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C. R. I. Clayton
University of Surrey, U.K.

J. Milititsky
U.F.R.G.S. Porto Alegre, Brazil

L. J. L. Carvalho
Higgs and Hill plc, U.K.

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Foundation Design and Construction for a Large Mill Complex

C. R. I. Clayton, J. Milititsky, and L. J. L. Carvalho

University of Surrey, U.K., U.F.R.G.S. Porto Alegre, Brazil, Higgs and Hill plc, U.K.

SYNOPSIS Very little good field data exists concerning the performance of heavily loaded end-bearing piles on thin layers of weak rock. The problems associated with the foundations for silos are often severe, since loads are normally heavy, and allowable differential settlements are often very small. The paper describes just such problems, associated with the construction of a flour mill complex, where the principal problems were associated with the uncertainties of pile performance. On the basis of the uniaxial unconfined compressive strength of the supporting rock the end-bearing piles supporting silos within the mill appeared to be overloaded. A programme of slow maintained load pile tests demonstrated that the piles in fact performed very well. Long term settlement records of the loaded structure have confirmed this.

INTRODUCTION

This paper describes the design, construction and long-term behaviour of the piled foundations of a large mill complex in England. The complex, containing heavily loaded silo buildings some 50m high, was successfully constructed on difficult subsoil, consisting of approximately 9m of made ground, a layer of 2-3m of limestone, and beneath it a very stiff to hard fissured silty clay extending to depth.

THE STRUCTURE

The structure is a flour mill, with associated wheat and flour silos, tempering bins, warehouse, bulk tanker outload, and office block. The total construction cost, including machinery, was of the order of £14 million sterling, in 1982. Figure 1 shows the completed mill, with the Wheat Silo at the left hand side and the Flour Silo at the right hand side of the photograph. The two silos presented the principal design problems. The Wheat Silo was the most heavily loaded structure, with approximate dimensions and loadings shown below (see also Figure 2):

Foundation slab: 33.2m x 15.9m x 1.6m thick
Silo area: 26.3m x 15.9m
Height: 45 m
No. of bins: 15
Design wheat load: 9000 t
Probable max. wheat load: 8100 t
Structural dead load: 6000 t

The remainder of the foundation slab is occupied by grain elevators and pre-clean equipment, which do not receive an appreciable live load. The 15 silo bins are square in cross-section and were slip-formed using cast-in-situ reinforced concrete. The silo bins commence 5.4 m and terminate 34.4 m above the foundation slab, and are supported either by

columns set in the external silo walls, or internally by 10 cruciform reinforced concrete columns.

SITE INVESTIGATION

An initial routine site investigation was carried out using the normal techniques, for the U.K., of light percussion cable boring, 100 mm diameter thick-walled open drive hammered sampling, and standard penetration testing. In addition one rotary hole was made using double tube swivel type corebarrels and air flush. The site was known to be a recently infilled ironstone quarry, and the subsoil identified by the first investigation was:

| Description | Thickness |
|---|-----------|
| Made Ground—generally firm brown or grey CLAY with scattered gravel | 8.4–9.7m |
| Rock—moderately weak to moderately strong LIMESTONE | 2.5–3.9m |
| Clay—dark grey silty CLAY of soft to hard consistency (Lias Clay) | to depth |

In an investigation for a nearby site the average undrained shear strength of the fill was found to be 51 kN/m².

FOUNDATION DESIGN CONSIDERATIONS

Whilst the Silo bins form a relatively rigid structure, the supporting columns and foundation slabs are particularly sensitive to differential settlement. It was apparent from preliminary analyses that the structure could not tolerate significant settlement if this produced bending deflections across the foundation slab, without suffering structural distress. It was essential therefore to restrict the differential settlement to about



Fig. 1. Photograph of completed mill complex.

