Fifteen plate tests were carried out in seven of the trialholes. These tests were made by jacking two 200mm diameter circular plates into opposing sidewalls of the trialhole. All tests were made at the in situ moisture content of the soil using test procedures described by Wrench (1984).

A programme of laboratory testing was conducted which included Atterberg Limit tests, grading analyses and oedometer tests made both at the in situ moisture content and after saturation of the sample.

The trialholes showed that the site is underlain by granites of the Archean complex. These granites were found to have weathered to varying depths due to preferential weathering along joint planes. The geological profile encountered across the site was generally uniform and may be summarized from surface as follows:

0 - 0,8m Topsoil/Fill: generally sand and gravel;

0,8 - 1,1m Hillwash: loose to medium dense sand;

1,1 - 1,5m Pedogenic zone: medium dense gravel in a matrix of sand;

1,5 - 11m Residual Granite:
* loose, silty fine sand;
* medium dense becoming very dense with depth, silty sand;

below 11m Granite: very soft rock and soft rock, massive granite with pegmatite veins.

No groundwater was encountered in the holes.

4 EVALUATION OF THE CAUSES OF SETTLEMENT

The laboratory tests showed that the soils residual from the granite have a collapsible grain structure and an average collapse potential of about 1%. The degree of saturation (Sr) of the in situ residual granite samples ranged from 27% to 62% with a mean of 42%. The dependance of collapse potential on Sr is well documented (Jennings & Knight 1975) and the critical Sr of residual granite soil (ie that value above which negligible additional collapse is expected) is reported to be about 50% (Schwartz 1985).

The samples were all overconsolidated and preconsolidation pressures of between 400 to 800 kPa were measured in the oedometer tests. Average properties of the weathered granite soils is given in Table 1.

Depth range (metres)	No. of samples	Dry density (kg/m^3)	Initial Void Ratio	Collapse Potential %
1,5 - 4,0	17	1558	0,66	1,09
4,1 - 6,0	5	1541	0,65	0,65
6,1 - 10,0	2	1540	0,64	0,38

TABLE 1 : Average properties of residual granite soils

The plate tests were analyzed in accordance with the rectangular hyperbola method (Wrench 1984) to obtain the partially saturated modulus. The results showed modulus values ranging from 10 to 60 MPa. A general increase in modulus with depth was evident.

A settlement analysis was carried out for a 2,5m square footing founded at a depth of 1,5m below surface (typical of the situation at Block B).

Using average parameters from the laboratory and field tests the analysis predicted consolidation settlements of 25mm for the soils at their in situ moisture content. Additional settlements of 27mm were indicated should the soils become saturated. The settlement predictions were made using an average modulus of 20MPa. The plate and oedometer tests showed varying moduli across the site and with depth. Consolidation and collapse settlements of up to 90mm were predicted for the south eastern corner of Block B where lower modulus and larger collapse potential values were measured.

The observed settlements at the south eastern portion of Block B were close to the predicted values. Over the remainder of Block B, however, the measured settlements were smaller than predicted suggesting that additional settlements may be expected should the soils become saturated.

After comparing the predicted and observed settlements and the location of the collector drain, it was concluded that there was a high probability of additional large settlements occurring under Block B. The drain leading under Block B was identified as the likely source of water causing saturation of the soils beneath the footings. It was therefore recommended that remedial measures be implemented to stabilize the building and prevent further movement.

5 DESIGN OF REMEDIAL MEASURES

The trialholes all refused on granite rock of at least soft rock consistency. Available geological information (Brink 1983) indicates that this rock type is massive and that the consistency invariably improves with depth.

After evaluation of various alternatives it was concluded that Block B should be underpinned using piles taken through the potentially collapsible residual granite soils and founded in the granite rock. The owner required that piling be undertaken without taking the first and second floors out of use. This requirement meant that the piling system adopted had to be both quiet and vibration free. A maximum head room of only 2,5m was available in the ground floor section of the building. After reviewing available pile types the Ropress micropile was selected since the pile meets the required technical specifications and the noise and vibration requirements.