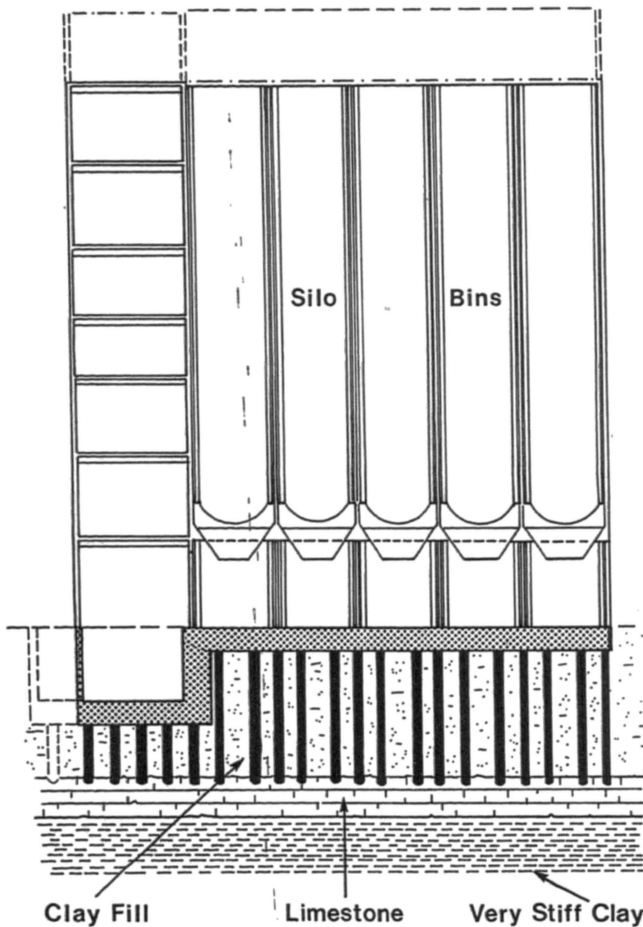


Fig. 2. Wheat Silo - Longitudinal Section

4.5 mm across the foundation in order to contain bending stresses and thus keep the slab reinforcement to manageable and practical proportions. Yet the Silo imposes an average stress of 350 kN/m^2 at foundation slab level, is of considerable area, and was to be constructed on fill overlying relatively thin, poor quality rock, beneath which lay clay.

It was clear that careful foundation design was required, and the most favourable option appeared to be a piled foundation. The presence of rock at relatively shallow depths suggested the use of driven piles. As it was necessary to proceed rapidly with foundation construction, tenders were invited for piles from three specialists piling contractors. After careful appraisal an offer using about 700 precast and "in-situ" shell piles was accepted, as offering a number of advantages detailed below.

Rock Core

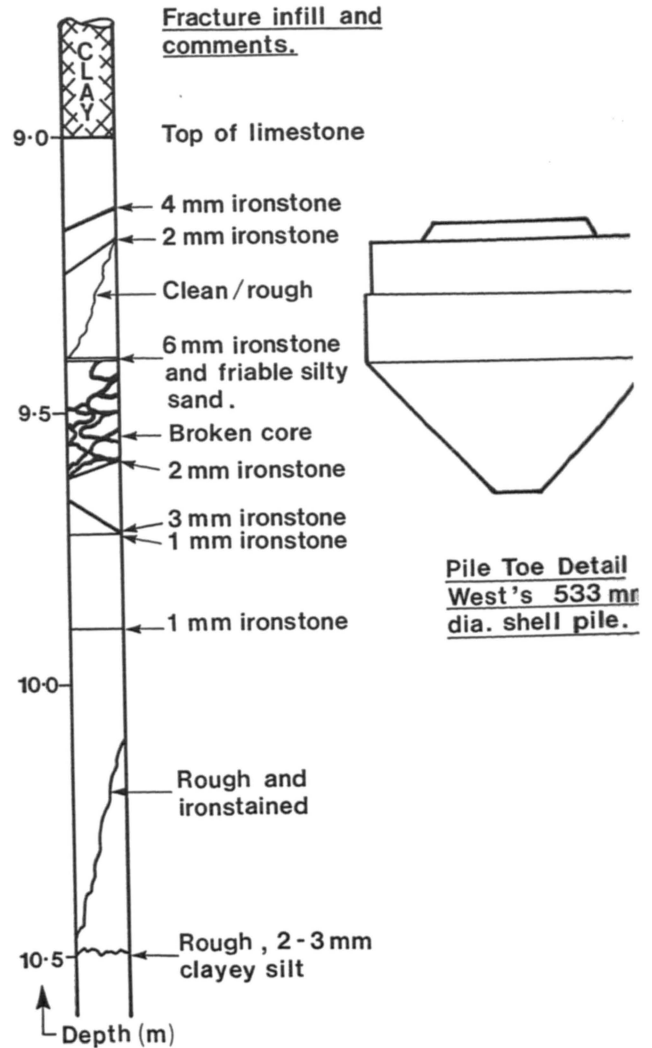


Fig. 3. Fracture log for a typical core, and pile toe section, to same scale

The piles have a conical toe (Figure 3), which were expected to bed better into the rock than, for example, a pile with a flat base. Because shell piles were to be used, the integrity of the piles could be guaranteed with less site quality control (a significant advantage on a fast track project such as this), and the high strength concrete shells also provide additional pile strength and resistance to sulphate attack. In addition, the concrete

core could be poured to required cut-off levels, rather than pile platform level.

The major advantages of the shell piles from the geotechnical point of view, however were that

- (a) although the toe stresses were high, they were less than offered by other piling contractors, and
- (b) the offer included pre-boring, monitoring, and re-driving should pile heave become a problem due to adjacent pile installation.

PILE FOUNDATION DESIGN

Six pile design problems were required to be solved:

- (i) The end-bearing resistance available to the piles from the rock
- (ii) The undrained shear strength of the clay beneath the rock
- (iii) The thickness of the rock, and its likely variation across the site
- (iv) The minimum rock thickness sufficient to prevent the piles punching through the rock into the underlying clay
- (v) The magnitude of negative skin friction on the piles
- (vi) The magnitude of differential settlements.

FURTHER SITE INVESTIGATIONS

To solve the problems posed by the ground and the problems required to be solved in the design of the piled foundation to meet the strict criteria for settlement, a further phase of site investigations was initiated to refine the parameters available for design and prediction of behaviour of the foundation.

An investigation was carried out during piling, and consisted of 6 drillholes. Openholing was carried out to just above anticipated rockhead, followed by continuous rotary coring to depths of up to 37 m below ground level. Coring was carried out using P and S sized double tube swivel type corebarrels with large handset diamond bits, Mylar liners, and a thick bentonite mud flush. Excellent core recovery was achieved, especially in the Lias Clay where total core recoveries of 100 % were normal.

The limestone, which had formed the floor of the former quarry, was found to vary in thickness between 2.12 m and 3.10 m. On an adjacent site four rotary drillholes carried out after the mill construction was complete proved thicknesses of rock between 2.70 m and 2.90 m. Nowhere was the rock found to be as thick as the maximum value previously indicated by light percussion boring.

23 uniaxial unconfined compressive strength tests were carried out on 38 mm diameter soaked specimens of rock prepared from the core. The results are shown in Figure 4, plotted as a function of distance below rockhead. The results of tests carried out on 25 mm diameter soaked specimens from the adjacent site are also shown on Figure 2. The minimum unconfined compressive strengths, near to the top of the rock, were of the order of 2 MN/m²; this material was both weak and friable and

specimens were difficult to prepare. Figure 3 shows a fracture log for a typical core, together with a section through a 533 mm diameter West shell pile toe.

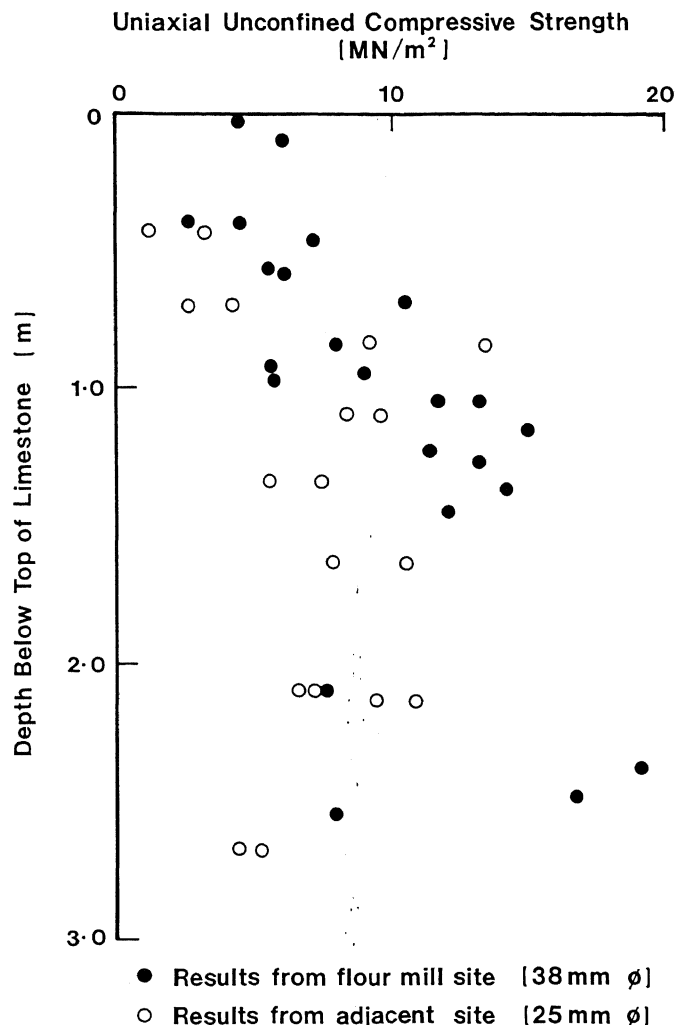


Fig. 4. Uniaxial unconfined compressive strengths of the rock

The underlying Lias Clay was found almost universally to be very stiff to hard in consistency. Undrained triaxial tests made on 90 mm diameter specimens prepared from the rotary core gave shear strengths of between 100 kN/m² and 450 kN/m². The average undrained shear strength for the Lias lying within 10 m of the underside of the rock was found to be 225 kN/m². The entire length of clay core from the deepest drillhole was split, and pocket penetrometer tests were made every 10 cm down the core, together with sample description. In only 18 out of approximately