To underpin an existing building the pile is installed by first drilling a 175mm diameter hole through the existing footing and attaching a threaded stainless steel sleeve to the footing. The pile shaft is formed by drilling a 150mm diameter borehole through the sleeve and into the underlying soils and rock. The pile is concreted by placing a grouting manchette into the pile hole and pumping grout in stages into the annular space between the manchette and the borehole sidewalls. A mechanical locking device is used to connect the pile to the sleeve in the footing. A sketch showing a typical completed pile is given in Fig 3.

Evaluation of the loads required for underpinning the columns under Block B were complicated since the designers wished to lift parts of the building to relieve stresses on certain elements. In such applications the load required to lift a column is frequently substantially larger than the design load on the column. After discussion with the structural engineers it was decided to design the micropiles to carry an axial load of 50 Tonnes.

6 BACK ANALYSIS OF PILE TEST RESULTS

A feature of the micropile is that each pile is test loaded during the transfer of load onto the pile. The contract document required that load-deformation plots be prepared for each pile during this operation.

Eighty four piles were installed under thirteen bases in groups of four, six or eight piles. Detailed measurements were made of the drilling depths, the material type and the grout volumes placed.

Pile tests were carried out on each pile after allowing sufficient time for the grout to harden. Load was applied in stages up to 120% of the working load, the pile unloaded in stages and then reloaded to 75% of the working load. Settlements at the head of the pile were measured and an allowance for the elastic shortening of the steel shaft made. Results for three typical tests are presented in Fig 4.

All of the tests showed initial loading curves having a hyperbolic shape. This result is typical of pile tests carried out in the elastic range of settlement. For these types of results the loading curve may be analyzed by transforming the data and plotting to yield a straight line relationship the slope of which allows the ultimate pile capacity to be estimated. The transformed plots of the three pile tests presented above are shown in Fig 5. Transformed plots were made for ten of the pile tests and the ultimate pile capacity of each pile estimated. The results of these analyses are summarized in Table 2 which also shows the settlement of each pile at the working load and the residual deflection on completely unloading the pile. Estimates were made of the length of the rock socket formed in very soft rock and in soft rock granite by analyzing the drilling records. These lengths are also given on the table.