250 tests did the unconfined compressive strength fall below 300 kN/m^2 , and these results were clearly associated with the penetration of drilling fluid. Oedometer tests indicated drained Young's moduli of the order of $25\text{-}35 \text{ MN/m}^2$ for an appropriate stress increase, while a limited number of drained triaxial tests on 90 mm specimens gave a more realistic average reload Young's Modulus value of 65 MN/m^2 .

Groundwater was found to lie at or below the level of the top of the limestone.

PILE LAYOUT AND DESIGN

In the U.K. it is common for pile selection to be determined largely by the piling subcontractor who makes the successful bid. In this instance the piling subcontractor was chosen not only because he submitted a competitive tender, but also because his tender included for preboring and re-driving (if necessary) of the driven piles that he proposed to use. This was attractive because it was anticipated that close pile spacings would be required beneath the heavily loaded silos, and that pile heave as a result of soil displacement during adjacent pile driving could be a problem in a large pile group.

Based on a fully flexible structure, maximum pile loads for the fully loaded Wheat Silo were calculated to be between 85 t (corner piles) and 105 t (internal piles), without allowance for negative skin friction. The stiffness of the foundation slab and superstructure, however, meant that the load on piles supporting edge and corner columns would be increased, perhaps by a factor of two. Initially each internal column was estimated to transmit 950 t, and 9 piles were to be used to support it. External columns had loads of about 580 t, and were to be supported by 6 piles. As a result of soil-structure interaction analyses an additional 3 piles were added to each external column group.

The final pile layout for the Wheat Silo foundation slab is shown in Figure 5. 207 No. 533 mm diameter piles were used beneath the Wheat Silo itself, whilst a further 37 No. piles were used to support the remainder of the structure. Typical centre-centre pile spacing was 1.50 m, giving a spacing/diameter ratio of 2.81. Centre-centre pile spacings in the external groups were as close as 1.06 m. Piles were driven by a 6 t hammer falling through 1.0 m, to a set of 10 blows for the last 10-20 mm of pile penetration.

The nominal pile capacity quoted by the piling contractor was 110-120 t. This figure is obtained from considerations of the concrete used in the pile. Permissible concrete stresses in driven and cast-in-situ piles in the U.K. are normally restricted to 25 % of the 28 day minimum works cube strength (CP 2004). For the standard 1:2:4 mix given in CP 114:1957 this corresponded to 5.2 MN/m^2 , since this mix produced 21 N/mm^2 concrete. 5.2 MN/m^2 on a 533 mm diameter pile gives a capacity of 118 t. It is generally felt that this magnitude of toe stress will not produce problems on rock.

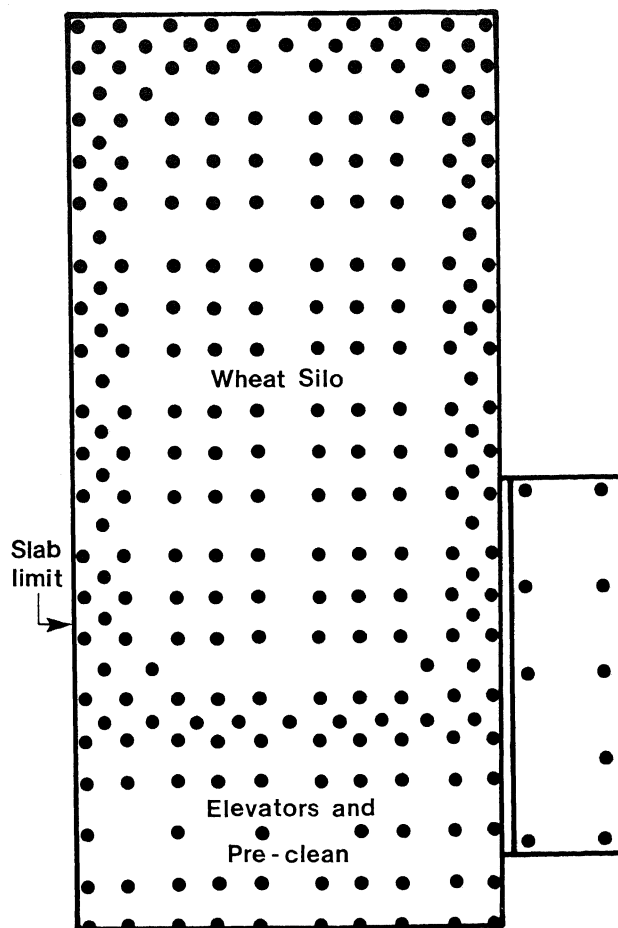


Fig. 5. Final pile layout for Wheat Silo

When safe pile toe stresses are to be calculated it is normal to obtain the stress level using the unconfined compressive strength of the rock. Because the rock generally fractured it is common to recommend that the pile toe stress does not exceed either (a) $1/3$ rd to $1/5$ th of the unconfined compressive strength of the rock (for example, see Bowles (1978)), or (b) $1/4$ of the 28 day minimum works cube strength of the concrete.

The performance of piles in rock is highly dependent on the way in which the rock is modified by pile installation. Information relating to end-bearing capacity is available for socketed piles in the Proceedings of the International Conference on Structural Foundations on Rock (1981), and a comprehensive survey of field tests is given in Williams and Pells (1981). For driven piles, however, there is no information relating to full-scale field tests. A number of experimental studies have been carried out to assess the bearing capacity of small diameter steel dowels perpendicular to the surface of intact rock (Ladanyi (1968) Rehnman and Broms (1970,1971)) which indicate that the maximum bearing capacity is of the order of 5 - 15 times the unconfined compressive strength of the rock (Figure 6

Fractures in the rock are known to reduce bearing capacity, but the relationship between loaded area, fracture spacing and the openness of the fracture remains unknown. For piles driven to rock, the situation is further complicated because the exact area of contact, the depth of penetration, as well as the variability of rock quality are largely unknown. Reliable determination of load capacity can only come from experience.

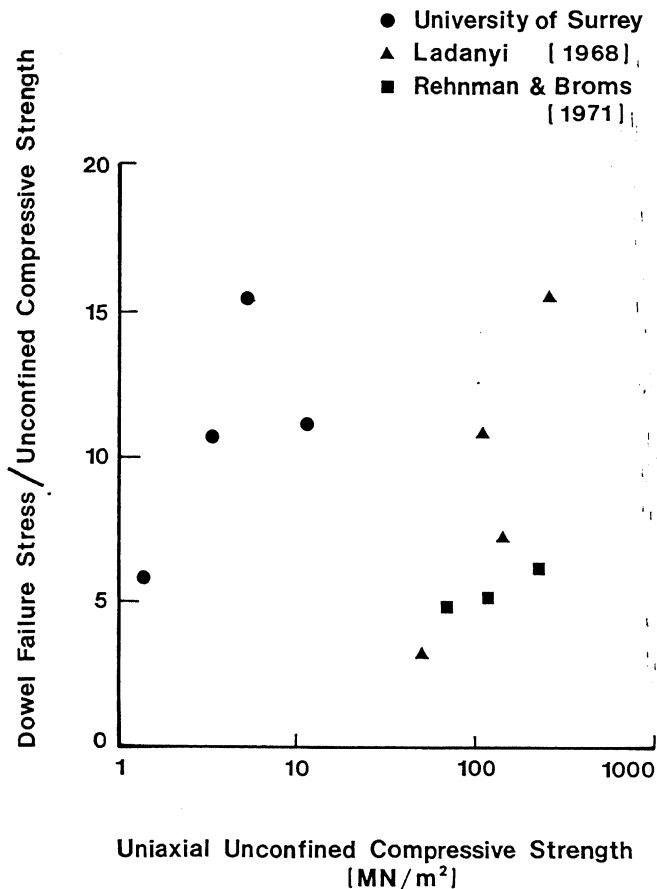


Fig. 6. Results of dowel tests on intact specimens of different rock types

It was quite clear, therefore, that the pile design might be inadequate since the uniaxial unconfined compressive strength at the top of the rock was of the order of 1-5 MN/m² the rock was fractured, and the proposed working toe stress was 5.2 MN/m². Furthermore, the maximum pile load due to the dead load plus the live (wheat and wind) load was calculated as 105 t, which did not take into account the possibility of negative skin friction due to the settlement of the fill following pile installation. Based on an undrained shear strength for the fill of 50 kN/m², a 9 m single pile might expect to attract a maximum downdrag of about 40-75 t (depending on the adhesion factor), but in a group at 1.5 m c/c (both ways) the maximum weight of soil available was estimated at only 37 t. Thus the maximum anticipated pile toe load was of the order of

5 times the minimum unconfined compressive strength of the rock upon which it was to bear. To overcome this problem 13 pile tests were carried out.