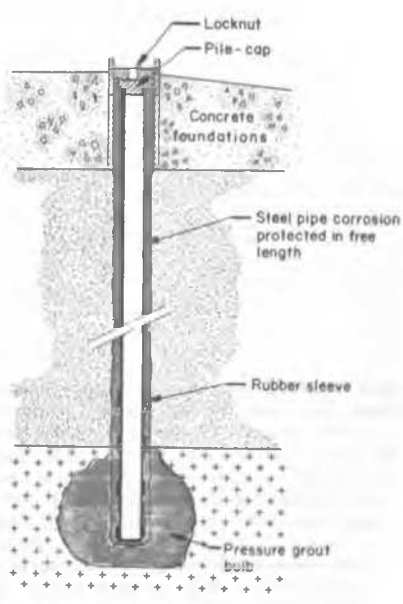

Fig 3 : Configuration of Ropress Pile

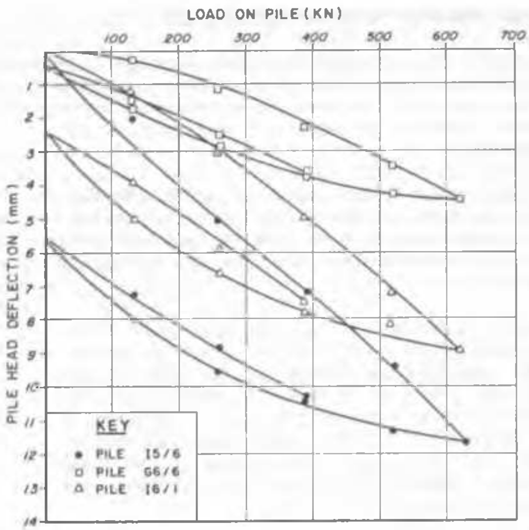

Fig 4 : Typical Pile Test Results at Block B

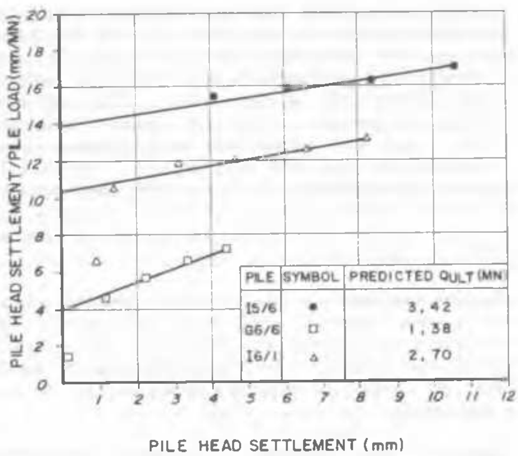

Fig 5 : Transformed Plots of the Pile Test Results

TABLE 2 : Predicted ultimate pile capacities and test data for ten typical Micropiles on the site.

Pile	Pile Settlement (mm) At 500kN	After Unloading	Socket length (m) V.Soft Rock granite	Soft Rock granite	Predicted Q(ult) kN
G4/1	9,2	5,1	2,7	1,5	3900
G5/1	5,4	0,5	3,0	1,5	2250
G5/5	9,4	2,0	3,1	1,4	2150
G6/6	3,3	0,4	3,0	0,8	1380
G7/3	6,8	0,6	2,9	3,2	1400
H4/1	5,6	0,8	2,4	2,2	2110
H5/7	5,2	0,1	4,3	1,3	2090
I5/1	5,9	0,7	5,8	1,5	1850
I5/6	9,3	5,7	3,8	2,9	3420
I6/1	6,5	2,2	5,9	1,3	2700

The ultimate pile capacity was also estimated using the predictive method proposed by Williams, Johnston and Donald (1980). The method allows the proportion of load carried by base resistance and in side shear in the rock socket to be estimated. Calculations were made for a 5 metre socket in the granite rock and using the measured settlements at working load from the appropriate pile tests. Estimates of the degree of jointing and of the unconfined compressive strength of the rock were also made. The method makes use of relationships derived from tests on Melbourne sandstones and, although not checked for other rock types, the above authors propose its general use for designing bored piles in rock.

The calculations suggest that approximately 90% of the pile load is carried in side shear in the rock socket. Average mobilized side stresses in the socket of 240 kPa and end bearing stresses of 1650 kPa are predicted. The method also allows the ultimate pile capacity to be predicted and showed that the piles should carry a load of 1560 kN. This may be compared with the ultimate pile capacities estimated from back-analysis of the pile test results (Table 2). Four of the piles have socket lengths of about 5m and show estimated capacities of approximately 2000 kN.

The stability of Ropress piles with respect to buckling and bursting resistance have been studied by Mascardi (1982). This work showed that only in the weakest of soils, such as very loose silts and peats, is the possibility of failure through insufficient lateral restraint feasible. The plate tests indicate that considerable lateral restraint is provided by the granite soils. A sufficient factor of safety against buckling was therefore obtained.

After construction and curing of the piles, load was transferred from the underpinned bases onto the pile groups by jacking pairs of piles. This operation required calculation of the maximum load that could be applied to the structure at each column to avoid applying too great an uplift and inducing additional cracking in the structure.

CONCLUSIONS

The piling solution adopted to underpin the building proved to be efficient and met the client's requirements of minimum noise and vibration. Analysis of the pile test results showed that a factor of safety in excess of 2,5 was achieved for bearing capacity and that the piles carry their load primarily by mobilizing side shear in the rock socket. The amount of load carried in side shear in the soil is unknown but the analyses suggest that it is probably small.

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ACKNOWLEDGEMENTS

The authors wish to acknowledge the assistance received from Mr P Sagato of Rodio, who controlled the installation of the piles and carried out the pile tests.