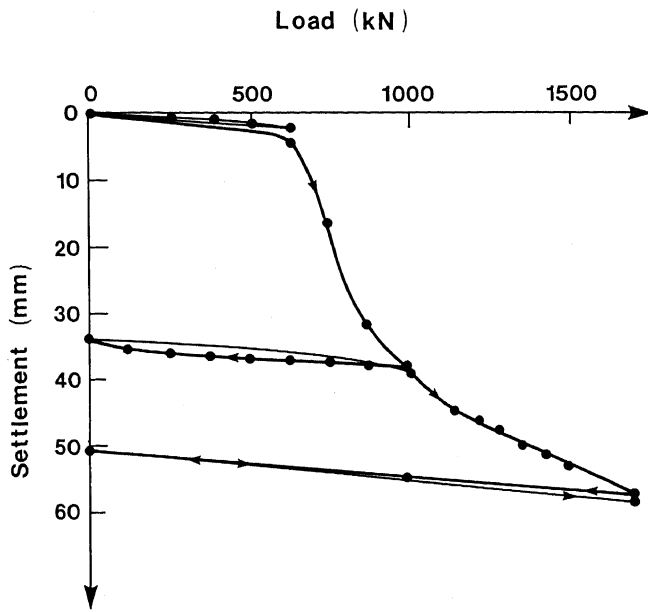
The other problems anticipated before the second phase of site investigation proved to be less intractable. The combination of rock thickness and strength of the underlying clay was thought to be adequate, provided that the piles did not penetrate the top of the rock by any significant amount. The first site investigation had clearly very significantly underestimated the undrained strength of the Lias Clay, had overestimated its variability, and had suggested that the thickness of the limestone was much more variable than it subsequently proved to be.

Although the total settlements anticipated for the Wheat Silo were of the order of 15 times the maximum differential settlement giving bending that could be tolerated, it was thought that the actual differential settlements giving bending would be tolerable. The EI value of the slab was approximately 10⁷ kNm²/m width, but it was estimated that the superstructure might increase the overall structural stiffness to about 10⁸ kNm²/m width. Since the Lias Clay had been proved to be very uniform, it was anticipated (on the basis of simple soil-structure interaction analysis) that for this upper value of stiffness bending across the width of the Wheat Silo foundation slab would amount to less than 1 mm. For the lower value of stiffness, bending of 7 mm was anticipated, falling below 4.5 mm for a stiffness of 2x10⁷ kNm²/m width.

OBSERVATIONS

During pile installation both the piling contractor and the supervising staff showed considerable reluctance to pre-bore the piles. In order to establish that pre-boring was not necessary the piling contractor drove three 444 mm diameter pile in line at 4.2 m centres, and by conventional levelling established that no measureable pile heave occurred. A considerable number of piles were then driven without pre-boring before the design engineer could reverse the decision, which he did on the basis that 10 % of the fill would need to be displaced or compressed in the areas beneath the silos; even though the made ground had been placed without compaction it did not contain noticeable air voids and therefore heave appeared inevitable in the closely spaced pile groups if pre-boring did not take place. Subsequently pre-boring took place to 6 m below ground level.

Observations of selected piles during driving indicated that the piles were penetrating up to 0.3 m into the top of the rock before the set was reached. Typically the penetration was between 0.1 m and 0.15 m, demonstrating that sufficient rock remained beneath the toes of the piles to spread the load onto the Lias Clay.



533 mm ϕ
 10.1 m long
 6 tonne hammer dropping 1.0m
 10 blows for last 20 mm penetration

Fig. 7. Results of first slow maintained pile load test

Since the end-bearing pressures of the piles could not be justified on the basis of laboratory test results and calculations, a programme of slow maintained load testing was carried out. 9 piles were tested to 1.5 times their nominal capacity. The first pile to be tested (Figure 7) failed at a load of 65 t, and underwent 38 mm of settlement under a load of 100 t. After a settlement of about 45 mm, however, bearing capacity improved, indicating that this pile had heaved away from the rock. This pile was in a group of about 70 piles with a spacing/diameter ratio 2.72 in one direction and 3.75 in the other which had not been pre-bored, and the obvious explanation was that a shaft friction of 65 t had been mobilised, and that the pile had then re-seated itself upon the top of the rock. Therefore some 320 piles were re-driven. During re-driving the pile heads were levelled and it became clear that 6m of preboring had not entirely eliminated pile heave. 90 % of all the pile movements recorded during retapping were less than 80 mm, with 50 % less than 30 mm. Subsequent pile load tests gave satisfactory results, with maximum settlements under 165 t ranging from 5.5 to 8.0 mm, and residual settlements after unloading between 0.4 and 1.7 mm.

Because of the uncertainties regarding the shedding of load by a relatively rigid structure onto the outer piles of the group,

combined with the problems of determining the end bearing capacity of the piles on the rock a further three 533 mm diameter piles were installed outside the silo areas and tested to 2½ times their nominal capacity. Maximum settlements under 250 t ranged from 8.8 to 10. mm, with residual settlements after unloading of between 1.4 and 1.7 mm.

MONITORING SETTLEMENTS

Construction of the Wheat Silo took place between October 1981 and May 1982. During construction settlements were, rather unsatisfactorily, measured using conventional levelling. Precise levelling using Building Research Establishment settlement station (Cheney (1974)) commenced at the end of construction but before the silo was loaded with wheat. Computer records of individual loadings within the silos gave a precise idea of the progress of loading and unloading, and it was found that settlements followed upon loading within a period of less than one month. Thus consolidation was barely detectable. The maximum wheat load applied to date has been 7000 t, and total settlements of the Wheat Silo foundation slab at this load are shown on Figure 8. Maximum settlements of the order of 16 mm have been observed, with the silo tilting away from the lightly loaded elevator and pre-clean area. Longitudinal twisting of the slab has occurred, but bending deflections across the slab appear to be less than 0.8 mm.

DISCUSSION AND CONCLUSIONS

The initial routine site investigation showed that light percussion boring cannot estimate the thickness of thin rock layers with adequate accuracy. Furthermore, while coring with air flush and double tube swivel type corebarrel failed to provide satisfactory estimates of the undrained shear strength of a very stiff hard clay, the use of bentonite flush, mud liners, and large diameter corebarrels was sufficient to produce very good quality samples for laboratory testing.

Pile heave will be a serious problem where closely spaced piles are used to support structures sensitive to differential settlement. In the present case the fill is reasonably homogeneous, and penetration into high strength stratum did not occur. Despite this, and the fact that the volumetric displacement ratio was of the order of 6 - 1 times the critical limit suggested by Brierley and Thompson (1972), the observed pile heaves were very much less than would be predicted by Hagerty and Peck's (1971) approach. On the other hand Cole's (1972) method predicts pile heaves for the pile in Figure 7 of between 4 mm and 50 mm depending on the sequence of driving, which is in good agreement with the observed settlement upon first loading. For the more closely spaced pile groups beneath the Wheat Silo, heaves of the order of 65 - 105 mm are estimated by this method. During re-driving, 82 % of the piles settled less than 65 mm, and 96 % settled less than 105 mm.

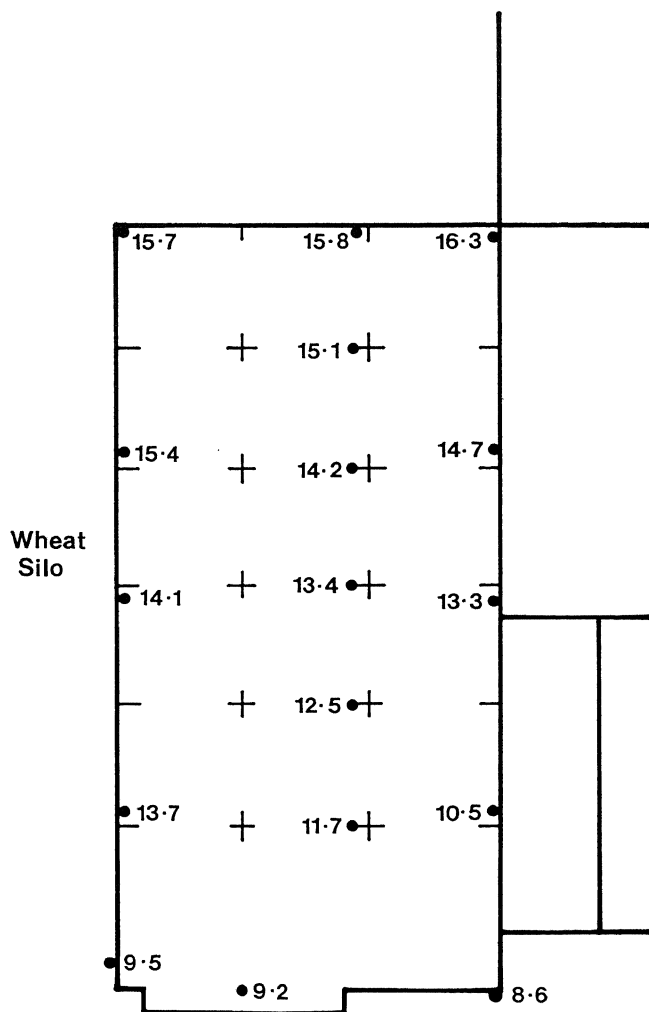


Fig. 8. Total settlements of Wheat Silo under 7000 t live load (4/10/83)

Partial preboring was not enough to prevent pile heave. In hindsight it was felt that preboring should have been taken to within 0.5 m of rockhead, but even so it would have been prudent to redrive. It has been argued (Cole (1972), Young and Thorburn (1981)) that limiting pile heave by decreasing pile displacement is impractical, but it is equally clear that if preboring is not carried out then re-driving may give rise to a second phase of pile heave. Therefore a combination of preboring, monitoring and re-driving is essential when closely spaced piles are to be driven through clay.

The first pile load tests indicated shaft adhesion contributing about 65 t to the pile capacity, at a settlement of 2-3 mm (or about 0.5-1.0% of the pile diameter). These figures are in accordance with full mobilisation of the undrained shear strength of the made ground, at displacements in common with those observed elsewhere. (Whitaker and Cooke (1966)).

The three 250 t pile load tests therefore

probably applied pile toe stresses of the order of 8.1 MN/m^2 , with maximum concrete stresses within the pile shaft of 11.0 MN/m^2 . Although a slight acceleration of settlements between loads of 235 t and 250 t in two of the three tests perhaps indicates that pile failure was not far off, the rock sustained stresses which must conservatively be estimated at 3 times its uniaxial unconfined compressive strength. The rock mass was by no means intact, and yet the piles behaved well.

It should be noted that where negative skin friction is anticipated, the maximum load for load tests on end bearing piles requires very careful consideration. For example, in this case, a 533 mm diameter pile designed to accept a structural load of 75 t might transfer 110 t to the rock once negative skin friction had been fully mobilised, while a pile test to 1.5 times the structural load would impose 110 t at the top of the pile, but only 45 t on the rock at the toe of the pile. Such a test would clearly be inadequate.

Subsequent observations of the loaded structure have demonstrated that the piles have performed satisfactorily under sustained load. The maximum bending deflections have been of the order of 0.8 mm, which confirms the importance of the superstructure in determining the overall structural stiffness.

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REFERENCES

- Bowles, J.E. (1978), "Foundation Analysis and Design, 750 pp., McGraw-Hill, New York, 2nd Ed.
- Brierley, G. and D. Thompson (1972), Discussion. Proc. A.S.C.E., S.M.F. Div., vol. 98, SM11, 1273-1275.
- Cheney, J.E. (1974), "Techniques and Equipment Using the Surveyor's Level for Accurate Measurement of Building Movement", Proc. Symp. Field Instrumentation in Geotech. Engng, Butterworths, London, 85-99.
- Cole, K.W. (1972), "Uplift on Piles Due to Driving Displacements", Civ. Engng. and Pub. Wrks. Review, Vol. 67, March, 263-269.
- CP114 (1957), "The Structural Use of Reinforced Concrete in Buildings", British Standards Institution, London.
- CP2004 (1972), "Code of Practice for Foundations", British Standards Institution, London.
- Hagerty, D.J. and R.B. Peck (1971), "Heave and Lateral Movements Due to Pile Driving", Proc. A.S.C.E., Journ. S.M.F. Div., Vol. 97, SM11, 1513-1532.

- Ladanyi, B. (1968), "Rock Failure under Concentrated Loading", 10th Symp. on Rock Mech., Austin, Texas.
- Pells, P.J.N. (ed.) (1981), Proceedings of the International Conference on Structural Foundations on Rock", Sydney, May 1980.
- Rehman, S.-E. and B. Broms (1970), "Bearing Capacity of End-bearing Piles Driven into Rock", Proc. 2nd Cong. Int. Soc. Rock Mech., Belgrade, Vol. 2, 1-8.
- Rehman, S.-E. and B. Broms (1971), "Bearing Capacity of Piles Driven into Rock", Can. Geotech. Journ., Vol. 8, 151-162.
- Whitaker, T. and R.W. Cooke (1966), "An Investigation of the Shaft and Base Resistances of Large Bored Piles in London Clay", Proc. I.C.E. Symp. on Large Bored Piles, London, 7-49.
- Williams, A.F. and P.J.N. Pells (1981), "Side Resistance of Rock Sockets in Sandstone, Mudstone, and Shale", Can. Geotech. Journ., Vol. 18, 502-513.
- Young, F.E. and S. Thorburn (1981), "Heave and Downdrag" in "Piles and Foundations", (Young, F.E. (ed.)), Thomas Telford Ltd., London, 152-156.